IRC:SP:13-2022

GUIDELINES FOR THE DESIGN OF SMALL BRIDGES AND CULVERTS

INDIAN ROADS CONGRESS
2022

CONTENTS

		Page
List of Plates		(vi)
Personnel of the Bridges Specifications & Standards Committee		(vii)
Introduction		(ix)
Article 1	General Aspects	1
Article 2	Site Selection & Inventory	2
Article 3	Collection of Design Data	3
Article 4	Empirical and Rational Formulae for Peak Run-off from Catchment	5
Article 5	Estimating Flood Discharge from the Conveyance Factor and Slope of the Stream	13
Article 6	Design Discharge	17
Article 7	Alluvial Streams Lacey's Equations	18
Article 8	Linear Waterway	20
Article 9	Normal Scour Depth of Streams	21
Article 10	Maximum Scour Depth	26
Article 11	Depth of Foundations	28
Article 12	Span and Vertical Clearance	29
Article 13	Geometric Standards, Specifications and Quality Control	31
Article 14	Structural Details of Small Bridges and Culverts	36
Article 15	Elements of the Hydraulics of Flow through Bridges	38
Article 16	Afflux	43
Article 17	Worked out Examples on Discharge Passed by Existing Bridges from Flood Marks	47
Article 18	Overtopping of the Banks	51
Article 19	Pipes and Box Culverts	52
Article 20	Protection Work and Maintenance	59
Article 21	Raft Foundations	61
Article 22	CD. Works in Black Cotton Soils	62
Article 23	Box Cell Structures	63
REFERENCES		64
APPENDICES		
APPENDIX-A	**Filling Behind Abutments, Wing and Return Walls**	65

LIST OF PLATES

Plate No.	Description	Page
1.	Chart for Time of Concentration	67
2.	Run-off Chart for Small Catchments	68
3.	Hydraulic Mean Depth R (METRES)	69
4.	Typical Method of Determination of Weighted Mean Diameter of Particles (d_m)	70
5.	Abutment and Wing Wall Sections for Culverts	71
6.	Details of segmental Masonry Arch Bridges without Footpaths- Effective Span 6m and 9m	72
7.	Typical Details of Floor Protection Works for Box Cell Structures - General Arrangement	73
8.	RCC Pipe Culvert with Single Pipe of 1.2 Metre Dia and Concrete Cradle Bedding for Heights of Fill Varying from 4.0 m to 8.0 m	74
9.	RCC Pipe Culvert with Single Pipe of 1.2 Metre Dia and First class Bedding for Heights of Fill Varying from 0.6 m-4.0 m	75
10.	RCC Pipe Culvert with 2 Pipes of 1.2 Metre dia and Concrete Cradle Bedding for Heights of Fill Varying from 4.0-8.0 m	76
11.	RCC Pipe Culvert with 2 Pipes of 1.2 Metre dia and First class Bedding for Heights of Fill Varying from 0.6-4.0 m	77
12.	Circular and Rectangular Pipes Flowing Full	78

ARTICLE 1

GENERAL ASPECTS

1.1. GENERAL: Occurrence of culverts and small bridges on roads and highways depends upon the type of region and terrain. The location, size and other details of such structures should be decided judiciously to cater for the requirements of discharge and balancing of water level on either side of road embankment. Number of culverts in 1 km length of road in India varies from one (flat country) to three in undulating regions whereas one small bridge (upto 30m) is found within 1 to 4 km length of the road. Number of culverts in hilly/undulating terrain is generally more than in plain region.

1.2. DEFINITIONS

1.2 1. Bridge: A bridge is a structure having a total length above 6 m for carrying traffic or other moving loads across a channel, depression, road or railway track or any other obstruction.

1.2.2. Minor Bridge: A minor bridge is a bridge having a total length of upto 60 m.

1.2.3. Small Bridge: A small bridge is a bridge where the overall length of the bridge between the inner faces of dirt walls is upto 30 m and individual span is not more than 10 m.

1.2.4. Culvert: Culvert is a structure having a total length of upto 6 m between the outer faces of walls measured at right angles. Cross drainage structures with pipes will be termed as culvert, irrespective of length.

1.3. THE SMALL BRIDGES AND CULVERTS CAN BE OF FOLLOWING TYPES:

a) RCC Hume Pipes
b) RCC slab on masonry/concrete abutment and piers
c) Stone slab on masonry/concrete abutment and piers
d) RCC box cell structure
e) RCC/masonry arches on masonry/concrete abutment and piers

Stone slabs can be used upto 2 m span when good quality stones having 200 mm thickness are available.

1.4. DESIGN PHILOSOPHY

1.4.1. General

Bridges shall be designed for specified limit states to achieve the objectives of constructability, safety, serviceability, with due regard to issue of economy, aesthetics and sustainability. Provision of IRC:112 shall be followed for design of concrete structures. Sizes of opening in culverts shall be fixed in a manner which provides sufficient space for inspection and maintenance and to avoid clogging. The limit states specified herein are intended to provide for a constructible, serviceable bridge, capable of safely carrying design loads for a specified lifetime as given in IRC:5. Small bridges and culverts need not be checked for seismic effects.

1.4.2. Limit States

1.4.2.1. Small bridges and Culverts are as important in the infrastructure as a major bridge. Each structural component of the small bridge / culvert shall therefore satisfy the requirements of design as spelt out in various IRC codes and standards as applicable.

1.4.2.2. Serviceability Limit State (SLS): The serviceability limit state shall be considered as per IRC:112.

1.4.2.3. Ultimate Strength Limit State (ULS): Strength limit state shall be considered as per IRC:112

1.4.2.4. Accidental Limit States: The accidental limit state shall consider as per IRC:112

1.4.3. Riding Quality

In order to improve the riding quality, number of expansion joints should be minimized. It is preferable to go for Integral structures / continuous structures.

1.5. STANDARD DESIGNS

1.5.1. Standard designs drawings for slab bridges and RCC Boxes published by Ministry of Road Transport & Highways are presently under revision because of change indesign philosophy from working stress method to limit state design besides incorporating changes in the codal provisions. Hence the revised drawings of slab bridges and RCC boxes, whenever published, shall be applicable for adoption.

1.5.2. Mean while parameters of loadings for design of small bridges and culverts are furnished below:

1.5.3. H.P. Culverts: RCC pipe culverts having minimum 1200 mm diameter of type NP4 conforming to IS:458. PSC pipes ofNP4 type conforming to IS:784 may also be used for H.P. culverts.

1.5.4. Provision of bridge and culvert with respect to Catchment Area: It is generally found that when catchment area is upto 1 sq. km (100 hectares) a culvert is required and for catchment area more than 1 sq. km (100 hectares), a small bridge will be necessary.

ARTICLE 2

SITE SELECTION & INVENTORY

2.1. SELECTION OF SITE: Normally selection of site for culverts and small bridges is guided by road alignment. However, where there is choice, select a site:

(i) Which is situated on a straight reach of stream, sufficiently down stream of bends;
(ii) Which is sufficiently away from the confluence of large tributaries as to be beyond their disturbing influence;
(iii) Which has well defined banks;
(iv) Which make approach roads feasible on the straight; and
(v) Which offers a square crossing.

2.2. EXISTING DRAINAGE STRUCTURES: If, there is an existing road or railway bridge or culvert over the same stream within 500 m from the selected site, the best means of ascertaining the maximum discharge is to calculate it from data collected by personal inspection of the existing structure. Intelligent inspection and local inquiry will provide very useful information, namely, marks indicating the maximum flood level, the afflux, the tendency to scour, the probable maximum discharge, the likelihood of collection of brushwood during floods, and many other particulars. It should be seen whether the existing structure is too large or too small or whether it has other defects. The size and other parameters of existing structures must be recorded. Also, whether the road has been overtopped must be enquired and recorded.

2.3. Inventory should also include taking notes on channel conditions from which the silt factor and the co-efficient of rugosity can be computed.

ARTICLE 3

COLLECTION OF DESIGN DATA

3.1. In addition to the information obtained by inventory of an existing structure, the design data described in the following paragraphs have to be collected. What is specified here is sufficient only for small bridges and culverts. For larger structures, detailed instructions contained in IRC 5 the Standard Specifications & Code of Practice for Bridges –General Features of Design and IRC SP 54 Project Preparation Manual for Bridges should be followed.

3.2. **CATCHMENT AREA:** When the catchment, as seen from the "topo" (G.T.) sheet, is less than 1.25 sq. km in area, a traverse should be made along the watershed. Larger catchments can be read from the 1 cm = 500 m to po maps of the Survey of India by marking the watershed in pencil and reading the included area by placing a piece of transparent square paper over it.

Catchment area can be delineated using topo-sheets of appropriate scale developed by Survey of India. For flat areas where ground elevation difference is very less, catchment area and other physiographic parameters can be estimated by using digital elevation model(DEM) and GIS tools. Catchment area may also be calculated by software's like BHUVAN developed by ISRO /or by Google Earth. images in any GIS software or 3-Dimensional CAD formats, which can be used to delineate even small catchment areas.

3.3. **CROSS-SECTIONS:** For a sizable stream, at least three cross-sections should be takenat right angles to the river alignment, namely, one at the selected site, one upstream and another downstream of the site, all to the horizontal scale of not less than 1 cm to 10 m or 1/1000 and with an exaggerated vertical scale of not less than1 cm to 1 m or 1/100. Approximate distances, upstream and downstream of the selected site of crossing at which cross-sections should be taken are given in **Table 3.1.**

Table 3.1

Catchment Area	Distance (u/s and d/s of the crossing) at which cross-sections should :be taken
1. Upto 3.0sq.km	100 m
2. From 3.0 to 15 sq. km	300 m
3. Over 15 sq. km	500 m

The cross-section at the proposed site of the crossing should show level at close intervals and indicate outcrops of rocks, pools, etc. Often an existing road or a cart track crosses the stream at the site selected for the bridge. In such a case, the cross-section should not be taken along the center line of the road or the track as that will not represent the natural shape and size of the channel. The cross-section should be taken at a short distance on down stream of the selected site.

3.4. In the case of very small streams (catchments of 40 hectares or less) one cross section may do but it should be carefully plotted so as to represent truly the normal size and shape of the channel on a straight reach.

3.5. **HIGHEST FLOOD LEVEL:** The highest flood level should be ascertained by judicial local observation, supplemented by local enquiry, and marked on the cross-sections.Design HFL corresponding to design flood of a given return period can be found from stage-discharge Curve

3.6. LONGITUDINAL SECTION: The longitudinal section should extend upstream and down stream of the proposed site for the distances indicated in Table 3.1 and should show levels of the bed, the low water level and the highest flood level.

3.7. VELOCITY OBSERVATION: Attempts should be made to observe the velocity during an actual flood and, if that flood is smaller than the Design peak flood, the observed velocity should be suitably increased. The velocity thus obtained is a good check on the accuracy of that calculated the oretically. Simplest way is to use a float to find surface velocity (Vs). Mean flow velocity is 0.8Vs.

3.8. TRIAL PIT SECTIONS:

3.8.1. Where the rock or some firm undisturbed soil stratum is not likely to be far below the alluvial bed of the stream, a trial pit should be dug down to such rock or firm soil. But if there is no rock or undisturbed firm soil for a great depth below the stream bed level, then the trial pit may be taken down roughly 2 to 3 meter below the lowest bed level. The location of each trial pit should be shown in the cross-section of the proposed site. The trial pit section should be plotted to show the kind of soils passed through. However, depth of trial pit in soils shall be minimum2 m for culverts and 3 m for small bridges.

For more detailed investigation procedure given in IRC:78 may be referred to.

3.8.2. For Pipe culverts, one trial pit is sufficient. The result should be inserted on the cross-section. '

3.9. Return period or frequency of peak run off.

Return periods or frequency of peak run off for design of small bridges and culverts shall be considered as under:

(i) Small bridges – 100 years

(ii) Culverts -- 50 years

3.10. ISO-PLUVIAL MAPS: Iso-pluvial maps prepared jointly by CWC, RDSO, IMD & MORTH and published by CWC for different regions titled “Flood Estimation Report” give 24 hr. rainfall value of different return periods e.g. 25, 50 and 100 yr. return period. Table-4.1 can be used to convert 24 hr. rainfall of given frequency (to be read from Iso-Pluvial maps) to rainfall corresponding to time of concentration to be determined for the given catchment.

ARTICLE 4

EMPIRICAL AND RATIONAL FORMULAE FOR PEAK RUN-OFF FROM CATCHMENT

4.1. Although records of rainfall exist to some extent, actual records of floods are seldom available in such sufficiency as to enable the engineer accurately to infer the worst flood conditions for which provision should be made in designing a bridge. Therefore, recourse has to be taken to the oretical computations. In this Article some of the most popular empirical formulae are mentioned.

4.2. DICKENS FORMULA

$$Q = CM^{3/4} \qquad ...(4.1)$$

Where

Q = the peak run-off in m^3/s and M is the catchment area in sq. km

C = 11 - 14 where the annual rainfall is 60 - 120 cm

= 14-19 where the annual rainfall is more than 120 cm

= 22 in Western Ghats

4.3. RYVE'S FORMULA : This formula was devised for erstwhile Madras Presidency.

$$Q = CM^{2/3} \qquad ...(4.2)$$

Where

Q = Run-off in m^3/s and M is the catchment area in sq. km

C = 6.8 for areas within 25 km of the coast

= 8.5 for areas between 25 km and 160 km of the coast

= 10.0 for limited areas near the hills

4.4. INGLI'S FORMULA: This empirical formula was devised for erstwhile Bombay Presidency

$$Q = \frac{125M}{\sqrt{M+10}} \qquad ...(4.3)$$

Where

Q = maximum flood discharge in m^3/s

M = the area of the catchment in sq. km

4.5. These empirical formulae involve only one factor viz. the area of the catchment and all the so many other factors that affect the run-off have to be taken care of in selecting an appropriate value of the co-efficient. This is extreme simplification of the problem and cannot be expected to yield accurate results.

4.6. A correct value of C can only be derived for a given region from an extensive analytical study of the measured flood discharges vis-a-vis catchment areas of streams in the region. Any value of C will be valid only for the region for which it has been determined in this way. Each basin has its own singularities affecting run-off. Since actual flood records are seldom available, the formulae leave much to the judgment of the engineer. Also, since the formulae do not consider rainfall, they are unreliable. Many other similar empirical formulae are in use but none of them encompasses all possible conditions of terrain and climate. Since the formulae do not consider rainfall, they are unreliable. C varies also With size of catchment area

4.7. RATIONAL FORMULAE FOR PEAK RUN-OFF FROM CATCHMENT

4.7.1. In recent years, hydrological studies have been made and theories set forth which comprehend the effect of the characteristics of the catchment on run-off. Attempts also have been made to establish relationships between rainfall and run-off under various circumstances. Some elementary account of the rationale of these theories is given in the following paragraphs.

4.7.2. Main factors: The size of the flood depends on the following major factors.

Rainfall

(1) Intensity
(2) Distribution in time and space
(3) Duration

Nature of Catchment

(1) Area
(2) Shape
(3) Slope
(4) Permeability ofthe soil and vegetable cover
(5) Initial state of wetness

4.7.3. **Relation between the intensity and duration of a storm:** Suppose in an individual storm, F cm of rain falls in T hours, then over the whole interval of time T, the mean intensity I will be F/T cm per hour. Now, within the duration T, imagine a smaller time interval t (Fig. 4.1). Since the intensity is not uniform through-out, the critical mean intensity reckoned over the time interval t (placed suitably within T) will be higher than the mean intensity i.e. I taken over the whole period. Mean intensity i.e. I=F/T in cm/hr is less than Ic which is the critical intensity of irainfall in cm/hr corresponding to time of concentration, tc in hrs

It is also known that the mean intensity of a storm of shorter duration can be higher than that of a prolonged one.

In other words, the intensity of a storm is some inverse function of its duration. It has been reasonably well established that

$$\frac{i}{I} = \frac{T+C}{t+c} \qquad \text{...(4.5a)}$$

Where c is a constant

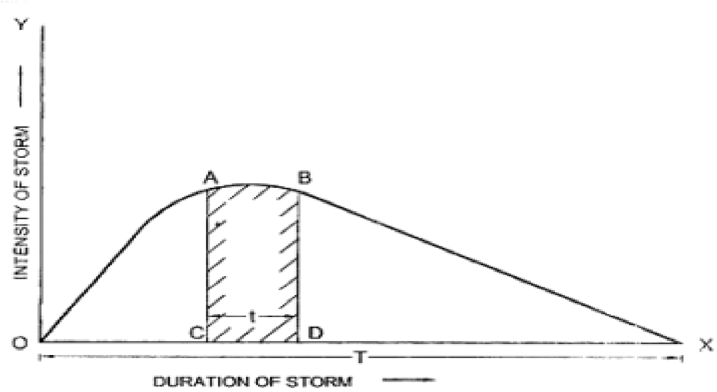


Fig. 4.1

6

Analysis of rainfall statistics has shown that for all but extreme cases, c=1[5]* when time is measured in hours and precipitation in cm.

$$\frac{i}{I} = \frac{T+1}{t+1} \qquad \text{...(4.5b)}$$

$$or\ i = I\left(\frac{T+1}{t+1}\right) \qquad \text{...(4.5c)}$$

Also

$$i = \frac{F}{T}\left(\frac{T+1}{t+1}\right) \qquad \text{...(4.5d)}$$

Thus, if the total precipitation F and duration T of a storm are known then the intensity corresponding to t, which is a time interval within the duration of the storm can be estimated.

*Refers to the number of the publication in the Bibliography.

4.7.4. For an appreciation of the physical significance of this relationship, some typical cases are considered below.

Take an intense but brief storm which drops (say) 5 cm of rain in 20 minutes. The average intensity comes to 15 cm per hour. For a short interval t of, say 6 minutes, within the duration of the storm the intensity can be as high as

$$i = \frac{F}{T}\left(\frac{T+1}{t+1}\right)$$

$$= \frac{5}{0.33}\left(\frac{0.33+1}{0.1+1}\right) = 18.2 \text{ cm per hour} \qquad \text{...(4.6)}$$

Storms of very short duration and 6 minute intensities within them (and, in general, all such high but momentary intensities of rainfall) have little significance in connection with the design of culverts except in built-up areas where the concentration time can be very short (see para 4.7.5.1) due to the rapidity of flow from pavements and roofs.

Next consider a region where storms are of medium size and duration. Suppose 15 cm of rain falls in 3 hours. The average intensity works out to 5 cm per hour. But in time interval of one hour within the storm the intensity can be as much as

$$= \frac{15}{3}\left(\frac{3+1}{I+1}\right) = 10 \text{ cm per hour} \qquad \text{...(4.7)}$$

For the purpose of designing waterway of bridge such a storm is said to be equivalent of a"*one hour rainfall of 10 cm*".

Lastly, consider a very wet region of prolonged storms, where a storm drops, say, 1 8 cm of rain in 6 hours. In a time interval of one hour within the storm the intensity can be as high as

$$= \frac{18}{6}\left(\frac{6+1}{I+1}\right) = 10.5 \text{ cm per hour} \qquad \text{...(4.7)}$$

Thus such a storm is equivalent of a "one hour storm of 10.5 cm".

7

4.7.5. "One-hour rainfall" for a region for designing waterway of bridges / culverts: Since a bridge should be designed for peak run-off resulting from the severest storm (in the region) that occurs once in 100 years, following procedure may be adopted for design of waterway. Let the total precipitation of that storm be F cm and duration T hours. Consider a time interval of one hour somewhere within the duration of the storm. The precipitation in that hour could be as high as

$$= \frac{F}{T}\left(\frac{T+1}{1+1}\right)$$

Or

$$= \frac{F}{2}\ \left(1 + \frac{1}{T}\right)\text{cm}$$

Hence the design of the bridge will be based on a "one-hour rainfall of say I_0 cm", where ...(4.8)

$$I_0 = \frac{F}{2}\ \left(1 + \frac{1}{T}\right)\text{cm}$$

Suppose **Fig. 4.1** represents the severest storm experienced in a region. If *t* represents one hour, then the shaded area ADBC will represent I_0.

It is convenient and common that the storm potential of a region for a given period of years should be characterized by specifying the "one-hour rainfall" I_0 of the region for the purpose of designing the waterways of bridges in that region.

I has to be determined from F and T of the severest storm. That storm may not necessarily not necessarily be the most prolonged storm. The correct procedure for finding I_0 is to take a number of really heavy and prolonged storms and work out / from F and T of each of them. The maximum of the values of/ thus found should be accepted as "one hour rainfall" of the region for designing bridges.

I_0 of a region does not have to be found for each design problem. It is a characteristic of the whole region and applies to a pretty vast area subject to the same weather conditions. I_0 of a region should be found once for all and should be known to the local engineers.

Heaviest rainfall in mm/hour in current years for a particular place may be obtained from Meteorological Department of the Government of India.

Start with I_0 and then modify it to suit the concentration time (see next para) of the catchment area in each specific case. This will now be explained.

Since Design flood for bridges is 100 year and that of culvert is 25 to 50 year, rainfall corresponding to those return periods only should be used for estimating design peak flood. Design rainfall can be found from Intensity-duration curves for different return periods provided such curve are available from IMD for the region in which bridge/culvert is located. However, such curves are rarely available. Iso-Pluvial maps, as mentioned in para 3.10, can also be used to convert 24 hour rainfall to rainfall corresponding to time of concentration to be determined for the given catchment.

4.7.5.1. Time of concentration (t_c): The time taken by the run-off from the farthest point on the periphery of the catchment (called the critical point) to" reach the site of the culvert is called the "concentration time". In considering the intensity of precipitation it was said that the shorter the duration considered the higher the intensity will be. Thus safety would seem to lie in designing for a high intensity corresponding to a very small interval of time. But this interval should not be shorter than the concentration time of the catchment under consideration, as otherwise the flow from distant parts of the catchment will not be able to reach the bridge in time to make its contribution in raising the peak discharge. Therefore, when examining a particular catchment, only the intensity corresponding to the duration equal to the concentration period (**t_c**) of the catchment, needs to be considered.

4.7.5.2. Estimating the concentration time of a catchment (t_c): The concentration time depends on (1) the distance from the critical point to the structure; and (2) the average velocity of flow. The slope, the roughness of the drainage channel and the depth of flow govern the later. Complicated formulae exist for deriving the time of concentration from the characteristics of the catchment. For our purpose, however, the following simple relationship [11] will do

$$t_c = \left(0.87 \times \frac{L^3}{H}\right)^{0.385} \qquad ...(4.9)$$

t_c = the concentration time in hours

L = the distance from the critical point to the structure in km.

H = the fall in level from the critical point to the structure in m.

L and H can be found from the survey plans of the catchment area and t calculated from Equation (4.9).

Plate 1 contains graphs from which t_c can be directly read for known values of L and H.

4.7.6. The critical or design intensity: The critical intensity for a catchment is that maximum intensity which can occur in a time interval equal to the concentration time t_c of the catchment during the severest storm (in the region) of a given frequency I_c.

Since each catchment has its own t_c it will have its own I_c

If we put t = t_c in the basic equation (4.5d) and write I_c for the resulting intensity, we get

$$I_c = \frac{F}{T}\left(\frac{T+1}{tc+1}\right) \qquad ...(4.10a)$$

Combination this with equation (4.8), we get

$$I_c = I_o\left(\frac{2}{t_c+1}\right) \qquad ... (4.10b)$$

4.7.7. Calculation of run-off: A precipitation of I_c cm per hour over an area of A hectares, will give rise to run-off

Where

$Q = 0.028\,A\,Icm^3/s$

To account for losses due to absorption etc. introduce a co-efficient P.

Then

$$Q = 0.028\,PAI_c \qquad ...(4.11)$$

Where

Q = run-off in m^3/s
A = area of catchment in hectares
I_c = critical intensity of rainfall in cm per hour
P = co-efficient of run-off for the catchment characteristics

The principal factors governing P are: (i) porosity of the soil, (ii) area, shape and size of the catchment, (iii) vegetation cover, (iv) surface storage viz. existence of lakes and marshes, and(v) initial state of wetness of the soil. Catchments vary so much with regard to these characteristics that it is evidently impossible to do more than generalize on the values of P. Judgment and experience must be used in fixing P. Also see Table 4. 1 for guidance.

Table 4.1 Maximum Value of P in the Formula $Q = 0.028\,PAI_c$

Steep, bare rock and also city pavements	0.90
Rock, steep but wooded	0.80
Plateaus, lightly covered	0.70
Clayey soils, stiff and bare	0.60
-do- lightly covered	0.50
Loam, lightly cultivated or covered	0.40
-do- largely cultivated	0.30
Sandy soil, light growth	0.20
-do- covered, heavy brush	0.10

4.7.8. Relation between intensity and spread of storm: Rainfall recording stations are points in the space and therefore the intensities recorded there are point intensities. Imagine an area around a recording station. The intensity will be highest at the center and will gradually diminish as we go farther away from the center, till at the fringes of the area covered by the storm, intensity will be zero. The larger the area considered the smaller would be the mean intensity. It is, therefore, logical to say that the mean intensity is some inverse function of the size of the area.

If I is the maximum point intensity at the center of the storm then the mean intensity reckoned over an area "a" is some fraction "f " of I. The fraction f depends on the area "a" and the relation is represented by the curve in **Fig. 4.2** which has been constructed from statistical analysis[5].

In hydrological theories it is assumed that the spread of the storm is equal to the area of the catchment. Therefore in **Fig. 4.2** the area "a" is taken to be the same as the area of the catchment.

The effect of this assumption can lead to errors which, on analysis have been found to be limited to about 12 per cent [5]

4.7.9. The-off formula: Introducing the factor f in the Equation 4.11 we get,

$$Q = 0.028\,P\,f\,AI_c \quad \text{... (4.12)}$$

$$Q = 0.028\,PfAI_o \left(\frac{2}{t_c+1}\right) \quad \text{... (4.13)}$$

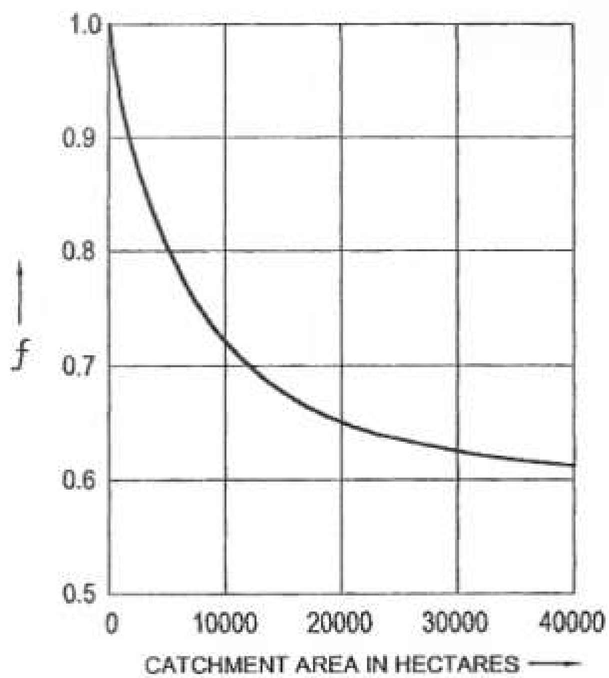


Fig. 4.2 'f' curve

$$Q = \frac{0.028\,AI_o\,2\,f\,P}{t_c+1} \quad \text{... (4.14)}$$

$$= A\,I_o\,\lambda$$

Where $\lambda = \dfrac{0.056\,f\,P}{t_c+1}$ ... (4.14a)

In the equation 4.14(a), I_o measures the role played by the clouds of the region and λ that of the catchment in producing the peak run-off.

