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GUIDELINES FOR THE DESIGN OF SMALL BRIDGES AND CULVERTS

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PREFACE

The Paper entitled "Guidelines for the Design of Small Bridges and Culverts" by Shri Goverdhan Lal who retired as Additional Director General to the Government of India (Roads), was published as Paper No. 167 in Volume XVIII-Part 2 of the Journal of the Indian Roads Congress. One of the main objectives of the Author in preparing this Paper was to help the Highway Engineers in the country to do the planning and design of culverts and small bridges for road projects correctly and expeditiously. The object of the Author seems to have greatly been achieved as there has been continuous demand for this Paper and every Highway Engineer has been wanting to possess it as a book of reference. The Indian Roads Congress is, therefore, indebted to the Author for his labour and ingenuity in making the above mentioned contribution which has not only helped the members of the profession, but has also assisted in accelerating the progress of road development in the country.

While the Paper in its original form has still great value, the Indian Roads Congress felt that if it is revised in the light of the revised IRC Bridge Codes and the experiences gained since its first publication, it will make the Paper almost an uptodate text-book or manual for the design of culverts and small bridges. Shri Goverdhan Lal, the Author of the Paper has kindly permitted the Indian Roads Congress to revise this Paper and bring it out as a Special Publication of the Congress. The Indian

Roads Congress express their high appreciation and thanks to Shri Goverdhan Lal for this kind permission. They also express their thanks to the Director General (Road Development) for kindly getting the manuscript of this Special Publication examined in the Roads Wing and for the useful suggestions offered. We are sure that this new publication "Guidelines for the Design of Small Bridges and Culverts" would prove as a useful book of reference to the Highway Engineers and also a guide to the engineering students.

CONTENTS

APPENDIX

PLATES

SYNOPSIS

The discussion begins by stressing the importance of collecting sufficient data before designing is started. Methods for calculating flood discharges from the catchment area and rainfall, from the conveyance factor and slope of the channel, and from flood marks on existing structures have been outlined and suggestions made for fixing the design discharge (Articles 3—7).

After giving an outline of the "Lacey's Theory of Flow in Incoherent Alluvium", the procedure for designing the linear waterway and estimating the normal depth of scour for alluvial streams has been included. This is followed by a discussion on the normal scour depth of quasi-alluvial streams and the effect of contraction on the scour depth. Rules for calculating the maximum depth of scour and designing the depth of foundations have been included. Their application has been illustrated with worked out examples.

Notes on fixing the length and number of spans and on some important structural details of minor bridges come next.

The hydraulics of flow through bridges and its applications to the calculation of discharges through existing bridges as well as of afflux through proposed bridges have been dealt with in some detail. Examples have been worked out in illustration.

Brief notes on how to deal with overflows have been added. The discussion concludes with Articles on the feasibility of pipe and box culverts and the hydraulics of designing them.

Sixteen Plates have been added at the end to facilitate routine computation.

INTRODUCTION

Road project estimates prepared for the Five-Year Plan are sometimes based on insufficient data and this causes delay in sanctioning them. While speed in execution of the Plan is essential, the quality of work has to be maintained.

The designing of a culvert or a small bridge is not a small matter to be despised by engineers. It must be remembered that the number of culverts and small bridges in any road project is so large that, together, they account for a considerable portion of the total expenditure.

The object of this Special Publication is to draw attention to some aspects of designing culverts and small bridges for road projects. What is said here might appear simple and commonplace but experience has shown that serious mistakes are made in these simple matters and that leads to waste through unsafe and uneconomical designs.

Observations in this Special Publication apply only to small bridges and culverts. Larger structures require more detailed design procedure.

1

Site Inspection

ARTICLE 1

SITE INSPECTION

1.1. **Selection of Site.** Where there is any choice, select a site :

(*i*) which is situated on a straight reach of the stream, sufficiently below bends ;

(*ii*) which is so far away from the confluence of large tributaries as to be beyond their disturbing influence ;

(*iii*) which has well-defined banks ;

(*iv*) which makes approach roads feasible on the straight; and

(*v*) which offers a square crossing as far as possible.

In siting small bridges and culverts, due consideration should be given to the geometrics of the approach alignment and the latter should essentially govern the selection of site unless there are any special problems of bridge design.

1.2. **Existing Drainage Structures.** If, by chance, there is an existing road or railway bridge or culvert over the same stream and not very far away 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 enquiry 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. All these should be carefully recorded.

1.3. Inspection should also include taking notes on channel conditions from which the silt factor and the co-efficient of rugosity can be estimated.

2

The Essential Design Data

ARTICLE 2

THE ESSENTIAL DESIGN DATA

2.1. In addition to the information obtained by personal inspection 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 the Indian Roads Congress Standard Specifications & Code of Practice for Road Bridges—Section I, Clauses 100-102, should be followed.

2.2. **Catchment Area.** When the catchment, as seen from the "topo" sheet, is less than about 1.25 sq.km. in area, a traverse should be made along the watershed with a chain and compass. Larger catchments can be read from the 1 cm = 500 m topo maps of the Survey of India by marking the watershed in pencil and reading the included area by placing over a piece of transparent square paper.

2.3. **Cross-sections.** As a rule, for a sizable stream, three cross-sections should be taken, 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 metres or 1/1000 and with an exaggerated vertical scale of not less than 1 cm to 1 metre or 1/100. Approximate distances, upstream and downstream of the selected site of crossing at which cross-sections should be taken are as under.

TABLE 1

Catchment area	*Distance (u/s & d/s of the crossing) at which cross-sections should be taken*
1. 2.5 sq. km.	150 m
2. From 2.5 to 10 sq. km.	300 m
3. Over 10 sq. km.	400 m to 1600 m

The cross-section at the proposed site of the crossing should show levels 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 centre line of the road or the track as that will not represent the natural shape and size of the channel. Instead, the cross-section should be taken a short distance upstream or downstream of the selected site.

2.4. 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.

2.5. **The Maximum H.F.L.** The maximum high flood level should be ascertained by intelligent local observation, supplemented by local enquiry, and marked on the cross-sections.

2.6. **Longitudinal Section.** The longitudinal section should extend upstream and downstream of the proposed site for the distances indicated in Table 1 and should show levels of the bed, the low water surface and the H.F.L.

2.7. **Velocity Observation.** Attempts should be made to observe the velocity during an actual flood and, if that flood is smaller than the maximum flood, the observed velocity should be suitably increased. The velocity thus obtained is a good check on the accuracy of that calculated theoretically.

2.8. **Trial Pit Sections.** 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 two times the maximum depth of the (existing or anticipated) scour line. 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.

2.9. For very small culverts, one trial pit will do. The results can be inserted on the cross-section.

2.10. In Conclusion. No satisfactory designing can be done unless the minimum essential design data listed above are collected and recorded.

These data can be collected with very little effort. Failure to collect these can only result in designs being based on guess work and such designs are likely to be either unnecessarily costly or to result in the failure of the structure when built.

3

Empirical Formulae for Peak Run-off from Catchment

ARTICLE 3

EMPIRICAL FORMULAE FOR PEAK RUN-OFF FROM CATCHMENT

3.1. 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 theoretical computations.

In this Article some of the most popular empirical formulae are mentioned.

3.2. **Dickens' Formula**

$$Q=CM^{\frac{3}{4}} \qquad \ldots(3.2)$$

where, Q=the peak run-off in cu. m./sec and M is the catchment area in sq. km.

C=11–14 where the annual rainfall is 60-120 cm;
14—19 in Madhya Pradesh;
22 in Western Ghats.

3.3. **Ryve's Formula**

$$Q=CM^{\frac{2}{3}} \qquad \ldots(3.3)$$

This formula was devised for Madras.

Q=run-off in cu. m./sec. and M is the catchment area in sq. km.

C=6.8 for areas within 25 km of the coast
8.5 „ between 25 and 16C km of the coast
10.0 for limited areas near the hills.

3.4. **Inglis' Formula**

$$Q=\frac{125\ M}{\sqrt{M+10}}$$

where Q = maximum flood discharge in cu. m./sec.

M = the area of the catchment in sq. km.

The formula was devised for Bombay Presidency.

3.5. 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.

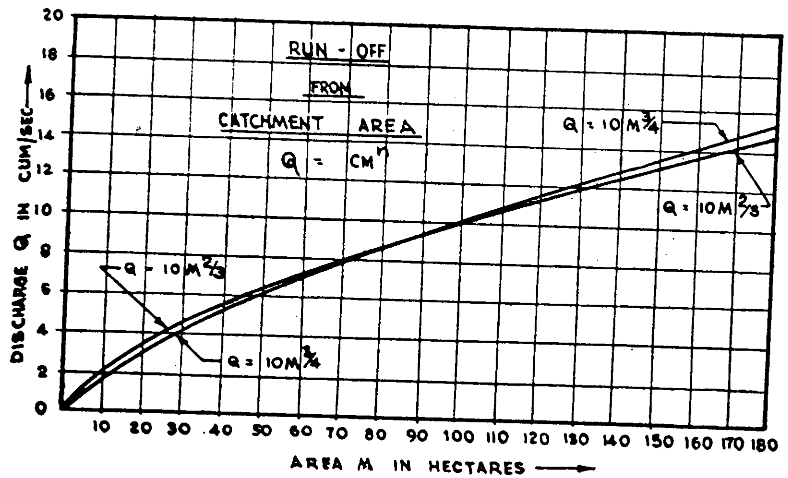


Fig. 1

3.6. 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. Many other similar empirical formulae are in use but none of them encompasses all possible conditions of terrain and climate.

3.7. To assist routine computations, Figs. 1 and 2 have been included in which curves have been plotted to represent the equation

$Q = 10\ M^n$. Two curves, one for each of the common values of n, *viz.*, $\frac{2}{3}$ and $\frac{3}{4}$, have been drawn.

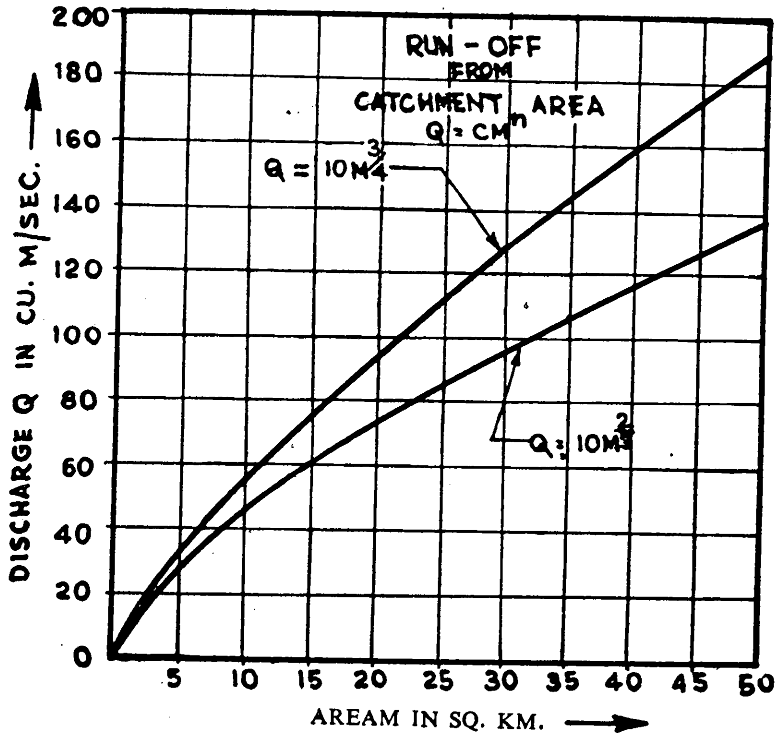


Fig. 2

4

Rational Formulae for Peak Run-off from Catchment

ARTICLE 4

RATIONAL FORMULAE FOR PEAK RUN-OFF FROM CATCHMENT

4.1. 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.

4.2. **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 the catchment

(4) Area
(5) Shape
(6) Slope
(7) Permeability of the soil and vegetable cover
(8) Initial state of wetness

4.3. **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 main intensity I will be $\frac{F}{T}$ cm per hour. Now, within the duration T, imagine a smaller time interval t (Fig. 3). Since the intensity is not uniform throughout, the mean intensity reckoned over the time interval t (placed suitably within T) will be higher than the mean intensity I taken over the whole period.

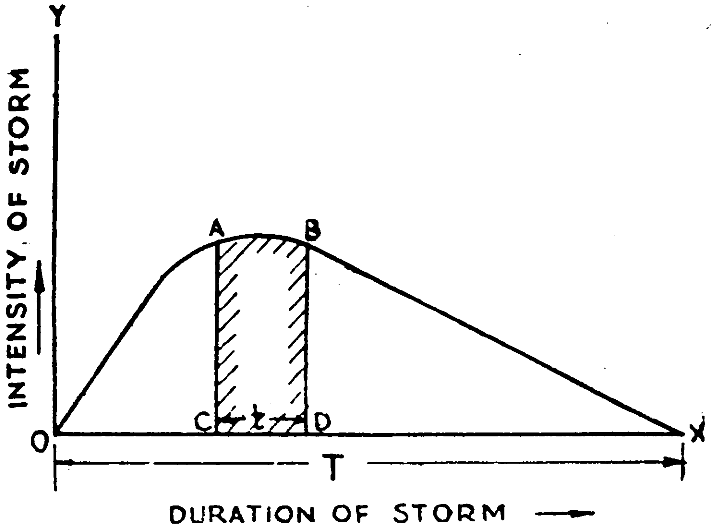


Fig. 3

We also know 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

$$\frac{i}{I}=\frac{T+c}{t+c}$$

where c is a constant.

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.

Thus,

$$\frac{i}{I}=\frac{T+1}{t+1}$$

$$\text{or } i=I.\left(\frac{T+1}{t+1}\right) \qquad \ldots(4.3_a)$$

Also,

$$i=\frac{F}{T}.\left(\frac{T+1}{t+1}\right) \qquad \ldots(4.3_b)$$

*Refers to the number of the book in the Bibliography given at the end of the Publication.

Thus, if we know the total precipitation F and duration T of a storm, we can estimate the intensity corresponding to t, which is a time interval within the duration of the storm.

4.4. For an appreciation of the physical significance of this relationship, let us consider some typical cases.

First 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

$$i=\frac{F}{T}\left(\frac{T+1}{t+1}\right)=\frac{5}{0.33}\left(\frac{0.33+1}{0.1+1}\right)=18.2 \text{ cm per hour.}$$

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 4.6) 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 a time interval of one hour within the storm the intensity can be as much as :

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

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, 18 cm of rain in 6 hours. In a time interval of one hour within the storm the intensity can be as high as :

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

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

4.5. "One-hour Rainfall" for a Region for Designing Waterway, of Bridges. Suppose we decide that a bridge should be designed for the peak run-off resulting from the severest storm (in the region) that occurs once in 50 years or any other specified period. 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

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

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

$$I_0=\frac{F}{2}\left(1+\frac{1}{T}\right) \qquad ...(4.5)$$

Suppose Fig. 3 represents the severest storm experienced in a region. If t represents one hour, then the shaded area $ABCD$ will represent I_0.

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

I_0 has to be determined from the F and T of the severest storm. That storm may 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 I_0 from the F and T of each of them. The maximum of the values of I_0 thus found should be accepted as "the one hour rainfall" of the region for designing bridges.

The 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.

The Meteorological Department of the Government of India, have supplied the heaviest rainfall in mm/hour experienced by various places in India. This chart is enclosed as Appendix "A" and the values indicated therein, may be adopted for I_0, in absence of other suitable data.

We 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.

4.6. 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 we are examining a particular catchment, we need only consider the intensity corresponding to the duration equal to the concentration period (t_c) of the catchment.

4.7. Estimating the Concentration Time of a Catchment (t_c) The concentration time depends on (1) the distance from the critical point to the culvert; and (2) the average velocity of flow. The latter is governed by the slope and the roughness of the drainage channel and the depth of flow. Complicated formulae exist for deriving the time of concentration from the characteristics of the catchment. For our purposes, however, the following simple relationship [11] will do.

$$t_c=\left(0.87 \times \frac{L^3}{H}\right)^{0.385} \qquad ...(4.7)$$

where t_c=the concentration time in hours.

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

H=the fall in level from the critical point to the culvert in metres.

L and H can be found from the survey plans of the catchment area and t_c calculated from Equation (4.7).

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

4.8. **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. Call it 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 (Equation 4.3$_b$) and write I_c for the resulting intensity, we get

$$I_c=\frac{F}{T}\left(\frac{T+1}{t_c+1}\right) \qquad ...(4.8_a)$$

Combining this with Equation (4.5), we get

$$I_c=I_0\left(\frac{2}{t_c+1}\right) \qquad ...(4.8_b)$$

4.9. Calculation of Run-off

A precipitation of I_c cm per hour over an area of A hectares, will give rise to a run-off

$$Q=0.028\ A\ I_c \text{ cu. m./sec.}$$

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

$$Q=0.028\ P\ A\ I_c \qquad ...(4.9)$$

where Q=max. run-off in cu. m./sec.

A=area of catchment in hectares.

I_c=critical intensity of rainfall in cm per hour.

P=percentage 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, (*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 generalise on the values of P. Judgment and experience must be used in fixing P. Also see Table 2 for guidance.

TABLE 2

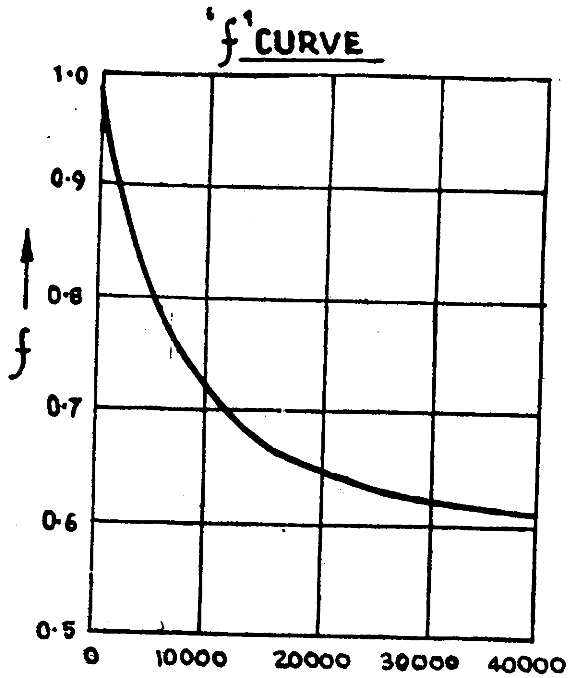
MAXIMUM VALUE OF P IN THE FORMULA Q=0.028 PAI$_c$

Steep, bare rock. 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

4.10. **Relation between Intensity and Spread of Storm.** We have so far deliberately eschewed any mention of the spread of the storm. Rainfall recording stations are points in the space and therefore the intensities recorded there are point intensities. Imagine an area round a recording station. The intensity will be highest at the centre and will gradually diminish as we go farther away from the centre, till at the fringes of the area covered by the storm, intensity will be zero. The larger the area considered the smaller will 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 centre 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 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 the area 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].