It should be clear from the foregoing discussion that the components of λ, are function of A, Land H of the catchment.

4.7.10. Resume of the Steps for Calculating the Run-Off

Step 1: Note down A in hectares, L in km and H in metres from the survey maps of the area.

Step 2: Estimate I_o for the region, preferably from rainfall records failing that from local knowledge.

$$I_o = \frac{F}{2}\left(1 + \frac{1}{T}\right)$$

Where F is rainfall in cm dropped by the serverest storm in T hours.

Step 3 : See **Plate 1** and read values of t_c, P, and f for known values of L, H and A.

Then calculate

$$\lambda = \frac{0.056 f P}{t_c + 1}$$

Step 4: Calculate $Q = A I_o \lambda$

4.7.11. **Example:** Calculate the run-off for designing a bridge across a stream.

Given Catchment: L = 5 km; H = 30 metres; A = 10 sq. km= 1000 hectares. Loamy soillargely cultivated.

Rainfall: The severest storm that is known to have occurred in 20 years resulted in 1 5 cm of rain in 2.5 hours.

Solution:

$$I_o = \frac{F}{T}\left(\frac{T+1}{t+1}\right) = \frac{15}{2.5}\left(\frac{2.5+1}{1+1}\right) = 10.5 \text{ cm per hour}$$

From **Plate 1,** $t_c = 1.7$ hours; $f = 0.97$; $P = 0.30$

$$\lambda = \frac{0.056 f P}{t_c + 1} = \frac{0.056 \times 0.97 \times 0.30}{1.7 + 1} = 0.006$$

$$Q = A I_o \lambda = 1000 \times 10.5 \times 0.006 = 63.6 \text{ m}^3\text{/s}$$

4.7.12. **Run-off curves for small catchment areas (Plate 2) :** Suppose the catchment areas A in hectares and the average slope S of the main drainage channel are known. Assuming that the length of the catchment is 3 times its width, then both L and H [as defined in para (4.7.5.2)], can be expressed in terms of A and S and then t_c calculated from equation (4.9).

Also for small areas, f may be taken equal to one, then vide para 4.7.9.

$$Q = P I_0 A \left(\frac{0.056}{tc+1}\right)$$

For I_0 = 1 cm, the equation becomes,

$$Q = PA \left(\frac{0.056}{tc+1}\right) \text{...(4.15)}$$

Hence Q can be calculated for various values of P, A and S. This has been done and curves plotted in **Plate 2.**

Plate 2 can be used for small culverts with basins upto 1500 hectares or 1 5 sq. km. The value of run-off read from **Plate 2** are of "One Hour rainfall", I_0, of one cm. These values have to be multiplied by the I_0 of the region. An example will illustrate the use of this Plate.

4.7.13. **Example:** The basin of a stream is loamy soil largely cultivated, and the area of the catchment is 10 sq. km. The average slope of the stream is 10 per cent. Calculate the run-off (I_0, the one hour rainfall of the region is 2.5 cm).

Use Plate 2. For largely cultivated loamy soil P = 0.3 vide the Table in set in **Plate 2**.

Enter the diagram at A = 10 sq. km = 1000 hectares; move vertically up to intersect the slopeline of 10 per cent. Then, move horizontally to intersect the OO line join the intersection with P = 0.3 and extend to the run-off (q) scale and read.

$q = 10.2\ m^3/s$

Multiply with I_0.

$Q = 10.2x2.5 = 25.5\ m^3/s$

4.7.14. **In conclusion:** The use of empirical formulae should be done with due diligence. The average designer who cannot rely so much on his intuition and judgment should go by the rational procedure outlined above.

The data required for the rational treatment, viz., A, L and H can be easily read from the survey plans. As regards I_0 it should be realized that this does not have to be calculated for each design problem. This is the storm characteristic of the whole region, with pretty vast area, and should be known to the local engineers.

Complicated formulae, of which there is abundance, have been purposely avoided in this Article. Indeed, for a terse treatment, the factors involved are so many and their interplay so complicated that recourse need be taken to such treatment only when very important structures are involved and accurate data can be collected. For small bridges, the simple formulae given here will give sufficiently accurate results.

ARTICLE 5

ESTIMATING FLOOD DISCHARGE FROM THE CONVEYANCE FACTOR AND SLOPE OF THE STREAM

5.1. In a stream with rigid boundaries (bed and banks) the shape and the size of the cross-sectionis significantly the same during a flood as after its subsidence. If the HFL is plotted and the bed slope is measured, it is simple to calculate the discharge.

5.2. But a stream flowing in alluvium, will have a larger cross sectional area when in flood than that which may be surveyed and plotted after the flood has subsided. During the flood the velocity is high and, therefore, an alluvial stream scours its bed, but when the flood subsides, the velocity diminishes and the bed progressively silts up again. From this it follows that before we start estimating the flood conveying capacity of the stream from the plotted cross-section, we should ascertain the depth of scour and plot on the cross-section the average scoured bed line that is likely to prevail during the high flood.

5.3. The best thing to do is to inspect the scour holes in the vicinity of the site, look at the size and the degree of incoherence of the grains of the bed material, have an idea of the probable velocity of flow during the flood, study the trial bore section and then judge what should be taken as the probable average scoured bed line.

5.4. Calculation of Velocity: Plot the probable scoured bed line. Measure the cross-sectional area A in m^2 and the wetted perimeter P in m. Then calculate the hydraulic mean depth, Rby the formula.

$R = \frac{A}{P}$ (in m) …..(5.1)

Next, measure the bed slope S from the plotted longitudinal section of the stream. Velocity can then be easily calculated from one of the many formulae. To mention one, viz., the Manning's formula:

$V = \frac{I}{n} (R^{2/3}, S^{1/2})$ …..(5.2)

Where

V = Mean velocity of flow(the velocity in m/s considered uniform throughout the cross-section) (to be undone)

R = the hydraulic mean depth

S = the energy slope which may be taken equal to the bed slope, measured over a reasonably long reach, say 500 m or moreover 500m

n = the rugosity co-efficient

For values of n, see Table 5.1. Judgment and experience are necessary in selecting a proper value of n for a given stream.

Table 5.1 Rugosity Co-efficient, n

Surface	**Perfect**	**Good**	**Fair**	**Bad**
Natural Streams				
(1) Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.03	0.033
(2) Same as (1), but some weeds and stones	0.03	0.033	0.035	0.04
(3) Winding, some pools and shoals, clean	0.035	0.04	0.045	0.05
(4) Same as (3), lower stages, more ineffective slope and sections	0.04	0.045	0.05	0.055
(5) Same as (3), some weeds and stones	0.033	0.035	0.04	0.045
(6) Same as (4), stoney sections	0.045	0.05	0.055	0.06
(7) Sluggish river reaches, rather weedy or with very deep pools	0.05	0.06	0.07	0.08
(8) Very weedy reaches	0.075	0.1	0.125	0.15

5.5. Calculation of Discharge

$$Q = A.V. \tag{5.3}$$

$$Q = \frac{A.R.^{2/3} S^{1/2}}{n} \tag{5.4}$$

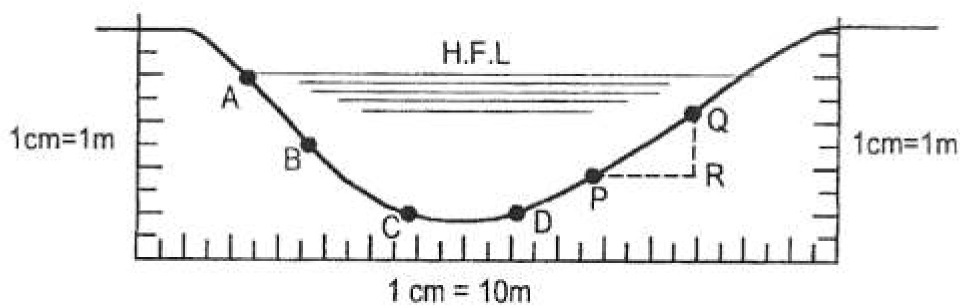
$$Q = \lambda\ S^{1/2} \tag{5.5}$$

Where, $\lambda = \dfrac{AR^{2/3}}{n}$

λ is a function of the size, shape and roughness of the stream and is called its conveyance factor. Thus, the discharge carrying capacity of a stream depends on its conveyance factor and slope.

5.6. When the cross-section is not plotted to the natural scale (the same scale horizontally and vertically), the wetted perimeter cannot be scaled off directly from the section and has to be calculated. Divide up the wetted line into a convenient number of parts, AB, BC and CD, etc. **(Fig. 5.1).** Consider one such part, say PQ, let PR and QR be its horizontal and vertical projections.

Then PQ = $\sqrt{PR^2 + QR^2}$V Now, PR can be measured on the horizontal scale of the given cross-section and QR on the vertical. PQ can then be calculated. Similarly, the length of each part is calculated. Their sum gives the wetted perimeter.



5.7. If the shape of the cross-section is irregular as happens when a stream rises above its banks and shallow overflows are created (Fig. 5.2) it is necessary to subdivide the channel into two or three sub-sections. Then R and n are found for each sub-section and their velocities and discharges computed separately.

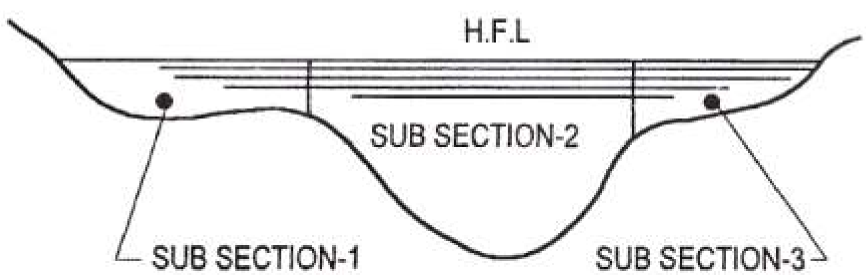


Fig. 5.2

Where further elaboration is justified, corrections for velocity distribution, change of slope, etc. may be applied. Books on Hydraulics give standard methods for this.

5.8. **Velocity Curves:** To save time in computation, curves have been plotted in **Plate 3.** Given R, S and n, velocity can be read from this plate.

5.9. **BETTER MEASURE THAN CALCULATE VELOCITY:** It is preferable to observe the velocity during a high flood. When it is not possible to wait for the occurrence of high flood, the velocity may be observed in a moderate flood and used as a check on the theoretical calculations of velocity. In making velocity observations, the selected reach should be straight, uniform and reasonably long.

5.10. The flood discharge should be calculated at each of the three cross-sections, which as already explained in para 3.3 should be plotted for all except very small structures. If the difference in the three discharges, thus, calculated is more than 10 per cent the discrepancy has to be investigated.

ARTICLE 6

DESIGN DISCHARGE

6.1. Estimated Flood Discharge from Flood Marks on an Existing Structure

6.1.1. Having collected the necessary information from inspection as mentioned in para 2.2, the discharge passed by an existing structure can be calculated by applying an appropriate formula. In Article 15 some formulae for calculating discharges from flood marks on existing bridges are discussed. Worked out examples have been included in Article 17.

6.1.2. Distinct water mark on bridge piers and other structures can be easily found immediately following the flood. Sometimes these marks can be identified years afterwards but it is advisable to survey them as soon after the flood is possible. Turbulence, standing wave and slashing may have caused a spread in the flood marks but the belt of this spread is mostly narrow and a reasonably correct profile of the surface line can be traced on the sides of piers and faces of abutments. This is perhaps the most reliable way of estimating a flood discharge because in the formulae discussed in Article- 15 the co-efficient involved have been accurately found by experiments.

6.2. FIXING DESIGN DISCHARGE

6.2.1. **The recommended rule:** Flood discharges can be estimated in three different ways as explained in Para 4.1 to 6.1.2. The values obtained should be compared. The highest of these values should be adopted as the design discharge Q, provided it does not exceed the next highest discharge by more than 50 per cent. If it does, restrict it to that limit. That is 1.5 times of second highest.

6.2.2. **Sound economy:** The designer is not expected to aim at designing a structure of such copious dimensions that it should pass a flood of any possible magnitude that can occur during the lifetime of the structure. Sound economy requires that the structure should be able to pass easily floods of a specified frequency and that extraordinary and rare floods should pass without causing excessive damage to the structure or the road.

6.2.3. The necessity for this elaborate procedure for fixing Q arises for sizeable structures. As regards small culverts, Q may be taken as the discharge determined from the run-off formulae.

ARTICLE 7

ALLUVIAL STREAMS LACEY'S EQUATIONS

7.1. The section of a stream, having rigid boundaries, is the same during the flood and after its subsidence. But this is not so in the case of streams flowing within, partially or wholly, erodible boundaries. In the latter case, a probable flood section has to be evolved from the theoretical premises for the purpose of designing a bridge; it is seldom possible to measure the cross-section during the high flood.

7.2. **WHOLLY ERODIBLE SECTION. LACEY'S THEORY**: Streams flowing in alluvium are wide and shallow and meander a great deal. The surface width and the normal scoured depth of such streams have to be calculated theoretically from concepts which are not wholly rational. The theory that has gained wide popularity in India is "Lacey's Theory of Flow in Incoherent Alluvium". The salient points of that theory, relevant to the present subject, are outlined here.

7.3. A stream, whose bed and banks are composed of loose granular material, that has been deposited by the stream and can be picked-up and transported again by the current during flood, is said to flow through incoherent alluvium and may be briefly referred to as an alluvial stream. Sucha stream tends to scour or silt up till it has acquired such a cross-section and (more particularly) sucha slope that the resulting velocity is "non-silting and non-scouring". When this happens the stream be comes stable and tends to maintain the acquired shape and size of its cross-section and the acquired slope. It is then said: "to have come to regime" and can be regarded as stable.

7.4. **LACEY'S EQUATION**: When an alluvial stream carrying known discharge, Q has come to regime, it has a regime wetted perimeter P, a regime slope S, and regime hydraulic mean depth R, In consequence, it will have a fixed area of cross-section A and a fixed velocity V. For these regime characteristics of an alluvial channel, Lacey suggests [18] the following relationships. It should be noted that the only independent entities involved are Q and K_{sf}. The K_{sf} is called silt factor and its value depends on the size and looseness of the grains of the alluvium. Its value is given by the formula:

$Ksf = 1.76\sqrt{(dm)}$ To find weighted mean dia of particles following sieves are used 5.6 mm,4 mm,2.8 mm,1 mm, 0.425 mm, 0.180 mm, and 0.075 mm ....(7.1a)

where d_m is the weighted mean diameter of the particles in mm. Table 7.1 gives values of K_{sf} for different bed materials.

(Typical method of determination of weighted mean diameter of particles (d_m) as given in ***Appendix-2*** of IRC:5 is reproduced in **Plate 4**).

(a) Regime Cross-Section

$P = 4.8Q^{½}$ ...(7.1b)

(This may vary from $4.5\,Q^{½}$ to $6.3\,Q^{½}$ according to local conditions)

$$R = \frac{0.473\,Q^{1/3}}{K_{sf}^{\ 1/3}} \qquad \text{... (7.1c)}$$

$$S = \frac{0.0003\,f^{5/3}}{\cancel{K_{sf}^{\ 1/6}}}\ Q^{1/6} \qquad \text{... (7.1d)}$$

18

(a) Regime Velocity and Slope

$$V = 0.44Q^{1/6}\ K_{sf}^{1/3} \qquad \text{... (7.1e)}$$

$$A = \frac{2.3\ Q^{5/6}}{K_{sf}^{1/3}} \qquad \text{... (7.1f)}$$

Table 7.1 Silt Factor K_{sf} in Lacey's Equations[18] = 1.76$\sqrt{dm}$

Type of bed material	d_m	K_{sf}
Coarse silt	0.04	0.35
Silt/fine sand	0.081 to 0.158	0.5 to 0.6
Medium sand	0.233 to 0.505	0.8 to 1.25
Coarse sand	0.725	1.5
Fine bajri and sand	0.988	1.75
Heavy sand	1.29 to 2.00	2.0 to 2.42

7.5. THE REGIME WIDTH AND DEPTH: Provided a stream is truly regime , it is destined to come to regime according to Lacey. It will then be stable and have a section and slope conforming to his equations. For wide alluvial streams the stable width W can be taken equal to the wetted perimeter P of Equation (7.1a).

That is

$W = P = 4.8\ Q^{1/2}$ (applicable for wide stream only) ... (7.2a)

Also, the normal depth of scour D on a straight and unobstructed part of a wide stream may be taken as equal to the hydraulic mean radius R in Equation (7. 1 c). Hence,

$$D = \frac{0.473\ Q^{1/3}}{K_{sf}^{1/3}} \qquad \text{...(7.2b)}$$

ARTICLE 8

LINEAR WATERWAY

8.1. **THE GENERAL RULE FOR ALLUVIAL STREAMS:** The linear waterway of a bridge/culverts across a wholly alluvial stream should normally be kept equal to the width required for stability, viz., that given by Equation (7.2a).

8.2. **UNSTABLE MEANDERING STREAMS:** A large alluvial stream, meandering over a wide belt, may have several active channels separated by land or shallow sections of nearly stagnant water. The actual (aggregate) width of such streams may be much in excess of the regime width required for stability. In bridging such a stream it is necessary to provide training works that will contract the stream. The cost of the latter, both initial and recurring, has to be taken into account in fixing the linear waterway.

8.3. In the ultimate analysis it may be found in some such cases, that it is cheaper to adopta linear waterway for the bridge/culverts somewhat in excess of the regime width given by Equation (7.2a). But as far as possible, this should be avoided. When the adopted linear waterway exceeds the regime width it does not follow that the depth will become less than the regime depth D given by Equation (7.2b). Hence, such an increase in the length of the bridge/culverts does not lead to any counter vailing saving in the depth of foundations. On the contrary, an excessive linear waterway can be detrimental in so far as it increases the action against the training works.

8.4. **CONTRACTION TO BE AVOIDED:** The linear waterway of the bridge/culverts across an alluvial stream should not be less than the regime width of the stream. Any design that envisages contraction of the stream beyond the regime width, necessarily has to provide for much deeper foundation. Much of the saving in cost expected from decreasing the length of the bridge/culverts may be eaten up by the concomitant increase in the depth of the substructure and the size of training works. Hence, except where the section of the stream is rigid, it is generally troublesome and also futile from economy consideration to attempt contracting the waterway.

8.5. **STREAMS NOT WHOLLY ALLUVIAL:** When the banks of a stream are high, well defined, and rigid (rocky or some other natural hard soil that cannot be affected by the prevailing current) but the bed is alluvial, the linear waterway of the bridge/culverts should be made equal to the actual surface width of the stream, measured from edge to edge of water along the HFL on the plotted cross-section. Such streams are later referred to as quasi-alluvial.

8.6. **STREAMS WITH RIGID BOUNDARIES:** In wholly rigid streams the rule of para 8.5 applies, but some reduction in the linear waterway may, across some streams with moderate velocities, be possible and may be resorted to, if in the final analysis it leads to tangible savings in the cost of the bridge.

8.7. As regards streams that overflow their banks and create very wide surface widths with shallow side sections, judgment has to be used in fixing the linear waterway of the bridge. The bridge/culverts should span the active channel and detrimental afflux avoided. See also Article 18.

20

ARTICLE 9

NORMAL SCOUR DEPTH OF STREAMS

9.1. ALLUVIAL STREAMS

9.1.1. What is the significance of the Normal Scour Depth, If a constant discharge were passed through a straight stable reach of an alluvial stream for an indefinite time, the boundary of its cross-section should ultimately become cosine curveelliptical.

This will happen when regime conditions come to exist. The depth in the middle of the stream would then be the normal scour regime depth.

In nature, however, the flood discharge in a stream does not have indefinite duration. For this reason the magnitude and duration of the flood discharge carried by it would govern the shape of the flood section of any natural stream. Some observers have found that curves representing the natural stream sections during sustained floods have sharper curvature in the middle than that of an ellipse. In consequence, it is believed that Lacey's normal depth is an under estimate when applied to natural streams subject to sustained floods. However, pending further research, Lacey's equations may be applied.

9.1.2. As discussed later in Article 11, the depth of foundations is fixed in relation to the maximum depth of scour, which in turn is inferred from the normal depth of scour. The normal depth of scour for alluvial streams is given by Equation (**7.2b**), so long as the bridge bridge / culverts does not contract the stream beyond the regime width W given by Equation (**7.2a**).

9.1.3. If the linear waterway of the bridge for some special reason, is kept less than the regime width of the stream, then the normal scour depth under the bridge will be greater than the regime depth of the stream (**Fig. 9.1**).

Where

W = the regime width of the stream

L = the designed waterway; when the bridge is assumed to cause contraction L is less than W

D = The normal scour depth when L = W

D' = The normal mean depth of scour depth under the bridge with L less than W

According to Clause 106.9.3.1 of IRC 5

$$d_{sm} = 1.34\left[\frac{{D_b}^2}{k_{sf}}\right]^{1/3}$$

Where

D_b = discharge in m^3/s perm width

k_{sf} = silt factor for material obtained upto deepest anticipated scour.

= $1.76\sqrt{dm}$, dm being the weighted mean diameter of particles in mm.

d_{sm} = mean depth of scour depth in m.

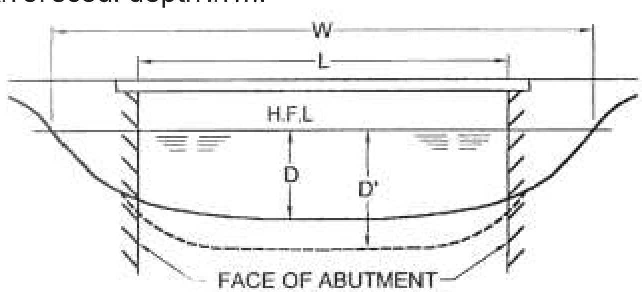


Fig. 9.1

The value of D_b shall be total design discharge divided by the effective linear waterway between abutments or guide bunds.

This formulae take into account the effect of contraction and, therefore, no further modification arc needed. When the bed is protected by apron and curtain wall, the scour considerations will be different as discussed in Article-20.

9.2. QUASI-ALLUVIAL STREAMS

9.2.1. **Some streams are not wholly alluvial** : A stream may flow between banks which are rigid in so far as they successfully resist erosion, but its bed may be composed of loose granular material which the current can pick-up and transport. Such a stream may be called quasi-alluvial to distinguish it, on the one hand, from a stream with wholly rigid boundaries and, on the other, from aw holly alluvial stream. Since such a stream is not free to erode its banks and flatten out the boundaries of its cross-section as a wholly alluvial stream does, it does not acquire the regime cross-sectionwhich Lacey's equations prescribe.

9.2.2. It is not essential that the banks should be of rock to be in erodible. Natural mixtures of sand and clay may, under the influence of elements, produce material hard enough to defy erosion by the prevailing velocity in the stream.

Across a stream section, the natural width of which is nowhere near that prescribed by Lacey's theory, it is expected to find that the banks, even though not rocky are not friable enough to be treated as incoherent alluvium for the application of Lacey's Theory. Such cases have, therefore, got to be discriminated from the wholly alluvial streams and treated on a different footing.

9.2.3. In any such case the width *W*of the section, being fixed between the rigid banks, canbe measured. But the normal scour depth *D* corresponding to the design discharge Q has to be estimated theoretically as it cannot be measured during the occurrence of high flood.

9.2.4. **When the stream width is large compared to depth :** In Article-5, for calculating the discharge of the stream from its plotted cross-section, the probable scoured bed line (para 5.3)was drawn.

When the stream scours down to that line it should be capable of passing the discharge calculated there, say q m^3/s. But the discharge adopted for design, Q, may be anything upto 50 percent more than q (see para 6.2.1). Therefore, the scour bed line will have to be lowered further. Suppose the normal scour depth for Q is D and that for q is d, then,

$$\boxed{D=d\left(\frac{Q}{q}\right)^{3/5}} \quad \text{...(9.2)}$$

Derived from Manning's Formula

Since d is known, D can be calculated. This relationship depends on the assumption that the width of the stream is large as compared, with its depth, and therefore, the wetted perimeter is approximately equal to the width and is not materially affected by variations in depth. It also assumes that the slope remains unaltered.

$$Q = \text{area x velocity}$$

$$= RPCR^{2/3}\,S^{1/2}$$

$$= K\;R^{5/3} \quad \text{...(9.3)}$$

where K is a constant.

Hence, R varies as $Q^{3/5}$. Since in such streams R is very nearly equal to the depth, therefore, D varies as $Q^{3/5}$. Hence, the equation (9.2).

From the above relationship it follows that if Q is 1 50 per cent of q, D will be equal to 127 percent of d.

9.2.5. Alternatively, the normal depth of scour of wide streams may be calculated as under.If the width of the stream is large as compared with its depth, then W may be taken as P and D as R.

$$Q = \text{area x velocity}$$

$$= (PR)\,V = (WD)\,V, \text{ where V is the mean velocity} \qquad ...(9.4)$$

$$D = \frac{Q}{WV}$$

When Actual Velocity is Observed during high flood - this formula may be less frequently used

Now W is the known fixed width of the stream. If the velocity V has actually been observed (para 5.9), then D can be calculated from the above equation. For mean velocity, refer relevant clause in IRC:6.

9.2.6. Suppose the velocity has not been actually measured during a flood, but the slope S is known.

$$Q = \text{area x velocity}$$

$$= \frac{(RP)\,R^{2/3}\;S^{1/2}}{n}$$

$$= \frac{WS^{1/2}\;D^{5/3}}{n} \qquad ...(9.5)$$

Knowing Q, W and S, D can be calculated from this equation.

For quickness, velocity curves in **Plate 3** can be used. Assume a value of R and fix a suitable value of the rugosity co-efficient n appropriate for the stream. Corresponding to these values and the known slope, read the velocity from **Plate 3**. Now calculate the discharge (= VRW). If this equals the design discharge Q, then the assumed value of R is correct. Otherwise, assume another value of R and repeat. When the correct value of R has been found, take D equal to R. (See the worked out Example in **Article-16**).

9.2.7. The procedure described above can be applied if either the slope of the stream or the actual observed velocity is known. If either of these are not known, the following procedure for approximate calculation of the normal scour depth can be applied.