Catchment area in hectares

Fig. 4

4.11. **The Final Run-off Formula.** Introducing the factor f in the Equation 4.9 we get,

$$Q=0.028\ P.f.A.I_c \qquad \text{...}(4.11_a)$$

Also, combining with Equation (4.8_b)

$$Q=0.028\ P.f.A.I_0\left(\frac{2}{t_c+1}\right)$$

$$=0.028\ A.I_0.\left(\frac{2.f.P}{t_c+1}\right)$$

$$=A\ I_0\ \lambda \qquad \text{...}(4.11_b)$$

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

In the last equation, I_0 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 paras that the components of λ are functions of A, L and H of the catchment.

4.12. 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 the I_0 for the region preferably from rainfall records : failing that from local knowledge. $I_0=\frac{F}{2}\cdot\left(1+\frac{1}{T}\right)$, where F is cm of rain dropped by the severest 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.

$$\text{Then calculate } \lambda=\frac{0.056\,fP}{t_c+1}$$

Step 4. Calculate $Q=A.I_0.\lambda$.

4.13. **Example.** Calculate the peak run-off for designing a bridge across a stream, given :

Catchment : $L=5$ km; $H=30$ metres
$A=10$ sq. km.$=1000$ hectares. Loamy soil largely cultivated.

Rainfall : The severest storm that is known to have occurred in 20 years dropped 15 cm of rain in 2.5 hours.

$$\text{Solution.}\quad I_0=\frac{15}{2.5}\left(\frac{2.5+1}{1+1}\right)10.5\ \text{cm/hour}$$

From **Plate 1**,

$t_c=1.7$ hours; $f=0.97$; P$=0.30$

$$\therefore\ \lambda=\frac{0.056\times0.97\times0.30}{1.7+1}=0.006$$

$$Q=1000\times10.5\times0.006=63.6\ \text{cu. m./sec.}$$

4.14. **Run-off Curves for Small Catchment Areas (Plate 2).** Suppose we know the catchment areas A in hectares and the average slope S of the main drainage channel. If we assume that the length of the catchment is 3 times its width, then both L and H (as defined in para 4.7) can be expressed in terms of A and S and then t_c calculated from Equation (4.7).

Also for small areas, f may be taken equal to one. Then vide para 4.11,

$$Q=P.I_0A\left(\frac{0.056}{t_c+1}\right)$$

For $I_0=1$ cm, this becomes,

$$Q=P.A\left(\frac{0.056}{t_c+1}\right) \qquad \ldots(4.14)$$

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

4.15. **Plate 2** can be used for small culverts with basins upto 1500 hectares or 15 sq. km. The values of run-off read from **Plate 2** are for "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.

4.16. **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, $P=0.30$ vide the Table inset in Plate 2.

Enter the diagram at $A=10$ sq. km.$=1000$ hectares; move vertically up to intersect the slope line of 10 per cent. Then, move horizontally to intersect the 00 line; join the intersection with $P=0.3$ and extend to the run-off (q) scale and read

$$q=10.2 \text{ cu. m./sec.}$$

Multiply with I_0

$$Q=10.2\times 2.5=25.5 \text{ cu. m./sec.}$$

4.17. **In Conclusion.** The use of empirical formulae should be avoided. They are primitive and are safe only in the hands of the expert. 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 quite an 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 should do.

5

Estimating Flood Discharge from the Conveyance Factor and Slope of the Stream

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-section is significantly the same during a flood as after its subsidence. If we carefully plot the H.F.L. and measure the bed slope 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 sq.m. and the wetted perimeter P in m. Then, calculate the hydraulic mean depth $R=\frac{A}{P}$ m.

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.*, Manning's formula:

$$V=\frac{R^{\frac{2}{3}}\quad S^{\frac{1}{2}}}{n} \qquad \text{... (5.4a)}$$

where,

V=the velocity in m per sec. considered uniform throughout the cross-section;

R=the hydraulic mean depth;

S=the energy slope which may be taken equal to the bed slope, measured over a reasonably long reach;

n=the rugosity co-efficient.

For values of n, see Table 3 below. Judgment and experience are necessary in selecting a proper value of n for a given stream.

TABLE 3

Rugosity Co-efficient

Value of "n" in the formula $V=\frac{R^{\frac{2}{3}} . S^{\frac{1}{2}}}{n}$

Surface	Perfect	Good	Fair	Bad
Natural streams				
(1) Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
(2) Same as (1), but some weeds and stones ...	0.030	0.033	0.035	0.040
(3) Winding, some pools and shoals, clean ...	0.035	0.040	0.045	0.050
(4) Same as (3), lower stages, more ineffective slope and sections ...	0.040	0.045	0.050	0.055
(5) Same as (3), some weeds and stones ...	0.033	0.035	0.040	0.045
(6) Same as (4), stony sections ...	0.045	0.050	0.055	0.060
(7) Sluggish river reaches, rather weedy or with very deep pools ...	0.050	0.060	0.070	0.080
(8) Very weedy reaches ...	0.075	0.100	0.125	0.150

5.5. **Calculation of Discharge**

$$Q=A.V.$$

$$=\frac{A.R^{\frac{2}{3}} . S^{\frac{1}{2}}}{n} \quad \text{... (5.5)}$$

$$=\lambda.S^{\frac{1}{2}} \quad \text{... (5.6)}$$

where $\lambda=\frac{A . R^{\frac{2}{3}}}{n}$

Now 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 horizontally and vertically), the wetted perimeter cannot be scaled off directly from the section and has to be calculated.

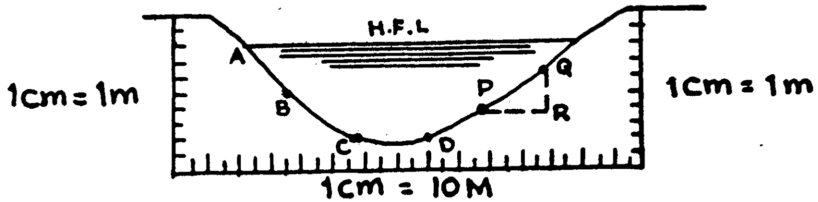


FIG. 5

Divide up the wetted line into a convenient number of parts, AB, BC, CD, etc. (Fig. 5). Consider one such part, say, PQ. Let PR and QR be its horizontal and vertical projections. Then, $PQ=\sqrt{(PR^2+QR^2.)}$ 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 each of the other parts 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. 6), 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.

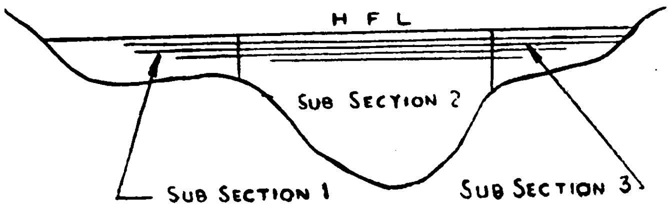


Fig. 6

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 actually to observe the velocity during a high flood. When it is not possible to wait for the occurrence of a 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 2.3, 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.

6

Estimating Flood Discharge from Flood Marks on an Existing Structure

ARTICLE 6

ESTIMATING FLOOD DISCHARGE FROM FLOOD MARKS ON AN EXISTING STRUCTURE

6.1. Having collected the necessary information from inspection as mentioned in para 1.2, the discharge passed by an existing structure can be calculated by applying an appropriate formula. In Article 17, some formulae for calculating discharges from flood marks on existing bridges are discussed. Worked out examples have been included in Article 19.

6.2. Distinct water marks 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 as possible. Turbulence, standing wave and splashing 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.

7

Fixing Design Discharge

these does not include empirical formulae which article 3

ARTICLE 7

FIXING DESIGN DISCHARGE

7.1. **The Recommended Rule.** Flood discharges can be estimated in three different ways as explained in Articles 4 to 6. 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.

7.2. **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 (say once in 20 years for small bridges and 100 years for big ones) and that extra-ordinary and rarer floods should pass without doing excessive damage to the structure or the road.

7.3. 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.

8

Alluvial Streams-Lacey's Equations

ARTICLE 8

ALLUVIAL STREAMS—LACEY'S EQUATIONS

8.1. 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 theoretical premises for the purposes of designing a bridge ; it is seldom possible actually to measure the cross-section during the high flood.

8.2. **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.

8.3. 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. Such a stream tends to scour or silt up till it has acquired such a cross-section and (more particularly) such a slope that the resulting velocity is "non-silting and non-scouring". When this happens the stream becomes 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.

8.4. **Lacey's Equations.** When an alluvial stream carrying a known discharge Q cu. m. per sec has come to regime, it has a regime wetted perimeter P metre, a regime slope S, and a regime hydraulic mean depth R metre. In consequence, it will have a fixed area of cross-section A sq. m. and a fixed velocity V metre per sec.

For these regime characteristics of an alluvial channel, Lacey suggests [18] the following relationships. It should be noted that the only independent entities involved in them are Q and f. The "f" 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 : $f=1.76\sqrt{m}$, where 'm' is the mean diameter of the particles in millimetre. Table 4 gives values of "f" for different bed materials.

(a) Regime Cross-section

$$P=4.8Q^{\frac{1}{2}} \quad \ldots(8.4_a)$$

$$R=\frac{0.473.Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \quad \ldots(8.4_b)$$

$$S=\frac{0.0003\,.\,f^{5/3}}{Q^{\frac{1}{6}}} \quad \ldots(8.4_c)$$

(b) Regime Velocity and Slope

$$V=0.44Q^{\frac{1}{6}}\times f^{\frac{1}{3}} \quad \ldots(8.4_d)$$

$$A=\frac{2.3\,.\,Q^{5/6}}{f^{\frac{1}{3}}} \quad \ldots(8.4_e)$$

TABLE 4

SILT FACTOR 'f' IN LACEY'S EQUATIONS [18]

		m size of grain in mm	*f (silt factor)*
Silt			
	Very fine	00.081	00.500
	Fine	00.120	00.600
	Medium	00.233	00.850
	Standard	00.323	1.000
Sand			
	Medium	00.505	1.250
	Coarse	00.725	1.500

8.5. **The Regime Width and Depth.** Provided a stream is truly alluvial, 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 (8.4_a).

That is :

$$W=4.8\times Q^{\frac{1}{2}} \quad \ldots(8.5_a)$$

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 (8.4_b). Hence

$$D=\frac{0.473\,.\,Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \quad \ldots(8.5_b)$$

8.6. **Curves for D.** In Fig. 7 and 8 curves have been plotted

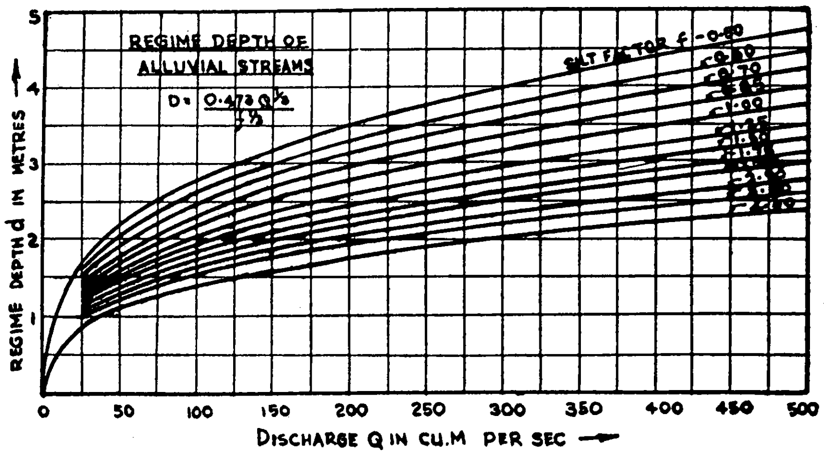


Fig. 7

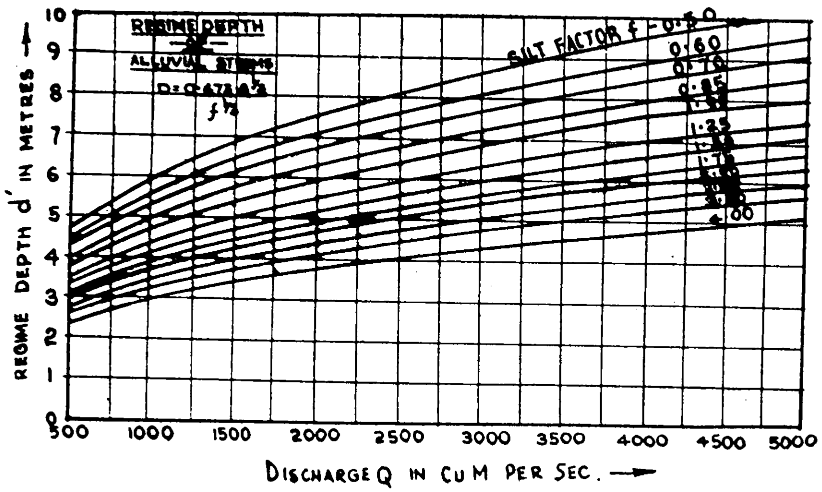


Fig. 8

from Equation (8.5_b). Knowing Q and f for an alluvial stream, the regime depth D can be directly read off these curves.

9

Linear Waterway of Bridges

ARTICLE 9

LINEAR WATERWAY OF BRIDGES

9.1. **The General Rule for Alluvial Streams.** The linear waterway of a bridge across a wholly alluvial stream should normally be kept equal to the width required for stability, *viz.*, that given by Equation (8.5_a).

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

9.3. In the ultimate analysis it may be found, in some such cases, that it is cheaper to adopt a linear waterway for the bridge somewhat in excess of the regime width given by Equation (8.5_a). 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 (8.5_b). Hence such an increase in the length of the bridge does not lead to any countervailing 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.

9.4. **Contraction to be Avoided.** The linear waterway of the bridge 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 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.

9.5. **Streams not Wholly Alluvial.** When the banks of a stream are high, well defined, and rigid (rocky or of some other natural hard soil that cannot be affected by the prevailing current) but the bed is alluvial, the linear waterway of the bridge should be made equal to the actual surface width of the stream, measured from edge to edge of water along the H.F.L., on the plotted cross-section.

Such streams are later referred to as quasi-alluvial.

9.6. **Streams with Rigid Boundaries.** In wholly rigid streams the rule of para 9.5 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 saving in the cost of the bridge.

9.7. As regards streams which 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 should span the active channel and detrimental afflux avoided. See also Article 20.

10

The Normal Scour Depth of Alluvial Streams

ARTICLE 10

THE NORMAL SCOUR DEPTH OF ALLUVIAL STREAMS

10.1. **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 elliptical.

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

In nature, however, the flood discharge in a stream does not have indefinite duration. For this reason the shape of the flood section of any natural stream would be governed by the magnitude and duration of the flood discharge carried by it. 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. We will, however, apply Lacey's Equations, pending further research.

10.2. As discussed later in Article 14, 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 (8.5_b) so long as the bridge does not contract the stream beyond the regime width W given by Equation (8.5_a).

10.3. If the linear waterway of the bridge is, for some special reason, 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).

$$D' = D\left[\frac{W}{L}\right]^{0.61} \qquad \ldots(10.3)$$

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 scour depth under the bridge with L less than W'.

This relationship is explained in Article 12.

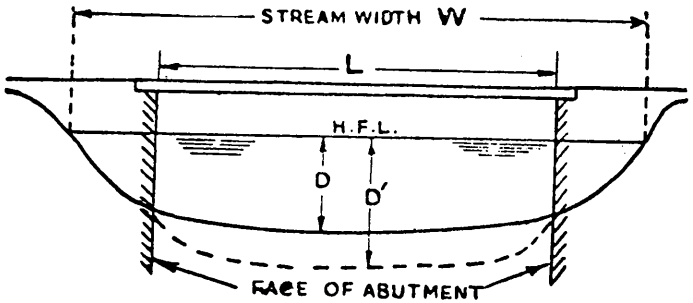


Fig. 9. This figure represents a hypothetical condition, i.e., uniform scour. D is the normal scour depth corresponding to stream width W and D' normal scour depth corresponding to reduced waterway under the bridge.

11

The Normal Scour Depth of Quasi-Alluvial Streams

ARTICLE 11

THE NORMAL SCOUR DEPTH OF QUASI-ALLUVIAL STREAMS

11.1. **Quasi-alluvial Streams.** 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 a wholly 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-section which Lacey's Equations prescribe.

11.2. It is not essential that the banks should be of rock to be inerodible. 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.

When we come across a stream section, the natural width of which is nowhere near that prescribed by Lacey's Theory, we should expect 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.

11.3. In any such case the width W of the section, being fixed between the rigid banks, can be 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 currency of a high flood.

11.4. **When the Stream Width is Large Compared to Depth.** In Article 5, for calculating the discharge of the stream from its plotted cross-section, we drew the probable scoured bed line (para 5.3).

When the stream scours down to that line it should be capable of passing the discharge calculated there, say q cu. m./sec. But the discharge adopted for design, Q, may be anything upto 50 per cent more than q (see para 7.1). 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,

$$D = d \times \left(\frac{Q}{q}\right)^{3/5} \qquad \ldots(11.4)$$

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 unalterd.

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

$$= (R.P) \;\; C.\, R^{\frac{2}{3}}\, S^{\frac{1}{2}}$$

$$= K.\, R^{5/3}$$

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 (11.4).

From the above relationship it follows that if Q is 150 per cent of q, D will be equal to 127 per cent of d.

11.5. **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, we may write W for P and D for R.

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

$$= (P.R) \times V = (W.D).\, V.$$

$$i.e.,\quad D = \frac{Q}{W.V}$$

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.

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

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

$$= (R \times P) \times \frac{R^{\frac{2}{3}}\, S^{\frac{1}{2}}}{n}$$

$$= \frac{W \times S^{\frac{1}{2}} \times D^{5/3}}{n} \qquad \ldots(11.6)$$

Knowing Q, W and S, we can calculate D 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 ($= V \times R \times W$). If this equals the design discharge Q, 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 para 18.8).

11.7. **Another Formula.** The procedure described above can be applied if we know either the slope of the stream or the actual observed velocity. If we do not know either of these, we can apply the following procedure for approximate calculation of the normal scour depth.