Suppose the wetted perimeter of the stream is P and its hydraulic mean depth R. If Q is its discharge, then,

$$Q = \text{area x velocity}$$

$$= (PR)\,[CR^{2/3}\,S^{1/2}] \qquad ...(9.6a)$$

Now, if this stream, carrying the discharge Q, had been wholly alluvial, with a wetted perimeter P_1, and hydraulic mean depth R_1 for regime conditions, then,

$$Q = (P_1R_1)\,[CR^{2/3}\,S^{1/2}] \qquad ...(9.6ab)$$

Also, for a wholly alluvial stream Lacey's Theory would give:

$$P_1 = 4.8\, Q^{1/2} \qquad \text{... (9.6c)}$$

$$R_1 = \frac{0.473\, Q^{1/3}}{K_{sf}^{1/3}} \qquad \text{... (9.6d)}$$

Equating values of Q in (9.6a) and (9.6b), and rearranging we get

$$\frac{R}{R_1} = \left(\frac{P_1}{P}\right)^{3/5} \qquad \text{... (9.6e)}$$

Now substituting values of P_1 and R_1 from equations (9.6c) and (9.6d) in (9.6e), we get

$$R = \frac{1.21\, Q^{0.63}}{K_{sf}^{0.33}\ P^{0.60}} \qquad \text{... (9.6f)}$$

If the width W of the stream is large compared with its depth D, then writing W for P and D for R in equation (9.6f).

$$D = \frac{1.21\, Q^{0.63}}{K_{sf}^{0.33}\ W^{0.60}} \qquad \text{... (9.7)}$$

Thus, if the design discharge Q, the natural width W, and the silt factor K_{sf} are known, the normal scour depth D can be calculated from Equation (9.7).

The above reasoning assumes that the slope at the section in the actual case under consideration is the same as the slope of the hypothetical (Lacey's) regime section, carrying the same discharge. This is not improbable where the stream is old and its bed material is really incoherent alluvium. But if there is any doubt about this, the actual slope must be measured and the procedure given in para 9.2.6 applied.

9.2.8. **When the stream is not very wide:** If the width of the stream is not very large as compared with its depth, then the methods given above will not give accurate enough results. In such a case draw the probable scoured bed line on the plotted cross-section, measure the area and the wetted perimeter and calculate R.

Corresponding to this value of R and the known values of S and n, read velocity from **Plate 3**. If the product of this velocity and the area equals the design discharge, the assumed scoured bed line is correct. Otherwise, assume another line and repeat the process. Then measure D.

9.2.9. **Effect of contraction on normal scour depth:** If, for some special reason, the linear waterway L of a bridge across a quasi-alluvial stream is kept less than the natural unobstructed width W of the stream (**Fig. 9.1**), then the normal scour depth under the bridge D^1 will be greater than the depth D ascertained above for the unobstructed stream. Covered by the relationship:

$$D' = 1.34 \left(\frac{D_b^2}{K_{sf}}\right)^{1/3} \qquad \text{... (9.8)}$$

Because D_b, of L case will be more than D_b, of W case.

9.3. **Scour in Clay and Bouldary Strata**: There are no rational methods for assessment of scour in clay or bouldary strata. Guidelines for calculating silt factor for bed materials consisting of clay and boulders as given in IRC-5 may be adopted and are reproduced in paras 9.3.1 and 9.3.2.

9.3.1. Scour in clay: Scour in clay is generally less than scour in sand. Normally in field ~~weget~~ we get a mixture of sand and clay at many places. For the purpose of assessment following definition ofsand and clay can be given.

Sand - Where ϕ is equal to or more than 15° even if c (Cohesion of soil) is more than 0.2 kg/cm^2

(Silt factor K_{sf} will be calculated as per provisions of para 7.4 or Table 7.1).

Clay- Where ϕ is less than 15°& c (Cohesion of soil) is more than 0.2 kg/cm^2

Scour in sand of above definition can be calculated by the formulae given earlier. In clay instead of silt factor (K_{sf}) clay factor (K_{sfc}) is adopted -

$$K_{sfc} = F\,(1+\sqrt{(c)}\,)$$

Where

c= Cohesion in kg/cm^2 and

$F = 1.5$ for $\phi \geqslant 10° < 15°$ ...(9.9)

F is 1.50, if 10d <= phi <15d,
1.75, if 5d <= phi < 10d
2.00, if phi <= 5d

$= 1.75$ for $\phi \geqslant 5° < 10°$

$= 2.0$ for $\phi < 5°$

Scour depth (dsm) = $1.34\,(D_b^{\,2} / K_{sfc})^{1/3}$

D_b= discharge per unit width

9.3.2. **Bouldary strata:** There is no rational method to assess scour in bouldary strata of boulders or pebbles. In the absence of any formula K_{sf} may be determined as per IRC 5and adopted. If, say, mean size of particles is d_m

Then, $K_{sf} = 1.76\,(d_m)^{1/2}$

Scour depth = $1.34 \times \left[\frac{D_b^{\,2}}{K_{sf}}\right]^{1/3}$ ...(9.10)

It is, however, better to investigate depth of foundations adopted in past for similar foundation and decide depth on the basis of precedence. Protection work around foundations in the form of curtain wall and apron or garland blocks should be provided, when the foundation is laid on bouldary strata.

25

ARTICLE 10

MAXIMUM SCOUR DEPTH

10.1. In considering bed scour, we are concerned with alluvial and quasi-alluvial streams only and not with streams which have rigid beds.

10.2. In natural streams, the scouring action of the current is not uniform all along the bed width. It is not so even in straight reaches. Particularly at the bends as also round obstructions to the flow, e.g., the piers of the bridge, there is deeper scour than normal. In the following paragraphs, rules for calculating the maximum scour depth are given. It will be seen that the maximum scour depth is taken as a multiple of the normal scour depth according to the circumstances of the case.

10.3. In order to estimate the maximum scour depth, it is necessary first to calculate the normal scour depth. The latter has already been discussed in detail. To summarise what has been said earlier, the normal scour depth will be calculated as under:

(i) **Alluvial Streams.** Provided the linear waterway of the bridge is not less than the regime width of the stream, the normal scour depth D is the regime depth as calculated from Equation (7.2b).

(ii) **Streams with Rigid Banks but Erodible Bed**. Provided the linear waterway of the bridge is not less than the natural unobstructed surface width of the stream, the normal scour depth d is calculated as explained in Article 9.

10.4. **RULES FOR FINDING MAXIMUM SCOUR DEPTH:** The rules for calculating the maximum scour depth from the normal scour depth are:

Rule (1) : For average conditions on a straight reach of the stream and when the bridge is a single span structure, i.e. it has no piers obstructing the flow, the maximum scour depth should be taken as 1.27 times the normal scour depth, modified forthe effect of contraction where necessary.

Rule (2) : For bad sites on curves or where diagonal current exist or the bridge is multi-span structure, the maximum scour depth should be taken as 2 times the normal scour depth, modified for the effect of contraction when necessary.

10.5. The finally adopted value of maximum scour depth must not be less than the depth (below HFL) of the deepest scour hole that may be found by inspection to exist at or near the site of the bridge.

The following example will illustrate the application of the rules in para 10.4 above.

10.6. Example, A bridge is proposed across an alluvial stream (K_{sf}= 1.2) carrying adischarge of 50 m^3/s. Calculate the depth of maximum scour when the bridge consists of (a)3 spans of 6 m and (b) 3 spans of 8 m

Regime surface width of the stream

$W = 4.8Q^{1/2} = 4.8 \times 50^{1/2} = 33.94m$

Regime depth

$$D = 0.473 \frac{Q^{1/3}}{K_{sf}^{1/3}} = \frac{0.473x50^{1/3}}{(1.2)^{1/3}} = 1.64 \text{ m}$$

Maximum scour depth

(a) when span (3x6 m), $D_b$the discharge per metre width is

50/18, i.e., 2.778 cumecs

d_{sm}= 1 .34 $(2.778^2/I .2)^{1/3}$ = 2.49 m

(i) Maximum depth of scour for pier

= 2_{dsm}= 2x2.49 = 4.98 m

(ii) Maximum depth of scour for abutment

= 1.27 d $_{sm}$ = 1.27x2.49 = 3. 16m

(b) When span is 3 x 8 m, $D_b$the discharge per metre width is 50/24, i.e., 2.083 cumecs

d $_{sm}$= 1.34 $(2.083^2/I .2)^{I/3}$ = 2.055 m

(i) Maximum depth of scour for pier

= $2d_{sm}$= 2x2.055 = 4.11 m

(ii) Maximum depth of scour for abutment

= $1.27d_{sm}$ = 1.27x2.055 = 2.61 m

10.7. For small bridges across alluvial channel having multiple spans, the foundation levels for abutments should be kept capped the same as that of pier for following reasons:

(i) In case of small spans, the scour hole around pier could extend up to abutment.

(ii) Abutment foundation at higher level may create a surcharge effect over thefoundation of adjacent pier.

(iii) In case of outflanking of the bridge the abutment in any case has to be designed for scour all around condition.

ARTICLE 11

DEPTH DEPTH OF OPEN FOUNDATIONS FOR BRIDGES OF FOUNDATIONS

11.1. The following rules should be kept in view while fixing the depth of bridge foundations:

Rule (1) In Soil. The embedment of foundations in soil shall be based on assessment of anticipated scour. Foundations may be taken down to a comparatively shallow depth below the bed surface provided good bearing stratum is available and foundation is protected against scour. The minimum depth of open foundations shall be upto stratum having adequate bearing capacity but not less than 2.0m below the scour level or protected scour level.

Rule (2) In Rocks. When a substantial stratum of solid rock or other material not erodible at the calculated maximum velocity is encountered at a level higher than or a little below that given by Rule (1) above, the foundations shall be securely anchored into that material. This means about 0.6 m into hard rocks with an ultimate crushing strength of 10 MPa or above and 1.5 m in all other cases.

Rule (3) All Beds. The pressure on the foundation material must be well within the safe bearing capacity of the material.

These rules enable one to fix the level of the foundations of abutments and piers.

11.2. The above rules are applicable for open foundations only. For deep foundations like well, and pile foundations, wherever adopted depending upon site requirements depth of foundations shall be worked out as per IRC: 78.

28

ARTICLE 12

SPAN AND VERTICAL CLEARANCE

12.1. As a rule, the number of spans should be as small as possible, since piers obstruct flow. Particularly, in mountainous regions, where torrential velocities prevail, it is better to span from bank to bank using no piers if possible.

12.2. **LENGTH OF SPAN:** In small structures, where open foundations can be laid and solid abutments and piers raised on them; it has been analyzed that the following approximate relationships give economical designs.

For Masonry arch bridges S = 2 H

For RCC Slab Bridges S = 1.5 H

Where

S = Clear span length in metre.

H = Total height of abutment or pier from the bottom of its foundation to its top in metre. For arched bridges it is measured from foundation to the intrados of the key stone.

12.3. **VERTICAL CLEARANCE:** After fixing the depth of foundations Df, the vertical clearance is added to it to get H. The minimum vertical clearance shall be provided as per **Table 12.1**.

Table 12.1

Discharge in m^3/s	Minimum vertical clearance in mm
Upto 0.30	150
upto 3.0	450
Above 3 and upto 30	600
Above 30 and upto 300	900

For openings of culverts having arched decking, the clearance below the crown of the intrados of arch shall not be less than 1/10 of the maximum depth of water plus 1/3 of the rise of arch intrados.

Further to keep the free board of approaches not less than 1750 mm, the vertical clearance in slab/box cell bridges may be increased suitably.

In designing culverts for roads across flat regions where streams are wide and shallow (mostly undefined dips in the ground surface), and in consequence the natural velocities of flow are very low, the provision of clearance serves no purpose. Indeed it is proper to design such culverts on the assumption that the water at the inlet end will pond up and submerge the inlet to a pre determined extent. This will be discussed in **Article 19**.

In case of structure over artificial channels or canals, etc. the minimum vertical clearance should be taken 600 mm above the Full Supply Level.

12.4. **THE NUMBER OF SPANS:**

12.4.1. If the required linear waterway L is less than the economical span length it has to be provided in one single span.

12.4.2. When L is more than the economical span length (S) the number of spans (N) requiredis tentatively found from the following relation:

$$L = NS$$

12.4.3. Since N must be a whole number (preferably odd) S has to be modified suitably. In doing so it is permissible to adopt varying span lengths in one structure to keep as close as possible to the requirements of economy and to cause the least obstructions to the flow.

12.5. To facilitate inspection and carrying out repairs, the minimum vent height of culverts should normally be 1500 mm. The vent size of irrigation culverts may be decided considering theactual requirements and site condition. For pipe culverts minimum diameter should be minimum dia of pipe as per different specifications of IRC should be followed. for pipe culverts, the minimum diameter of pipes should be 1200mm 1.20 m..

12.6. If a large number of small bridges and culverts are required to be provided in a project, it is important that the span lengths or box sections should be standardized, so that the repetitive use of false work or use of precast method of construction. may be appropriately adopted such design would make substantial reduction in overall construction period of the project.

30

ARTICLE 13

GEOMETRIC STANDARDS, SPECIFICATIONS AND QUALITY CONTROL

13.1. Details of small bridges and culverts of probable spans and heights conforming to latest IRC codes and guidelines are incorporated with a view to cut short the time in preparation of estimates and design of culverts and attain uniform standards and quality control in the work.

13.2. GEOMETRIC STANDARDS

13.2.1. IRC standards: Standards contained in IRC:73 and IRC:86 are adopted for Geometric Standards. The overall widths adopted for culverts and small bridges for 2-lane carriage way are as follows.

NH and SH	-	12 m Minimum
MDR	-	8.4 m

13.2.2. Design loads for 2-lane roadway: Design loading for culverts and small bridges should be as below:

Village Road and ODR (Rural Roads)	-	2-lanes IRC Class A or 70R whichever gives worst effect
NH, SH and MDR	-	Depending upon the carriageway width refer IRC-6

13.2.3. Width of roadway: The width of a culvert and small bridge (along the direction ~~of low~~ of flow) should be such that the distance between the outer faces of the parapets will equal the full designed width of the formation of the road. Any proposed widening of the road formation in the near future should also be taken into account in fixing the width of the structure. In case of high banks, the length of culvert should be judiciously decided to avoid high face walls.

13.2.4. In small bridges, the width (parallel to the flow of the stream) should be sufficient to give a minimum clear carriageway of 4.25 m for a single-lane bridge and 7.5 m for a two-lane bridge between the inner faces of the kerbs or wheel guards. Extra provision should be made for foot paths, etc., if any are required. Width of bridges and culverts shall be at least equal to width of road way in the section.

13.2.5. Siting of structures and gradients: Culverts and small bridges must be sited on the straight alignment of roads. If the Nalla is crossing the road at angles other than right angle, either skew culverts and small bridges should be provided or, if economical, the Nalla should be suitably trained. The same gradient of road may be provided on the bridges and culverts for these. If the bridges these are situated at change of gradient (hump), the profile of vertical curve should be given in wearing coat. Alternatively, the profile could be given in the deck itself. It shall be ensured that T the bearing surface of deck slab on the abutment/piercap shall be horizontal. cap should be horizontal.

13.3. DESIGN:

13.3.1. Road top level: For maintaining the geometric standards of the road, culverts and small bridges should be constructed simultaneously or prior to with the earthwork for road as otherwise there would be the following two disadvantages.

(1) Practically, every culvert and small bridge becomes a hump on the road and geometric of the road is affected.

31

(2) Duplicate work of consolidation of approaches giving rise to extra cost.
(3) Consolidation for approaches may not be done properly

13.3.2. Minimum span and clearance : From the consideration of maintenance of culverts, it is desirable that the span of slab culvert is kept minimum 2 m and height 1.5 m and diameter of pipes 1.0 2 m. Culverts of small span or diameter are found to get choked due to silting and also cause difficulty in cleaning.

13.3.3. Pipe culverts: Pipe culverts shall conform to IS category NP3/NP4. The cushion between the top of the pipe and the road level shall not be less than 600 mm. First class bedding consisting of compacted granular material can be used for height of fill upto 4 m and concrete cradle bedding upto a maximum height of fill upto 8 m.

Where cushion over the pipe is less than 600 mm throughout and/or partly over the culvert, encasing of full-length pipes shall be done by minimum 200mm thick M20 concrete.

For small size culverts, RCC pipe culverts with single pipe or up to 6 pipes placed side by side (with minimum clear distance of 600mm) depending upon For small size structures, RCC pipe culverts with single row or upto six rows of R.C.C. pipes, dependingupon the discharge may be used as far as possible, as they are likely to prove comparatively cheaper than slab culverts.

RCC pipes in two rows one above the other have also been used for small bridges on cost considerations, especially for providing waterway in breached section of roads.

13.3.4. RCC slab: RCC slab culverts and small bridges should be adopted where the founding strata is rocky or of better bearing capacity. In case where adequate cushion is not available for locating pipe culvert RCC slab culvert should be adopted. RCC slab culverts/bridges are also useful for cattle crossing during dry weather.

13.3.5. RCC box cell structures: In a situation where bearing capacity of soil is low, RCC Box type culvert should be preferred.

13.3.6. Balancing culverts: Balancing culvert are to be located at points on L section of the road where down gradients meet. These balancing culverts balance the discharge from either side of the road. Observation of the road alignment during rains also gives a good idea about location of balancing culverts.

13.4. NUMBERING OF CULVERTS AND SMALL BRIDGES:

For details reference may be made to "Recommended Practice for Numbering Bridges and Culverts", **IRC:7.**

13.5. GENERAL DESIGN ASPECTS AND SPECIFICATIONS: The type design of pipe culverts and RCC slab culverts and slab bridges given here are based on following general aspects. Course drubble stone masonry for substructure and parapet walls is generally found to be economical in comparison to mass concrete substructure. The masonry below or above the ground level should be as per IRC:40. If bricks having minimum crushing strength of 7 Mpa are available, these can also be used for culverts.

13.5.1. Parapet wall and railing : For culverts, where parapet walls are provided they shall be of plain concrete M1l 5 grade or brick or store masonry with 450 mm top width. In case of pipe culverts no parapet walls are needed and guard stones would be adequate except for culverts on hill roads. Guard stones provided shall be of size 200x200x600 mm. Railings as given in Standard Drawings of MORT&H may also be provided for culverts and small bridges. Railings or parapets shall have a minimum height above the adjacent roadway or footway safety kerb surface of 1.1 m less one half the horizontal width of the top rail or top of the parapet. Crash barriers may be provided when they are found functionally required. Crash barriers when provided shall conform to provisions in IRC:5 and while adopting MORT&H standard drawings, the design of deck slab shall be checked for provision of crash barriers.

13.5.2. **Wearing coat:** Normally, the wearing surfaces of the road shall be carried over the culverts/small bridges. For low category road which do not have bituminous surfaces, concrete wearing coat of average 75 mm or bituminous wearing coat is provided. On the small bridges wearing coat is provided as per IRC:5need be adopted and approach profile may be suitably graded.

13.5.3. **Approach slab:** Approach slab can be dispensed with in case of culverts. For Small bridges approach slab as per IRC:5shall be provided.

13.5.4. **Deck slab:** Grade of concrete shall be as per IRC 112. M25 concrete for moderate and M30 concrete for severe conditions of exposure and high strength deformed bars conforming to IS: 1786 are specified for the deck slabs.

13.5.5. **Expansion joint:** For spans upto 10m small bridges, pre moulded bituminous sheet (like, shalitex board) of 20 mm thickness are required to be provided.

13.5.6. **Pier/abutment cap/coping :coping:** The minimum thickness of reinforced cap over solid PCC/RCC substructure shall be 200 mm and that in case of masonry substructure shall not be less than 500 mm. The minimum grade of concrete shall be M 25 and M 30 for moderate and severe conditions of exposure respectively. However, the coping over the returns may be of M 15 grade and thickness not less than 100 mm.

13.5.7. **Section of pier abutment and returns:** The abutment and pier sections should be so designed as to withstand safely the worst combination of loads and forces as specified in the **IRC:6**.

13.5.8. **Top width of pier/abutment:** In respect of masonry and concrete piers/abutments minimum width at top of pier and abutments for slab bridges just below the caps shall be as per **Table 13.1**. Tar paper bearings shall be provided between abutment/peir cap and RCC slab for spans upto 10 m.

Table 13.1

Span (in m)	Minimum width at top of abutment/pier (mm)
2.0	500
3.0	500
4.0	1000
5.0	1000
6.0	1200
8.0	1200
10.0	1200

If the velocity flow is more than 4.5 m/s and river carries abrasive particles, it is advisable to design section of foundation and pier considering their effect. A sacrificial layer of brick/stone masonry of suitable thickness and height shall be provided irrespective of total height of substructure.

In the case of arch bridges, the top width of abutments and piers should be adequate to accommodate skew decks and to resist the stresses imposed under the most unfavorable conditions of loading.

13.5.9. Return walls or wing walls: Wing walls are generally at 45° angle to the abutment and are also called as splayed wing walls. Walls parallel to road are called as return walls.

Where embankment height exceeds 2 m, splayed return walls may be preferred. The length of straight return should normally be 1.5 times the height of the embankment. Where the foundations of the wing walls can be stepped up, having regard to the soil profile, this should be done for the sake of economy. Quite often short return walls meet the requirements of the site and should be adopted.

The top width of wing walls and returns shall not be less than 450 mm.

i. Layout of wing walls for skew bridges/culverts shall be prepared keeping in view the height and normal distance from the road to the wing wall

ii. Slope of top of wing wall to be such that height of wall matches with the slope of embankment

13.5.10. Weepholes and water spouts: Adequate number of weep holes at spacing not exceeding 1 m in horizontal and vertical direction should be provided to prevent any accumulation of water and building up of the hydrostatic pressure behind the abutment and wing walls. The weep holes should be provided at about 1 50 mm above low water level or ground level whichever is higher. Weep holes shall be provided with 100 mm dia AC pipes for structures in plain/reinforced concrete, brick masonry and stone in masonry. For brick and stone masonry structures, rectangular weep holes of 80 mm wide and 150 mm height may also be provided. Weep holes shall extend through the full width of the concrete/masonry with slope of about 1 vertical to 20 horizontals towards the drainage face.

In case of stone masonry, the spacing of weep holes shall be adjusted to suit the height of the course in which they are formed. The sides and bottom of the weep holes in the interior shall be made up with stones having fairly plain surface.

For spans more than 5 m one water spout of ~~1001 00 mm~~ 100 x 100 mm dia. in the center of the slab located on either side of the deck shall be provided. The spacings of drainage spouts shall not exceed 10 m.

In case of one side camber, the number of drainage spouts shall be doubled and location suitably adjusted.

13.5.11. Foundation concrete: Foundation concrete shall not be less than M 15 grade. If the foundation level is below water table, 10 per cent extra cement is to be added in concrete. The minimum thickness depth of footing shall be 300 mm. For foundation resting on rock a leveling course of at least 150 mm thickness in M 15 grade of concrete shall be used.

13.5.12. Arches: The type of superstructure depends on the availability of the construction materials and its cost. An evaluation of the relative economics of RCC slabs and masonry arches should be made and the latter adopted where found more economical.

The masonry arches may be either of cement concrete blocks of M 15 or dressed stones or bricks in 1:3 cement mortar. The crushing strength of concrete, stone or brick units shall not be less than 105 kg/cm^2. Where stone masonry is adopted for the arch ring, it shall be either coursed rubble masonry or ashlar masonry.

34

13.5.13. Raft foundation: Raft foundations are found to be quite suitable for small bridges and culverts where the founding strata is soft and has SBC upto 10 t/m^2. The following aspects are to be kept in consideration.

(1) Raft foundations are suitable for all types of structures other than pipe culverts.

(2) Protection needs to be provided in the form of apron.

(3) Cut-off should be done first, i.e., before the raft. Immediately, after the raft is complete, aprons on U/s and D/s should be completed.

(4) Details of raft foundation are given in **Article 21.**

13.6. QUALITY CONTROL

13.6.1. Although, the work of culverts and small bridges is simple it is necessary to have quality control in the work of stone/brick masonry and concrete in deck slab, bar bending, etc. Reference may be made to "Guidelines on Quality Systems for Road Bridges", IRC:SP:47.

13.6.2. Specifications should be in accordance with "Specification for Road and Bridge Works" of Ministry of Road Transport and Highways published by Indian Roads Congress.

13.7. SETTING OUT OF CULVERTS AND SMALL BRIDGES: Setting out of culverts and small bridges should be done from 4 masonry/ concrete pillars, two in the direction of road and two along the stream, all placed along two center lines. The top of pillars in the direction of road should be at the proposed top level of deck slab. Two lines, one along the direction of stream and the other along the center line of road should be inscribed on one of the pillars and all distances should be measured with respect to these lines. The pillars should be placed sufficiently away from the zone of excavation.

13.8. MASONRY WORK

13.8.1. All masonry work shall conform to IRC:40. The mortar mix in case of cement sand shall be 1:3, 1:4 or 1:5, whereas, in case of cement lime sand it shall be 1.0:0.5:4.5.

13.8.2. Brick proposed to be used shall be of minimum compressive strength of 7 MPa. However, for rivers with velocity of 4.5 m/s and carrying highly abrasive particles, this shall be increased to 10 MPa.

13.8.3. Brick and stone masonry shall conform to IRC:40.

13.9. CONCRETE

13.9.1. According to IRC:112, the minimum grade of plain concrete is M 15 of concrete and that of RCC is M 20. The size of reinforcing steel metal to be used for RCC slabs and the grading of aggregates are specified in relevant codes and specifications. It is advisable to use power driven concrete mixer. Similarly, vibrators should also be made available. Furthermore, precast concrete cover blocks must be provided to ensure bottom cover to reinforcement. Water cement ratio must be limited to 0.45 maximum. Incase of use of Plasticiser w/c ratio can be restricted to 0.4. Size of coarse aggregate will be 20mm for RCC and upto 40 mm for plain concrete. Wherever feasible, precast construction may be adopted, which ensures quality and speed of construction.

13.10. Bar bending: Lengths of bars and numbers are given in standard drawings. Cutting of bars from available stock must be done carefully. Generally, tendency of cutting bars of required lengths and discarding pieces of shorter lengths give rise to greater wastages. Normally staggered overlaps to the extent of 25 per cent may be provided. Calculated quantities of steel are increased suitably to account for overlaps, its length conforming to IRC:112 Steel chairs should be provided for maintaining correct position of top bars.

ARTICLE 14

STRUCTURAL DETAILS OF SMALL BRIDGES AND CULVERTS

14.1. **ABUTMENT AND WING WALL SECTIONS:** For RCC slab culverts designed for IRC single lane of class 70 R loading or 2-lanes of IRC class A loading, the abutment, pier and wing wall sections upto 4 m height for a minimum bearing capacity of the soil of 1 6.5 t/m2 are given in **Plate 5**.These sections are not applicable for seismic zones IV and V shall be designed as per IRC 112..

The base widths of the abutment and the pier depend on the bearing capacity of the soil. The pressure at the toe of the abutment should be worked out to ensure that the soil is not over stressed.

The pier sections should be made preferably circular in the case of skew crossings.

14.2. Filling behind the abutments, wing walls and return walls shall confirm to IRC:78 as reproduced in Appendix B

14.3. **UNREINFORCED MASONRY ARCHES:** Plate 6 shows the details of arch ring of segmental masonry arch bridges without footpaths for spans 6 m and 9 m.

The section of abutment and pier for masonry arch bridges will have to be designed taking into account the vertical reaction, horizontal reaction and the moment at springing due to dead load and live load. **Table 14.1** gives the details of horizontal reaction, vertical reaction and moment at springing for arch bridges of span 6 m and 9 m and Table 14.2 gives the influence line ordinates for horizontal reaction, vertical reaction and moment at springing for a unit load placed on the arch ring.

Table 14.1 Vertical Reaction, Horizontal Reaction and Moment at Springing Due to Dead Load of Arch Ring Masonry, Fill Material and Road Crust for One Meter of Arch Measured Along the Transverse Direction (i.e. Perpendicular to the Direction of Traffic) for Right Bridges

Sl. No.	Effective Span(m)	Horizontal Reaction (Tonnes)	Vertical Reaction (Tonnes)	Moment at Springing (Tonne Metres)
(1)	6	9.35	10.92	(+) 0.30
(2)	9	17.40	21.00	(+) 0.47

Notes: 1. Unit weight of arch ring masonry, fill materials and the road crust is assumed as 2.24t/m^3.
2. Positive sign for moment indicates tension on the inside of arch ring.

Table 14.2 influence Line Ordinates for Horizontal Reaction (H) Vertical Reaction at Support (VA) and (VB) and Moment at Springing (MA) and (MB) for Unit Load, say 1 Tonne Located along the Arch Axis at an Angle 9 Degrees from the Radius OC. Rise of Arch is One Quarter of Span (Fig. 14.1)

Sl.No.	θ Degree tonnes	H in tonnes	VA in tonnes	VB in (tonnes-m)	MA (tonnes-m)	MB
(a)	Effective Span 6 m					
(1)	0	0.93	0.500	0.500	(-)0.2213	(-)0.2213
(2)	5	0.91	0.577	0.423	(-)0.1388	(-)0.2775
(3)	15	0.75	0.725	0.275	(+)0.0713	(-)0.3075
(4)	25	0.52	0.849	0.152	(+)0.2513	(-)0.2588
(5)	35	0.25	0.940	0.061	(+)0.3413	(-)0.1388
(6)	45	0.05	0.989	0.012	(+)0.2438	(-)0.0338
(7)	53°8'	0	1.000	0	0	0
(b)	Effective Span 9 m					
(1)	0	0.93	0.500	0.500	(-)0.3318	(-)0.3318
(2)	5	0.91	0.577	0.423	(-)0.2081	(-)0.4163
(3)	15	0.75	0.725	0.275	(+)0.1069	(-)0.4612
(4)	25	0.52	0.849	0.152	(+)0.3769	(-)0.3881
(5)	35	0.25	0.940	0.061	(+)0.5119	(-)0.2081
(6)	45	0.05	0.989	0.012	(+)0.3656	(-)0.0506
(7)	53°8'	0	1.000	0	0	0

Note: Positive sign for moment indicates tension on the inside of arch ring

14.4. RCC SLABS

14.4.1. The details of RCC slabs to be used for culverts and bridges at right crossings and skew crossings shall be designed as per IRC 112.

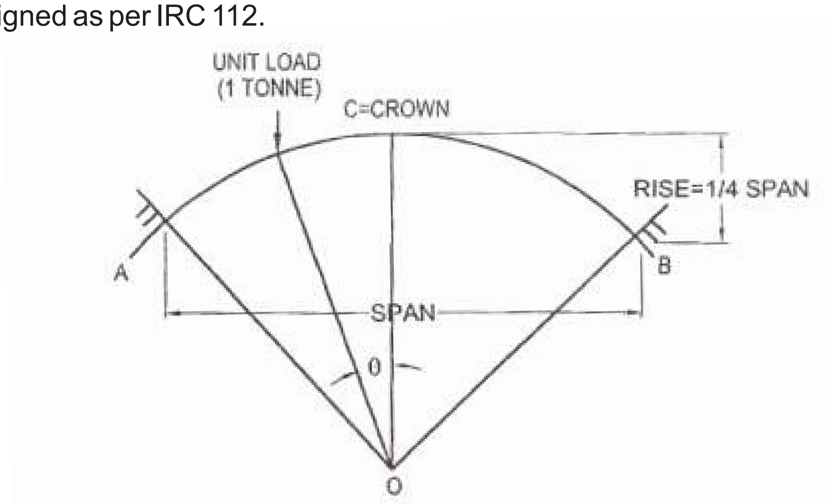


Fig. 14.1

14.5. BOX CELL STRUCTURES: The details for single cell box upto 8 m opening, for double cell upto 3 m opening of each cell and triple cell upto 3 m opening of each cell with and without earth cushion for varying bearing capacity may be designed as per IRC 112.

14.6. RCC PIPE CULVERTS: The details of pipe culverts of 1.2 m dia, with single or double pipes having cement concrete or granular materials in bed are given in **Plates 8- 11**

37

ARTICLE 15

ELEMENTS OF THE HYDRAULICS OF FLOW THROUGH BRIDGES

15.1. The formulae for discharge passing over broad crested weirs and drowned orifices have been developed ab initio in this section. These formulae are very useful for computing flood discharges from the flood marks left on the piers and abutments of existing bridges and calculating afflux in designing new bridges. It is necessary to be familiar with the rationale of these formulae to be able to apply them intelligently.

15.2. **BROAD CRESTED WEIR FORMULAE APPLIED TO BRIDGE OPENINGS: In Fig. 15.1,** X-X is the water surface profile, and Z-Z the total energy line. At Section 1, the total energy.

$$H = \frac{u^2}{2g} + D_u \qquad \text{... (15.1)}$$

At Section 2, let the velocity head AB be a fraction n of H, i.e.,

$$AB = \frac{v^2}{2g} = n\,H \qquad \text{... (15.2)}$$

Equating total energies at Sections 1 and 2 ignoring the loss of head due to entry and friction

$$H = AC = AB + BC = nH + BC$$

$$\therefore BC = (1\text{-}n)\,H \qquad \text{... (15.3)}$$

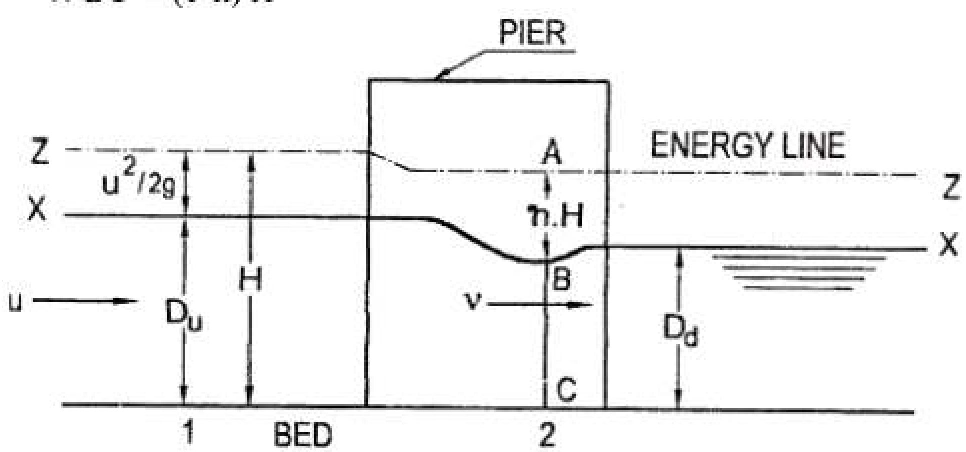


Fig. 15.1

The area of flow at Section 2,

a = BC x linear waterway

= (1-in)HL

Where L is the linear waterway. From Eq. (15.2) Velocity at Section 2

$v = (2gn\,H)^{½}$

Therefore, the discharge through the bridge

Q = av

$= (l\text{-}n)HL(2gnH)^{½}$

To account for losses in friction, a coefficient Cw may be introduced. Thus,

$Q = C_w (l-n)HL(2gnH)^{1/2}$

$= C_w\sqrt{2g}\, LH^{3/2}(n^{1/2} - n^{3/2})$ ...(15.4)

The depth BC adjusts itself so that the discharge passing through the section is maximum. Therefore, differentiating

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

$$\frac{1}{2} n^{-1/2} - \frac{3}{2} n^{1/2} = 0$$

$$\therefore n = \frac{1}{3}$$

Putting $n = \frac{1}{3}$ in Eq. (15.4) we get

$Q = 1.706\, C_w LH^{3/2}$ ... (15.5a)

C[illegible] with Eq. (15.1)

$Q = 1.706\, C_w L \left(D_u + \frac{u^2}{2g}\right)^{3/2}$ ... (15.5b)

Since AB is $\frac{1}{3}$ H, therefore, BC is $\frac{2}{3}$ H, or 66.7 per cent of H.

On exit from the bridge, some of the velocity head is reconverted into potential head due to the expansion of the section and the water surface is raised, so that D_d is somewhat greater than BC, i.e. greater than 66.7 per cent of H. In fact, observations have proved that, in the limiting condition,

D_d can be 80 per cent of D_u. Hence, the following rule:

"So long as the afflux (D_u - D_d) is not less than $\frac{1}{4}$ D_d, the weir formula applies, i.e., Q depends on D_u and is independent of D_d

The fact that the downstream depth D_d has no effect on the discharge Q, nor on the upstream depth D_u when the afflux is not less than $\frac{1}{4}$ D_d is due to the formation of the "Standing Wave"

The coefficient C_w may be taken as under:-

(1)	Narrow Bridge opening with or without floors	0.94
(2)	Wide bridge opening with floors	0.96
(3)	Wide bridge opening with no bed floors	0.98

39

15.3. THE ORIFICE FORMULAE: When the downstream depth, D_d is more than 80 per cent of the upstream depth D_u, the weir formula does not hold good, i.e. the performance of the bridge opening is no longer unaffected by D_d.

In Fig. 15.2, X-X is the water surface line and Z-Z the total energy line.

Apply Bernoulli's Equation to points 1 and 2, ignoring the loss of head (h) due to entry and friction.

$$D_u + \frac{u^2}{2_g} = D^1 + \frac{v^2}{2_g}$$

or

$$\frac{V^2}{2g} = D_u - D^1 + \frac{u^2}{2g}$$

Then

$$V = \left[\sqrt{2g}\,(D_u - D^1) + \frac{u^2}{2g}\right]$$

Put $\quad D_u - D = h^1$

Then,

$$v = \sqrt{2g}\left(h^1 + \frac{u^2}{2g}\right)^{1/2}$$

The discharge through the Section 2,

$$Q = a\ v$$

Substituting ...(15.6)

$$Q = LD^1\ \sqrt{2g}\left(h^1 + \frac{u^2}{2g}\right)^{1/2}$$

Now the fractional difference between D^1 and D_d is small. Put D_d for D^1 in Eq. (15.6). ...(15.7)

$$Q = LD_d\ \sqrt{2g}\left(h' + \frac{u^2}{2g}\right)^{1/2}$$

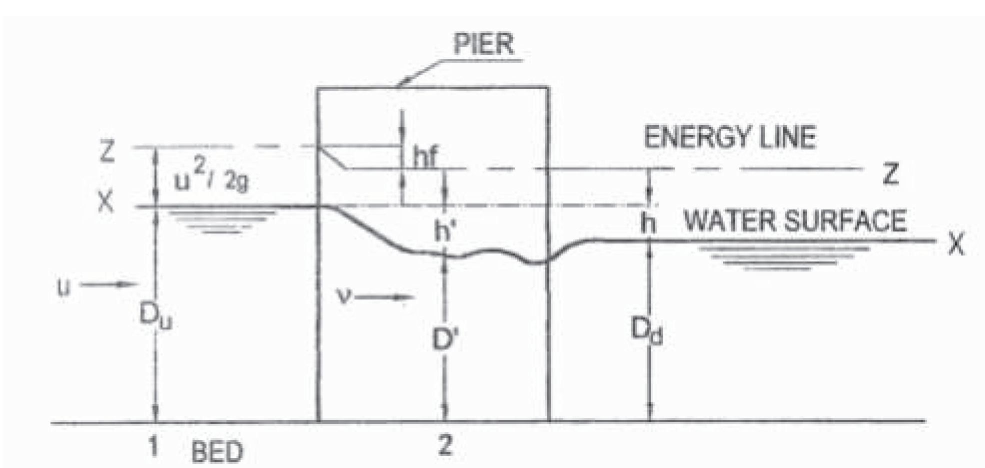


Fig. 15.2

40

In the field it is easier to work in terms of $h = D_u - D_d$ instead of h'. But h is less than n as on emergence from the bridge the water surface rises, due to recovery of some velocity energy as potential head. Suppose $eu^2/2g$ represents the velocity energy that is converted into potential head.

Then

$$h' = h + \frac{eu^2}{2g}$$

Substituting in equation (15.7)

$$Q = LD_d \sqrt{2g} \left(h + (e+1) \frac{u^2}{2g} \right)^{1/2}$$

Now introduce a co-efficient C_0 to account for losses of head through bridge, we get.

$$Q = C_0 \sqrt{2g}\, LD_d \left(h + (1+e) \frac{u^2}{2g} \right)^{1/2} \quad \text{h is Afflux} \qquad \ldots (15.8)$$

For values of e and C_0, see **Figs. 15.3 and 15.4**[10]

15.4. **IN CONCLUSION:** Let us get clear on some important points (1) In all these formulae D_d is not affected in any way by the existence of the bridge. It depends only on the conveyance factor and slope of tail race. D_d has, therefore, got to be actually measured or calculated from area - slope data of the channel as explained already in Article 7.

(2) The Weir Formula applies only when a standing wave is formed, i.e., when the afflux ($h = D_u - D_d$) is not less than ¼ D_d.

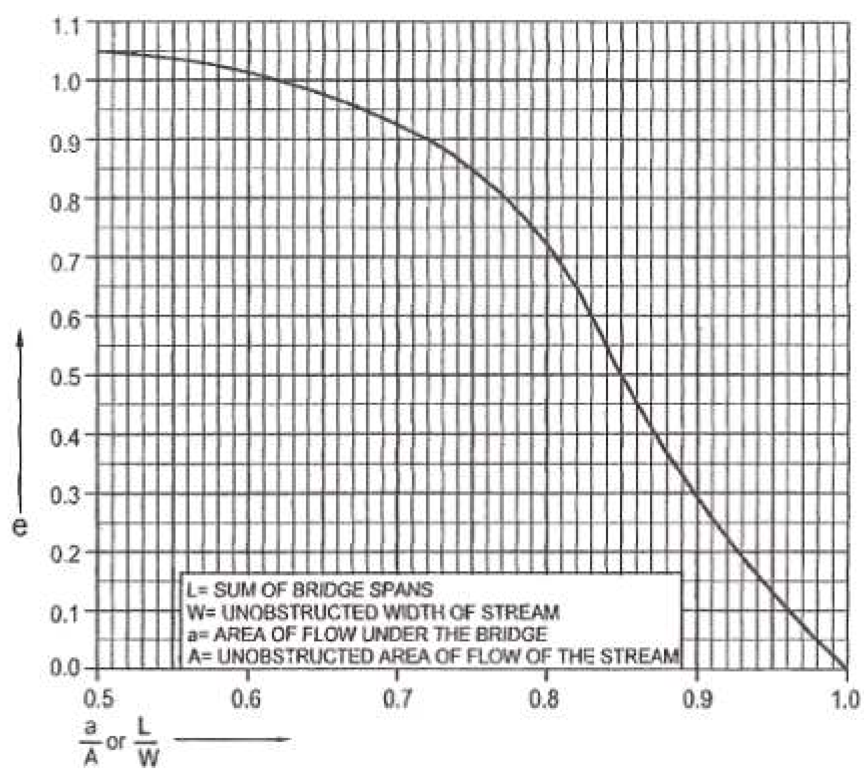


Fig. 15.3 Elements of the hydraulics of flow through bridges
The orifice formula coefficient "e"

(3) The Orifice Formulae with the suggested values of C_Q and e should be applied when the afflux is less than ¼ D_d.

15.5. Examples have been worked out in Articles 1 6 and 1 7 to show how these formulae can be used to calculate aflux and discharge under bridges.

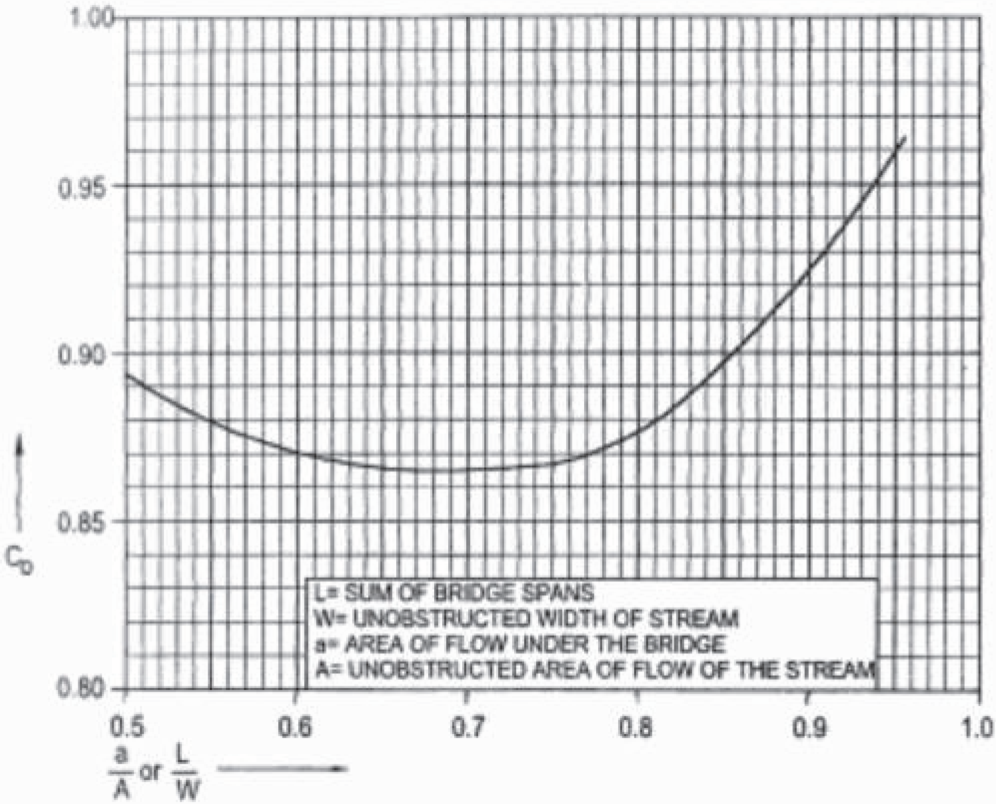


Fig. 15.4 The Orifice Formula Coefficient "C_o"

ARTICLE 16

AFFLUX

16.1. The afflux at a bridge is the heading up of the water surface caused by it. It is measured by the difference in levels of the water surfaces upstream and downstream of the bridge (Fig. 16.1)

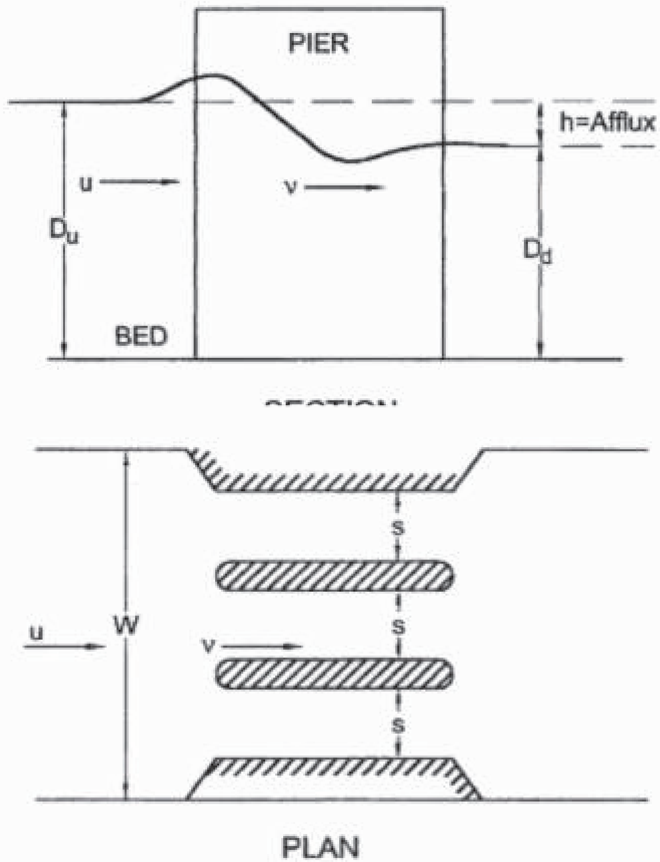


Fig. 16.1

16.2. When the waterway area of the openings of a bridge is less than the unobstructed natural waterway area of the stream, i.e., when the bridge contracts the stream, afflux occurs. Contraction of the stream is normally not done, but under some circumstances it is taken recourse to, if it leads to ponderable economy. Also, in the case of some alluvial streams in plains the natural stream width may be much in excess of that required for regime. When spanning such a stream, it has to be contracted to, more or less, the width required for stability by providing training works.

16.3. Estimating afflux is necessary to see its effect on the 'clearance' under the bridge, on the regime of the channel upstream of the bridge; and on the design of training works.

16.4. For calculating afflux we must know (1), the discharge Q, (2) The un obstructed width of the stream W, (3) the linear waterway of the bridge L, and (4) the average depth downstream of the bridge D_d.

16.5. **THE DOWNSTREAM DEPTH D_D IS NOT AFFECTED BY THE BRIDGE:** it is controlled by the conveyance factor and slope of the channel below the bridge. Also, the depth, that prevails at the bridge site before the construction of the bridge, can be assumed to continue to prevail just down

43

stream of the bridge after its construction. Thus, D_d is the depth that prevails at the bridge site before its construction. To estimate afflux, we must know D_d. In actual problems, D_d is either given or can be calculated from the data supplied.

16.6. **EXAMPLE:** A bridge, having a linear waterway of 25 m, spans a channel 33 m wide carrying a discharge of 70 m^3/s. Estimate the afflux when the downstream depth is 1 m.

D_d= 1 m; W = 33m;L = 25m

$$Q = C_o \sqrt{2g}\ LD_d \sqrt{\left(h + (1+e)\frac{u^2}{2g}\right)}$$

$$\frac{L}{W} = \frac{25}{33} = 0.757$$

Afflux Corresponding to this, $C_o = 0.867$, $e = 0.85$, $g = 9.8\ m/sec^2$

$$70 = 0.867 \times 4.43 \times 25 \times 1 \sqrt{h + \frac{1.85u^2}{2g}}$$

$$\therefore\ h + 0.0944u^2 = 0.53 \qquad \text{...(16.1)}$$

Also, just upstream of the bridge

$$Q = W\ (D_d + h)\ u$$

$$70 = 33\ (1 + h)\ u$$

$$h = \frac{70}{33u} - 1 \qquad \text{...(16.2)}$$

Subtracting for h from (16.2) in (16.1) and rearranging

$u = 0.0617\ u^3 + 1.386$ $u = 1.68$ m/sec   How is this cubic equation solved?

Substituting for u in (16.1)

h=0.263 m

Alternatively, assume that h is more than ¼ D_d

and apply the Weir formula

$Q = 1.706\ C_w\ LH^{3/2}$

$70 = 1.706 \times 0.94 \times 25 \times H^{3/2}$

H = 1.45m

$H = D_u + u^2/2g = D_u$ (approx.)

Or D_u =1.45 m (approx.)

Now,

$Q = W\ D_u\ u$

$\therefore$ 70 = 33 x 1.45 u

$\therefore$ u = 1.46; $u^2/2g$ = 0.1086 m

$H = D_u + u^2/2g$

44

i.e.

$1.45 = D_u + 0.1086$

$D_u = 1.3414$ m

$h = D_u - D_d = 1.3414 - 1.0 = 0.3414$ m

Adopt h = 0.3414 m. Since h is actually more than ¼ D_d, therefore, the value of afflux arrived by the Weir Formula is to be adopted.

16.7. EXAMPLE: The unobstructed cross-sectional area of flow of a stream of 90 m^2 and the width of flow is 30 m. A bridge of 4 - spans of 6 m clear is proposed across it. Calculate the afflux when the discharge is 280 m^3/s.

w = 30 m; L = 24 m, $D_d = \frac{90}{30} = 3.00$ m

The depth before the construction of the bridge is the depth downstream of the bridge after its construction. Hence, $D_d = 3.00$ m

$$\frac{L}{W} = \frac{24}{30} = 0.8$$

By the Orifice Formula the discharge through the bridge

$$280 = 0.877 \times 4.43 \times 24 \times 3.00 \times \sqrt{h + 1.72 \frac{u^2}{2g}}$$

$$280 = 279.7 \sqrt{h + 1.72 \frac{u^2}{2g}}$$

$$h + \frac{1.72\,u^2}{2g} = 1 \qquad \text{...(16.3)}$$

Now, the discharge just upstream of the bridge

$$280 = (3 + h)30u \qquad \text{...(16.4)}$$

Putting for h from (16.4) in (16.3) and rearranging

$u = 2.33 + .02195\,u^3$

L = 2.81 m/sec

Putting for u in (1 6.4)

$h = 0.32 \text{ m} < 1/4\,D_d$

16.8. EXAMPLE: A bridge of 3 spans of 8 m each is proposed across a stream, whose unobstructed width is 36 m, slope 1/2000 and discharge 400 m^3/sec. Calculate the afflux (n=0.03)(Fig. 16.2).

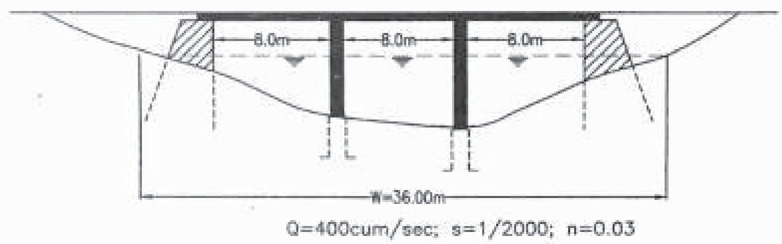


Fig. 16.2

We have first to find D_d,

Q = AV = (RP) V = RWV

$\therefore$ RV = $\frac{Q}{W} = \frac{400}{36}$ = 11.11

Knowing n = 0.03; S = 1/2000, read velocity for various values of R from **Plate 3** and select that pair whose product is 11.11. Thus, we get.