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

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

$$= (P.R) \quad (C.\, R^{\frac{2}{3}} \quad S^{\frac{1}{2}}) \qquad \ldots(i)$$

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,

$$Q = (P_1.\, R_1)\;(C.R_1^{\frac{2}{3}}\; .S^{\frac{1}{2}}) \qquad \ldots(ii)$$

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

$$P_1 = 4.8.\ Q^{\frac{1}{2}} \qquad \ldots(iii)$$

and
$$R_1 = \frac{0.473\ Q^{\frac{1}{3}}}{f^{\frac{1}{3}}} \qquad \ldots(iv)$$

Equating values of Q in (i) and (ii), and rearranging we get:

$$\frac{R}{R_1} = \left(\frac{P_1}{P}\right)^{3/5} \qquad \ldots(v)$$

Now substituting values of P_1 and R_1 from equations (iii) and (iv) in (v), we get

$$R = \frac{1.21.Q^{0.63}}{f^{0.33} \times P^{0.60}} \qquad \ldots(vi)$$

If the width W of the stream is large compared with its depth D, then we can write W for P and D for R in equation (vi). This will give :

$$D = \frac{1.21.Q^{0.63}}{f^{0.33} \times W^{0.60}} \qquad \ldots(11.7)$$

Thus, if the design discharge Q, the natural width W, and the silt factor f are known, the normal scour depth D can be calculated from Equation (11.7).

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 11.6 applied.

11.8. **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. Then measure D.

11.9. **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), then the normal scour depth under the bridge D' will be greater than the depth D ascertained above for the unobstructed stream. The following relationship will apply :

$$D' = D\left(\frac{W}{L}\right)^{0.61} \qquad \ldots(11.9)$$

For proof see Article 12.

12

The Effect of Contraction on the Normal Scour Depth

ARTICLE 12

THE EFFECT OF CONTRACTION ON THE NORMAL SCOUR DEPTH

12.1. In Fig. 10, W is the width, D the normal scour **depth,** and u the velocity of the stream in an unobstructed reach **upstream** of the bridge.

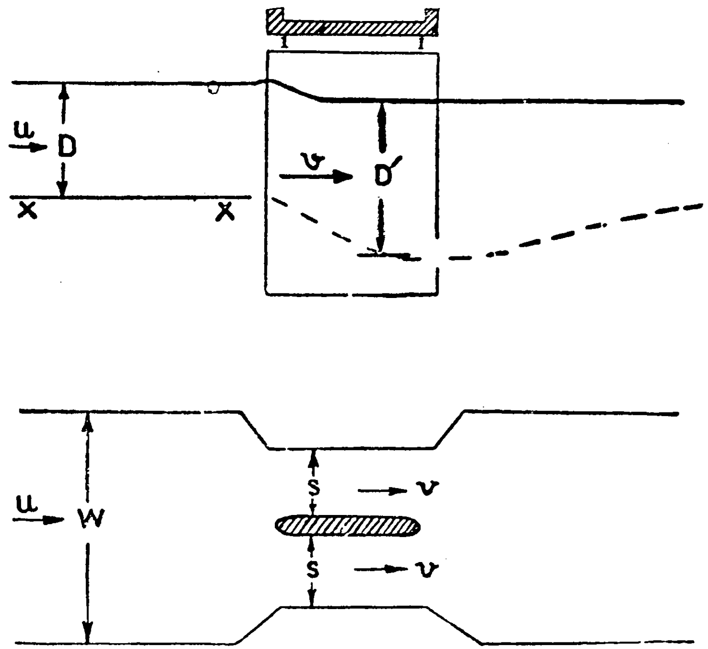


Fig. 10

If XX is the normal scour bed line and further scouring has ceased, then according to Kennedy's Silt Theory :

$$u = m \times (D)^{0.64} \qquad \ldots(i)$$

Since the vent-area of the bridge is smaller than that of the stream, therefore, when the water arrives at the bridge the initial velocity V

through the bridge will be greater than u. At that stage,

$$V=u.\frac{W}{L}$$

The initial high velocity V under the bridge is momentary. It starts further scouring and, in the process, itself begins to decrease (because further scouring means more area of flow through the bridge) till equilibrium is reached. If in that state of equilibrium, the depth under the bridge is D and the final reduced velocity v, then by Kennedy again,

$$v=m(D')^{0.64} \qquad ...(ii)$$

From equations (i) and (ii)

$$\frac{v}{u}=\left(\frac{D'}{D}\right)^{0.64} \qquad ...(iii)$$

Now, since the discharge at all sections is the same, therefore,

$$u.\ W.\ D=v.\ L.\ D'$$

where L is the linear waterway through the bridge,

$$i.e.,\ v=\left(\frac{W\ .\ D}{L\ .\ D'}\right)\times u \qquad ...(iv)$$

Substituting for v, from (iv) in (iii), we get :

$$D'=D\left(\frac{W}{L}\right)^{0.61} \qquad ...(12.1)$$

Knowing D, W and L we can calculate the increased depth of scour D' under the bridge from the above equation.

12.2. In this discussion it is tacitly assumed that scour develops uniformly and the bed line is lowered across the whole section of the bridge in a smooth curve. When that happens the depth D' will be the normal scour depth. In natural streams, however, scour is not uniform. At some points there is deeper scour than that at others. An expression for the deepest scour in such cases will be developed later (see para 13.5).

13

Maximum Scour Depth

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ARTICLE 13

MAXIMUM SCOUR DEPTH

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

13.2. 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.

13.3. 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 (8.5_b).

(*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 11.

(*iii*) **Modification of Normal Scour Depth.** When the bridge causes contraction, *i.e.*, L is less than W, both for alluvial and quasi-alluvial streams, the modified normal scour depth D' is given by :

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

where, L=linear waterway of the bridge.

W=regime width in the case of alluvial streams and unobstructed natural width in the case of quasi-alluvial streams.

13.4. **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 for the effect of contraction where necessary.

Rule 2 : For bad sites on curves or where diagonal currents exist or the bridge is a 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.

Rule 3 : For bridges causing contraction, the maximum scour depth obtained by Rule 1 or 2, should be compared with that given by the following equation and the greater of the two values adopted :

$$D_m = D\left(\frac{W}{L}\right)^{1.56}$$

where D_m is the maximum scour depth, and

D, L, W have the same significance as above. An explanation of this last Rule will now follow.

13.5. **Explanation of the Formula**

$$D_m = D\left(\frac{W}{L}\right)^{1.56}$$

We have to revert to the discussion in Article 12. There we assumed that scouring under the bridge was uniform and worked out an expression for the normal scour depth under the bridge. In natural streams scouring is not uniform and deeper scour develops at some points than at others (Fig. 11). To work out what the maximum scour will be under such conditions we assume that, as scour develops, the velocity through deepened parts, P and Q in Fig. 11, does not decrease. More water can rush into portions of deeper scour from the sides and the initial velocity through the bridge, *viz.*, V is maintained in those portions.

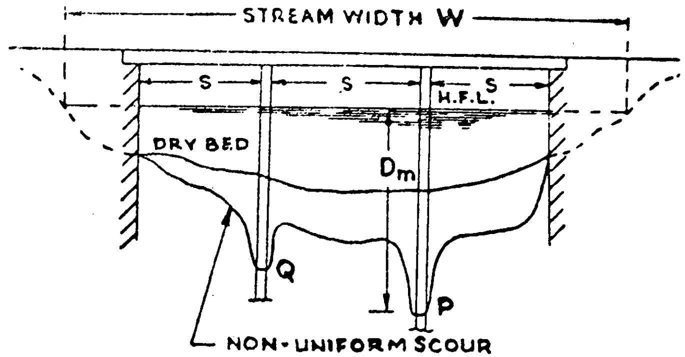


Fig. 11

$$V = \frac{W}{L} \cdot u \qquad \ldots(i)$$

When the maximum scour depth D_m has developed there will be equilibrium between V and D_m.

Hence, by Kennedy,

$$V = m(D_m)^{0.64} \qquad \ldots(ii)$$

Also, as in Article 11

$$u = m(D)^{0.64} \qquad \ldots(iii)$$

From (ii) and (iii)

$$\frac{V}{u} = \left(\frac{D_m}{D}\right)^{0.64} \qquad \ldots(iv)$$

Substituting for V from (i) in (iv) and rearranging

$$D_m = D\left(\frac{W}{L}\right)^{1.56} \qquad \ldots(13.5)$$

13.6. It is easy to prove that, where Rule 1 applies, the maximum scour depth given by it governs so long as the linear waterway of the bridge L is not less than 65 per cent of W. Beyond that limit, the maximum scour depth given by Equation (13.5) is greater.

But where Rule 2 applies, the maximum scour depth, given by it, will govern where L is anything down to 47 per cent of W. Since contractions below this limit are not resorted to in practice, checking by Rule 3 becomes needless.

Examples at the end of this Article will make this clear.

13.7. The finally adopted value of D_m must not be less than the depth (below H.F.L.) of the deepest scour hole that may be found by inspection to exist at or near the site of the bridge.

The following examples will illustrate the application of the rules in para 13.4 above.

13.8. **Example 1.** A bridge is proposed across an alluvial stream ($f=1.2$) carrying a discharge of 600 cu. m./sec. Calculate the depth of maximum scour when the bridge consists of :

(a) 3 spans of 15, 25 and 15 m;
(b) 5 spans of 15 m each;
(c) 4 spans of 30 m each.

Regime surface width of the stream

$$W=4.8\,Q^{\frac{1}{2}}=4.8\times 600^{\frac{1}{2}}=117.5 \text{ m}$$

Regime depth

$$D=0.473\frac{Q^{\frac{1}{3}}}{f^{\frac{1}{3}}}=\frac{0.473\times 600^{\frac{1}{3}}}{(1.2)^{\frac{1}{3}}}=3.76 \text{ m}$$

Case (a). Rule 2 applies, since the bridge has piers:

$$D_m=2\times D'=2\times D(W/L)^{0.61}$$

$$=2\times 3.76\left(\frac{117.5}{55}\right)^{0.61}=11.9\ m$$

For check, calculate D_m by Equation (13.5)

$$D_m=D(W/L)^{1.56}=3.76\left(\frac{117.5}{55}\right)^{1.56}=12.3\ m$$

Adopt 12.3 m say 12.5 m

[Note.—In this case Equation (13.5) governs because the contraction exceeds 47 per cent. Compare para 13.6].

Case (b). Rule 2 applies :

$$D_m=2D'=2D\,(W/L)^{0.61}$$

$$=2\times 3.76\times\left(\frac{117.5}{75}\right)^{0.61}=9.9\ m$$

Check by Equation (13.5).

$$D_m=D(W/L)^{1.56}=3.76\left(\frac{117.5}{75}\right)^{1.56}=7.6\ m$$

Adopt 9.9 m say 10.0 m

Case (c). Rule 2 applies

$$D_m=2D=2\times 3.76=7.52\ m$$

Equation (13.5) will not apply as $L=W$

Adopt 7.52 m say 7.5 m

13.9. **Example 2.** A stream with rocky banks has a width of 90 m. Its bed is alluvial ($f=1.2$) and discharge 600 cu. m./sec. Calculate the maximum scour depth under a bridge (a) a single arch span of 60 m, (b) three spans of total linear waterway 60 m.

By Equation (11.7) :

$$D=1.21Q^{0.63}/(f^{0.33}\times W^{0.60})$$

$$=\frac{1.21\times 600^{0.63}}{1.2^{0.33}\times 90^{0.6}}$$

4.35 m

Case (a). By Rule 1 :

$$D_m=1.5D'=1.5D(W/L)^{0.61}$$

$$=1.5\times 4.35\times\left(\frac{90}{60}\right)^{0.61}=8.36\ m$$

Check by Equation (13.5),

$$D_m=D(WL)^{1.56}=4.35\left(\frac{90}{60}\right)^{1.56}=8.2\ m$$

Adopt 8.25 m

[Note : When contraction is 67 per cent, Rule 1 and Equation (13.5) give nearly equal values of D_m].

Case (b). By Rule 2 :

$$D_m = 2 \times D' = 2 \times D \times (W/L)^{0.61}$$

$$= 2 \times 4.35 \times \left(\frac{90}{60}\right)^{0.61} = 11.15\ m$$

Check by Equation (13.5),

$$D_m = D(W/L)^{1.56} = 4.35\left(\frac{90}{60}\right)^{1.56} = 8.2\ m$$

Adopt 11.15 m

[Note : In this example, the data do not mention the velocity or the slope of the stream, yet Equation (11.7) enables us to find the normal depth D of the unobstructed stream].

14

Depth of Bridge Foundations (Df)

ARTICLE 14

DEPTH OF BRIDGE FOUNDATIONS (D_f)

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

Rule (1) Erodible Beds. The foundations shall be taken down to a depth below the maximum high flood level one-third greater than the calculated depth of maximum scour subject to a minimum depth below the scour line of *two metres for arched bridges, and 1.2 metres for other bridges.*

Rule (2) Hard Beds. 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 0.3 metre or so into rock and about 0.6 metre or so into other hard material).

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

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

14.2. The rules mentioned in the last para apply when no bed floor is provided under the structure and the stream is free to scour as it may. The structure is then said to have deep foundations. It is sound to design deep foundations as far as circumstances permit.

14.3. In the case of small culverts, however, bed floors may be provided. In this connection the following is considered sound practice for culverts on erodible soil ;

Keep the top of the floor about 0.3 metre below the bed level. Take the foundations of the abutment 1.25 metres below the top of the floor. Provide an upstream curtain wall 1 to 1.5 metres deep and a downstream curtain wall 1.5 to 2.5 metres deep from the top of the floor. The precise depth of curtain walls, within the stated limits, will depend on the velocity of flow through the structure and erodibility of the bed material.

This rule may be followed when it is intended to avoid calculation of maximum scour depths for want of sufficient site data.

15

Length of Clear Span and Number of Spans

ARTICLE 15

LENGTH OF CLEAR SPAN AND NUMBER OF SPANS

15.1. 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.

15.2. Considering only the variable items, the cost of superstructure increases and that of the substructure decreases with an increase in the span length. The most economical span length is that, which satisfies the equation.

$$\left.\begin{array}{c}\textbf{Cost of variable parts in the}\\ \textbf{superstructure}\end{array}\right\} = \left\{\begin{array}{l}\textbf{Cost of the}\\ \textbf{substructure}\end{array}\right.$$

15.3. Where the bridge is big, and difficult conditions are anticipated in the sinking of foundations, accurate and detailed examination of the economic aspect is essential and any deviation from the ideal case has to be justified strongly.

15.4. **Length of Span.** In small structures, where open foundations can be laid and solid abutments and piers raised on them, it has been analysed that the following approximate relationships give economical designs.

For masonry arched bridges :

$$S=2H \quad \text{...(15.4}a\text{)}$$

For R.C.C. slab bridges:

$$S=1.5H \quad \text{...(15.4}b\text{)}$$

where S=clear span length in metres.

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

15.5. We have earlier found the depth of foundations, D_f. We should now fix the vertical clearance, which, added to D_f, will give H. The following minimum vertical clearance shall be provided:

Discharge in m³/sec	Minimum vertical clearance in mm
upto 0.3	150
0.3 to 3.0	450
3 to 30	600
30 to 300	900
300 to 3000	1200
Above 3000	1500

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 predetermined extent. This will be discussed later in detail (Articles 21 and 22).

15.6. **The Number of Spans.** If the required linear waterway L is less than the economical span length it has to be provided, as it is, in one single span.

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

$$L=NS.$$

15.8. 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 obstruction to the flow.

15.9. To facilitate inspection and carrying out of repairs, the minimum vent-dimension of culverts should normally be 750 mm. The vent size of irrigation culverts may be decided considering the actual requirements and site conditions.

16

Structural Details of Minor Bridges and Culverts

ARTICLE 16

STRUCTURAL DETAILS OF MINOR BRIDGES AND CULVERTS

16.1. Abutment and Pier Sections

The abutments and pier sections should be so designed as to withstand safely the worst combination of loads and forces as specified in the IRC Code of Practice for Road Bridges. Bridges and Culverts on National Highways should be designed for one lane of Class 70R or two lanes of Class 'A' loading, whichever produces more severe stresses in the various members under construction.

For culverts with small spans designed for IRC Class 70R loading or 2 lanes of IRC Class 'A' loading, the abutment sections may be adopted from Plate 4. In case of bridges, detailed designs have to be carried out to arrive at the safe sections.

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 overstressed.

In scourable beds, the length of the pier should be kept minimum and preferably made circular in the case of skew bridges.

16.2. Wing Walls and Return Walls

Long and hefty wing walls should be avoided. 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 case and should be adopted. Plate 4 gives the sections of wing walls and return walls for various spans and heights.

16.3. Unreinforced Masonry Arches

The type of superstructure depends on the availability of the materials of construction and their cost. An evaluation of the relative economics of R.C.C. 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 1:3:6 or dressed stones or bricks in 1:3 cement mortar. The crushing strength of concrete, stone or brick units shall not be less than 106 kg per sq.m. Where stone masonry is adopted for the arch ring, it shall be either coursed rubble masonry or ashlar masonry. Plate 5 shows the details of arch ring of segmental masonry arch bridges without footpaths for spans 6 m, 9 m and 12 m.

The sections of abutments and piers 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 5 gives the details of horizontal reaction, vertical reaction and moment at springing for arch bridges of spans 6 m, 9 m and 12 m considered in Plate 5 and Table 6 gives the influence line ordinates for horizontal reaction, vertical reaction and moment at springing for a unit load placed on the arch ring.

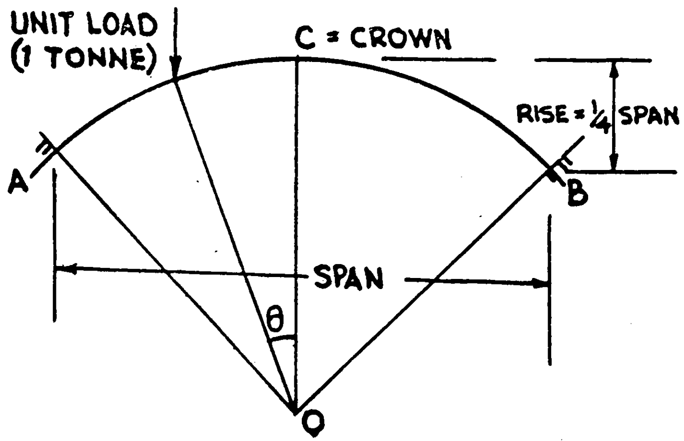
TABLE 5 VERTICAL REACTION, HORIZONTAL REACTION AND MOMENT AT SUPPORT DUE TO DEAD LOAD, DUE TO ARCH RING MASONRY, FILL MATERIAL AND ROAD CRUST FOR ONE METRE OF ARCH MEASURED ALONG THE TRANSVERSE DIRECTION (i.e., PERPENDICULAR TO THE DIRECTION OF TRAFFIC). RIGHT BRIDGES

Sl. No.	Span (metres) (Effective)	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
3.	12	27.00	33.60	(+) 0.62

Notes: 1. Unit weight of arch ring masonry, fill material and the road crust is assumed as 2.24 tonnes per cubic metre;

2. Positive sign for moment indicates tension on the inside of arch ring.

TABLE 6 INFLUENCE LINE ORDINATES FOR HORIZONTAL REACTION (H), VERTICAL REACTION AT SUPRORT (V_A) AND (V_B) AND MOMENT AT SPRINGING (M_A) AND (M_B) FOR UNIT LOAD, SAY 1 TONNE LOCATED ALONG THE ARCH AXIS AT AN ANGLE θ DEGREES FROM THE RADIUS OC. RISE OF ARCH IS ONE QUARTER OF SPAN



Sl. No.	θ degree	H in tonnes	V_A in tonnes	V_B in tonnes	M_A metre tonnes	M_B metre tonnes
	(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

(c) **Effective Span 12 m**

1.	0	0.93	0.500	0.500	(−)0.4425	(−)0.4425
2.	5	0.91	0.577	0.423	(−)0.2775	(−)0.5550
3.	15	0,75	0.725	0.275	(+)0.1425	(−)0.6150
4.	25	0.52	0,849	0.152	(+)0.5025	(−)0.5175
5.	35	0.25	0.940	0.061	(+)0.6825	(−)0.2775
6.	45	0.05	0.989	0.012	(+)0.4875	(−)0.0676
7.	53° 8	0	1.000	0	0	0

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

16.4. R.C.C. Slabs

In case a R.C.C. slab has to be used for culverts, its dimensions and reinforcement details should conform to those given in Plate 6. For R.C.C. slab bridges at square crossings, the dimensions and reinforcement details should conform to Plate 7, if without footpaths and to Plate 8, if with footpaths. For R.C.C. slab bridges at skew crossings, the dimensions and details of reinforcement are given in Plates 9 and 10 for slabs without footpaths. The dimensions and details of reinforcement of R.C.C. box cell bridges of 3 m span for different heights without footpaths are given in Plate 11.