R = 5.1

V = 2.18

Take D_d = R =5.1 m

Now, W = 36rn,L = 24 M, D_d= 5.1 m

$\frac{L}{W} = \frac{24}{36}$ = 0.67 Therefore, C_0 = 0.865; e = 0.95

By the Orifice Formula, the discharge through the bridge

$$Q = C_o \sqrt{2g}\, L\, D_d \left[h + (1+e)\frac{u^2}{2g}\right]^{1/2}$$

$$400 = 0.865 \times \sqrt{2 \times 9.8} \times 24 \times 5.1 \left[h + 1.95 \frac{u^2}{2g}\right]^{1/2}$$

$$0.8528 - \left[h + \frac{0.975u^2}{g}\right]^{1/2}$$

$$\text{or } h + 0.009\,u^2 = 0.7272 \qquad \text{... (16.5)}$$

The discharge just upstream of the bridge

$$400 = 36(5.1 + h)u$$

$$\text{i.e., } h = \frac{11.11}{u} - 5.1 \qquad \text{... (16.6)}$$

Put value for h from (16.6) in (16.5) and rearrange

$$u - 0.017\,u^3 = 1.90$$

$$\therefore u = 2.05 \text{ m/sec}$$

Put this value of u in (16.6), we get,

$$h = \frac{11.11}{2.05} - 5.1 = 0.31\,m$$

ARTICLE 17

WORKED OUT EXAMPLES ON DISCHARGE PASSED BY EXISTING BRIDGES FROM FLOOD MARKS

17.1. CALCULATING DISCHARGE BY THE WEIR FORMULAE

Example: The unobstructed width of a stream is 40 m. The linear waterway of a bridge across is 27 m. In a flood, the average depth of flow downstream of the bridge was 3.0 m and the afflux was 0.9 m. Calculate the discharge (Fig. 17.1).

$$\frac{h}{D_d} = \frac{0.90}{3} = 0.30$$

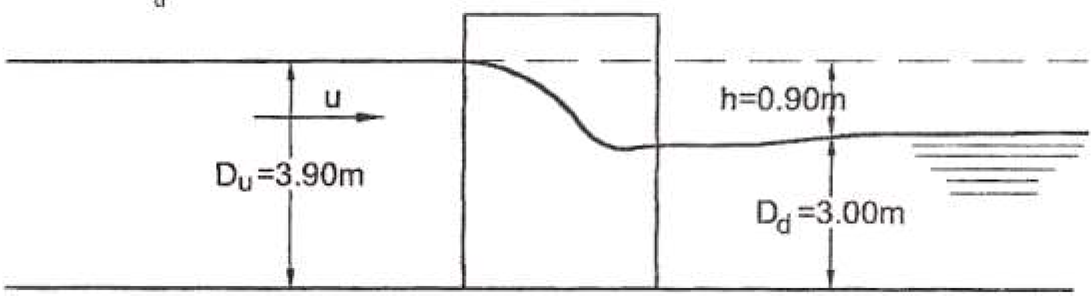


Fig. 17.1

Since h is more than 0.25 D_d, therefore, the Weir Formula will apply

w = 40 m; L = 27 m, h = 0.9 m

Let the velocity of approach be u m/sec. The discharge at a section just upstream of the bridge.

$$Q = u \times 3.9 \times 40 = 156u \qquad \text{...(17.1a)}$$

The discharge through the bridge by the Weir formula

$$Q = 1.706 \times 0.98 \times 27 \times \left(3.9 + \frac{u^2}{19.6}\right)^{3/2}$$

$$= 45.14 \left(3.9 + \frac{u^2}{19.6}\right)^{3/2} \qquad \text{...(17.1b)}$$

Equating values of Q from (17.1a) and (17.1b)

$$156\,u = 45.14 \left(3.9 + \frac{u^2}{19.6}\right)^{3/2}$$

Rearranging

$u^{2/3}$ -0.0222 u^2 = 1.70

or u = 2.45 m/sec ?

Putting the value of u in (17.1 a) or (17.1b) we get Q

Q – 156x2.45

47

$= 382\ m^3/sec$

Try the Orifice Formula

Discharge through the bridge by the Orifice Formula

$$Q = 0.85 \times 4.43 \times 27 \times 3 \sqrt{0.90 + 1.95 \frac{u^2}{19.6}}$$

$$= 305 \sqrt{0.090 + 0.1u^2} \qquad \text{... (17.1c)}$$

Discharge just upstream of the bridge

$$Q = 40 \times 3.9 \times u$$

$$= 156\ u \qquad \text{... (17.1d)}$$

Equating values of Q in (17.1c) and (17.1d)

$$305 \sqrt{(0.90 + 0.1u^2)} = 156\ u$$

Simplifying

$u = 2.36$

Substituting for u in (17.1c) and (17. Id) we get Q

$Q= 156 \times 2.36 = 368.16\ m^3/sec$

This result is about the same as given by the first method. In fact, the Orifice Formula, with the recommended value of C_o and e gives nearly correct results even where the conditions are appropriate for the Weir Formula. But the converse is not true.

17.2. CALCULATING DISCHARGE BY THE ORIFICE FORMULA

Example: The unobstructed width of a stream is 30 m and the linear waterway of the bridge across is 22 m. During a flood the average depth of flow downstream of the bridge was 1 .6 m and the afflux 0.10 m. Calculate the discharge (**Fig. 17.2**).

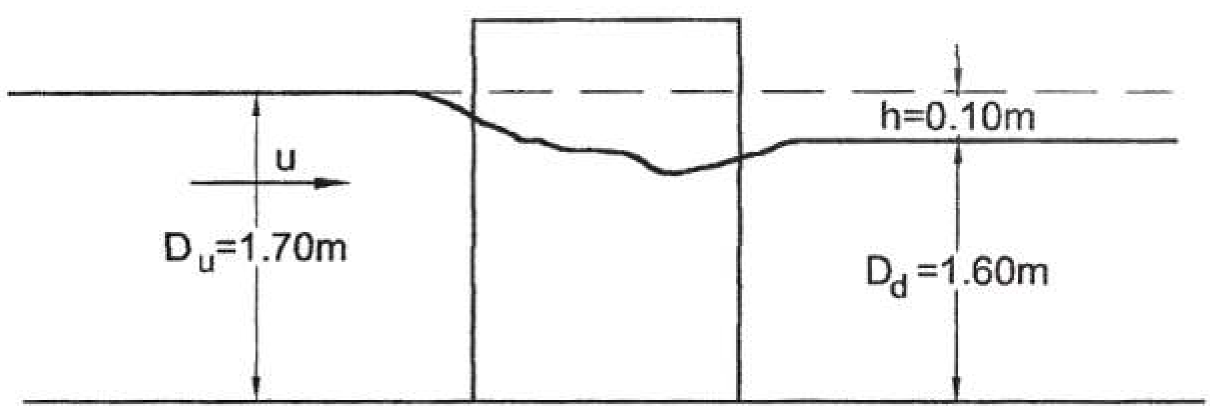


Fig. 17.2

48

Given: W = 30 m, L = 22 m, h 0.1m, Depth of flow =1.6 m. Let velocity of approach be u m/s. The discharge at a section just upstream of the bridge will be.

Q = ux 1.7x30 ...(17.2a)

Contraction =

$$Q = Co\sqrt{2g}\ x\ L\ x\ D_d \left| h + (1 + e)\ \frac{u^2}{2g} \right|^{1/2}$$

$$= 0.87 \times 4.43 \times 22 \times 1.6 \left| 0.1 + 1.9\ \frac{u^2}{19.6} \right|^{1/2}$$

$$= 135.66\ [0.1 + 0.097\ u^2]^{1/2} \quad \text{... (17.2b)}$$

Equating values of Q in (17.2a) and (17.2b)

$$51\ u = 135.66\,[0.1 + 0.097u^2]^{1/2}$$

$$u = 1.51\ m/s$$

Substituting for u in (1 7.2a) and (1 7.2b) to get Q

Q = 1.51 x 1.7 x 30

= 77.01 cu. m/sec

17.3. **THE BORDER LINE CASES:** An example will now follow to illustrate what results are obtained by applying the Weir Formula and Orifice Formulae to cases which are on the borderline, i.e., where the afflux is just ¼ D_d.

17.4. **EXAMPLE:** A stream whose unobstructed width is 35 m is spanned by a bridge whose linear waterway is 30 m. During a flood the average downstream depth was 2.6 m and the afflux was 0.65 m. Calculate the discharge (**Fig. 17.3**).

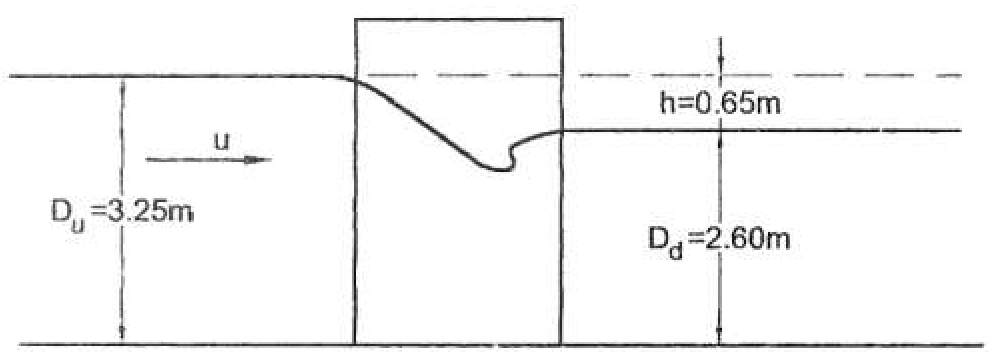


Fig. 17.3

$$\frac{h}{D_d} = \frac{0.65}{2.6} = 0.25$$

Since h is ¼ D_d, therefore, both the weir formula and Orifice formula should apply.

By the Weir Formula

If the velocity of approach is u, the discharge just upstream of the bridge.

$$Q = 35 \times 3.25 \times u = 113.75u \qquad \text{... (17.3a)}$$

The discharge through the bridge

$$Q = 1.706 \times 0.98 \times 30 \times \left(3.25 + \frac{u^2}{19.6}\right)^{3/2} \qquad \text{... (17.3b)}$$

Equating values of Q from (17.3a) and (17.3b)

$$113.75\ u = 50.16(3.25 + 0.051\ u^2)^{3/2}$$

u = 3.27 m/s

Put for u in (17.3a) or(17.3b)

Q= 1 13.75x3.2 =371.96 m^3/s

By the Orifice Formula

Contraction =

If u is the velocity of approach, the discharge just upstream of the bridge.

$Q = 35 \times 3.25u = 1\ 13.75u$ ...(17.3c)

The discharge under the bridge by the Orifice Formula

$Q = 0.906 \times 4.43 \times 30 \times 2.6\ (0.65 + 0.0735\ u^2)^{1/2}$

$= 310.98\ (0.65 + 0.0735\ u2)^{1/2}$

Equating values of Q from (17.3c) and (17.3d) and Squaring and rearranging

$113.75\ u = 310.98 \times (0.65 + 0.0735\ u2)^{1/2}$

$\therefore$ u = 3.27 m/s

Substituting for u in (1 7.3c) and (1 7.3d), we get Q

Q= 1 13.75 x3.27 =371.96 m^3/s

ARTICLE 18

OVERTOPPING OF THE BANKS

18.1. In plains where the ground slopes are gentle and the natural velocities of flow in streams are low, the flood water may spill over one or both the banks of the stream at places.

18.2. **HEIGHT OF APPROACH ROADS:** Consider the case where main channel carries the bulk of the discharge and a small fraction of it flows over the banks somewhere upstream of the bridge. If the overflow strikes high ground at a short distance from the banks, it can be forced back into the stream and made to pass through the bridge. This can be done by building the approach roads of the bridge solid and high so that they intercept the overflow. In this arrangement, the linear waterway of the bridge must be ample to handle the whole discharge without detrimental afflux. Also, the top level of the approach road must be high enough to prevent overtopping. If the velocity of the stream is V(m/s), the water surface level, where it strikes the road embankment, will bev^2/19.6(m) higher than HFL in the stream at the point, where the overflow starts. This arrangement is, therefore, normally feasible where the stream velocity is not immoderately high.

18.3. **SUBSIDIARY OR RELIEF CULVERTS:** Sometimes, however, the overflow spreads far and away from the banks. This is often the case in alluvial plains, where the ground level falls continuously away from the banks of the stream. In sue h cases, it is impossible to force the overflow back into the main stream. The correct thing to do is to pass the overflow through relief culverts at suitable points in the road embankment. These culverts have to be carefully designed. They should not be too small to cause detrimental ponding up of the overflow, resulting in damage to the road or some property, nor, should they be so big as to attract the main current.

18.4. **DIPS AND BREACHING SECTIONS IN APPROACH ROADS:** It is sometimes feasible as well as economical to provide permanent dips (or alternatively breaching sections) in the bridge approaches to take excessive overflows in emergencies. The dips or breaching sections have to be sited and designed so that the velocity of flow through them does not become erosive, cutting deep channels and ultimately leading to the shifting of the main current. However, since the state highways, national highways, and expressways are to be designed as all-weather roads, dips and breaching sections may be considered only for rural roads and major district roads.

18.5. **RETROGRESSION OF LEVELS:** Suppose water overflows a low bank some where upstream of the bridge and after passing through a relief culvert, rejoins the main stream some where lower down. When the flood in the main channel subsides, the ponded up water at the inlet of the subsidiary culvert gets a free fall. Under such conditions deep erosion can take place. A deep channel is formed, beginning at the outfall in the mains stream and retrogressing towards the culvert. This endangers the culvert. To provide against this, protection has to be designed downstream of the culvert so as to dissipate the energy of the falling water on the same lines as is done on irrigation falls. That Is a suitable cistern and baffle wall should be added for dissipating the energy and the issuing current should be stilled through a properly designed expanding flume.

ARTICLE 19

PIPES AND BOX CULVERTS

19.1. FEASIBILITY OF PIPE AND BOX CULVERTS FLOWING FULL

19.1.1. Some regions along plain consist of vast flat without any deep and defined drainage channels in it. When the rain falls, the surface water moves in some direction in a wide sheet of nominal depth. So long as this movement of water is unobstructed, no damage may occur to property or crops. But when a road embankment is thrown across the country intercepting the natural flow, water ponds up on one side of it. Relief has then to be afforded from possible damage from this ponding up by taking the water across the road through causeways or culverts.

19.1.2. In such flat regions the road runs across wide but shallow dips and, therefore, the most straight forward way of handling the surface flow is to provide suitable dips (i.e., cause ways) in the longitudinal profile of the road and let water pass over them.

19.1.3. There may, however, be cases where the above solution is not the best. Some of its limitations may be cited. Too many causeways or dips detract from the usefulness of the road. Also, the flow of water over numerous sections of the road, makes its proper maintenance problematic and expensive. Again, consider the case of a wet cultivated or waterlogged country (and flat plains are quite often swampy and waterlogged) where the embankment has necessarily got to be taken high above the ground. Frequent dipping down from high road levels to the ground produces a very undesirable road profile. And, even cement concrete slabs, in dips across a waterlogged country, do not rest evenly on the mud underneath them. Thus, it will appear that constructing culverts in such circumstances should be a better arrangement than providing dips or small causeways.

19.1.4. After we have decided that a culvert has to be constructed on a road lying across some such country, we proceed to calculate the discharge by using one of the run off formulae, having due regard to the nature of terrain and the intensity of rainfall as already explained in Article-4. But the natural velocity of flow cannot be estimated because (i) there is no defined cross-section of the channel from which we may take the area of cross-section and wetted perimeter and (ii) there is no measurable slope in the drainage line. Even where we would calculate or directly observe the velocity, it may be so small that we could not aim at passing water through the culvert at that velocity, because the area of waterway required for the culvert $(A+\frac{Q}{V})$ is prohibitively large. In such cases the design has to be based on an increased velocity of flow through the culvert and to create the velocity the design must provide for heading up at the inlet end of the culvert. Economy, in design being the primary consideration, the correct practice, indeed is to design a pipe or a box culvert on the assumption that water at the inlet end may head upto a predetermined safe level above the top of the inlet opening. This surface level of the headed up water at the upstream end has to be so fixed that the road bank should not be overtopped, nor any property in the flood plain damaged.

Next, the level of the downstream water surface should be noted down. This will depend on the size of the slope of the leading out channel and is normally, the surface level of the natural unobstructed flow at the site, that prevails before the road embankment is constructed.

After this we can calculate the required area of cross-section of the barrel of the culvert by applying the principles of hydraulics discussed in this Article.

19.1.5. The procedure set out above is rational and considerable research has been carried out on the flow of water through pipe and box culverts, flowing full.

19.1.6. In the past, use was extensively made of empirical formulae which gave the vent way area required for a culvert to drain a given catchment area. Dun's Drainage Table is one of the class and is purely empirical. This table is still widely used, as it saves the trouble of hydraulic calculations.

But it is unfortunate that recourse is often taken rather indiscriminately to such short cuts, even where other more accurate and rational procedure is possible and warranted by the expense involved. Dun's Table or other in that class, should NOT be used until suitable correction factors have been carefully evolved from extensive observations (in each particular region with its own singularities of terrain and climate) of the adequacy or otherwise of the existing culverts vis-a-vis their catchment area.

19.1.7. Considerations of economy require that small culverts, in contrast with relatively larger structures across defined channels, need not be designed normally to function with adequate clearance for passing floating matter. The depth of a culvert should be small and it does not matter if the opening stops appreciably below the formation level of the road. Indeed, it is correct to leave it in that position and let it function even with its inlet submerged. This makes it possible to design low abutments supporting an arch or a slab, or alternatively, to use round pipes or square box barrels.

19.1.8. High head wall should not be provided for retaining deep over-fills. Instead of this the length of the culverts should be increased suitably so that the road embankment, with its natural slopes, is accommodated without high retaining head walls.

19.1.9. Where masonry abutments supporting arches or slabs are designed for culverts functioning under "head", bed pavements must be provided. And, in all cases, including pipe and box culverts, adequate provision must be made at the exit against erosion by designing curtain walls. Where the exit is a free fall, a suitable cistern and baffle wall must be added for the dissipation of energy and stilling of the ensuring current.

19.2. HYDRAULICS OF THE PIPE AND BOX CULVERTS FLOWING FULL

19.2.1. **The permissible heading up at the inlet:** It has been explained already that where a defined channel does not exist and the natural velocity of flow is very low, it is economical to design a culvert as consisting of a pipe or a number of pipes of circular or rectangular section functioning with the inlet submerged. As the flood water starts heading up at the inlet, the velocity through the barrel goes on increasing. This continues till the discharge passing through the culvert equals the discharge coming towards the culvert. When this state of equilibrium is reached the upstream water level does not rise any higher.

For a given design discharge the extent of upstream heading up depends on the vent way of the culvert. The latter has to be so chosen that the heading up should not go higher than a predetermined safe level. The criterion for safety being that the road embankment should not be overtopped, nor any property damaged by submergence. The fixing of this level is the first step in the design.

19.2.2. **Surface level of the tail race:** It is essential that the HFL in the outfall channel near the exit of the culvert should be known. This may be taken as the HFL prevailing at the proposed site of the culvert before the construction of the road embankment with some allowance for the concentration of flow caused by the construction of the culvert.

19.2.3. **The operating head when the culverts flow full:** In this connection the cases that have to be considered are illustrated in **Fig. 19.1.** In each case the inlet is submerged and the culvert flows full. In case (a) the tail race water surface is below the crown of the exit and in case(b) it is above that. The operating head in each case is marked "H". Thus, we see that: "When the culvert flows full, the operating head, H, is the height of the upstream water level measured from the surface level in the tail race or from the crown of the exit of the culvert whichever level is higher".

19.2.4. **The velocity generated by "H"** : The operating head "H" is utilized in (i) supplying the energy required to generate the velocity of flow through the culvert (ii) Forcing water through the inlet of the culvert, and (iii) overcoming the frictional resistance offered by the inside wetted surface of the culvert.

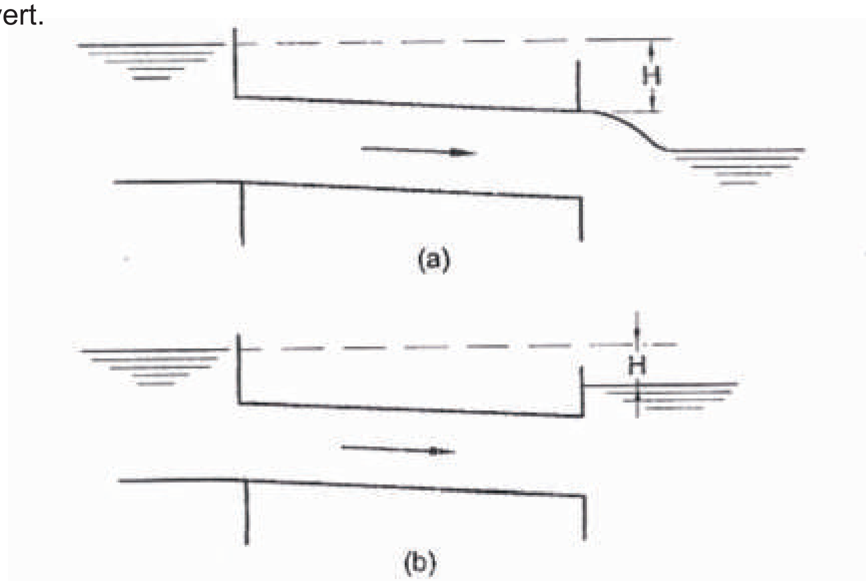


Fig. 19.1

If the velocity through the pipe is v, the head expended in generating is $\frac{v^2}{2g}$

As regards the head expended at the entry it is customary to express it as a fraction K_e of the velocity head $\frac{v^2}{2g}$ Similarly, the head required for overcoming the friction of the kf of the $\frac{v^2}{2g}$ From this it follows that:

$$H = [I + KQ + Kf]\,\frac{v^2}{2g} \qquad ...(19.1)$$

From this equation we can calculate the velocity v, which a given head H will generate in a pipe flowing full, if we know K_e and K_f.

19.2.5. Values of K_e and K_f : K_e principally depends on the shape of the inlet. The following values are commonly used:

K_e = 0.08 for bevelled or

Bell - mouthed entry

= 0.505 for sharp edgedentry ... (19.2)

As regards K_f it is a function of the Length L of the culvert, its hydraulic mean radius R, and the coefficient of rugosity n of its surface.

The following relationship exists between K_f and n:

$$K_f = \frac{14.85n^2}{R^{1/3}} \times \frac{L}{R} \qquad ...(19.3)$$

54

For cement concrete circular pipes or cement plastered masonry culverts of rectangular section, with the co-efficient of rugosity n = 0.015, the above equation reduces to:

$$K_f = \frac{0.0334L}{R^{1.33}} \quad \text{...(19.4)}$$

The graphs in **Fig. 19.2** are based on Equation 19.4. For a culvert of known sectional area and length, K_f can be directly read from these graphs.

19.2.6. Values of K_e and K_f modified through research: Considerable research has recently been carried out on the head lost in flow through pipes. The results have unmistakably demonstrated the following:-

The entry loss co-efficient K_e depends not only on the shape of the entry but also on the size"entry and the roughness of its wetted surface. In general, K_e, increases with an increase in the e of the inlet.

Also K_f the friction loss co-efficient, is not independent of K_e. Attempts to make the entry efficient repercuss adversely on the frictional resistance to flow offered by the wetted surface of the barrel. In other words, if the entry conditions improve (i.e. if K_e decreases), the friction of the barrel increases (i.e. K_f increases). This phenomenon can be explained by thinking of the velocity distribution inside the pipe. When the entry is square and sharp edged, high velocity lines are concentrated nearer the axis of the barrel, while the bell-mouthed entry' gives uniform distribution of velocity over the whole section of the barrel. From this it follows that the average velocity being the same in both cases, the velocity near the wetted surface of the pipe will be lower for square entry than for bellmouthed entry. Hence, the frictional resistance inside the culvert is smaller when the entry is square than when it is bell-mouthed. Stream lining the entry is, therefore, not an unmixed advantage.

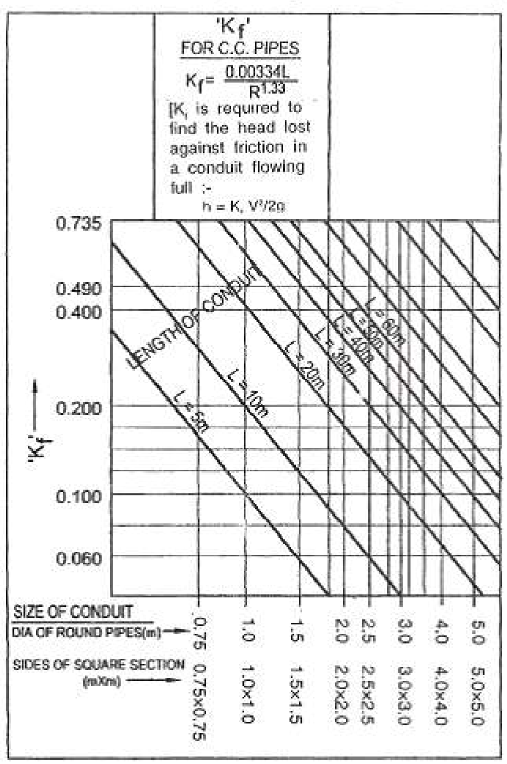


Fig. 19.2

Consequently, it has been suggested that the values of K_e, and K_f should be as given in Table 19.1

Table 19.1 Values of K_e and K_f [9]

Entry and friction	Circular pipes		Rectangular culverts	
	Square entry	Bevelled entry	Square entry co-efficient	Bevelled entry
K_e =	1.107 $R^{0.5}$	0.1	0.572 $R^{0.3}$	0.05
K_f=	$0.00394L/R^{1\text{-}2}$	$0.00394L/R^{1.2}$	$0.0035\ L/R^{1.25}$	$0.0035L/R^{1.25}$

19.2.7. Design calculations: We have said that

$H = (l + K_e + K_f)\ \frac{v^2}{2g}$

i.e. $v = 4.43\left(\frac{H}{1+K_e+K_f}\right)^{½}$

$Q = A \times 4.43\left(\frac{H}{1+K_e+K_f}\right)^{½}$

Suppose we know the operating head H and the length of the barrel L, and assume that the diameter of a round pipe or the side of a square box culvert is D.

From D calculate the cross-sectional area A and the hydraulic mean radius R of the culvert.

Now from R and L compute K_e and K_f using appropriate functions from **Table 19.1.** Then, calculate Q from Equation (19.5). If this equals the design discharge, the assumed size of the culvert is correct. If not, assume a fresh value of D and repeat.

19.2.8. Design chart (Plate 2712): Equation (19.5) may be written as

$$Q = \lambda \sqrt{2g}\,H \qquad \text{... (19.6)}$$

$$\lambda = \frac{A}{(1+K_e+K_f)1/2} \qquad \text{... (19.7)}$$

It is obvious that all components of λ in Equation (19.7) are functions of the cross-section, length, roughness, and the shape of the inlet of the pipe. Therefore, λ represents the conveying capacity of the pipe and may be called the 'Conveyance Factor'. The discharge, then depends on the conveyance factor of the pipe and the operating head. In **Plate 12**, curves have been constructed from equation (19.7) from which Q can be directly read for any known values of λ and H.

Also, in the same Plate, Tables are included from which X can be taken for any known values of (i) length, (ii) diameter in case of circular pipes or sides in case of rectangular pipes, and (iii) conditions of entry, viz., sharp-edged or round. The material assumed is cement, concrete and values of K_e and K_f used in the computation are based on functions in **Table 19.1.**

The use of **Plate12** renders the design procedure very simple and quick. Examples will now follow to illustrate.

19.2.9. Example data:

(1) Circular cement concrete pipe flowing full with bevelled entry

(2) Operating head = 1 m

(3) Length of the pipe = 25 m

(4) Diameter=1 m

Find the discharge.

See, in **Plate 12**, the Table for circular pipes with rounded entry.

For L=25 m and D=1 m, the conveyance factor

λ =0.618

Now refer to the curves in the same Plate. For λ = 0.6 1 8 and H= 1 m

Q=2.72 m^3 /sec

19.2.10. Example: Design a culvert consisting of cement concrete circular pipes with bevelled entry and flowing full, given: (Fig. 19.3).

Discharge	=10m^3/sec
R.L. of ground in metres	=100.00
H.F.L of tail race in metres	=100.80
Permissible heading up at inlet R.L.	=101.80
Length of culvert	=20 m

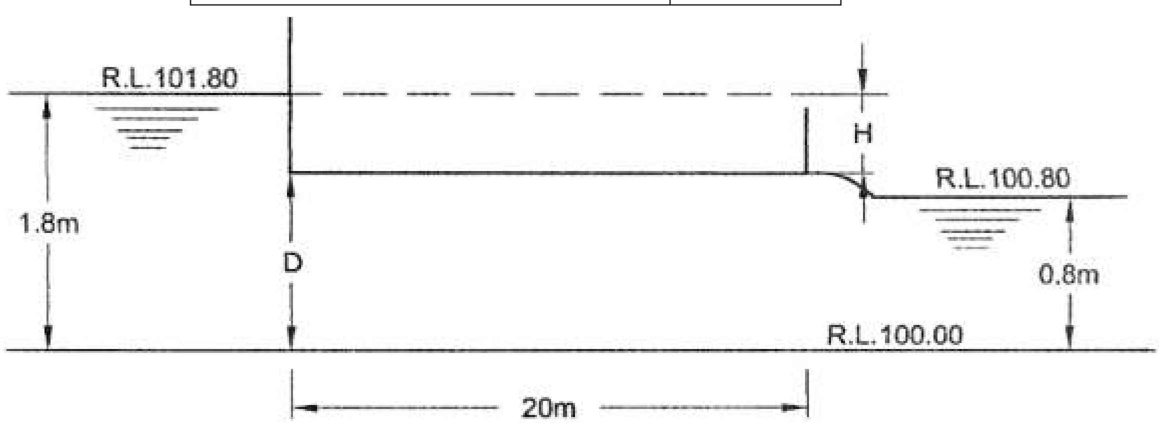


Fig. 19.3

Since we shall try pipes of diameters exceeding 0.8 m, the culvert will function as sketched:

Assumed value of D = (1) 1 m; (2) 1.5 m;

Corresponding

H= 1.8 -D = (1) 0.8 m; (2) 0.3 m;

Discharge per pipe

From **Plate 2712**, Q = (1) 2.54 m^3/s; (2) 3.5 m^3/s

Number of pipes

Require 10/Q = (1) 3.93; (2) 2.85

Say 4 Say 3

Hence, 4 pipes of 1 meter diameter will suit.

19.3. Improved Intake to Increase Culvert Capacity Culverts In dips and low height roads are often provided below ground. The portion of culvert lying under ground eventually silt up and its conveying capacity is drastically reduced. Improved intake design with proper design of inlet and outlet transitions connecting the culvert with the channel like a siphon in canal cross-drainage works will increase the conveying capacity of culvert. It is advisable to provide the inlet level of culverts at least 150 mm below lowest bed level at the section, so that water will always pass through the culvert.

19.4. "Scour in Culverts"

Where the bed is unprotected, scour depth in the abutments should be found as in the case of bridges on alluvial fine and coarse soil to find the depth of foundation. When the bed is rigid as in case of box culverts, curtain/ cut-off walls must be provided both upstream and downstream to protect against scour. Flexible stone pitching/stone gabions laid over geo-synthetic filter of length 3 to 4 times estimated scour depth should be provided downstream to arrest erosion of bed and banks where outlet velocity is high.

58

ARTICLE 20

PROTECTION WORK AND MAINTENANCE

20.1. FLOOR PROTECTION WORKS:

In case structures founded on erodible soil are protected against scour by floor protection works, the following is considered as sound practice.

20.1.1. For structures where adoption of shallow foundations becomes economical by restricting the scour, floor protection may be provided. The floor protection will comprise of rigid flooring with curtain walls and flexible apron so as to check scour, washing away or disturbance by piping action, etc. Usually performance of similar existing works is the best guide for finalizing the design of new works. However, the following minimum specification for floor protection shall be followed while designing new structures subject to the general stipulation that post protection works velocity under the structures does not exceed 2 m/s and the intensity of discharge is limited to $2m^3/m$ sec. In case it does not satisfies, design of floor protection work need to be done as per IRC 89

20.1.2. Suggested Specifications:

20.1.2.1. Excavation for laying foundation and protection works should be carried out as per specifications under proper supervision. Before laying the foundation and protection works the excavated trench should be thoroughly inspected by the Engineer-in-Charge to ensure that:

(a) There are no loose pockets, unfiled depressions left in the trench.

(b) The soil at the founding level is properly compacted to true lines and level.

(c) All concrete and other elements are laid in dry bed.

20.1.2.2. *Rigid flooring:* The rigid flooring should be provided under the bridge and it should extend for a distance of at least 3 m on upstream side and 5 m on downstream side of the bridge.

However, in case the splayed wing walls of the structure are likely to be longer, the flooring should extend upto the line connecting the end of wing walls on either side of the bridge.

The top of flooring should be kept 300 mm below the lowest bed level.

Flooring should consist of 150 mm thick flat stone/bricks on edge in cement mortar 1:3 laid over 300 mm thick cement concrete M20 grade laid over a layer of 150 mm thick cement concrete M15 grade. Joints at suitable spacings (say 20 m) may be provided.

20.1.2.3. *Curtain walls:* The rigid flooring should be enclosed by curtain walls (tied to the wing walls) with a minimum depth below floor level of 2 m on upstream side and 2.5 m on down stream side. The curtain wall should be in cement concrete M20 grade or brick/stone masonry in cement mortar 1:3. The rigid flooring should be continued over the top width of curtain walls. In this context, relevant provision in "Guidelines for design and construction of river training and control works for road bridges", **IRC: 89** is also referred.

20.1.2.4. *Flexible apron:* Flexible apron 1 m thick comprising of loose stone boulders (weighing not less than 40 kg) should be provided beyond the curtain walls for a minimum distance of 3 m on upstream side and 6 m on downstream side. Where required size stones are not economically available, cement concrete blocks or stones in wire crates may be used in place ofisolated stones. In this context, relevant provision in **IRC:89** is also referred.

59

20.1.2.5. Wherever scour is restricted by provision of flooring/flexible apron, the work of flooring/apronetc., should be simultaneously completed alongwith the work on foundations so that the foundation work completed is not endangered.

20.2. MAINTENANCE:

20.2. 1. The bridge structures are more susceptible to damages during monsoon. It is generally observed that following factors contribute mainly to damage.

(a) Choking of vents
(b) Wash outs of approaches
(c) Dislodgement of wearing course and cushion
(d) Scour on D/S (downstream)
(e) Silting on U/S (upstream)
(f) Collection of debris on approaches in cutting

20.2.2. To minimize the occurrence of above phenomena, it is necessary to take adequate steps as below:

(1) The vents should be thoroughly cleaned before every monsoon.

(2) The bridge vents should be cleared after the first monsoon flood as the flood carries maximum debris with it.

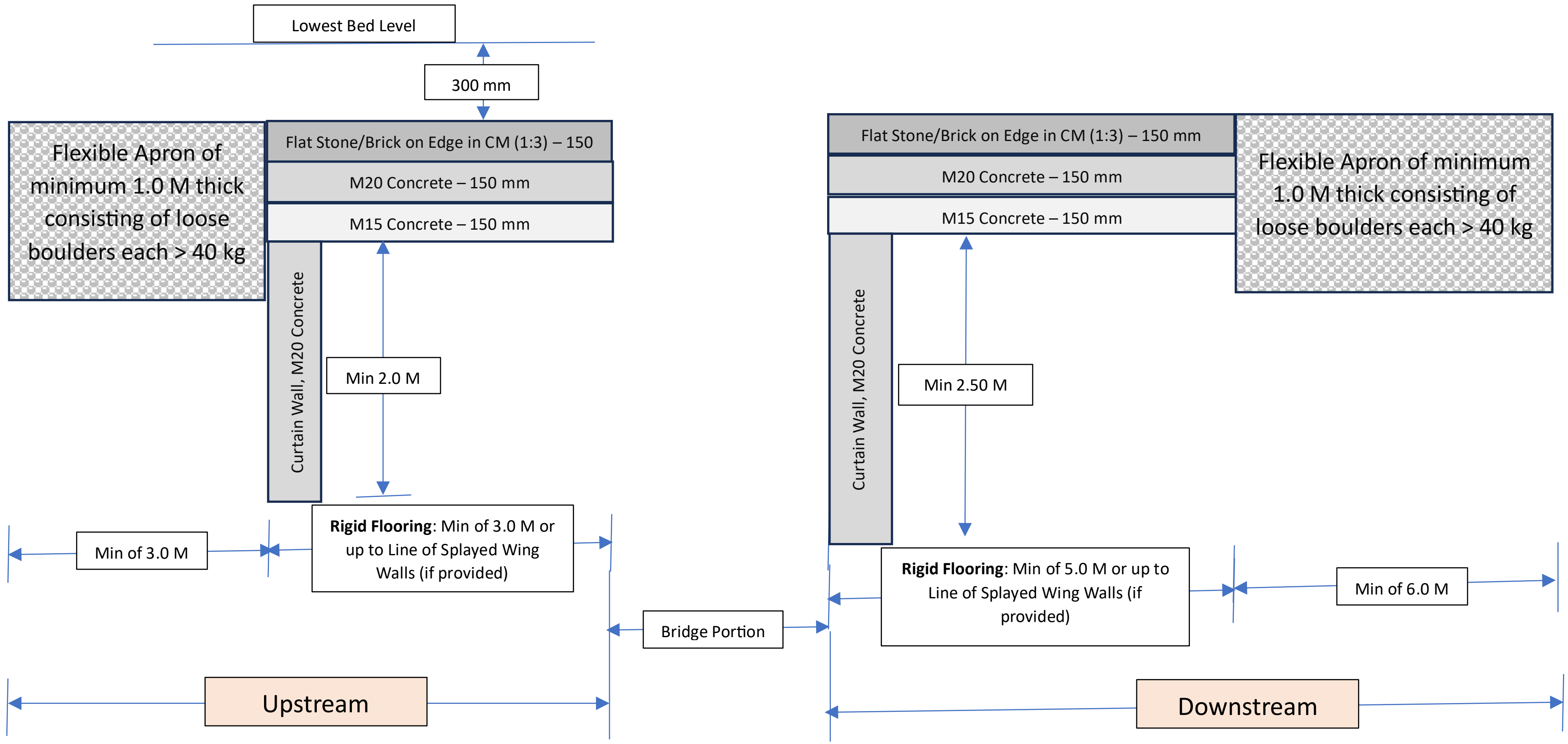
(3) Keep approaches almost matching with existing bank, i.e., cutting or embankment should be minimum to avoid wash outs of approaches.

(4) Disposal of water through side gutters shall be properly planned so that it does not damage the cross-drainage work proper.

(5) The wearing coat with cushion should be sufficiently stable and it should not get dislodged during floods.

(6) In the event of approaches being in cutting there is a tendency of whirling of water at the approaches. This leads to collection of debris in the approaches. After the floods recede, huge heap of debris is found on the approaches. This should be quickly cleared.

Typical Floor Protection Works for a Bridge as per Article 20 of IRC-SP-13 (2022)



Note:

- Curtain Walls can also be provided with Stone/Brick Masonry in C.M. (1:3)
- Curtain Walls should be tied to Wing Walls

ARTICLE 21

RAFT FOUNDATIONS

21.1. Raft foundation is preferred when the good foundable strata is not available within a reasonable depth. Thus, the sandy layer or sand and silty foundations warrant provision of raft foundation. While providing raft foundation, some important points should be kept in view.

21.1.1. Raft top should be kept 300 mm below the lowest bed level. This will ensure protection to raft and also would avoid silting tendency on U/S and scouring tendency on D/S. The raft will also not be subjected to stresses due to temperature variations.

21.1.2. U/S and D/S aprons should be provided in accordance with IRC 89 to protect the bridge from scouring or undermining. The width of U/S and D/S aprons should be 1.5 d_{sm} and 2.0 d_{sm} respectively (Fig. 21.1).

21.1.3. The depth of cut-off wall should be 30 cm below the scour level. The normal scour depth is worked out by the formula $d_{sm} = 1.34 \times \left(\frac{D_b^2}{K_{sf}}\right)$ (Refer Equation 9.1).

(Scour Depth need not be increased by any factor as in case of open foundations as stipulate din **IRC:78-2000**).

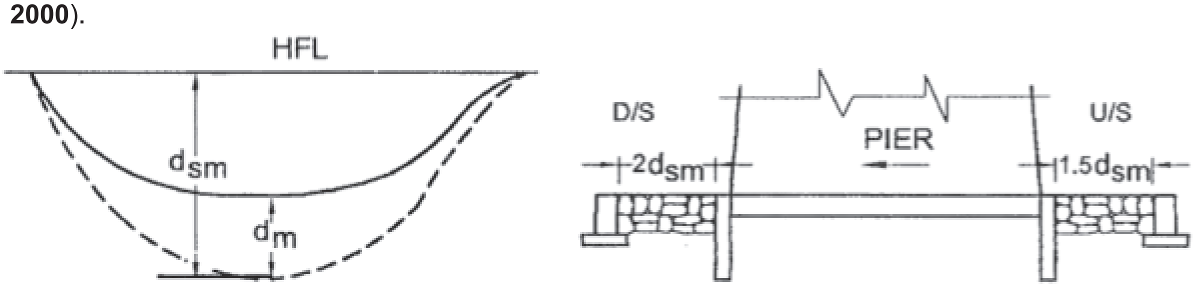
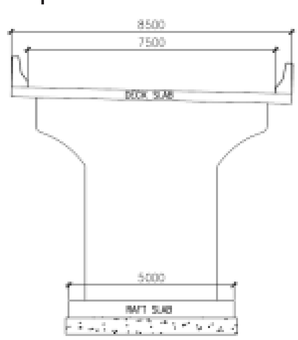


Fig. 21.1 Scour Depth and Apron Width for Raft

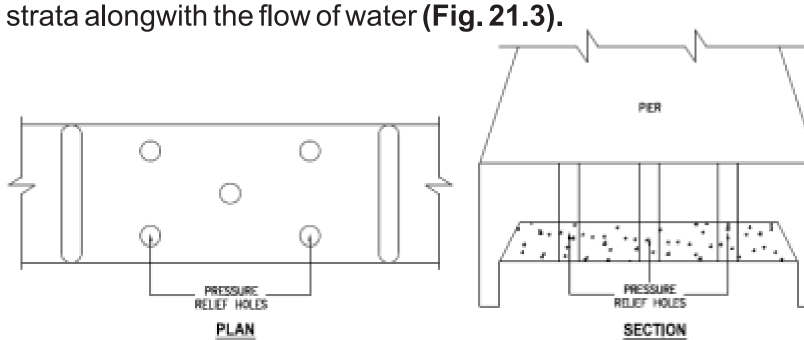
21.1.4. Longitudinal cut-off walls should be provided on U/S and D/S side and they should be connected by cross cut off walls. Longitudinal cut-off walls safeguard the bridge from scour whereas the cross-cut-off walls keep the longitudinal cut-off walls in position and also protect the bridge from scouring particularly due to out flanking.

21.1.5. The raft is generally as wide as the deck but in certain cases may be narrower than the deck **(Fig. 21.2).**

21.1.6. Pressure relief holes may be provided in the raft to relieve the raft from possible uplift pressure from below. The holes need to be carefully packed with graded filter material to prevent outflow of soil particles of the foundation strata alongwith the flow of water **(Fig. 21.3).**



RAFT SLAB NARROWER THAN DECK WIDTH



PRESSURE RELIEF HOLES IN RAFT SLAB

61

ARTICLE 22

HP CULVERTS IN BLACK COTTON SOIL

22.1. Generally, the black cotton (B.C.) soil is of expansive nature. As it comes in contact with water, the montmorillonite group cells expand. This phenomenon leads to heavy pressure on structure and the structure may develop cracks and fail. It is, therefore, necessary to safeguard the structure from the ill-effects of the damaging nature of the soil. It is desirable to cut the contact of expansive soil and the foundation structure. This can be achieved by providing a sandy media all around the foundation. Such non-expansive layer not only cuts the all around contact between soil and foundation but also absorbs energy of swelling and shrinking of foundation soil below the layer ofsand and keeps the foundation safe.

22.2. Since expansive soils have low SBC, ground improvement is required. The same can be done by introducing a granular layer of suitable thickness below the pipe which would help in load dispersion to a wider area and thus reduce the stress on the soil below the granular layer.. Such layer, improves Safe Bearing Capacity (SBC) of the strata to a considerable extent and safeguards the foundation from the adverse effects of the expansive soil also **(Fig. 22.1).**

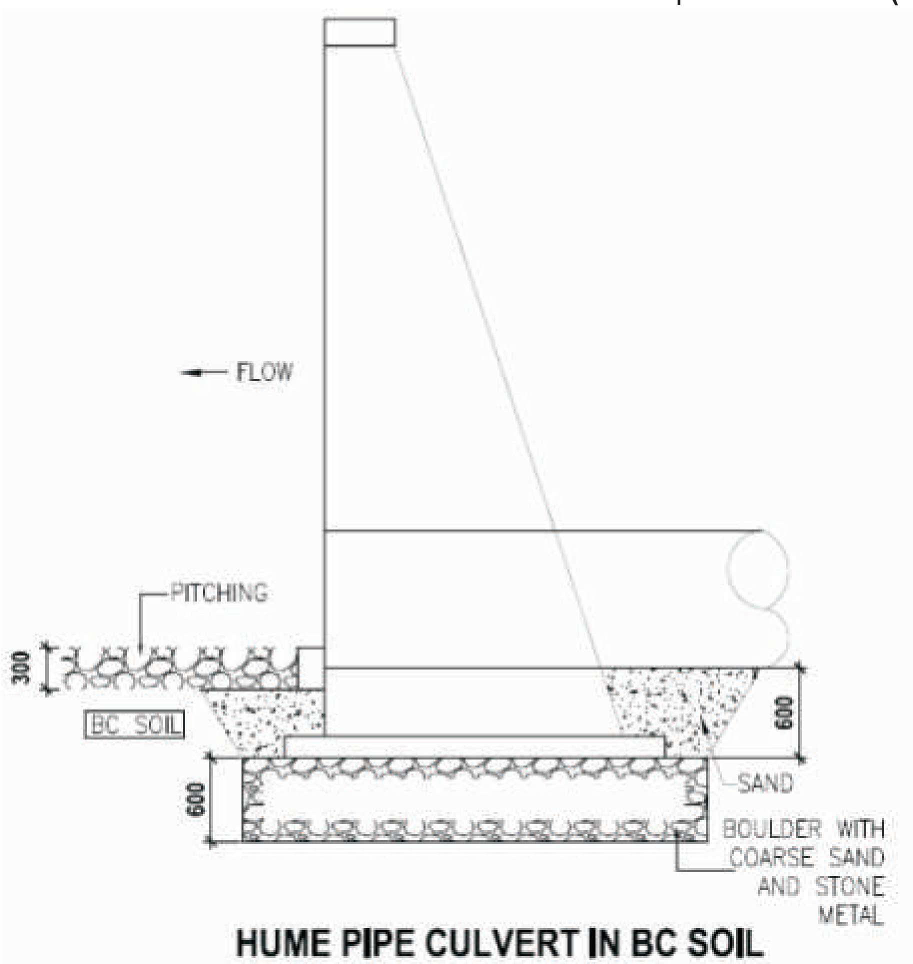


Fig. 22.1 Hume Pipe Culvert in BC Soil

ARTICLE 23

BOX CELL STRUCTURES

23.1. WHERE TO PROVIDE BOX STRUCTURES: Box structures are hydraulically efficient structures where thickness of walls and slab are small and there is least obstruction to flow. Box cell structures are most suitable for soils having SBC 8-12 T/m2.Box cell structures are also provided to keep the deck level lower. The box being a continuous structure, the thickness of top slab is less than that of a simply supported span of same length. Also, expansion joints at the deck ends are dispensed with.

When the river or Nalla has sandy bed and/or purely clayey strata, the independent foundations are likely to be deeper and this may enhance the cost of culverts and small bridges. Under these circumstances box culverts are found to be a better solution. Several such box cell structures have shown a good in service performance. Purely sandy soil or clayey strata may be at few places but mixed soils are available in several cases. Where ϕ value of mixed soil is less than 15°, it may be treated as a clayey soil. Similarly, where safe bearing capacity of soil is found to be less than 10 t/m^2, box culverts are most suitable for such type of soils.

23.2. FOUNDATION: Where there is a clayey stratum, top soil below box may be replaced by a layer of granular soil of suitable thickness like sandy murum and stone dust etc. Where there is purely clayey strata top 900 mm below box should have granular material, like, sandy murum, or stone dust or GSB.

Where there is murum and mixed soil having ϕ more than 1 5°, there is no need of providing sandy layer.

The box cell structures are of concrete of M25 grade for moderate and M30 grade for severe conditions of exposure with HYSD steel bars. Shall be designed as per IRC 112.

Box cell structures are to be provided with curtain walls and apron and these must be completed before floods. The best practice is to lay foundations of curtain wall and apron first and then lay box.

Appendix - A

FILLING BEHIND ABUTMENTS, WING AND RETURN WALLS

1. FILLING MATERIALS

The type of materials to be used for filling behind abutments and other earth retaining structures, should be selected with care. A general guide to the selection of soils is given in Table 1.

TABLE 1. GENERAL GUIDE TO THE SELECTION OF SOILS ON BASIS OF ANTICIPATED EMBANKMENT PERFORMANCE

Soil group according to IS:1498-1970		Visual description	Max. dry density range (kg/m^3)	Optimum moisture content range (per cent)	Anticipated embankment performance
Most probable	Possible				
GW. GP, GM, SW, HP		Granular materials	1850-2280	7-15	Good to Excellent
SB, SM, GM, GC, SM, SC		Granular materials with soil	1760-2160	9-18	Fair to Excellent
SP		Sand	1760-1850	19-25	Fair to Good
ML, MH, DL	CL, SM, SB, SC	Sandy Silts & Silts	1760-2080	10-20	Fair to Good

2. LAYING AND COMPACTION

2.1. Laying of Filter Media for Drainage

The filter materials should be well packed to a thickness of not less than 600 mm with smaller size towards the soil and bigger size towards the wall and provided over the entire surface behind abutment, wings or return walls to the full height.

Filter materials need not be provided in case the abutment is of spill through type.

2.2. Density of Compaction

Densities to be aimed at in compaction should be chosen with due regard to factors, such as, the soil type, height of embankment, drainage conditions, position of the individual layers and type of plant available for compaction.

Each compacted layer should be tested in the field for density and accepted before the operations for next layer are begun.

3. EXTENT OF BACKFILL

The extent of backfill to be provided behind the abutment should be as illustrated in Fig. 1.

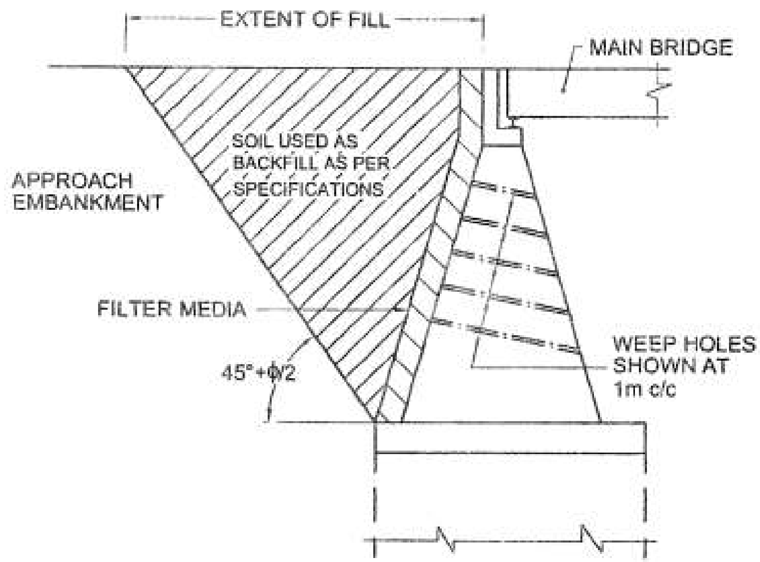


Fig. 1

4. PRECAUTIONS TO BE TAKEN DURING CONSTRUCTION

4.1. The sequence of filling behind abutments, wing walls and return walls should be so controlled that the assumptions made in the design are fulfilled and they should clearly be indicated in the relevant drawings. For example, if the earth pressure in front of the abutment is assumed in the design, the front filling should also be done simultaneously alongwith the filling behind abutment, layer by, and in case the filling behind abutment before placing the superstructure is considered not desirable, the filling behind abutment should also be deferred to a later date. In case of tie beams and friction slabs, special care should be taken in compacting the layer underneath and above them so that no damage is done to them by mechanical equipment.

4.2. Special precautions should be taken to prevent any wedging action against structures, and the slopes bounding the excavation for the structure should be stepped or strutted to prevent such wedging action.

4.3. Adequate number of weep holes not exceeding one metre spacing in both directions should be provided to prevent any accumulation of water and building up of hydrostatic pressure behind the walls. The weep holes should be provided above the low water level.