The above drawings are for structures designed for one lane of IRC Class 70 R or two lanes of IRC Class A loading, whichever is severer.

16.5. Width of Roadway

The length of a culvert (along the direction of flow) should be such that the distance between the outer faces of the parapets wil 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 culvert.

In small bridges, the length (parallel to the flow of the stream) should be sufficient to give a minimum clear roadway of 4.5 metres for a single lane bridge and 7.5 metres for a two lane bridge between the inner faces of the kerbs or wheel guards. Extra provision should be made for footpaths, etc., if any are required.

16.6. R.C.C. Pipe Culverts

For small size structures, R.C.C. pipe culverts with single or double pipes, depending upon the discharge may be used as far as possible, as they are likely to prove comparatively cheaper than slab culverts. The details of pipe culverts of 1 metre dia., with single or double pipes having cement concrete or granular materials in bed, may be taken from Plates 12 to 15, for IRC Class 70R or 2 lanes of Class A loading.

17

Elements of the Hydraulics of Flow through Bridges

ARTICLE 17

ELEMENTS OF THE HYDRAULICS OF FLOW THROUGH BRIDGES

17.1. The formulae for discharges 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.

17.2. **Broad Crested Weir Formula applied to Bridge Openings**

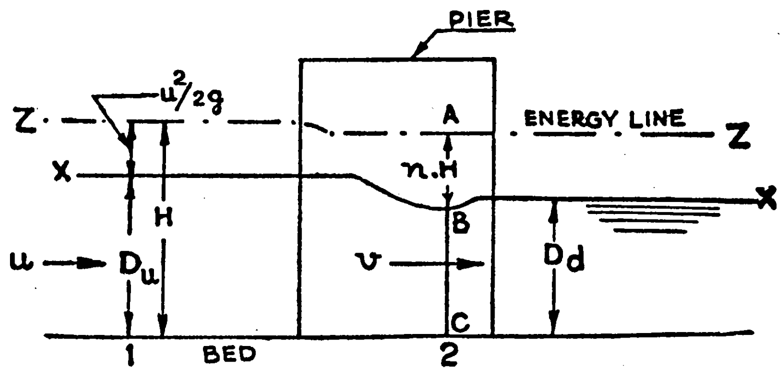


Fig. 12

In Fig. 12, *XX* is the water surface profile, and *ZZ* the total energy line.

At Section 1, the total energy,

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

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

$$AB = \frac{v^2}{2g} = n.H \qquad \ldots (ii)$$

Ignoring the loss of head due to entry and friction, and equating total energies at Sections 1 and 2 :

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

Hence, the depth $BC = (1-n)\ H$ $\qquad \ldots (iii)$

The area of flow at Section 2,

$$a = BC \times \text{linear waterway}$$
$$= (1-n)\ H \times L.$$

Velocity at 2, from (ii),

$$v = (2gn\ H)^{\frac{1}{2}}$$

Therefore, the discharge through the bridge

$$Q = a.v.$$
$$= (1-n)\ H.L. \times (2gn\ H)^{\frac{1}{2}}$$

To account for losses in friction, a co-efficient C_w may be introduced. Thus,

$$Q = C_w\ (1-n)\ H.L.\ (2gn\ H)^{\frac{1}{2}}$$
$$= C\sqrt{2g}.\ L.\ H^{3/2}\ (n^{\frac{1}{2}} - n^{3/2}) \qquad \ldots (iv)$$

The depth BC adjusts itself so that the discharge passing through it is maximum. In that condition,

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

i.e., $\qquad \frac{1}{2}.n^{-\frac{1}{2}} - {}^{3}/_{2}n^{\frac{1}{2}} = O$, *i.e.*, $n = \frac{1}{3}$.

Putting $n = \frac{1}{3}$ in (iv)

$$Q = 1.706\ C_w.\ LH^{3/2} \qquad \ldots (17.2_a)$$

Combining with (i),

$$Q = 1.706\ C_w.L\left(Du + \frac{u^2}{2g}\right)^{3/2} \qquad \ldots (17.2_b)$$

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 co-efficient 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

17.3. The Orifice Formula

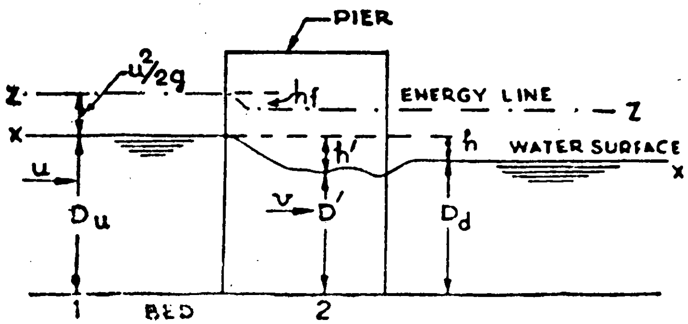


Fig. 13

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. 13, XX is the water surface line and ZZ the total energy line.

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

$$D_u+\frac{u^2}{2g}=D'+\frac{v^2}{2g}$$

or

$$\frac{v^2}{2g}=D_u-D'+\frac{u^2}{2g}$$

$$v=\sqrt{2g}\left(D_u-D'+\frac{u^2}{2g}\right)^{\frac{1}{2}}$$

Put $D_u-D'=h'$

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

The discharge through the Section 2,

$$Q=a\,.\,v.$$

$$=L.D'.\sqrt{2g}\left(h'+\frac{u^2}{2g}\right)^{\frac{1}{2}} \qquad \text{...(i)}$$

Now the fractional difference between D' and D_d is small. Put D_d for D' in (i).

$$Q=L\,.\,D_d\,.\sqrt{2g}\left(h'+\frac{u^2}{2g}\right)^{\frac{1}{2}} \qquad \text{...(ii)}$$

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

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

Substituting in (ii)

$$Q=L.D_d\sqrt{2g}\left(h+\overline{e+1}\,\frac{u^2}{2g}\right)^{\frac{1}{2}}$$

Now introduce a co-efficient C_o to account for losses of head through the bridge. We get :

$$Q=C_o.\sqrt{2g}.L.D_d\left(h+\overline{1+e}\,\frac{u^2}{2g}\right)^{\frac{1}{2}} \qquad \text{...(17.3)}$$

For values of e and C_o, see Figures 14_b and 15_c [10].

— COEFFICIENT "e" —
— FOR —
— THE ORIFICE FORMULA —

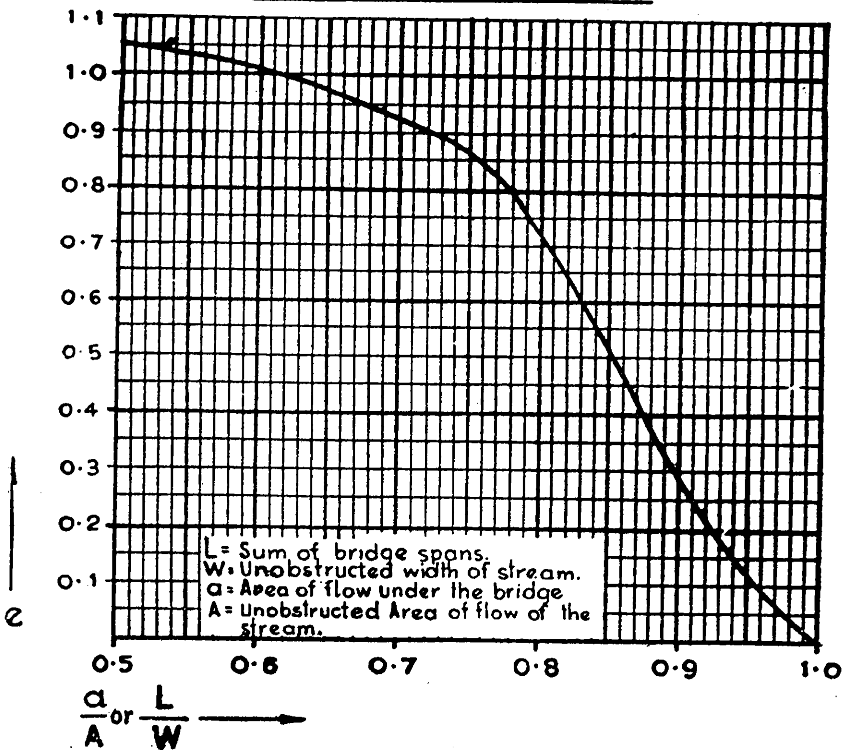


Fig. 14

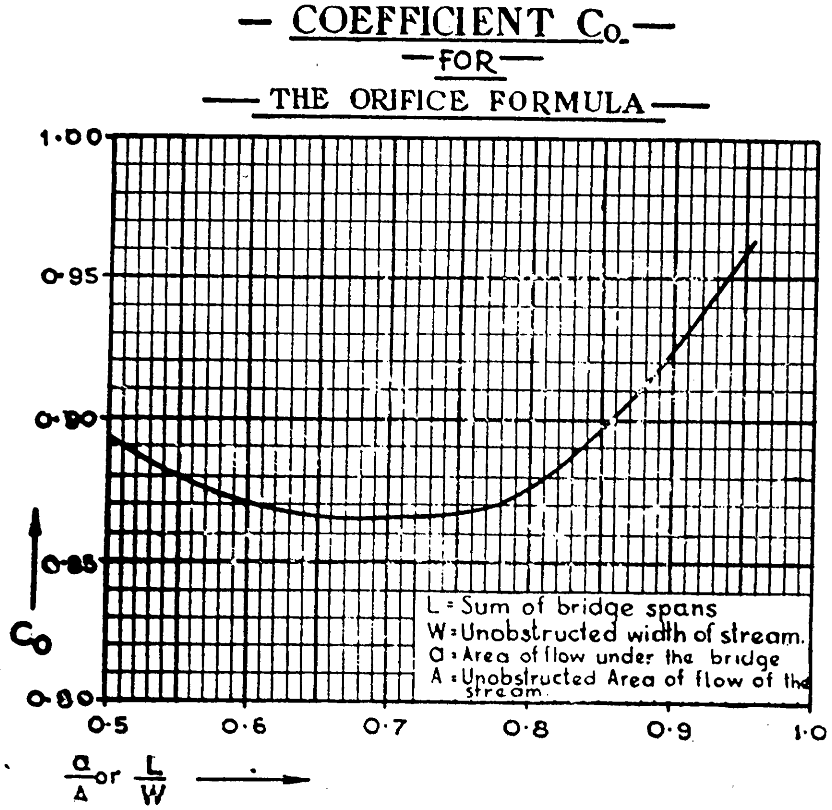


Fig. 15

17.4. **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 11.

(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 $\frac{1}{4}$ D_d.

(3) The Orifice Formula with the suggested values of C_o and e should be applied when the afflux is less than $\frac{1}{4}$ D_d.

17.5. Examples have been worked out in Articles 18 and 19 to show how these formulae can be used to calculate afflux and discharge under bridges.

18

Afflux

ARTICLE 18

AFFLUX

18.1. 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).

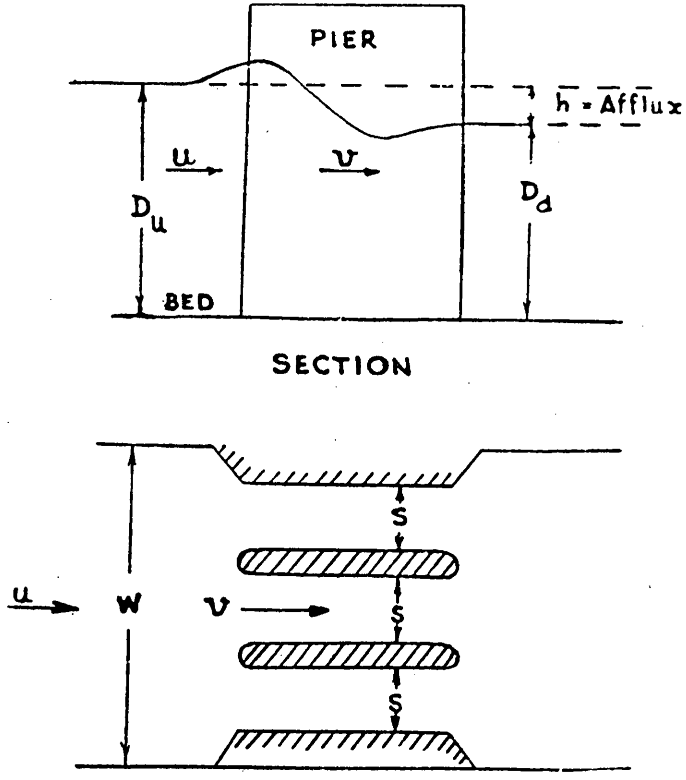


Fig. 16

18.2. 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.

18.3. 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.

18.4. For calculatiug afflux we must know (1) the discharge Q, (2) the unobstructed width of the stream W, (3) the inner waterway of the bridge L, and (4) the average depth downstream of the bridge D_d.

18.5. 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 downstream 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.

18.6. **Example.** A bridge, having a linear waterway of 25 m, spans a channel 33 m wide carrying a discharge of 70 cu. m./sec. Estimate the afflux when the downstream depth is 1 metre.

$D_d = 1$ m; $W = 30$ m; $L = 25$ m .

Discharge through the bridge by the Orifice Formula,

$$Q = C_o \sqrt{2_g} . L . D_d \sqrt{\left(h + \overline{1+e} \frac{u^2}{2_g}\right)}$$

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

Corresponding to this, $C_o = 0.867$, $e = 0.85$, $g = 9.8$ m/sec²

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

$\therefore h + 0.0944u^2 = 0.53$ .. (i)

Also, just upstream of the bridge

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

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

or $h = \dfrac{70}{33u} - 1$ ...(ii)

Substituting for h from (ii) in (i) and rearranging

$u - 0.0617\ u^3 = 1.386$ $\therefore u = 1.68$ m/sec

Substituting for u in (i)

$$h = \frac{70}{33 \times 1.68} - 1$$
$$= 0.263 \text{ m}$$

Alternatively

Assume that h is more than $\frac{1}{4} D_d$ and apply the Weir Formula

$$Q = 1.706 \times C_w . L H^{3/2}$$

$$70 = 1.706 \times 0.92 \times 25 \times H^{3/2}$$

$\therefore\ H = 1.47$ m

Now, $H = D_u + \dfrac{u^2}{2g} = D_u$ (approx.)

$\therefore\ D_u = 1.47$ m approx.

Now, $Q = W \times D_u \times u$, *i.e.*, $70 = 33 \times 1.47\ u$

$\therefore\ u = 1.44;\ \dfrac{u^2}{2g} = 0.1055$ m

$$H = D_u + \frac{u^2}{2g}$$

i.e., $1.47 = D_u + 0.1055$

$D_u = 1.3645$ m

$$h = D_u - D_d = 1.3645 - 1.0 = 0.3645$$

Adopt $h = 0.365$ m.

Since h is actually more than $\frac{1}{4}D_d$, therefore the value of afflux arrived at by the Weir Formula is to be adopted.

18.7. **Example.** The unobstructed cross-sectional area of flow of a stream is 265 sq. m. and the width of flow is 90 m. A bridge of 4 spans of 18 m clear is proposed across it. Calculate the afflux when the discharge is 650 cu. m./sec.

$$W = 90 \text{ m};\ L = 72 \text{ m};\ D_d = \frac{265}{90} = 2.944 \text{ m}$$

The depth before the construction of the bridge is the depth downstream of the bridge after its construction. Hence $D_d = 2.944$ m.

$$\frac{L}{W} = \frac{72}{90} = 0.8. \text{ Therefore } C_0 = 0.877 \text{ and } e = 0.77.$$

By the Orifice Formula the discharge through the bridge

$$650 = 0.877 \times 4.43 \times 72 \times 2.94 \times \sqrt{h + 1.72\frac{u^2}{2g}}$$

$$650 = 825\sqrt{h + 0.0877\,u^2}$$

$$i.e.\ h + 0.0877\,u^2 = 0.62 \qquad \text{(i)}$$

Now, the discharge just upstream of the bridge

$$650 = 90\,(2.944 + h)_u$$

or $$h = \frac{65}{9u} - 2.944 \qquad \text{...(ii)}$$

Putting for h from (ii) in (i) and rearranging

$$u - 0.0246\,u^3 + 2.02$$

$$u = 2.34 \text{ m/sec}$$

Putting for u in (ii)

$$h = 0.137 \text{ m}$$

18.8. **Example.** **A bridge of 5 spans of 36 m each is proposed across a stream, whose unobstructed width is 250 m, slope 1/2000 and discharge 1800 cu.m./sec. Calculate the afflux ($n = 0.03$).**

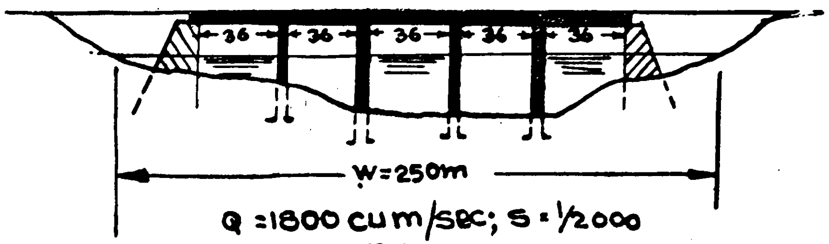


FIG - 17

We have first to find D_d.

$$Q = A.V = (R.P)V = R.W.V.$$

$$\therefore\ R.V = \frac{Q}{W} = \frac{1800}{250} = 7.2$$

Knowing $n = 0.03$; $S = 1/2000$, read velocity for various values of R from **Plate 3** and select that pair whose product is 7.2. **Thus** we get

$$R = 3.9$$

$$V = 1.85$$

Take $D_d = R = 3.9$ m

Now, $W = 250$ m; $L = 180$ m; $D_d = 3.9$ m

$$\frac{L}{W} = \frac{180}{250} = 0.72, \text{ therefore } C_o = 0.87;\ e = 0.90$$

By the Orifice Formula, the discharge through the bridge.

$$1800 = 0.87 \times 4.43 \times 180 \times 3.9\sqrt{h + 1.9\frac{u^2}{2g}}$$

$$i.e.,\quad h + 0.097\,u^2 = 0.442 \qquad \text{...(i)}$$

The discharge just upstream of the bridge

$$1800=250\,(3.9+h)\,u$$

i.e.,
$$h=\frac{7.2}{u}-3.9 \qquad \ldots\text{(ii)}$$

Put for h from (*ii*) in (*i*) and rearrange

$$u-0.0224\,u^3=1.66$$

$$\therefore\ u=1.8\ \text{m/sec.}$$

Put this value of u in (*ii*), we get:

$$h=0.442-0.314=0.128\ \text{m.}$$

19

Worked out Examples on Discharge Passed by Existing Bridges from Flood Marks

ARTICLE 19

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

19.1. Calculating Discharge by the Weir Formulae

Example. The unobstructed width of a stream is 70 m. The linear waterway of a bridge across it is 47 m. In a flood, the average depth of flow downstream of the bridge was 3.4 m and the afflux was 0.95 m. Calculate the discharge.

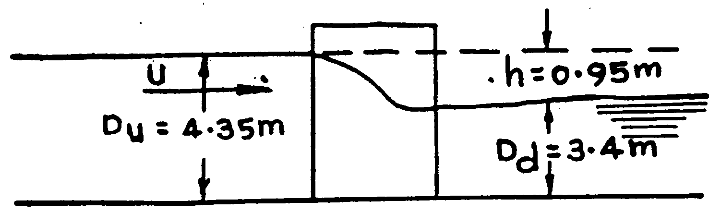


Fig. 18

$$\frac{h}{D_d} = \frac{0.95}{3.4} = 0.303$$

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

$$W = 70 \text{ m}; \quad L=47 \text{ m}; \quad h=0.95 \text{ m}$$

Let the velocity of approach be u m/sec. The discharge at a section just upstream of the bridge (Fig. 18)

$$Q=u \times 4.35\times 70=304.5u \qquad \text{...(i)}$$

The discharge through the bridge by the Weir Formula

$$Q=1.706\times 0.98\times 47\left(4.35+\frac{u^2}{19.6}\right)^{3/2}$$

$$=78.5\left(4.35+\frac{u^2}{19.6}\right)^{3/2} \qquad \text{...(ii)}$$

Equating values of Q from (*i*) and (*ii*)

$$304.5u=78.5\left(4.35+\frac{u^2}{19.6}\right)^{3/2}$$

Rearranging $u^{\frac{2}{3}}-0.0206u^2=1.76$

or $u=2.64$ m/sec.

Putting the value of u in (*i*) or (*ii*) we get Q.

$$Q=304.5\times2.64$$
$$=804 \text{ cu. m./sec.}$$

Try the Orifice Formula

$$\frac{L}{W}=\frac{47}{70}=0.67$$

$$\therefore \quad C_0=0.865\ ;\ e=0.95$$

Discharge through the bridge by the Orifice Formula

$$Q=0.85\times4.43\times47\times3.4\sqrt{0.95+1.95\frac{u^2}{19.6}}$$
$$=602\sqrt{0.95+0.1u^2} \qquad ...(i)$$

Discharge just upstream of the bridge

$$Q=70\times4.35\times u$$
$$=304.5\,u \qquad ...(ii)$$

Equating values of Q in (*i*) and (*ii*)

$$602\sqrt{(0.95+0.1u^2)}=304.5u$$

Simplifying and squaring

$$u^2=\frac{0.95}{0.155} \qquad \therefore \quad u=2.48 \text{ m/sec}$$

Substituting for u in (*i*) or (*ii*) we get Q.

$$Q=304.5\times2.48=756 \text{ cu. m./sec.}$$

This result is about the same as given by the first method. In fact, the Orifice Formula, with the recommended values of C_0 and e gives nearly correct results even where the conditions are appropriate for the Weir Formula. But the converse is not true.

19.2. Calculating Discharge by the Orifice Formula

Example. The unobstructed width of a stream is 130 m and the linear waterway of a bridge across it 105 m. During a flood the average depth of flow downstream of the bridge was 2.6 m and the afflux 0.10 m. Calculate the discharge.

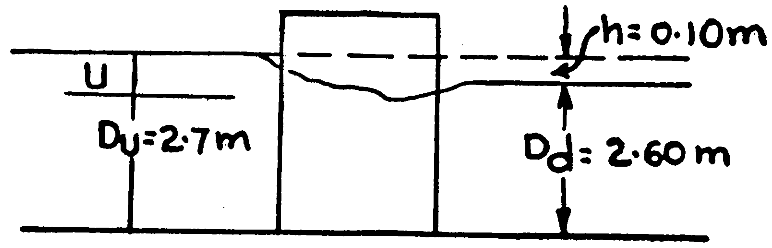


FIG - 19

Given : $W=130$ m; $L=105$ m; $h=0.1$ m; Let the velocity of approach be u. The discharge at a section just upstream of the bridge

$$Q=u\times2.7\times130=351u \qquad ...(i)$$

$$\text{Contraction}=\frac{a}{A}=\frac{L}{W}=\frac{105}{130}=0.80$$

Corresponding to this, $C_0=0.875$ and $e=0.72$. The discharge under the bridge, by the Orifice Formula,

$$Q=0.875\times4.43\times2.6\times105\sqrt{\left(0.1+1.72\frac{u^2}{19.6}\right)}$$
$$=1060\sqrt{0.1+0.0877u^2)} \qquad ...(ii)$$

Equating values of Q in (*i*) and (*ii*)

$$351\,u=1060\sqrt{0.10+0.0877u^2}$$

Squaring and rearranging

$$u^2=\frac{0.1}{0.0213} \qquad \therefore \quad u=2.17 \text{ m/sec.}$$

Subsitute for u in (i) or (ii) to get Q.

$$Q=351\times 2.17=760 \text{ cu. m./sec.}$$

19.3. **The Border-line Cases.** An example will now follow to illustrate what results are obtained by applying the Weir Formula and Orifice Formula to cases which are on the border line, *i.e.*, where the afflux is just $\frac{1}{4}$ D_d.

19.4. **Example.** A stream, whose unobstructed width is 230 m is spanned by a bridge whose linear waterway is 200 m. During a flood the average downstream depth was 2.5 m and the afflux was 0.065 m. Calculate the discharge.

$$\frac{h}{D_d}=\frac{0.065}{2.6}=0.25$$

Since h is nearly $\frac{1}{4}$ D_d, therefore both the Weir Formula and Orifice Formula should apply.

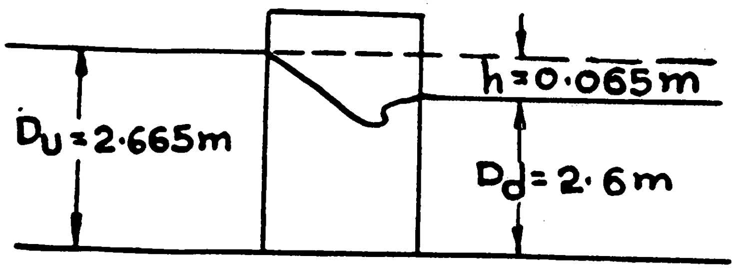


Fig. 20

By the Weir Formula

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

$$Q=230\times 2.665\times u=614u \qquad \text{...(i)}$$

The discharge through the bridge

$$Q=1.706\times 0.98\times 200\left(2.665+\frac{u^2}{19.6}\right)^{3/2} \qquad \text{...(ii)}$$

Equating values of Q from (i) and (ii)

$$614u=334\ (2.665+0.051\ u^2)^{3/2}$$

Rearranging

$$u^{\frac{2}{3}}-0.0374u^2=1.78 \qquad \therefore \quad u=3.2 \text{ m/sec.}$$

Put for u in (i)

$$Q=614\times 3.2=1935 \text{ cu. m./sec.}$$

By the Orifice Formula

$$\frac{a}{A}=\frac{L}{W}=\frac{200}{230}=0.87$$

$$\therefore \quad C_o=0.906 \qquad e=0.44$$

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

$$Q=230\times 2.665u=614u \qquad \text{...(i)}$$

The discharge under the bridge by the Orifice Formula

$$Q=0.906\times 4.43\times 200\times 2.6\ \ (0.065+0.0735u^2)^{\frac{1}{2}}$$

$$=2090\ (0.065+0.735\ u^2)^{\frac{1}{2}} \qquad \text{...(ii)}$$

Equating values of Q from (i) and (ii)

$$614\ u=2090\ (0.065+0.0735\ u^2)^{\frac{1}{2}}$$

$$0.0855\ u^2=0.065+0.0735\ u^2$$

Squaring and rearranging

$$u^2=\frac{0.065}{0.0120}=5.42$$

$$=u2.33 \text{ m/sec.}$$

Substituting for u in (i) or (ii) we get Q

$$Q=614\times 2.33=1430 \text{ cu. m./sec.}$$

20

Overtopping of the Banks

ARTICLE 20

OVERTOPPING OF THE BANKS

20.1. 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.

20.2. **Height of Approach Roads.** Consider the case where the bulk of the discharge is carried by the main channel 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 roads must be high enough not to be overtopped. If the velocity of the stream is Vm/sec., the water surface level, where it strikes the road embankment, will be $\frac{V^2}{19.6}$ m higher than the H.F.L. in the stream at the point, where the overflow starts. This arrangement is, therefore, normally feasible where the stream velocity is not immoderately high.

20.3. **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 such 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.

20.4. **Permanent 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.

20.5. **Retrogression of Levels**. Suppose water overflows a low bank somewhere upstream of the bridge and after passing through a relief culvert, rejoins the main stream somewhere lower down. When the flood in the main channel subsides, the ponded up water at the inlet of the subsidiary culvert gets a free overfall. Under such conditions deep erosion can take place. A deep channel is formed, beginning at the outfall in the main 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.

21

Feasibility of Pipe and Box Culverts Flowing Full

ARTICLE 21

FEASIBILITY OF PIPE AND BOX CULVERTS FLOWING FULL

21.1. Many a plain is one 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.

21.2. In such contourless regions the road runs across wide but shallow dips and therefore the most straightforward way of handling the surface flow is to provide suitable dips (*i.e.*, causeways) in the longitudinal profile of the road and let water pass over them.

21.3. 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 not infrequently 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.

21.4. 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 the 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 $\left[A=\frac{Q}{V}\right]$ will be prohibitively large. In such cases, the design has to be based on an increased velocity of flow through the culvert and to create that 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 up to 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 and 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 Article 22.

21.5. 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.

21.6. In the past, use was extensively made of empirical formulae which gave the ventway area required for a culvert to drain a given catchment area. Dun's Drainage Table is one of that 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 others 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 areas.

21.7. 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.

21.8. Nor should high headwalls be provided for retaining deep over-fills; instead, the length of the culverts should be increased suitably so that the road embankment, with its natural side slopes, is accommodated without high retaining headwalls.

21.9. 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 overfall, a suitable cistern and baffie wall must be added for the dissipation of energy and stilling of the ensuing current.

22

Hydraulics of the Pipe and Box Culverts Flowing Full

133

ARTICLE 22

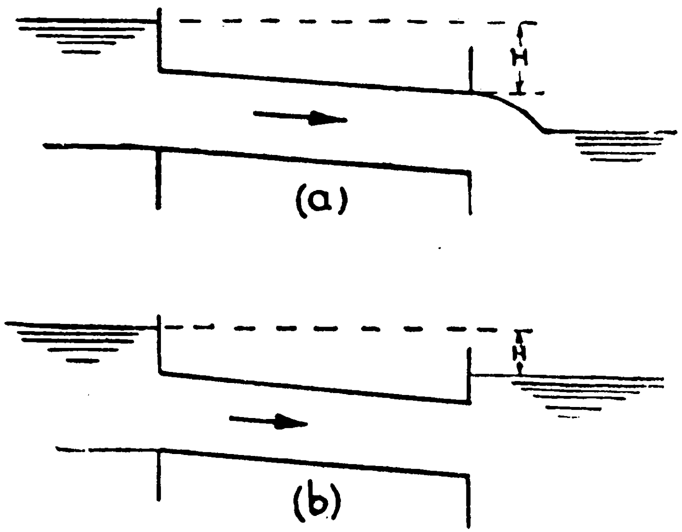
HYDRAULICS OF THE PIPE AND BOX CULVERTS FLOWING FULL

22.1. **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 ventway 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.

22.2. **Surface Level of the Tail Race.** It is essential that the H.F.L. in the outfall channel near the exit of the culvert should be known. This may be taken as the H.F.L. 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.

22.3. **The Operating Head when the Culverts Flow Full.** In this connection the cases that have to be considered are illustrated in Fig. 21. In each case the inlet is submerged and the culvert is flowing 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



The Operating Head 'H' when the Culvert Flows Full
Fig. 21

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 is higher".

22.4. **The Velocity Generated by "H".** The operating head "H" is utilised 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.

If the velocity through the pipe is v, the head expended in generating it 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 pipe is expressed as a fraction K_f of $\frac{v^2}{2g}$. From this it follows that

$$H=[1+K_e+K_f]\,\frac{v^2}{2g} \qquad \ldots(22.4)$$

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.

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

$$\left.\begin{aligned} K_e &= 0.08 \text{ for bevelled or bell-mouthed entry} \\ &= 0.505 \text{ for sharp edged entry} \end{aligned}\right\} \qquad \ldots(22.5_a)$$

As regards K_f it is a function of the length L of the culvert, its hydraulic mean radius R, and the co-efficient of rugosity n of its surface.

The following relationship exists between K_f and n :

$$K_f=\frac{14.85n^2}{R^{\frac{1}{3}}}\cdot\frac{L}{R} \qquad \ldots(22.5_b)$$

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 :

$$K_f-\frac{0.00334L}{R^{1.33}} \qquad \ldots(22.5_c)$$

The graphs in Fig. 22 are based on Equation 22.5_c. For a culvert of known sectional area and length, K_f can be directly read from these graphs.

22.6. **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 of entry and the roughness of its wetted surface. In general, K_e increases with an increase in the size of the inlet.

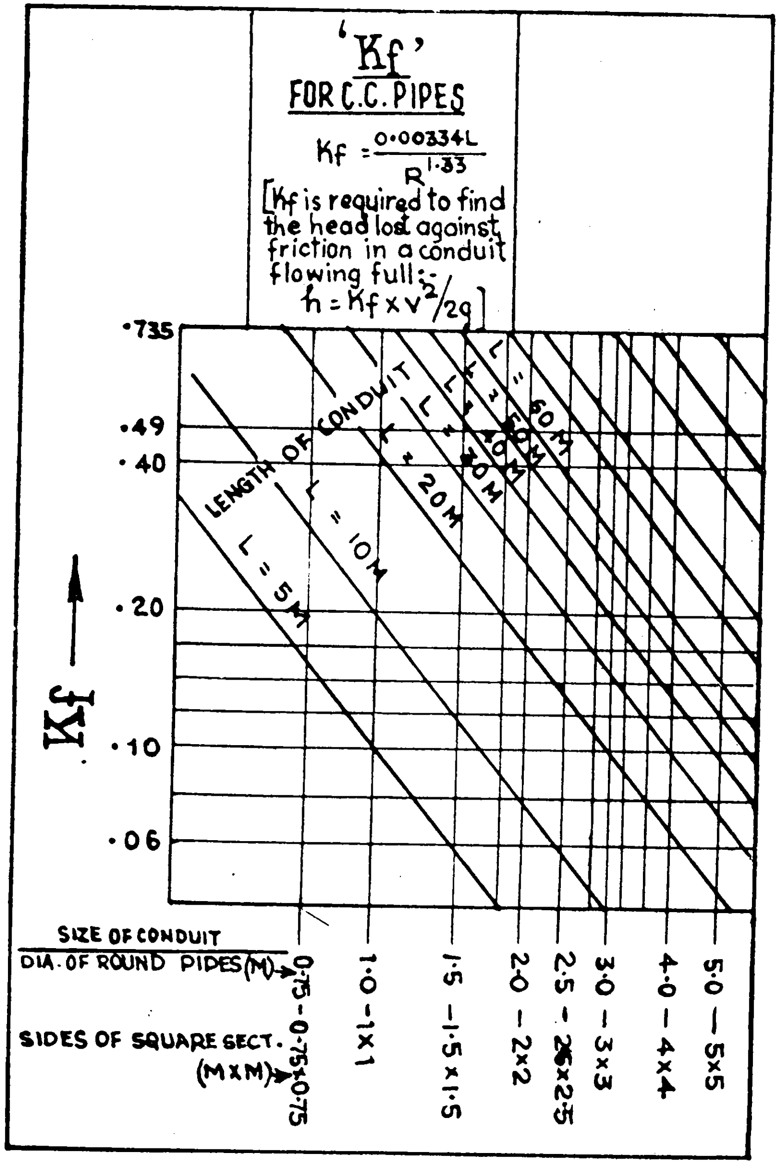


Fig. 22

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 bell-mouthed 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.

Consequently, it has been suggested that the values of K_e and K_f should be as given in Table 5.

TABLE 5

Values of K_e and K_f [9]

Entry & friction co-efficients	*Circular pipes*		*Rectangular culverts*	
	Square entry	*Bevelled entry*	*Square entry*	*Bevelled entry*
K_e=	$1.107\ R^{0.5}$	0.1	$0.572\ R^{0.3}$	0.05
K_f=	$0.00394L/R^{1.2}$	$0.00394L/R^{1.2}$	$0.00335L/R^{1.25}$	$0.00335L/R^{1.25}$

22.7. **Design Calculations**

We have said that

$$H = \left[1 + K_e + K_f \right] \frac{v^2}{2g}$$

i.e., $v=4.43\left(\frac{H}{1+K_e+K_f}\right)^{\frac{1}{2}}$

Hence,

$$Q=A\times 4.43\left(\frac{H}{1+K_e+K_f}\right)^{\frac{1}{2}} \qquad ...(22.7)$$

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 5. Then, calculate Q from Equation (22.7). If this equals the design discharge, the assumed size of the culvert is correct. If not, assume a fresh value of D and repeat.