PLATE-1

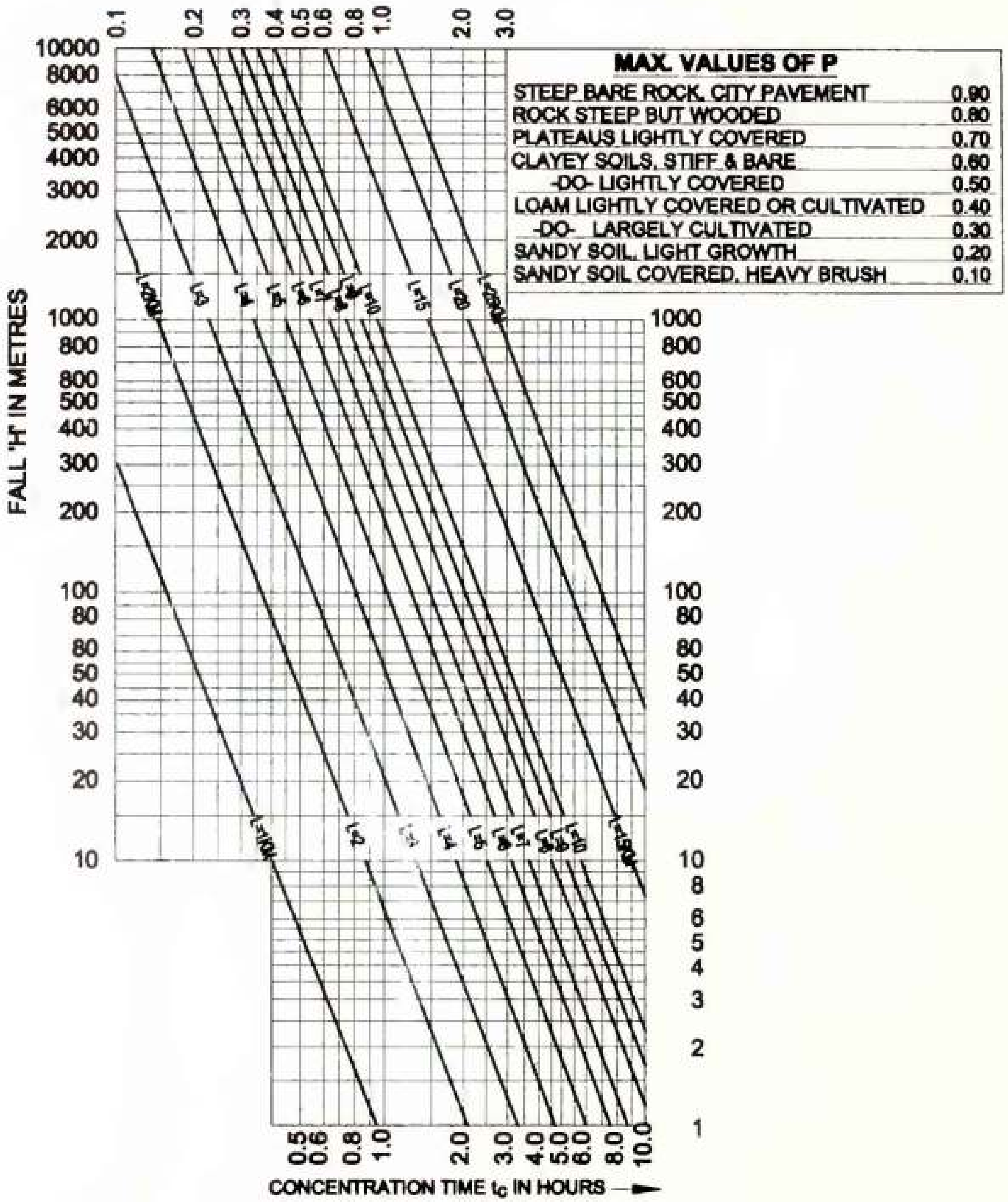
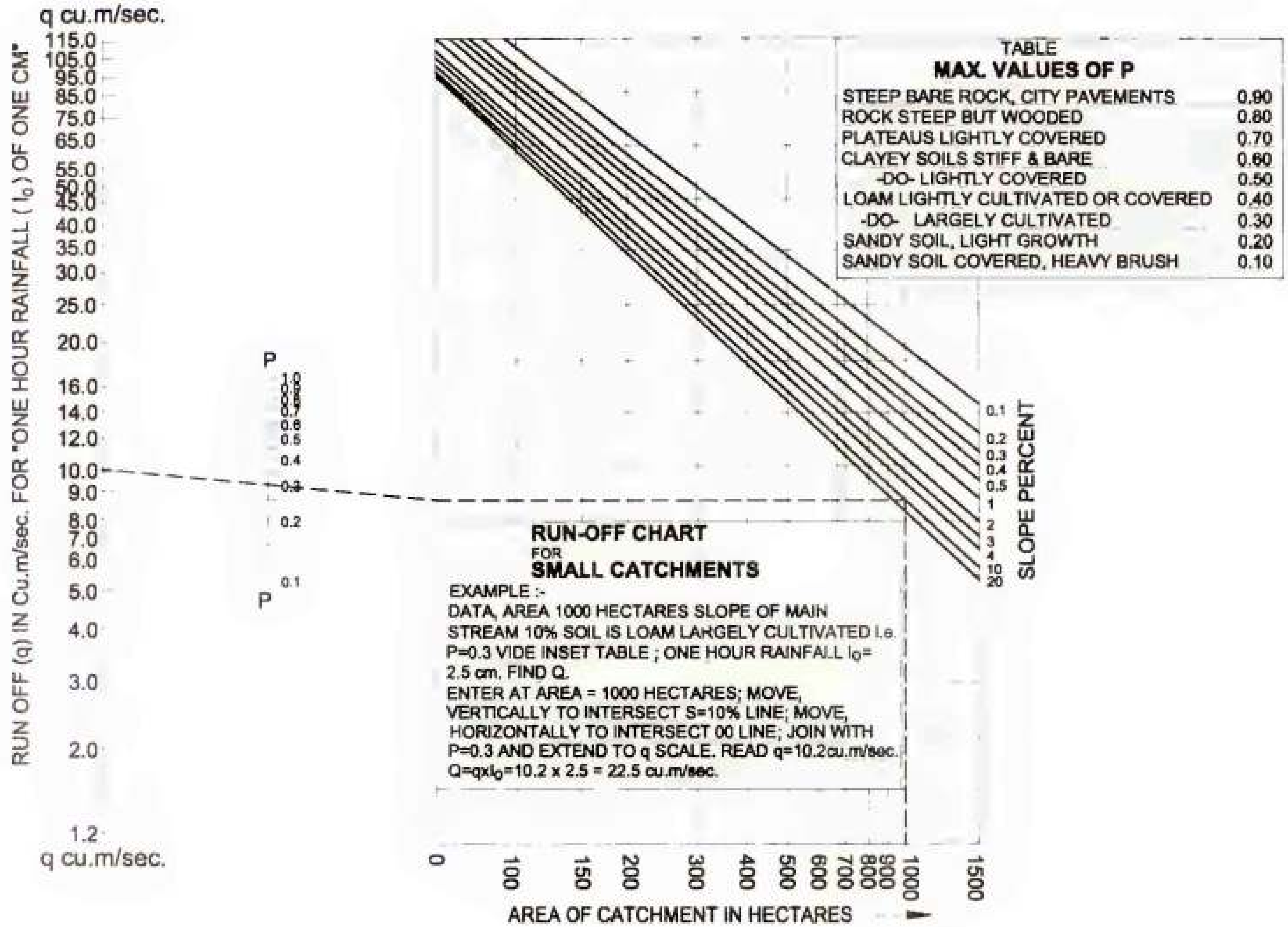


Chart for Time of Concentration

PLATE-2



Run-off Chart for Small Catchments

PLATE-3

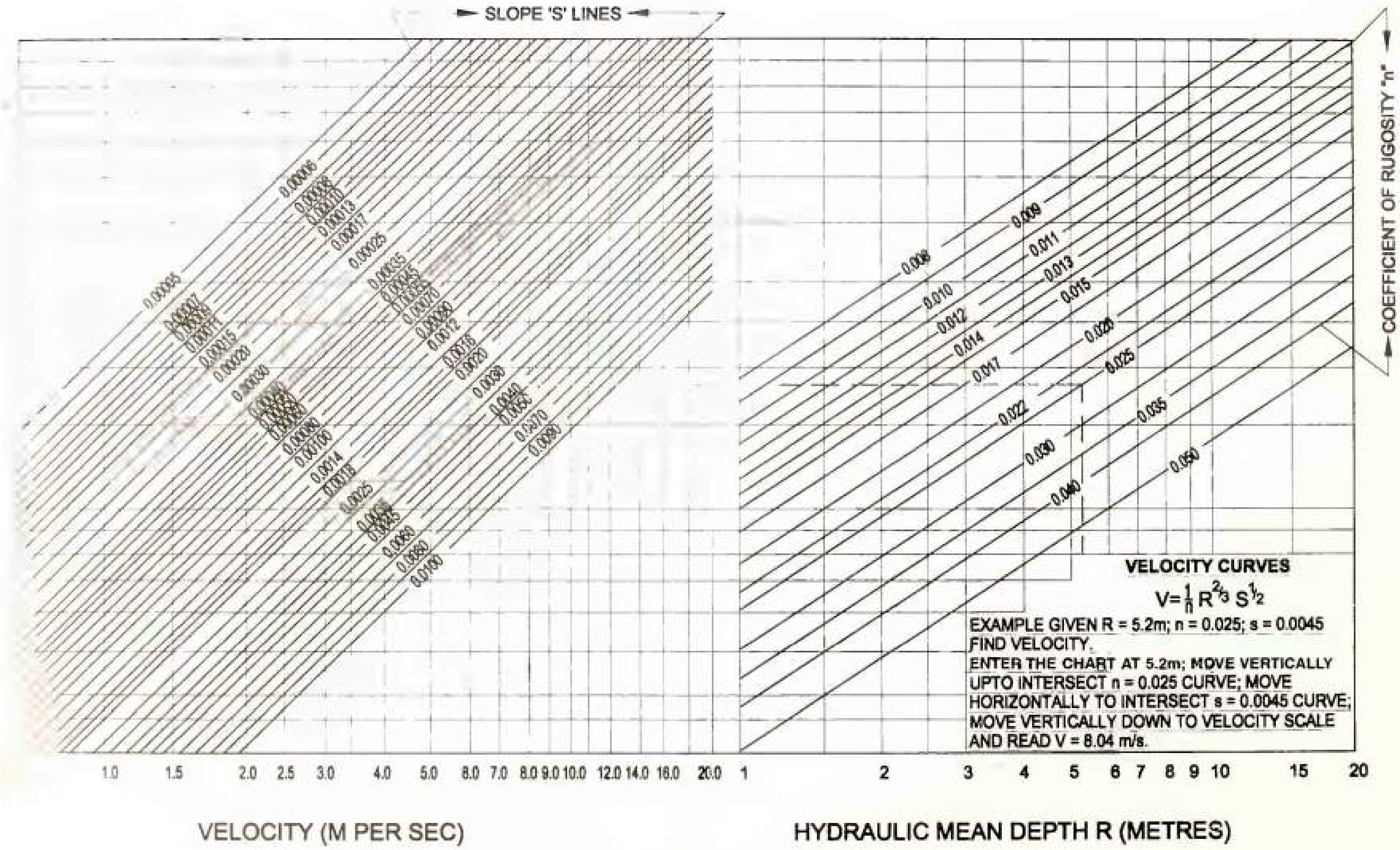

SLOPE 'S' LINES
COEFFICIENT OF RUGOSITY "n"
0.008
0.009
0.010
0.011
0.012
0.013
0.014
0.015
0.017
0.020
0.022
0.025
0.030
0.035
0.040
0.050
VELOCITY CURVES
$V=\frac{1}{n}R^{2/3}S^{1/2}$
EXAMPLE GIVEN R = 5.2m; n = 0.025; s = 0.0045
FIND VELOCITY.
ENTER THE CHART AT 5.2m; MOVE VERTICALLY
UPTO INTERSECT n = 0.025 CURVE; MOVE
HORIZONTALLY TO INTERSECT s = 0.0045 CURVE;
MOVE VERTICALLY DOWN TO VELOCITY SCALE
AND READ V = 8.04 m/s.
1.0 1.5 2.0 2.5 3.0 4.0 5.0 8.0 7.0 8.0 9.0 10.0 12.0 14.0 16.0 20.0
1 2 3 4 5 6 7 8 9 10 15 20
VELOCITY (M PER SEC)
HYDRAULIC MEAN DEPTH R (METRES)

PLATE-4

Representative disturbed samples of bed materials shall be taken at every change of strata upto the maximum anticipated scour depth. The sampling should start from 300 mm below the existing bed. About 500 gms of each of the representative samples so collected shall be sieved by a set of standard sieves and the weight of soil retained in each sieve is taken. The results thereof are then tabulated. A typical test result is shown below (Table A & B).

TABLE A

Sieve Designation	Sieve Opening (mm)	Weight of Soil retained (gm)	Per cent retained
5.60 mm	5.60	0	0
4.00 mm	4.00	0	0
2.80 mm	2.80	16.90	4.03
1.00 mm	1.00	76.50	18.24
425 micron	0.425	79.20	18.88
180 micron	0.180	150.40	35.86
75 micron	0.75	41.00	9.78
Pan	–	55.40	13.21
Total		419.40	100.00

TABLE B

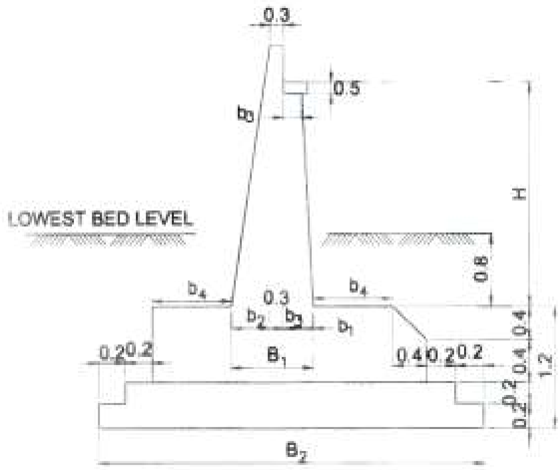
Sieve No.	Average size (mm)	Percentage of weight retained	Column (2) x column (3)
(1)	(2)	(3)	(4)
4.00 to 2.80 mm	3.40	4.03	13.70
2.80 to 1.00 mm	1.90	18.24	34.66
1.00 to 425 micron	0.712	18.88	13.44
425 to 180 micron	0.302	35.86	10.83
180 to 75 micron	0.127	9.78	1.24
75 micron and below	0.0375	13.21	0.495
			74.365

Weighted mean diameter $d_m = \frac{74.365}{100}$

$= 0.74365$ Say 0.74

TYPICAL METHOD OF DETERMINATION OF WEIGHTED MEAN DIAMETER OF PARTICLES (d_m)

PLATE-5



SECTION OF ABUTMENT

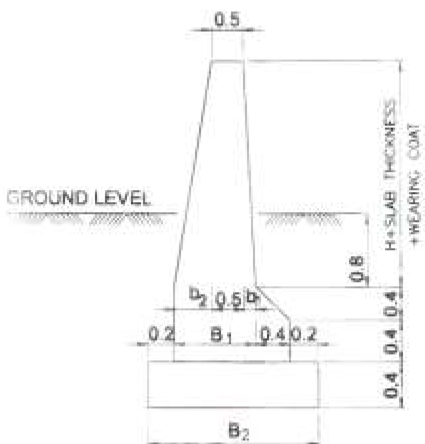
TABLE OF DIMENSIONS FOR ABUTMENT

Effective Span	5m and 6m					3m, 2m, 1.5m and 1m						3.5m and 4m					
H	2.0m	2.5m	3.0m	3.5m	4.0m	1.5m	2.0m	2.5m	3.0m	3.5m	4.0m	1.5m	2.0m	2.5m	3.0m	3.5m	4.0m
b_1	0.2	0.25	0.3	0.35	0.4	0.15	0.2	0.25	0.3	0.35	0.4	0.15	0.2	0.25	0.3	0.35	0.4
b_2	0.6	0.85	1.0	1.2	1.4	0.5	0.7	0.95	1.1	1.25	1.4	0.5	0.7	0.95	1.1	1.25	1.4
b_3	1.0	1.0	1.0	1.0	1.0	0.5	0.5	0.5	0.5	0.5	0.5	1.0	1.0	1.0	1.0	1.0	1.0
b_4	–	0.1	0.2	0.4	0.5	–	–	0.1	[illegible]	[illegible]	0.5	–	–	0.1	0.2	0.3	0.5
B_1	2.1	2.4	2.8	2.85	3.1	1.45	1.7	2.0	2.20	2.4	2.8	1.95	2.2	2.5	2.7	2.9	3.1
B_2	3.3	3.8	4.2	4.65	5.3	2.65	2.9	3.4	3.8	4.2	4.8	3.15	3.4	3.9	4.3	4.7	5.3

TABLE OF DIMENSIONS FOR WING WALL (HIGH END)

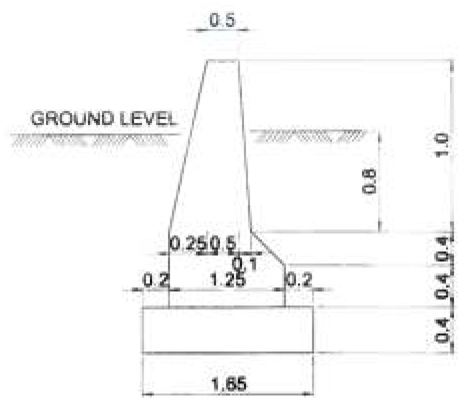
SPAN	UPTO 2 METRES						3 METRES					
H	1.50	2.00	2.50	3.00	3.50	4.00	1.50	2.00	2.50	3.00	3.50	4.00
b_1	0.18	0.23	0.28	0.33	0.38	0.43	0.19	0.24	0.29	0.34	0.39	0.44
b_2	0.45	0.57	0.70	0.82	0.95	1.07	0.46	0.59	0.71	0.84	0.96	1.09
B_1	1.13	1.30	1.48	1.65	1.83	2.00	1.15	1.33	1.50	1.68	1.85	2.03
B_2	1.93	2.10	2.28	2.45	2.65	2.80	1.95	2.13	2.30	2.48	2.65	2.83

SPAN	4 METRES						5 METRES					6 METRES				
H	1.50	2.00	2.50	3.00	3.50	4.00	2.00	2.50	3.00	3.50	4.00	2.00	2.50	3.00	3.50	4.00
b_1	0.19	0.24	0.29	0.34	0.39	0.44	0.25	0.30	0.35	0.40	0.45	0.25	0.30	0.35	0.40	0.45
b_2	0.48	0.60	0.73	0.85	0.98	1.10	0.62	0.75	0.87	1.00	1.13	0.63	0.75	0.88	1.00	1.13
B_1	1.17	1.34	1.52	1.69	1.87	2.04	1.37	1.55	1.72	1.90	2.08	1.38	1.55	1.73	1.90	2.08
B_2	1.97	2.14	2.32	2.49	2.67	2.84	2.17	2.35	2.52	2.70	2.88	2.18	2.35	2.53	2.70	2.88



WING WALL SECTION

AT HIGH END



WING WALL SECTION

AT LOW END

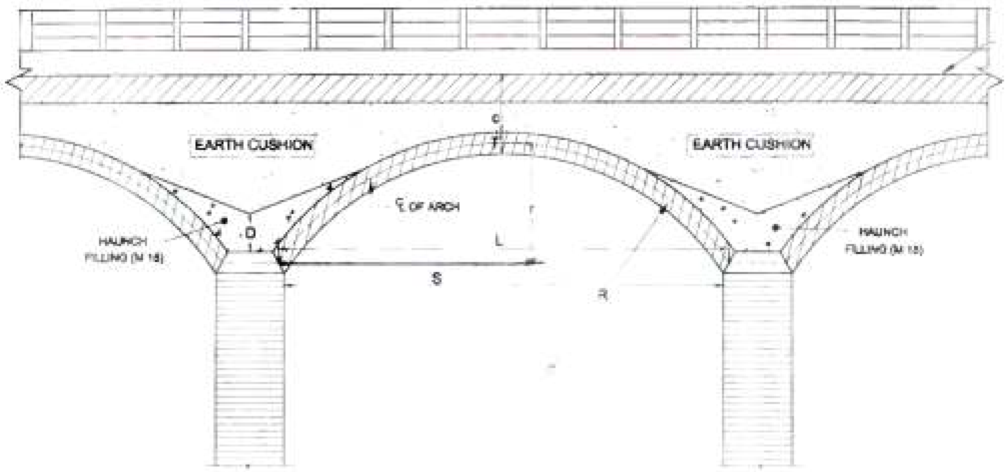
(FOR ALL SPANS)

NOTES :-

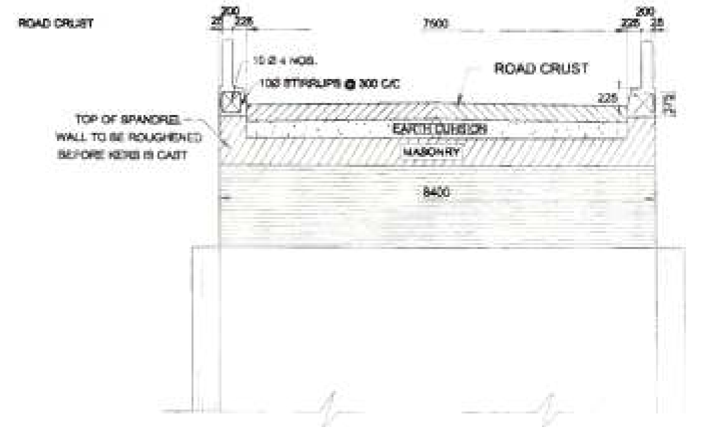
1. ABUTMENT AND WING WALL SECTIONS ARE APPLICABLE FOR A MINIMUM BEARING CAPACITY OF THE SOIL OF 16.5 t/m^2. FOR SOIL HAVING LOWER BEARING CAPACITY THE SECTIONS SHOULD BE INCREASED SUITABLY.
2. ABUTMENT AND WING WALL SECTION FOR INTERMEDIATE HEIGHTS TO BE ADOPTED SUITABLY.
3. THE VARIOUS DIMENSIONS TO BE SUITABLY ADJUSTED TO SUIT THE SIZE OF BRICKS WHERE NECESSARY.
4. THE SECTIONS ARE APPLICABLE FOR CULVERTS DESIGNED FOR IRC CLASS 70R OR 2 LANES OF CLASS A LOADING, WHICHEVER IS MORE SEVERE, WITHOUT PROVISION OF APPROACH SLABS.
5. THESE SECTIONS ARE NOT APPLICABLE TO SEISMIC ZONE IV AND V.
6. THE SECTIONS SHALL BE IN CEMENT CONCRETE M 15, BRICK MASONRY IN CEMENT MORTAR 1:3 OR COURSED RUBBLE MASONRY (IInd SORT) IN CEMENT MORTAR 1:3. THE FOUNDATION CONCRETE SHALL BE IN CEMENT CONCRETE M 15.

ABUTMENT AND WING WALL SECTIONS FOR CULVERTS

PLATE-6



SECTIONAL ELEVATION



CROSS SECTION AT THE CROWN OF ARCH

TABLE

EFFECTIVE SPAN (L) METRES	6	9
CLEAR SPAN (S) METRES	5.572	8.512
RISE (r) MILLIMETRES	1500	2250
RADIUS OF CENTRE LINE(R) (MILLIMETRES)	3750	5625
CUSHION ABOVE CROWN (C) (MILLIMETRES)	610	760
ARCH THICKNESS (T) (MILLIMETRES) (UNIFORM SECTION FROM SPRINGING TO CROWN)	535	610
DEPTH OF HAUNCH FILLING AT PIER & ABUTMENT $D=\frac{r+T}{2}$ (MILLIMETRES)	1018	1430

GENERAL NOTES :-

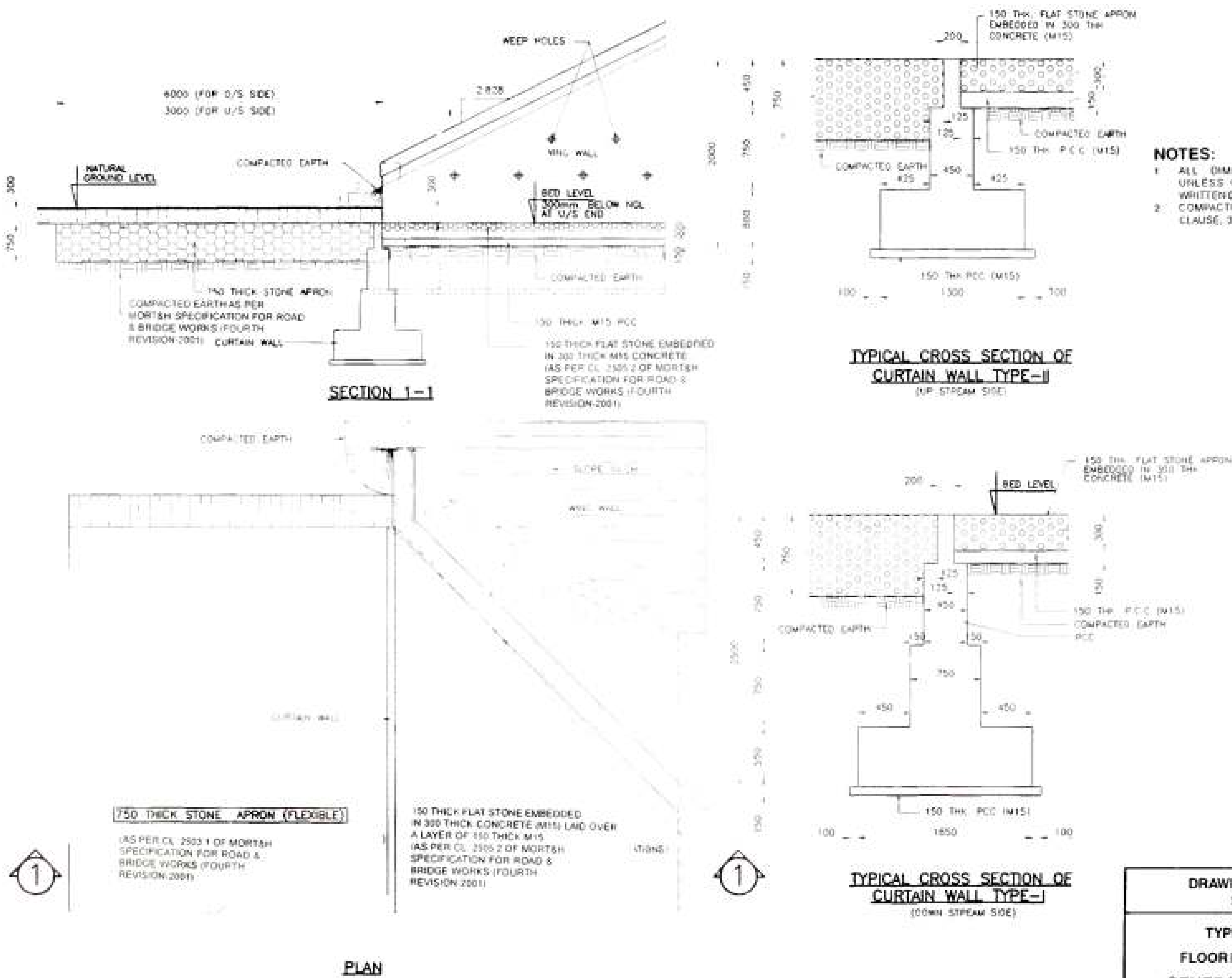
1. SPECIFICATIONS :- I.R.C. STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES SECTION I, II AND IV.
2. DESIGN LIVE LOAD :- I.R.C. CLASS A LOADING TWO LANES OR CLASS '70-R' LOADING ONE LANE.
3. MATERIAL :- THE MASONRY OF THE ARCH RING MAY CONSIST OF EITHER CONCRETE BLOCKS (M 15) OR DRESSED STONES OR BRICKS IN (1:3) CEMENT MORTAR. THE CRUSHING STRENGTH OF STONE OR BRICK UNITS SHALL NOT BE LESS THAN 10.5 MPa. WHERE STONE MASONRY IS ADOPTED FOR THE ARCH RING, IT SHALL BE EITHER COURSED RUBBLE MASONRY OR ASHLAR MASONRY.
4. DESIGN STRESSES :- (MASONRY OF ARCH RING) PERMISSIBLE TENSILE STRESS, PERMISSIBLE COMPRESSIVE STRESS — AS SPECIFIED IN I.R.C. BRIDGE CODE SECTION IV (2002).
5. RAILINGS :- AS PER DETAILS APPROVED.