22.8. Design Chart (Plate 13)

Equation (22.7) may be written as

$$Q=\lambda\sqrt{2g\ H} \qquad ...(22.8_a)$$

where

$$\lambda=\frac{A}{(1+K_e+K_f)^{\frac{1}{2}}} \qquad ...(22.8_b)$$

It is obvious that all components of λ in Equation (22.8_b) 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 16**, curves have been constructed from equation (22.8_a) 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 λ can be taken for any known values of (1) length, (2) diameter in case of circular pipes or sides in case of rectangular pipes, and (3) 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 7.

The use of **Plate 16** renders the design procedure very simple and quick. Examples will now follow to illustrate.

22.9. 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 16**, the Table for circular pipes with rounded entry.

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

$$\lambda=0.618$$

Now refer to the curves in the same **Plate**. For $\lambda=0.618$ and $H=1$ m.

$$Q=2.72 \text{ cu. m./sec.}$$

22.10. Example. Design a culvert consisting of cement concrete circular pipes with bevelled entry and flowing full, given : (see Fig. 23).

Discharge	= 10 cu.m./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

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

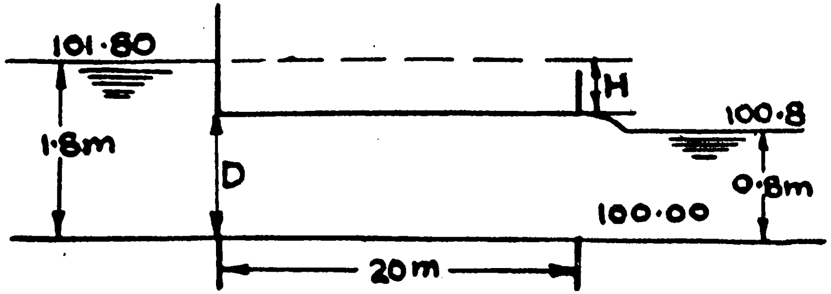


Fig. 23

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 16**, Q = (1) 2.54 cu.m./sec.; (2) 3.5 cu.m./sec.

Number of pipes required $10/Q$ = (1) 3.93; (2) 2.85
say 4 say 3

Hence 4 pipes of 1 metre diameter will suit.

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Appendix

HEAVIEST RAINFALL IN ONE HOUR (mm)

(All hours in I.S.T.)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
1. **Agartala** (1953-1966)												
mm	14.0	32.0	32.6	39.6	65.5	60.0	66.0	61.3	54.9	51.3	33.0	7.9
Date	10	21	30	29	19	15	29	27	27	2	23	30
Time	14-15	12-13	23-24	7-8	17-18	13-14	12-13	13-14	23-24	16-17	11-12	3-4
Year	1957	1958	1959	1962	1960	1961	1964	1965	1964	1956	1966	1956
2. **Ahmedabad** (1951-1966)												
mm	3.6	2.5	2.0	17.5	11.8	42.5	59.7	61.0	80.0	25.9	20.0	1.3
Date	6	27	20	5	13	17	3	9	2	1	25	29
Time	12-13	8-9	19-20	17-18	17-18	19-20	19-20	22-23	0-1	15-16	8-9	20-21
Year	1953	1956	1954	1963	1963	1960	1956	1954	1958	1955	1963	1960
3. **Aligarh** (1950)												
mm	8.1	0	5.1	2.8	5.6	24.4	50.8	—	27.4	2.3	0	0.5
Date	24	—	14	1	30	23	5	—	1	11	—	24
Time	16-17	—	20-21	23-24	17-18	0-1	12-13	—	14-15	22-23	—	22-23
4. **Allahabad** (1948-1966)												
mm	16.5	13.2	29.5	19.0	16.0	60.2	54.5	74.8	64.5	25.5	9.7	6.3
Date	28	3	21	28	29	30	28	16	10	22	1	31
Time	20-21	23-24	20-21	21-22	4-5	11-12	2-3	13-14	5-6	18-19	13-14	8-9
Year	1958	1956	1950	1962	1959	1951	1962	1961	1956	1959	1956	1953
5. **Amini Devi** (1964-66)												
mm	5.7	7.5	0	5.8	12.2	37.3	30.9	49.5	52.7	40.0	24.4	24.0
Date	25	12	—	20	29	1	10	13	6	7	12	7
Time	16-17	14-15	—	16-17	12-13	5-6	0-1	1-2	1-2	5-6	6-7	0-1
Year	1966	1965	—	1966	1965	1966	1966	1965	1964	1966	1966	1966
6. **Amritsar** (1951-66)												
mm	14.5	10.7	9.9	6.9	15.0	28.0	51.3	50.0	74.4	32.5	9.5	12.5
Date	13	23	15	18	11	27	24	26 30 19	2	4	5	12
Time	12-13	21-22	22-23	18-19	15-16	1-2	5-6	14-15 3-4 11-12	1-2	2-3	23-24	20-21
Year	1961	1954	1952	1960	1966	1960	1956	1961, 1962 1966	1964	1955	1959	1963
7. **Anantapur** (1965-66)												
mm	1.7	0	0	9.1	35.3	44.0	14.9	38.2	22.4	34.7	18.4	21.8
Date	30	—	—	20	1	7	25	29	19	2	17	9
Time	21-22	—	—	22-23	16-17	1-2	16-17	22-23	5-6	23-24	7-8	19-20
Year	1966	—	—	1965	1966	1966	1966	1966	1966	1966	1966	1965

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
8. Asansol (1953-66)												
mm	13.7	13.4	16.0	39.4	64.5	60.5	60.0	86.0	72.4	47.8	13.2	6.6
Date	21	19	11	25	14	28	9	20	26	20	7	28
Time	17-18	17-18	16-17	16-17	12-13	14-15	17-18	20-21	14-15	16-17	13-14	6-7
Year	1953	1965	1956	1963	1956	1965	1964	1960	1956	1958	1955	1954
9. Aurangabad (Chikalthana) (1952-1966)												
mm	16.7	5.5	16.3	6.2	27.5	60.5	44.2	33.5	37.1	23.0	21.0	20.2
Date	7	5	20	11	20	15	30	21	20	4	25	2
Time	18-19	13-14	17-18	18-19	16-17	22-23	19-20	2-3	22-23	14-15	18-19	8-9
Year	1965	1961	1954	1964	1961	1955	1954	1965	1952	1959	1958	1966
10. Bagdogra (1962-1965)												
mm	0	4.6	20.7	20.0	37.1	70.2	60.0	32.5	42.0	17.7	11.7	0.3
Date	—	7	20	29	8	13	29	15	14	5	4	8
Time	—	6-7	7-8	14-15	4-5	0-1	15-16	15-16	2-3	23-24	3-4	18-19
Year	—	1965	1964	1963	1963	1963	1964	1963	1964	1962	1963	1963
11. Bagra Tawa (1952-66)												
mm	12.8	9.4	18.8	11.2	44.5	59.7	38.5	63.2	63.0	30.0	12.0	8.9
Date	11	14	2	4	30	26	18	6	15	2	25	8
Time	3-4	7-8	16-17	15-16	3-4	18-19	12-13	23-24	17-18	10-11	22-23	11-12
Year	1966	1955	1957	1960	1959	1955	1960	1964	1961	1961	1963	1956
12. Bangalore Aerodrome (1954-66)												
mm	18.5	13.7	18.0	29.3	55.0	32.8	31.7	50.0	57.9	53.9	22.9	17.2
Date	31	2	24	5	5	6	2	30	13	3	7	8
Time	22-23	1-2	21-22	15-16	16-17	17-18	17-18	22-23	21-22	16-17	16-17	22-23
Year	1959	1959	1954	1965	1963	1960	1965	1964	1955	1956	1957	1965
13. Bangalore Central Observatory (1950-66)												
mm	7.8	20.0	12.5	35.7	44.8	61.0	59.2	50.8	48.0	50.8	27.4	39.0
Date	31	1	7	3	13	1	11	10	8	21 16	1	8
Time	21-22	22-23	22-23	0-1	21-22	22-23	17-18	0-1	22-23	14-15 23-24	22-23	21-22
Year	1959	1959	1957	1961	1961	1952	1952	1965	1964	1952 1956	1950	1965
14. Barakachar (1952-55)												
mm	4.6	3.6	3.6	19.8	3.1	32.0	50.8	32.3	53.3	18.5	0	0.5
Date	21	22	23	4	12	19	22	24	26	25	—	30
Time	1-2	4-5	16-17	23-24	17-18	19-20	19-20	19-20	0-1	20-21	—	3-4
Year	1953	1952	1955	1952	1955	1953	1954	1955	1954	1955	—	1954
15. Barahkshetra (1952-66)												
mm	15.2	15.4	32.3	32.9	54.9	88.5	68.0	78.0	61.5	83.8	10.0	7.8
Date	11	8	24	13	15	30	27	17	26	2	2	3
Time	19-20	5-6	16-17	0-1	22-23	23-24	21-22	13-14	11-12	17-18	8-9	0-1
Year	1957	1961	1953	1965	1958	1966	1964	1961	1963	1953	1963	1966

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
16. Barhi (1953-55)												
mm	13.5	7.5	3.7	8.8	22.0	56.0	39.0	41.9	49.2	31.5	10.8	8.7
Date	21	10	29 30	22	30	26	23	9	14	23	4	2
Time	5-6	8-9	17-18 21-22	21-22	15-16	21-22	13-14	21-22	17-18	22-23	17-18	23-24
Year	1955	1964	1965	1964	1964	1965	1964	1953	1964	1963	1963	1966
17. Barmul (1952-58)												
mm	3.1	24.1	11.9	24.9	34.5	53.3	66.5	45.7	74.9	26.9	6.3	3.8
Date	21	24	15	2	24	21	19	26	25	15	10	29
Time	23-24	20-21	19-20	18-19	14-15	19-20	18-19	18-19	20-21	14-15	20-21	21-22
Year	1953	1958	1956	1952	1956	1952	1953	1953	1956	1953	1953	1954
18. Baroda (1948-66)												
mm	4.6	4.6	6.6	2.5	37.6	71.4	52.8	66.5	44.7	42.4	12.7	2.0
Date	6	4	16	13	28	22	26	5	9	10	25	5
Time	17-18	10-11	18-19	15-16	23-24	4-5	7-8	0-1	19-20	10-11	11-12	1-2
Year	1953	1961	1962	1962	1956	1966	1957	1956	1960	1956	1963	1962
19. Barrackpore (1957-65)												
mm	22-1	14.7	34.3	31.5	54.5	43.0	48.5	58.2	56.5	54.0	20.5	0
Date	11	26	1	27	31	1	30	6	10	16	11	—
Time	0-1	8-9	20-21	20-21	20-21	15-16	18-19	2-3	12-13	16-17	23-24	—
Year	1957	1957	1960	1958	1959	1962	1964	1957	1961	1959	1958	—
20. Bhimkund (1957-66)												
mm	5.6	16.0	30.0	41.7	28.0	50.0	62.0	52.0	60.0	30.0	7.3	1.0
Date	26	7	9	2	27	23	27	8	11	14	3	8,2,1
Time	8-9	10-11	12-13	15-16	18-19	18-19	15-16	9-10	2-3	15-16	13-14	23-24 3-4 3-4
Year	1962	1961	1962	1965	1959	1961	1959	1963	1963	1966	1963	1962, 63,66
21. Bhopal (Bairagarh) (1953-66)												
mm	15.5	6.5	26.9	9.3	61.0	59.9	70.0	71.5	61.2	24.9	11.2	5.8
Date	11	5	24	12	30	26	1	7	25	2	15	8
Time	17-18	19-20	16-17	15-16	17-18	20-21	9-10	16-17	22-23	21-22	2-3	3-4, 14-15
Year	1966	1962	1957	1962	1956	1957	1964	1963	1961	1955	1966	1956
22. Bhubaneshwar (1964-1966)												
mm	13.7	19.3	11.0	20.0	25.0	46.0	30.0	45.5	43.0	30.0	16.8	5.8
Date	3	17	29	27	16	23	10	13	23	8	22	17
Time	13-14	16-17	14-15	19-20	18-19	14-15	19-20	17-18	18-19	17-18	11-12	5-6
Year	1966	1966	1965	1964	1964	1966	1964	1966	1964	1966	1966	1965

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
23. Bhuj (1964-1966)												
mm	1.7	0	5.5	0.6	0	16.1	48.8	46.0	17.5	7.5	0	0
Date	21	—	31	11	—	22	19	26	6	6	—	—
Time	4-5	—	17-18	6-7	—	16-17	21-22	15-16	16-17	17-18	—	—
Year	1964	—	1965	1965	—	1966	1966	1965	1966	1966	—	—
24. Bishungarh (1953-66)												
mm	19.6	8.0	14.2	18.5	40.0	36.0	59.5	43.0	39.0	59.0	13.2	6.3
Date	9	10	5	24	11	13	5	8	27	22	2	28
Time	15-16	8-9	23-24	14-15	13-14	15-16	0-1	22-23	22-23	0-1	5-6	4-5
Year	1957	1964	1957	1962	1963	1963	1961	1963	1963	1959	1956	1954
25. Bokaro (1951-66)												
mm	23.6	15.8	23.9	32.5	36.6	32.7	48.0	48.5	57.9	44.8	5.6	6.9
Date	9	7	9	19	15	13	11	1	6	4	11	29
Time	17-18	18-19	17-18	17-18	14-15	19-20	18-19	13-14	21-22	17-18	1-2	17-18
Year	1957	1961	1957	1951	1958	1966	1951	1953	1953	1959	1953	1954
26. Bombay (Colaba) (1948-66)												
mm	5.5	15.0	2.8	7.3	27.0	62.5	68.1	53.3	128.5	43.2	31.7	35.0
Date	26	5	26	28	16	27	14	5	22	11	22	5
Time	4-5	5-6	6-7	2-3	5-6	1-2	7-8	6-7	14-15	7-8	3-4	4-5
Year	1962	1961	1951	1959	1960	1958	1949	1957	1949	1948	1948	1962
27. Bombay (Santa Cruz) (1952-66)												
mm	6.3	11.6	0	1.0	17.8	83.3	91.4	55.9	57.0	44.2	10.0	16.4
Date	24	5	—	26	16	19	17	6	9	8	10	5
Time	7-8	10-11	—	3-4	5-6	5-6	6-7	11-12	23-24	2-3	17-18	5-6
Year	1965	1961	—	1954	1956	1953	1952	1954	1962	1960	1964	1962
28. Calcutta (Alipore) (1948-66)												
mm	14.2	33.2	24.4	43.9	54.9	51.0	61.5	59.0	48.3	41.7	24.9	4.6
Date	12,22	23	13	25	20	12	2	13	27	9	1	28
Time	3-4, 3-4	18-19	20-21	19-20	18-19	9-10	6-7	3-4	9-10	11-12	19-20	20-21
Year	1957, 1961	1958	1948	1952	1954	1959	1965	1966	1963	1948	1950	1954
29. Chambal (1963-66)												
mm	1.0	0.2	1.5	1.8	21.5	83.5	40.8	58.9	44.0	4.0	11.6	0.1
Date	14	12	30	29	26	20	6	26	5	16	21	26
Time	16-17	23-24	14-15	19-20	20-21	19-20	15-16	14-15	14-15	18-19	14-15	3-4
Year	1963	1966	1963	1963	1964	1965	1964	1965	1963	1963	1963	1963
30. Chandwa (1953-66)												
mm	13.5	10.8	12.2	6.2	21.3	55.3	46.0	37.1	29.0	38.1	5.8	20.5
Date	21	7	3	1	10, 9	10	23	15	12	20	2	8
Time	19-20	16-17	22-23	5-6	13-14 7-8	18-19	15-16	19-20	7-8	17-18	3-4	3-4
Year	1959	1961	1958	1965	1956 1963	1960	1960	1953	1959	1958	1956	1962

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
31. Cherrapunji (1940-61)												
mm	9.2	25.4	51.8	87.4	101.4	108.7	127.0	76.2	106.7	44.2	16.8	18.3
Date	22	10	30	29	4	17	11	4	15	4	18	6
Time	12-13	4-5	2-3	8-9	8-9	6-7	0-1	19-20	0-1	11-12	23-24	21-22
Year	1959	1950	1951	1948	1956	1949	1952	1957	1951	1951	1950	1954
32. Coimbatore (1963-66)												
mm	7.8	0	2.4	36.6	80.0	11.5	33.5	19.6	42.8	31.8	22.8	55.0
Date	14	—	25	18	24	26	28	30	24	16	23	7
Time	20-21	—	15-16	15-16	22-23	14-15	0-1	14-15	17-18	17-18	19-20	23-24
Year	1966	—	1963	1965	1965	1963	1964	1963	1966	1964	1965	1963
33. Dadeldhura (1959-66)												
mm	8.5	8.0	9.0	26.6	19.6	26.7	36.1	25.3	22.2	13.0	11.5	5.4
Date	20	24	20	13	17	8	8	6	2	13	7	13
Time	4-5	23-24	0-1	16-17	17-18	5-6	13-14	19-20	12-13	23-24	15-16	21-22
Year	1965	1962	1965	1963	1964	1961	1961	1962	1965	1961	1963	1963
34. Dhanbad (1952-1961)												
mm	23.1	13.7	12.9	14.5	34.0	58.0	73.7	54.3	43.0	30.5	4.8	7 6
Date	21	7	27	26	24	25	3	11	6	13	9 8	28
Time	16-17	20-21	14-15	3-4	14-15	14-15	1-2	21-22	11-12	11-12	0-1 11-12	6-7
Year	1953	1961	1955	1958	1954	1958	1953	1958	1958	1952	1953 1955	1954
35. Dhanwar (1957)												
mm	19.3	2.5	7.9	0	0	19.6	17.8	10.9	43.9	5.3	0	0
Date	15	22	22	—	—	26	20	7	15	12	—	—
Time	14-15	15-16	21-22	—	—	10-11	19-20	1-2	17-18	15-16	—	—
36. Dholpur (1962-66)												
mm	3.9	2.2	5.8	3.7	10.0	40.0	46.6	33.5	34.4	24.5	0	4.1
Date	15	12	4	6	12	16	25	21	19	18	—	29
Time	0-1	11-12	19-20, 22-23	14-15	9-10	4-5	17-18	13-14	16-17	0-1	—	5-6
Year	1963	1965	1962	1963	1966	1966	1966	1966	1964	1965	—	1966
37. Dibrugarh (Mohanbari) (1953-66)												
mm	9.4	25.6	26.0	36.0	50.0	92.5	56.5	50.0	60.0	36.8	20.3	9.3
Date	21	13	22	15	25	25	14	9,3	30	6	5	30
Time	23-24	6-7	21-22	3-4	0-1	23-24	2-3	1-2 1-2	0-1	0-1	23-24	14-15
Year	1961	1960	1964	1961	1960	1965	1965	1958 1960	1961	1956	1965	1959

38. Dum Dum (1948-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	13.5	30.2	28.2	66.0	50.0	63.0	68.1	56.9	64.9	40.1	13.2	6.1
Date	30	6	7	20	8	2	9	6	7	8	26	28 29
Time	7-8	2-3	18-19	16-17	14-15	17-18	13-14	11-12	2-3	13-14	7-8	21-22 16-17
Year	1959	1948	1949	1962	1963	1953	1953	1955	1960	1949	1951	1954

39. Dumri (1954-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	9.7	7.6	10.2	24.5	48.8	42.7	40.1	59.2	53.5	66.0	9.4	6.8
Date	29	27	26	22	31	6	15,25	25	26	22	25	2
Time	18-19	19-20	14-15	15-16	18-19	14-15	17-18 14-15	1-2	21-22	0-1	4-5	11-12
Year	1959	1958	1955	1964	1954	1958	1955	1956	1960	1959	1966	1966

40. Durgapur (1957-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	8.5	16.4	10.0	12.0	52.5	58.0	52.0	90.0	64.0	41.2	4.9	1.0
Date	22	5	20	21	12	8	28	21	18	21	3	1
Time	1-2	21-22	19-20	18-19	23-24	16-17	15-16	6-7	17-18	15-16	18-19	23-24
Year	1959	1961	1965	1962	1964	1969	1959	1964	1959	1964	1963	1966

41. Gangtok (1956-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	26.7	16.2	40.6	53.5	60.8	81.2	33.9	41.7	35.7	37.5	12.6	5.0
Date	5	24	22	22	16	9	10	30	22	14	17	14
Time	16-17	18-19	15-16	16-17	14-15	16-17	19-20	21-22	0-1	22-23	15-16	14-15
Year	1958	1963	1957	1963	1959	1966	1958	1958	1965	1966	1961	1961

42. Gannavaram (1963-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	1.3	0	10.0	19.0	26.0	39.0	30.8	51.4	61.4	38.6	19.8	2.3
Date	3	—	26	2	21	21	23	27	7	24	4	31
Time	6-7	—	12-13 13-14	20-21	1-2	21-22	18-19	15.16	6.7	3-4	13-14	15-16
Year	1966	—	1963	1965	1965	1964	1963	1966	1964	1963	1966	1964

43. Gauhati (1955-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	5.5	8.2	20.0	33.0	32.0	67.0	60.5	51.5	60.1	21.7	11.7	6.1
Date	30	23	30	30	9	19	9	28	26	8	30	15
Time	22-23	23-24	2-3	1-2	19-20	22-23	17-18	19-20	4-5	19-20	15-16	17-18
Year	1959	1964	1965	1961	1966	1958	1955	1960	1961	1959	1955	1956

44. Gaya (1948-66) (Except 61)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	14.5	7.1	28.2	15.7	48.0	49.2	40.6	69.9	58.1	38.6	12.7	5.1
Date	9	24	5	29	28	27	3	1	15	11	27	29
Time	23-24	21-22	18-19	2-3	16-17	19-20	22-23	14-15	16-17	21-22	4-5	18-19
Year	1957	1954	1957	1962	1963	1965	1953	1957	1962	1956	1948	1956

45. Gorkha (1956-61-64-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	11.7	7.0	5.6	29.3	27.9	43.9	62.0	43.2	49.4	25.9	49.3	7.1
Date	9	13	30	13	31	16	6	30	12	6	4	11
Time	22-23	13-14	9-10	17-18	23-24	0-1	9-10	22-23	3-4	5-6	19-20	18-19
Year	1957	1966	1958	1965	1957	1965	1965	1957	1966	1966	1965	1956

46. Gwalior (1963-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	3.5	4.0	1.3	1.5	1.8	25.0	29.5	62.5	48.5	0.3	3.0	9.5
Date	14	12	29	1	27	26	16	6	19	17	17	29
Time	20-21	20-21	20-21	22-23	23-24	13-14	13-14	14-15	18-19	20-21	2-3	8-9
Year	1963	1966	1965	1965	1964	1966	1963	1966	1964	1965	1963	1966

47. Hazaribagh (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	15.7	21.8	15.5	20.3	29.3	67.6	49.5	78.0	56.6	33.0	9.3	7.4
Date	16	13	5	15	7	12	21	10	2	5	16	29
Time	16-17	15-16	16-17	14-15	0-1	14-15	8-9	16-17	18-19	16-17	17-18	19-20
Year	1953	1958	1957	1952	1961	1953	1959	1966	1963	1953	1966	1956

48. Hirakud (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	8.9	33.0	24.9	11.5	20.0	82.3	64.0	68.0	55.0	40.6	5.5	3.0
Date	16	27	3	8	13	20	2	14	27	25	5	1
Time	3-4	2-3	22-23	4-5	21-22	19-20	23-24	16-17	4-5	3-4	14-15	18-19
Year	1957	1964	1958	1961	1963	1957	1953	1961	1964	1957	1961	1966

49. Hyderabad (Begumpet) (1948-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	3.6	20.8	38.3	40.0	30.2	42.9	43.7	101.6	32.1	47.0	32.8	24.0
Date	23	20	11	29	13	9	24	1	18	27	5	2
Time	0-1	14-15	13-14	18-19	23-24	0-1	23-24	0-1	19-20	13-14	20-21	21-22
Year	1953	1950	1957	1958	1965	1952	1953	1954	1960	1961	1948	1966

50. Imphal (1956-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	8.6	16.1	11.1	17.8	41.0	48.1	25.3	44.0	26.7	23.6	15.3	9.6
Date	12	26	3	24	9	16	9	11	14	17	13	15
Time	10-11	3-4	18-19	11-12	14-15	18-19	11-12	18-19	3-4	22-23	16-17	6-7
Year	1957	1964	1961	1966	1963	1958	1958	1959	1966	1956	1961	1965

51. Indore (1963-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	7.3	18.3	5.5	1.3	2.7	59.5	40.0	30.0	33.5	18.8	18.4	5.2
Date	11	26	26	30	30	26	6	25	18,1	18	13	12
Time	15-16	21-22	17-18	21-22	17-18	16-17	15-16	13-14	14-15 16-17	19-20	15-16	13-14
Year	1966	1963	1964	1963	1964	1964	1963	1963	1964 1966	1963	1966	1965

52. Jabalpur (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	25.9	12.7	26.7	10.0	50.0	77.3	73.7	67.5	52.8	28.0	21.6	21.3
Date	24	27	26	30	4	22	20	27	16	9	1	1
Time	16-17	16-17	13-14	19-20	17-18	20-21	0-1	8-9	4-5	21-22	4-5	12-13
Year	1955	1958	1957	1963	1966	1965	1952	1963	1954	1959	1956	1966

53. Jagdalpur (1953-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
mm	12.8	15.3	23.9	50.0	39.4	66.0	73.1	52.6	48.0	50.3	15.6	33.7
Date	12	20	23	26	13	27	29	1	30	2	4	2
Time	4-5	20-21	16-17	16-17	20-21	19-20	14-15	14-15	11-12	19-20	18-19	13-14
Year	1966	1962	1965	1964	1956	1959	1954	1954	1955	1954	1961	1962

54. Jaipur (1950-59)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	12.2	9.7	16.5	5.1	12.2	37.1	57.1	45.5	53.6	54.6	7.9	1.3
Date	27	20	5	10	21	15	2	11	27	3	28	21
Time	18-19	5-6	5-6	3-4	16-17	1-2	14-15	4-5	21-22	7-8	14-15	16-17
Year	1956	1954	1957	1958	1950	1951	1956	1955	1954	1956	1958	1958

55. Jaipur (Sanganer Aerodrome) (1959-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	6.5	24.5	5.3	9.1	24.8	48.0	49.0	54.0	23.8	22.5	12.6	2.3
Date	1	12	30	24	27	18	7	15	6	7	5	29
Time	10-11	14-15	20-21	13-14	19-20	5-6	18,19	13-14	2-3	13-14	17-18	21-22
Year	1961	1965	1966	1963	1964	1966	1962	1959	1961	1961	1959	1960

56. Jamshedpur (1948-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	14.0	29.3	19.3	22.6	54.4	85.9	53.5	61.7	53.2	34.3	17.8	11.9
Date	27	7	24	24	20	10	17	29	7	22	25	30
Time	21-22	21-22	18-19	18-19	15-16	0-1	16-17	21-22	8-9	22-23	23-24	20-21
Year	1949	1961	1951	1962	1949	1949	1964	1953	1964	1959	1948	1956

57. Jamui (1955-65)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	12.9	5.1	18.5	26.4	26.7	56.9	44.0	59.5	50.0	39.9	10.0	4.7
Date	9	7	30	22	31	25	19	13	25	4	28	12
Time	5-6	10-11	14-15	13-14	14-15	5-6	0-1	23-24	14-15	13-14	11-12	11-12
Year	1957	1961	1965	1964	1959	1957	1962	1964	1957	1957	1965	1961

58. Jawai Dam (1962-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	2.0	1.1	6.0	5.5	21.2	22.3	98.0	59.8	40.5	12.2	7.9	0.4
Date	2	12	30	22	13	28	19	20	19	17	25	26
Time	10-11	17-18	11-12	17-18	3-4	15-16	22-23	17-18	16-17	17-18	7-8	3-4
Year	1965	1965	1963	1963	1964	1963	1962	1965	1964	1963	1963	1963

59. Jharsuguda (1954-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	15.5	19.6	14.5	14.5	31.2	50.0	63.0	71.1	53.5	77.0	5.3	11.0
Date	16	24	29	11	13	22	3	25	6	3	2	11
Time	1-2	9-10	19-20	18-19	16-17	15-16	21-22	13-14	13-14	22-23	7-8	14-15
Year	1957	1958	1965	1966	1956	1958	1954	1957	1958	1954	1956	1961

60. Jodhpur (1948-65)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	13.2	4.1	17.0	5.4	17.8	27.9	60.0	52.0	50.8	25.7	7.5	2.1
Date	18	20	3	9	30	27	7	18	24	2	28	29
Time	16-17	23-24	23-24	22-23	3-4	17-18	16-17	4-5	13-14	22-23	11-12	20-21
Year	1948	1948	1962	1961	1951	1951	1964	1964	1954	1956	1958	1960

61. Tonk Dam Site (1952-53)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
mm	5.8	4.6	7.1	2.1	0	24.9	37.1	46.0	19.1	29.5	0	0
Date	20	5	15	15	—	29	3	24	4	15	—	—
Time	3-4	22-23	18-19	8-9	—	18-19	21-22	18-19	19-20	19-20	—	—
Year	1953	1953	1952	1953	—	1952	1952	1952	1952	1952	—	—

62. Kathmandu (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	5.6	16.0	11.2	23.5	24.4	44.0	41.7	33.2	35.3	27.4	5.1	3.5
Date	28,9	11	22	25	9	19	19	12	15	1	2	18
Time	22-23 23-24	16-17	4-5	16-17	23-24	1-2	19-20	22-23	14-15	2-3	22-23	6-7
Year	1956,57	1956	1956	1962	1956	1965	1952	1961	1963	1961	1952	1961

63. Khalari (1963-1966)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	0	6.0	10.0	19.2	22.5	30.0	37.5	63.2	21.8	30.5	6.5	7.5
Date	—	10	31	1	23	13	23	13	9	20	25	1
Time	—	9-10	18-19	14-15	14-15	13-14	23-24	16-17	0-1	23-24	3-4	19-20
Year	—	1964	1965	1965	1965	1963	1965	1966	1964	1964	1966	1966

64. Khijrawan (1958-1961)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	12.0	11.4	17.5	5.7	14.0	29.8	36.5	28.0	40.0	66.0	8.6	8.7
Date	7	24	20	7	24	13	1	23	23	5	23	12
Time	8-9	15-16	5-6	0-1	18-19	17-18	15-16	10-11	11-12	15-16	13-14	2-3
Year	1960	1958	1960	1961	1958	1958	1960	1961	1958	1960	1958	1961

65. Kodaikanal (1948-1966)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	35.6	20.0	38.1	68.6	83.3	24.9	40.6	30.0	40.0	65.5	30.8	25.1
Date	16	24	20	20	6	5	22	14	4	9	7	18,4
Time	19-20	2-3	21-22	18-19	16-17	17-18	12-13	15-16	19-20	18-19	0-1	15-16 18-19
Year	1948	1962	1962	1957	1964	1953	1964	1964	1964	1953	1959	1957 1961

66. Konar (1960-1964)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	5.6	17.4	5.8	14.0	32.5	58.7	41.4	50.0	41.0	27.1	2.5	8.0
Date	16	7	5	18	25	20	19	31	30	2	3	8
Time	3-4	18-19	21-22	13-14	12-13	16-17	13-14	10-11	21-22	20-21	14-15	11-12
Year	1963	1961	1962	1962	1961	1964	1964	1963	1963	1962	1963	1962

67. Luchipur (1963-1966)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	10.0	5.5	19.8	55.0	36.5	62.5	29.0	45.5	36.5	30.0	11.0	3.8
Date	3	20	20	25	26	29	9	14	7	21	4	1
Time	23-24	18-19	19-20	20-21	22-23	22-23	16-17	15-16	13-14	21-22	18-19	22-23
Year	1966	1965	1965	1964	1963	1965	1964	1965	1964	1963	1963	1966

68. Lucknow (Amausi) (1953-1966)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	12.9	8.6	14.5	10.2	40.0	50.0	70.0	63.7	59.3	39.0	8.0	9.5
Date	16	1	21	24	21	21	9	2	14	2	1	17
Time	1-2	1-2	17-18	16-17	1-2	13-14	3-4	13-14	6-7	19-20 23-24	21-22	23-24
Year	1953	1961	1960	1963	1964	1964	1960	1955	1958	1958	1963	1961

69. Madras (Meenambakkam) (1948-1966)

	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	24.5	8.4	13.2	35.3	52.6	49.9	36.4	62.2	52.6	49.0	61.0	43.7
Date	10	3	24	11	19	22	14	28	17	9	4	1
Time	7-8	0-1	20-21	14-15	3-4	3-4	23-24	2-3	4-5	0-1	2-3	16-17
Year	1963	1959	1963	1951	1952	1961	1966	1950	1956	1963	1957	1952

	Jan.	Feb	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
70. **Madras** (Nungambakkam) (1957-66)												
mm	26.2	15.5	10.0	33.9	24.5	48.2	38.8	38.8	44.7	74.5	47.7	30.4
Date	10	3	25	23	6	22	12	15	30	7	11	7
Time	7-8	10-11	13-14	10-11	2-3	3-4	22-23	3-4	0-1	2-3	21-22	10-11
Year	1963	1959	1963	1963	1958	1961	1961	1966	1960	1959	1961	1965
71. **Mahabaleshwar** (1948-1966)												
mm	5.1	0.5	19.3	34.8	30.2	50.8	45.2	38.5	41.2	45.2	29.0	16.8
Date	22	21	6	16	26	23	10	9	1	1	20	5
Time	10-21	14-15	17-18	17-18	14-15	23-24	22-23	16-17	10-11	15-16	21-22	9-10
Year	1948	1948	1948	1959	1956	1951	1965	1963	1958	1951	1951	1962
72. **Maithon** (1957-1966)												
mm	6.0	7.0	20.0	35.0	26.0	31.0	54.0	38.4	52.0	25.0	7.0	2.8
Date	3	19	23	25	11	30	30	8	10	20	3,4	2
Time	22-23	17-18	14-15	19-20	15-16.	9-10	20-21	3-4	16-17	13-14	18-19 19-20	11-12
Year	1966	1965	1965	1964	1958	1963	1965	1963	1966	1958	1963	1966
73. **Mangalore** (1953-1966)												
mm	5.6	0	31.5	24.9	71.8	58.0	43.5	47.0	29.5	56.0	38.5	43.3
Date	7	—	27	29	21	26	10	15	27	17	22	10
Time	2-3	—	23-24	19-20	0-1	5-6	4-5	12-13	2-3	3-4	18-19	14-15
Year	1954	—	1963	1956	1965	1961	1964	1962	1955	1963	1958	1965
74. **Marmagao** (1964-1966)												
mm	1.9	0.4	0.5	0	60.3	51.8	39.3	25.7	32.6	39.0	42.0	29.0
Date	4	1	1	—	3	6	8	20	27	9	13	11
Time	5-6	2-3	12-13	—	10-11	20-21	2-3	14-15	20-21	5-6	3-4	10-11
Year	1965	1966	1964	—	1966	1964	1964	1965	1965	1964	1966	1965
75. **Mawsynram** (1960-1966)												
mm	10.0	12.6	45.2	42.0	86.4	127.0	118.5	103.0	100.0	32.0	27.0	2.5
Date	4	22	1	29	19	11	7	2	13	20	5	12
Time	8-9	21-22	21-22	12-13	3-4	22-23	20-21	0-1	20-21	4-5	14-15	15-16
Year	1966	1964	1961	1962	1950	1966	1964	1964	1960	1964	1963	1966
76. **Minicoy** (1963-1966)												
mm	14.5	13.5	7.7	64.3	34.3	42.0	54.2	22.2	70.0	64.1	38.5	31.7
Date	9	10	13	24	4	3	24	10	26	23	19	5
Time	9-10	15-16	15-16	1-2	21-22	23-24	9-10	19-20	13-14	1-2	15-16	20-21
Year	1963	1963	1963	1963	1963	1964	1963	1965	1965	1965	1963	1965
77. **Mukhim** (1956-1966)												
mm	4.0	5.7	8.5	14.7	22.6	26.2	36.7	57.3	43.3	22.6	9.7	6.6
Date	4	11	19	14	28	30	14	20	15	7	6	31
Time	22-23	23-24	1-2	15-16	22-23	22-23	3-4	3-4	15-16	17-18	11-12	4-5
Year	1959	1959	1966	1963	1956	1964	1965	1965	1960	1961	1959	1960