NOTES:

1. THIS DRAWING IS NOT APPLICABLE TO BRIDGES LOCATED IN SEISMIC ZONES IV AND V.
2. THE RATIO OF RISE TO SPAN OF THE CENTRAL LINE OF ARCH RING SHALL BE 1/4.
3. SPECIFICATION FOR ROAD CRUST OVER THE ARCH BRIDGESS MAY BE SAME AS THAT ADOPTED FOR THE ADJACENT STRETCHES OF ROAD.
4. THE DIMENSIONS AND THE DIAMETRES OF REINFORCEMENT BARS ARE INDICATED IN MILLIMETRES EXCEPT WHERE SHOWN OTHERWISE.
5. FIGURED DIMENSIONS SHALL BE TAKEN INSTEAD OF SCALED DIMENSIONS.

DETAILS OF SEGMENTAL MASONRY ARCH BRIDGES WITHOUT FOOTPATHS EFFECTIVE SPAN 6m & 9m

PLATE 7

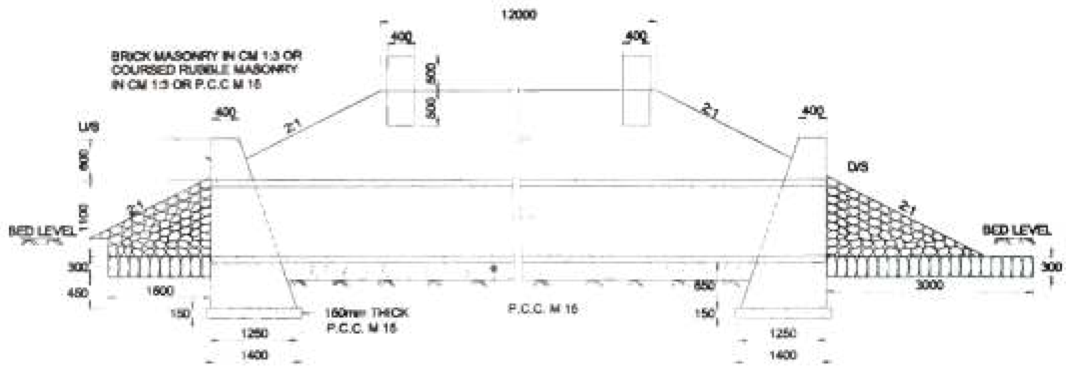


NOTES:

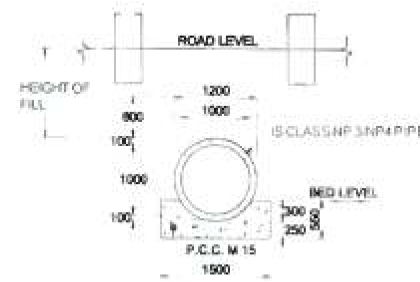
1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONED. ONLY WRITTEN DIMENSIONS ARE TO BE FOLLOWED.
2. COMPACTED EARTH SHOULD CONFORM TO CLAUSE, 305.2.1.5 OF MOST SPECIFICATIONS

DRAWINGS FOR BOX CELL STRUCTURES
TYPICAL DETAILS OF FLOOR PROTECTION WORKS **GENERAL ARRANGEMENT**

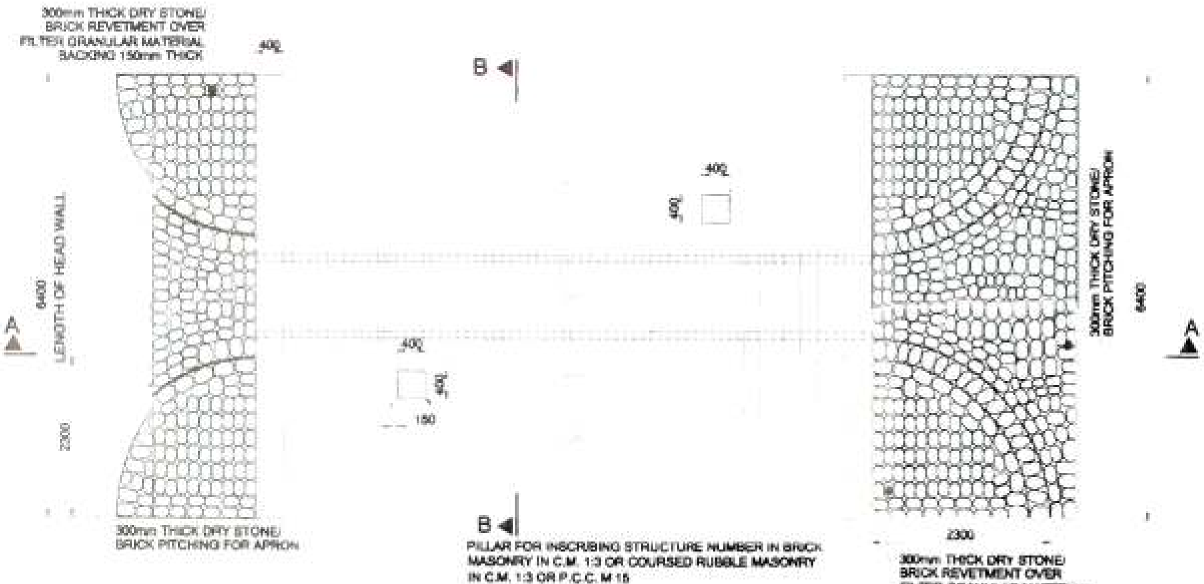
PLATE-8



SECTION AT 'AA'



SECTION AT 'BB'



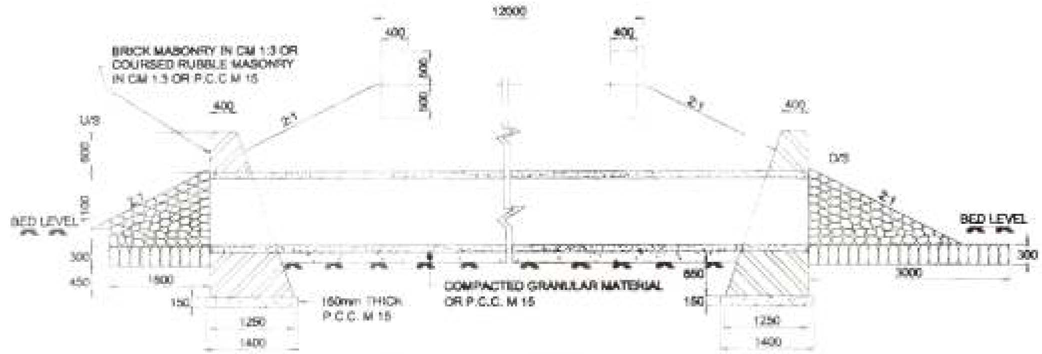
PLAN

NOTES :-

1. LONGITUDINAL SLOPE OF PIPE SHOULD BE MINIMUM 1 IN 1000
2. ALL DIMENSIONS IN MILLIMETRES EXCEPT WHERE OTHERWISE MENTIONED.
3. CONCRETE CRADLE BEDDING CAN BE USED FOR MAXIMUM HEIGHT OF FILL OF 8 METRES.

R.C.C. PIPE CULVERT WITH SINGLE PIPE OF 1 METRE DIA AND CONCRETE CRADLE BEDDING FOR HEIGHTS OF FILL VARYING FROM 4.0 m - 8.0 m

PLATE-9



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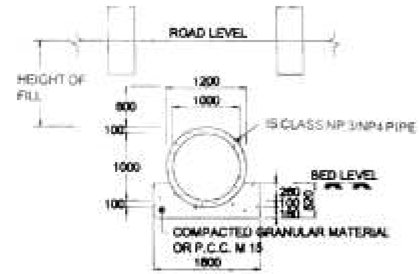
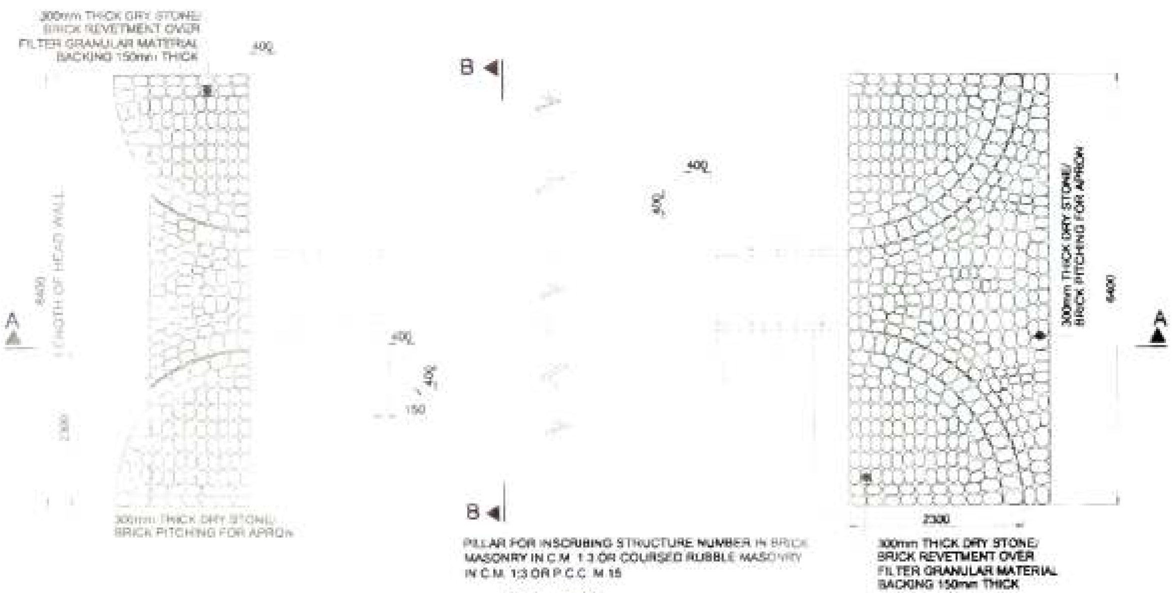


SECTION AT 'BB'

NOTES :-

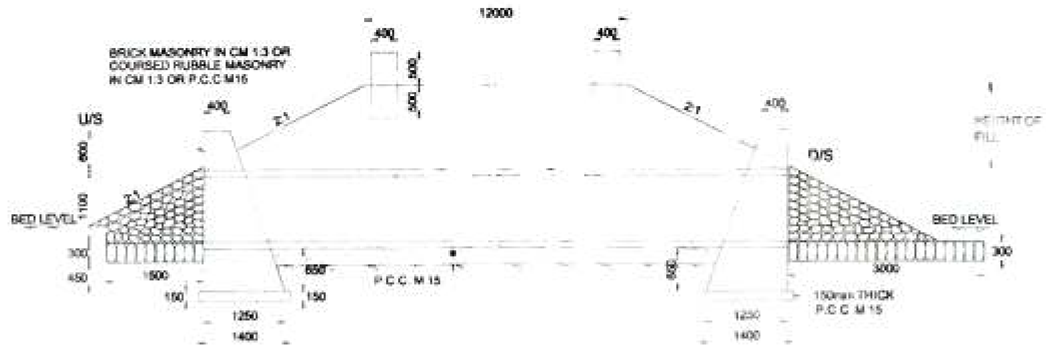
1. LONGITUDINAL SLOPE OF PIPE SHOULD BE MINIMUM 1 IN 1000
2. ALL DIMENSIONS IN MILLIMETRES EXCEPT WHERE OTHERWISE MENTIONED.
3. FIRST CLASS BEDDING CAN BE USED FOR MAXIMUM HEIGHT OF FILL OF 4 METRES.



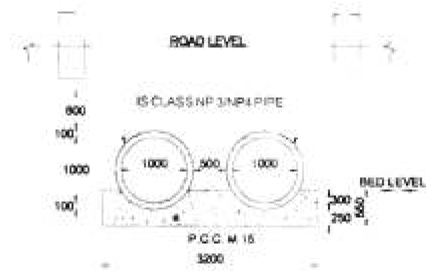
PLAN

R.C.C. PIPE CULVERT WITH SINGLE PIPE OF 1 METRE DIA AND FIRST CLASS BEDDING FOR HEIGHTS OF FILL VARYING FROM 0.6 m-4.0 m

PLATE-10



SECTION AT 'AA'



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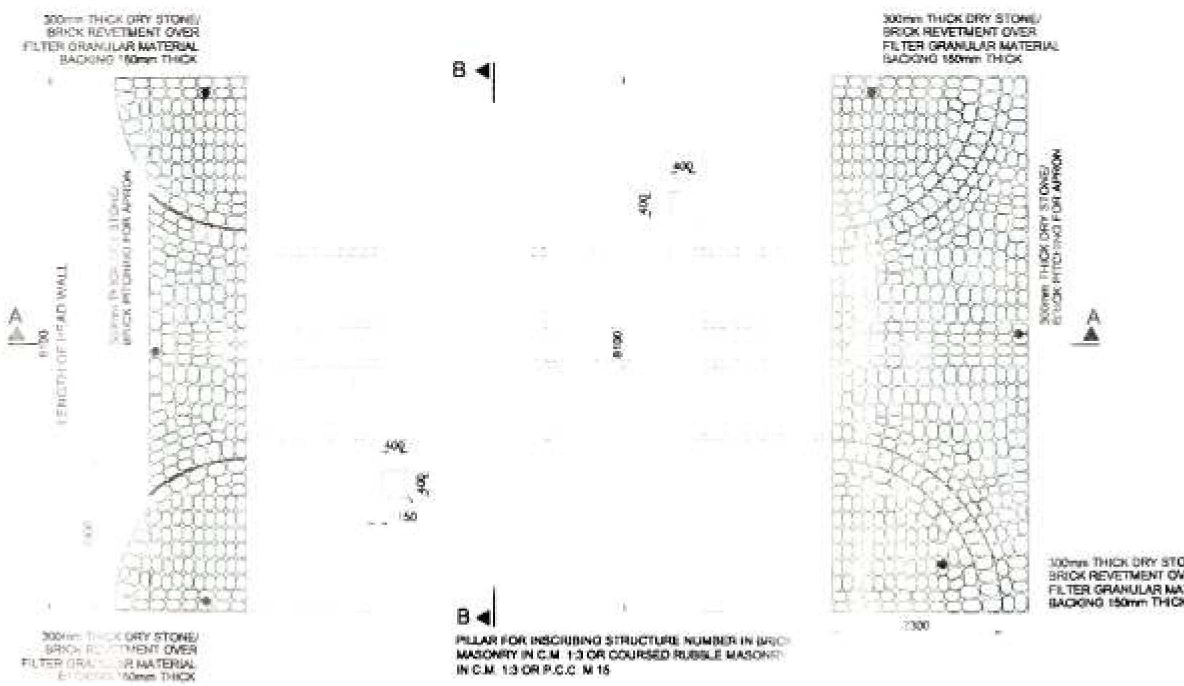


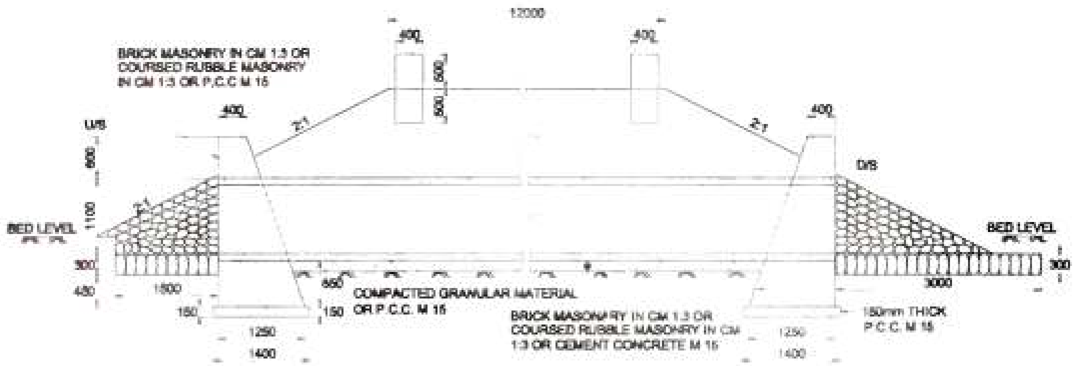
PLAN

NOTES :-

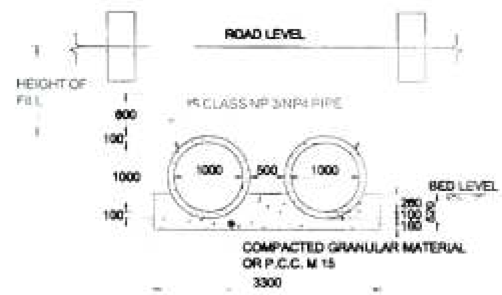
1. LONGITUDINAL SLOPE OF PIPE SHOULD BE MINIMUM 1 IN 1000
2. ALL DIMENSIONS IN MILLIMETRES EXCEPT WHERE OTHERWISE MENTIONED.
3. CONCRETE CRADLE BEDDING CAN BE USED FOR MAXIMUM HEIGHT OF FILL OF 8 METRES.

R.C.C. PIPE CULVERT WITH 2 PIPES OF 1 METRE DIA AND CONCRETE CRADLE BEDDING FOR HEIGHTS OF FILL VARYING FROM 4.0 m - 8.0 m

PLATE-11



SECTION AT 'AA'



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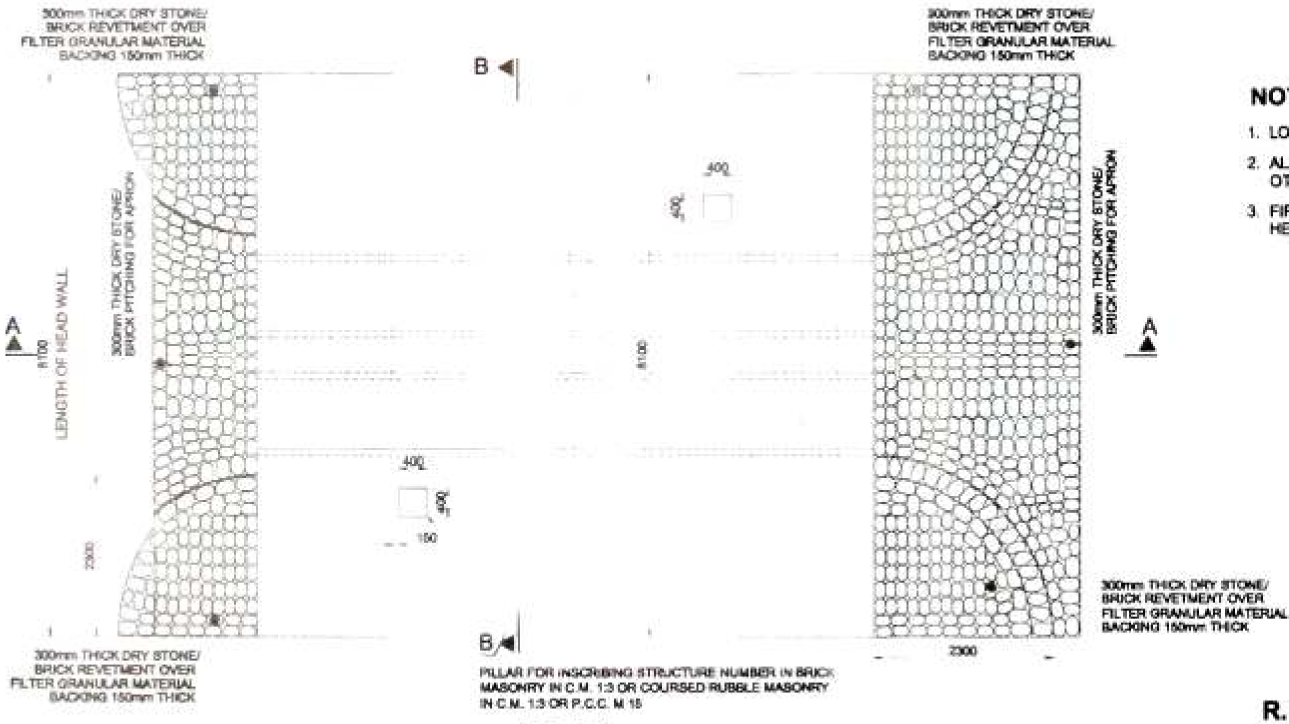


PLAN

NOTES :-

1. LONGITUDINAL SLOPE OF PIPE SHOULD BE MINIMUM 1 IN 1000
2. ALL DIMENSIONS ARE IN MILLIMETRES EXCEPT WHERE OTHERWISE MENTIONED.
3. FIRST CLASS BEDDING CAN BE USED FOR MAXIMUM HEIGHT OF FILL OF 4 METRES.

R.C.C. PIPE CULVERT WITH 2 PIPES OF 1 METRE DIA AND FIRST CLASS BEDDING FOR HEIGHTS OF FILL VARYING FROM 0.6 m-4.0 m

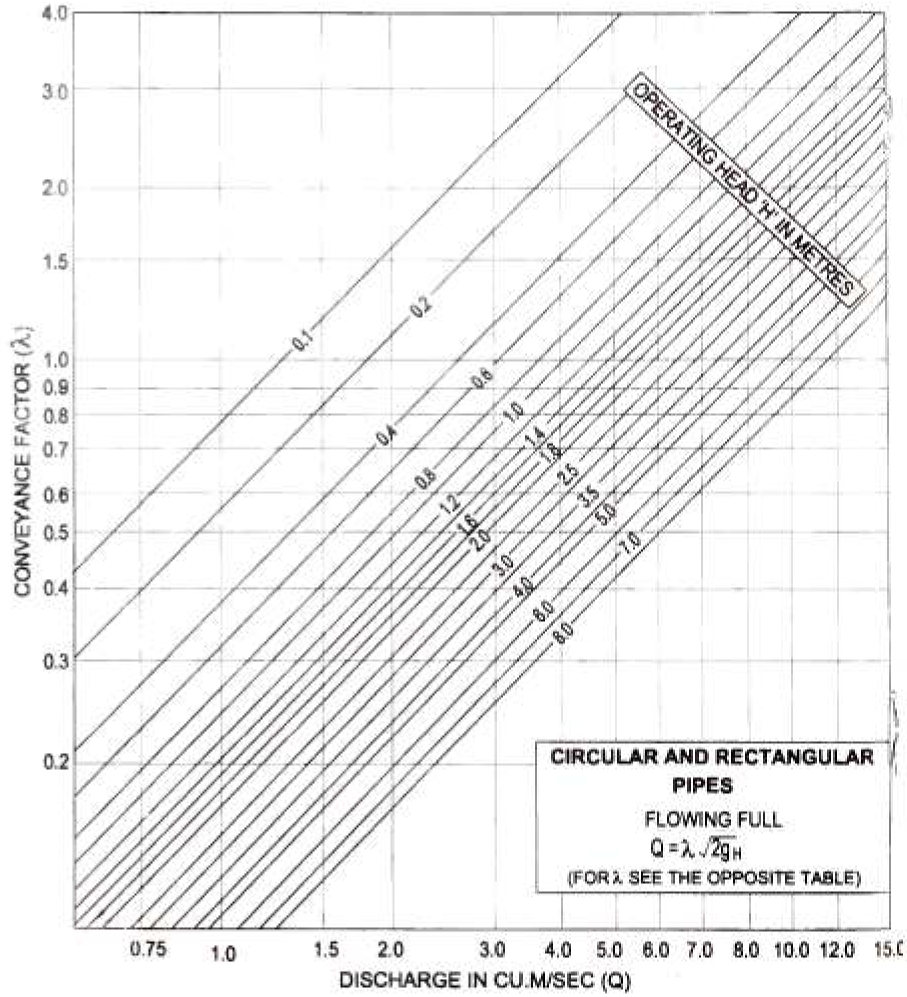
PLATE-12

CIRCULAR CULVERTS CONVEYANCE FACTOR λ IN THE FORMULA $Q = \lambda\sqrt{2gH}$

	Length "M" → / ↓ Diameter "M"	5	10	15	20	25	30	35	40	45	50	55	60
Entry Round Edged	0.75	0.394	0.373	0.356	0.339	0.326	0.313	0.302	0.292	0.283	0.225	0.267	0.26
	1.0	0.714	0.688	0.66	0.638	0.618	0.597	0.582	0.565	0.55	0.536	0.522	0.512
	1.5	1.637	1.60	1.56	1.52	1.485	1.45	1.42	1.395	1.365	1.34	1.315	1.295
	2.0	2.93	2.85	2.83	2.77	2.72	2.68	2.64	2.59	2.56	2.52	2.48	2.45
Entry Sharp Edged	0.75	0.381	0.333	0.319	0.308	0.297	0.288	0.279	0.271	0.263	0.257	0.251	0.245
	1.0	0.611	0.585	0.572	0.56	0.545	0.532	0.526	0.507	0.497	0.487	0.48	0.47
	1.5	1.34	1.316	1.295	1.275	1.255	1.235	1.216	1.195	1.175	1.165	1.145	1.135
	2.0	2.33	2.3	2.27	2.24	2.22	2.19	2.17	2.142	2.12	2.1	2.08	2.06

RECTANGULAR CULVERTS CONVEYANCE FACTOR λ IN THE FORMULA $Q = \lambda\sqrt{2gH}$

	Length "M" → / ↓ Ventway "M"	5	10	15	20	25	30	35	40	45	50	55	60
Entry Round Edged	0.75 x 0.75	0.516	0.488	0.466	0.445	0.427	0.412	0.397	0.384	0.373	0.362	0.352	0.393
	1.0 x 0.75	0.693	0.68	0.632	0.607	0.584	0.562	0.545	0.528	0.513	0.497	0.485	0.473
	1.0 x 1.0	0.935	0.90	0.867	3.837	0.81	0.788	0.765	0.745	0.727	0.71	0.693	0.877
	1.25 x 1.0	1.175	1.136	1.1	1.068	1.037	1.01	0.985	0.96	0.937	0.917	0.898	0.88
	1.25 x 1.25	1.47	1.43	1.385	1.35	1.316	1.285	1.252	1.226	1.2	1.175	1.15	1.13
	1.5 x 1.25	1.78	1.73	1.68	1.64	1.6	1.568	1.532	1.5	1.47	1.44	1.415	1.39
	1.5 x 1.5	2.14	2.08	2.03	1.99	1.95	1.91	1.87	1.836	1.8	1.765	1.74	1.71
Entry Sharp Edged	0.75 x 0.75	0.48	0.442	0.426	0.41	0.398	0.383	0.371	0.361	0.35	0.343	0.334	0.326
	1.0 x 0.75	0.615	0.593	0.572	0.563	0.536	0.52	0.505	0.492	0.48	0.468	0.457	0.447
	1.0 x 1.0	0.824	0.797	0.776	0.756	0.736	0.717	0.7	0.685	0.67	0.656	0.642	0.632
	1.25 x 1.0	1.03	1.006	0.98	0.955	0.933	0.914	0.896	0.877	0.86	0.844	0.828	0.815
	1.25 x 1.25	1.285	1.256	1.226	1.2	1.175	1.15	1.13	1.11	1.09	1.07	1.05	1.035
	1.5 x 1.25	1.545	1.51	1.48	1.45	1.425	1.4	1.37	1.35	1.33	1.31	1.29	1.27
	1.5 x 1.5	1.85	1.81	1.78	1.75	1.72	1.69	1.665	1.64	1.616	1.59	1.57	1.55



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