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
78. **Nagpur** (1948-1966)												
mm	29.0	9.4	28.5	19.8	37.8	78.0	65.4	51.8	53.6	31.5	12.2	21.8
Date	6	21	29	25	20	27	27	27	4	12	5	4
Time	17-18	4-5	16-17	1-2	18-19	3-4	9-10	15-16	5-6	14-15	15-16	17-18
Year	1960	1950	1957	1966	1962	1954	1960	1955	1954	1958	1948	1962
79. **Nandurbar** (1952-1966)												
mm	6.9	0	8.8	33.0	24.5	34.0	72.5	36.0	32.4	50.0	20.0	5.7
Date	14	—	28	29	16	24	2	29	18	9	3	4
Time	15-16	—	15-16	16-17	16-17	6-7	22-23	14-15	17-18	22-23	2-3	22-23
Year	1963	—	1963	1958	1965	1957	1963	1958	1964	1959	1959	1962
80. **New Delhi** (1948-1966)												
mm	28.7	16.6	19.1	5.1	13.5	35.5	73.0	49.5	79.3	26.4	8.4	8.6
Date	15	5	1	2	30	25	22	7	7	9	20	29
Time	18-19	23-24	18.19	8-9	15-16	18-19	16-17	10-11	8-9	2-3	13-14	18-19
Year	1953	1961	1952	1951	1950	1966	1965	1960	1948	1956	1957	1963
81. **North Lakhimpur** (1957-1966)												
mm	9.6	10.0	21.3	22.9	51.5	71.1	55.0	50.8	65.0	36.8	19.6	23.8
Date	20	16	28	16	8	9	21	24	28	10	3	17
Time	4-5	11-12	6-7	13-14	3-4	3-4	7-8	3-4	3-4	2-3	0-1	1-2
Year	1961	1960	1964	1964	1958	1959	1959	1964	1960	1965	1963	1965
82. **Okha** (1963-1966)												
mm	4.7	0	0	0	0	53.0	76.1	24.0	6.1	5.3	7.4	0.5
Date	2	—	—	—	—	22	19	27	6	15	25	29
Time	3-4	—	—	—	—	0-1	22-23	15-16	7-8	18-19	5-6	23-24
Year	1965	—	—	—	—	1966	1966	1964	1966	1963	1963	1964
83. **Okhaldunga** (1952-1966)												
mm	7.1	18.0	15.7	25.0	43.4	47.6	51.8	49.4	9.0	30.0	14.6	3.6
Date	10	22	24	13	20	27	10	19	21	3	12	16
Time	0-1	14-15	16-17	22-23	23-24	22-23	17-18	15-16	0-1	16-17	16-17	11-12
Year	1957	1954	1953	1963	1954	1961	1956	1963	1961	1964	1961	1955
84. **Palganj** (Giridih) (1953-57)												
mm	9.9	5.3	9.1	7.1	51.3	40.1	43.4	55.9	55.9	24.1	2.8	23.6
Date	21	2	27	10	28	13	14	27	6	9	7	29
Time	7-8	13-14	13-14	14-15	11-12	6-7	13-14	15-16	22-23	21-22	12-13	16-17
Year	1955	1956	1955	1955	1956	1956	1956	1957	1954	1954	1955	1954
85. **Panaji** (1965-66)												
mm	0.2	0.9	0	28.2	42.8	20.4	30.7	11.6	54.0	18.0	20.4	18.6
Date	4	1	—	16	3	16	19	31	22	2	14	11
Time	5-6	1-2	—	23-24	10-11	16-17	22-23	17-18	3-4	18-19	4-5	10-11
Year	1965	1966	—	1965	1966	1966	1965	1966	1965	1966	1966	1965
86. **Panambur** (Manalore Project) (1965-66)												
mm	4.0	0	0	11.0	29.2	37.3	31.4	26.0	21.5	33.2	22.5	17.5
Date	14	—	—	20	30	27	29	19	3	15	6	11
Time	2-3	—	—	1-2	0-1	0-1	18-19	21-22	2-3	1-2	15-16	10-11
Year	1966	—	—	1966	1965	1966	1965	1965	1966	1966	1965	1965

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.

87. Panchat Hill (1953-1966)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	17.3	10.0	15.3	25.3	41.6	70.6	65.5	60.0	62.0	38.7	9.9	12.7
Date	21	12	29	29	11	25	23	24	16	23	7	28
Time	1-2	12-13	19-20	4-5	18-19	15-16	17-18	16-17	17-18	22-23	10-11	6-7
Year	1954	1959	1965	1962	1962	1958	1954	1963	1953	1958	1955	1954

88. Pathankot (1957-1961)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	9.1	8.1	11.2	4.6	23.5	30.4	68.1	47.8	56.4	12.4	9.9	9.3
Date	20	6	29	2	29	21	5	8	6	6	16	22
Time	23-24	18-19	19-20	16-17 21-22	7-8	4-5	10-11	3-4	12-13	7-8	5-6	14-15
Year	1960	1961	1958	1958	1959	1961	1959	1960	1961	1959	1961	1958

89. Patna (1962-1965)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	14.0	17.5	4.3	10.1	30.0	38.5	32.3	59.0	45.0	55.9	2.6	3.0
Date	22	19	6	16	21	27	30	25	25	9	2	6
Time	5-6	18-19	5-6	22-23	8-9	23-24	5-6	2-3	5-6	8-9	4-5	4-5
Year	1964	1962	1963	1964	1964	1964	1964	1963	1965	1964	1963	1962

90. Pokhara (1956-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	15.4	15.8	11.2	27.3	31.5	61.5	73.0	49.0	59.8	40.0	38.8	8.5
Date	19	27	7	30	26	25	13	30	5	5	4	8
Time	17-18	18-19	18-19	15-16	14-15	20-21	23-24	14-15	19-20	16-17	16-17	20-21
Year	1961	1966	1957	1963	1962	1957	1966	1966	1965	1962	1965	1962

91. Poona (1948-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	10.2	0.2	23.4	34.5	40.2	43.4	39.2	~~35.0~~	42.3	47.1	19.8	7.3
Date	22	5	16	5	22	12	10	16	26	4	14,22	5
Time	8-9	13-14	18-19	18-19	15-16	21-22	22-23	18-19	17-18	21-22	0-1, 17-18	8-9
Year	1948	1961	1954	1953	1962	1953	1966	1965	1959	1958	1948, 1951	1962

92. Port Blair (1951-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	28.7	31.5	24.0	37.5	54.4	46.6	60.5	60.2	49.5	58.4	37.5	36.8
Date	7	9	10	27	25	9	20	3	23	21	23	22
Time	3-4	22-23	9-10	23-24	3-4	14-15	6-7	20-21	5-6	11-12	21-22	13-14
Year	1955	1956	1961	1961	1955	1965	1964	1953	1954	1951	1964	1965

93. Punasa (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	16.0	12.2	16.0	8.9	3.7	54.9	61.0	75.4	64.1	16.0	14.3	13.8
Date	10	9	10	7	16	20	21	4	2	11	25	4
Time	16-17	19-20	17-18	21-22	3-4	22-23	4-5	22-23	7-8	1-2	17-18	3-4
Year	1966	1952	1960	1957	1963	1955	1961	1955	1966	1961	1963	1962

94. Pupanki (Chas Road) (1953-56)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	21.3	3.1	2.0	7.1	11.2	26.2	27.4	67.1	77.2	16.3	5.1	6.1
Date	16	1	26	19	6	23	10	11	7	1	13	28
Time	2-3	2-3	15-16	15-16	15-16	17-18	13-14	12-13	8-9	16-17	19-20	5-6
Year	1953	1953	1955	1953	1953	1953	1955	1955	1954	1954	1953	1954

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.

95. Putki (1960-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	9.0	9.5	24.0	32.0	30.0	46.0	34.0	45.2	49.0	30.0	10.1	0
Date	3	10	29	25	7	30	16	3	2	3	4	—
Time	19-20	12-13	18-19	17-18	17-18	23-24	16-17	0-1	15-16	15-16	18-19	—
Year	1966	1964	1965	1964	1964	1963	1960	1953	1963	1963	1963	—

96. Raipur (1962-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	3.2	7.8	18.6	9.2	15.7	51.4	40.0	48.9	49.0	21.7	11.9	18.4
Date	6	28	31	17	19	18	11	23	20	19	23	1
Time	10-11	7-8	3-4	21-22	17-18	15-16	15-16	20-21	15-16	16-17	15-16	14-15
Year	1965	1963	1965	1962	1962	1966	1965	1965	1965	1964	1966	1966

97. Ramgarh (1953-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	19.8	16.1	16.6	15.6	26.5	43.6	45.0	42.8	55.6	36.5	5.1	11.0
Date	31	3	31	25	27	7	17	3	23	21	10	8
Time	15-16	6-7	17-18	16-17	17-18	6-7	22-23	18-19	17-18	10-11	21-22	8-9
Year	1953	1956	1959	1964	1959	1961	1957	1963	1965	1964	1953	1962

98. Sagar Island (1948-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	20.3	34.5	24.5	48.8	51.4	54.9	92.7	94.0	88.3	57.4	27.2	29.2
Date	17	6	24	7	2	10	18	15	21	30	18	29
Time	5-6	9-10	0-1	22-23	22-23	10-11	14-15	4-5	11-12	10-11	8-9	7-8
Year	1953	1948	1965	1949	1962	1950	1957	1963	1964	1962	1950	1954

99. Shillong (1957-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	9.1	8.4	36.0	28.0	36.0	43.2	31.2	30.4	57.5	22.5	6.5	5.3
Date	10	12	29	20	27	30	18	27	2	22	6	9
Time	9-10	21-22	15-16	15-16	13-14	13-14	2-3	21-22	18-19	23-24	11-12	15-16
Year	1957	1960	1964	1964	1966	1958	1960	1961	1958	1965	1961	1957

100. Sindri (1963-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	1.7	5.4	18.5	23.5	29.0	77.5	36.5	50.0	31.5	20.0	13.3	6.3
Date	3	20	29	25	21	30	9	24	15	8	4	1
Time	21-21	16-17	19-20	18-19	12-13	23-24	19-20	14-15	17-18	1-2	17-18	21-22
Year	1966	1965	1965	1964	1964	1963	1964	1963	1963	1963	1963	1966

101. Sonepur (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	10.0	13.2	11.5	10.2	13.5	38.0	76.0	78.2	61.0	34.5	2.9	9.0
Date	27	4	10	26,3	25	19	19	11	16	17	22	1
Time	16-17	9-10	5-6	16-17 22-23	18-19	18-19	6-7	21-22	18-19	17-18	5-6	18-19
Year	1966	1956	1962	1965 1958	1954	1966	1965	1953	1952	1958	1966	1966

102. Srinagar (1953-1966)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	—	—	7.4	10.0	10.7	8.8	17.3	22.0	10.0	10.2	7.8	4.1
Date	—	—	11	3	30	21	31	10	18	3	6	27
Time	—	—	15-16	4-5	1-2	16-17	9-10	5-6	17-18	0-1	20-21	12-13
Year	—	—	1954	1964	1966	1963	1966	1960	1965	195[illegible]	1959	1953

103. Shanti Niketan (1960-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	5.3	17.5	23.2	20.0	46.5	42.0	41.5	49.0	38.0	88.0	4.0	1.7
Date	4	6	29	26	11	23	28	29	7	21	21	1
Time	1-2	23-24	21-22	19-20	15-16	2-3	15-16	9-10	16-17	17-18	22-23	23-24
Year	1966	1961	1965	1964	1962	1962	1961	1960	1964	1964	1966	1966

104. Taplejung (1954-1956)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	7.4	11.0	18.1	16.7	37.0	49.8	32.9	31.0	28.0	59.2	5.5	7.2
Date	11	16	8	25	13	5	21	4	5	2	4,1	14
Time	19-20	1-2	20-21	15-16	16-17	17-18	7-8	16-17	1-2	14-15	4-5 15-16	19-20
Year	1957	1960	1963	1965	1959	1965	1962	1958	1959	1956	1963 1965	1963

105. Tehri (1956-1966)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	9.9	11.1	14.2	10.4	28.7	23.7	40.6	35.6	33.0	10.4	6.6	12.0
Date	9	4	19	7	29	6	19	24	17	9	21	27
Time	12-13	14-15	20-21	9-10	10-11	16-17	16-17	0-1	2-3	5-6	2-3	11-12
Year	1957	1965	1965	1957	1956	1963	1956	1960	1962	1956	1957	1962

106. Tezpur (1957-1964)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	10.0	7.6	19.1	25.1	53.3	52.0	63.0	50.0	48.5	32.8	10.6	4.0
Date	23	20	30	28	29	20	26	10	30	15	9	7
Time	14-15	21-22	16-17	10-11	3-4	5-6	23-24	0-1	6-7	7-8	3-4	7-8
Year	1959	1957	1959	1958	5957	1963	1963	1963	1961	1963	1959	1964

107. Thikri (1952-1966)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	10.9	0.8	3.0	7.7	30.0	55.9	39.3	61.0	61.0	35.6	8.0	30.0
Date	27	5	10	17	17	23	25	13	1	1	28	3
Time	20-21	6-7	19-20	18-19	16-17	20-21	19-20	15-16	14-15	19-20	20-21	0-1
Year	1955	1961	1960	1959	1960	1956	1960	1953	1954	1955	1958	1962

108. Tiliava Dam Site (1956-1966)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	9.3	9.9	8.0	16.8	21.6	50.0	33.0	80.0	40.0	35.6	12.8	2.8
Date	29	7	6	24	14	26	15	11	14,24	1	16	29
Time	22-23	15-16	21-22	12-13	13-14	21-22	15-16	16-17	17-18 4-5	22-23	15-16	19-20 20-21
Year	1959	1961	1960	1962	1956	1965	1962	1966	1964 1965	1959	1966	1956

109. Tiruchirappalli (1954-1966)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	29.7	4.7	27.4	41.6	41.1	30.0	46.3	55.5	68.7	77.7	31.2	20.2
Date	8	3	22	23	4	10	8	6	27	26	1	7
Time	17-18	14-15	1-2	19-20	20-21	18-19	20-21	21-22	21-22	20-21	17-18	14-15
Year	1963	1962	1962	1959	1954	1966	1965	1958	1962	1955	1961	1962

110. Trivandrum (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	53.0	59.5	96.3	71.0	51.8	50.5	25.0	16.9	40.0	68.0	45.2	69.8
Date	30	23	24	9	17	7	3	26	25	18	14	3
Time	0-1	17-18	6-7	20-21	14-15	4-5	3-4	2-3	3-4	2-3	14-15	15-16
Year	1962	1962	1954	1962	1957	1953	1964	1962	1966	1964	1953	1965

111. Vengurla (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	2.9	0	0.3	9.9	57.1	66.0	42.2	52.3	61.0	40.5	40.1	43.5
Date	4	—	15	18	20	12	15	7	2	3	5	2
Time	4-5	—	2-3	18-19	4-5	15-16	18-19	23-24	6-7	2-3	19-20	20-21
Year	1965	—	1954	1952	1955	1960	1953	1958	1966	1964	1962	1962

112. Veraval (1952-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	10.2	5.9	0	0.3	1.8	50.3	121.5	64.4	48.0	55.0	3.2	8.0
Date	31	4	—	14	11	18	2	26	12	13	25	4
Time	19-20	13-14	—	7-8	10-11	18-19	11-12	4-5	14-15	19-20	8-9	12-13
Year	1961	1961	—	1956	1966	1962	1960	1961	1958	1959	1963	1962

113. Visakhapatnam (1951-66)

	Jan.	Feb.	Mar.	Apr.	May	Jun.	July	Aug.	Sept.	Oct.	Nov.	Dec.
mm	40.0	40.6	16.0	45.5	27.4	63.0	48.5	56.7	52.0	47.0	45.2	30.0
Date	3	7	30	18	7	16	20	24	19	18	21	7
Time	4-5	9-10	15-16	22-23	6-7	5-6	20-21	4-5	2-3	14-15	19-20	21-22
Year	1966	1961	1957	1963	1955	1960	1951	1965	1959	1961	1966	1960

PLATE I

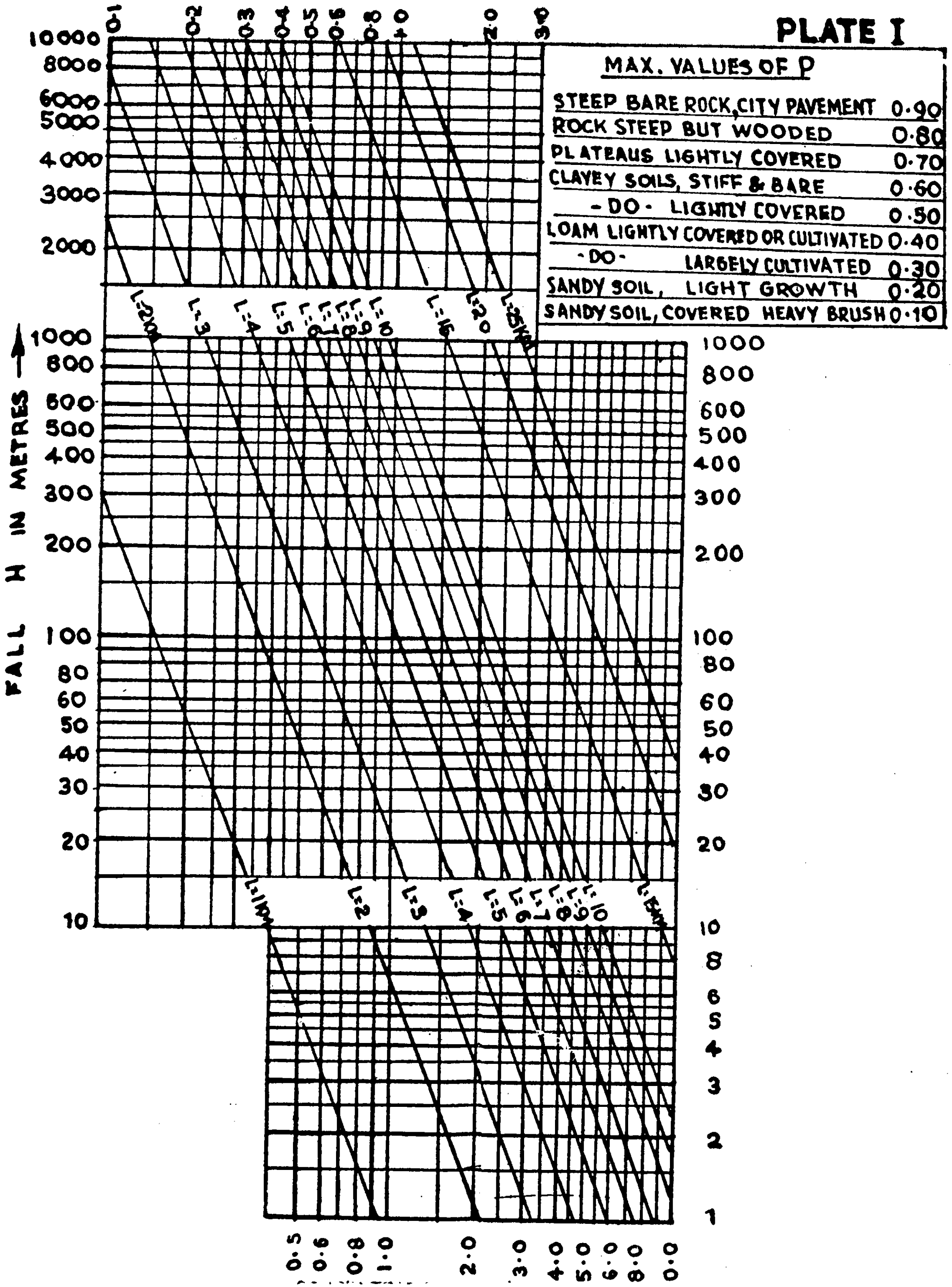
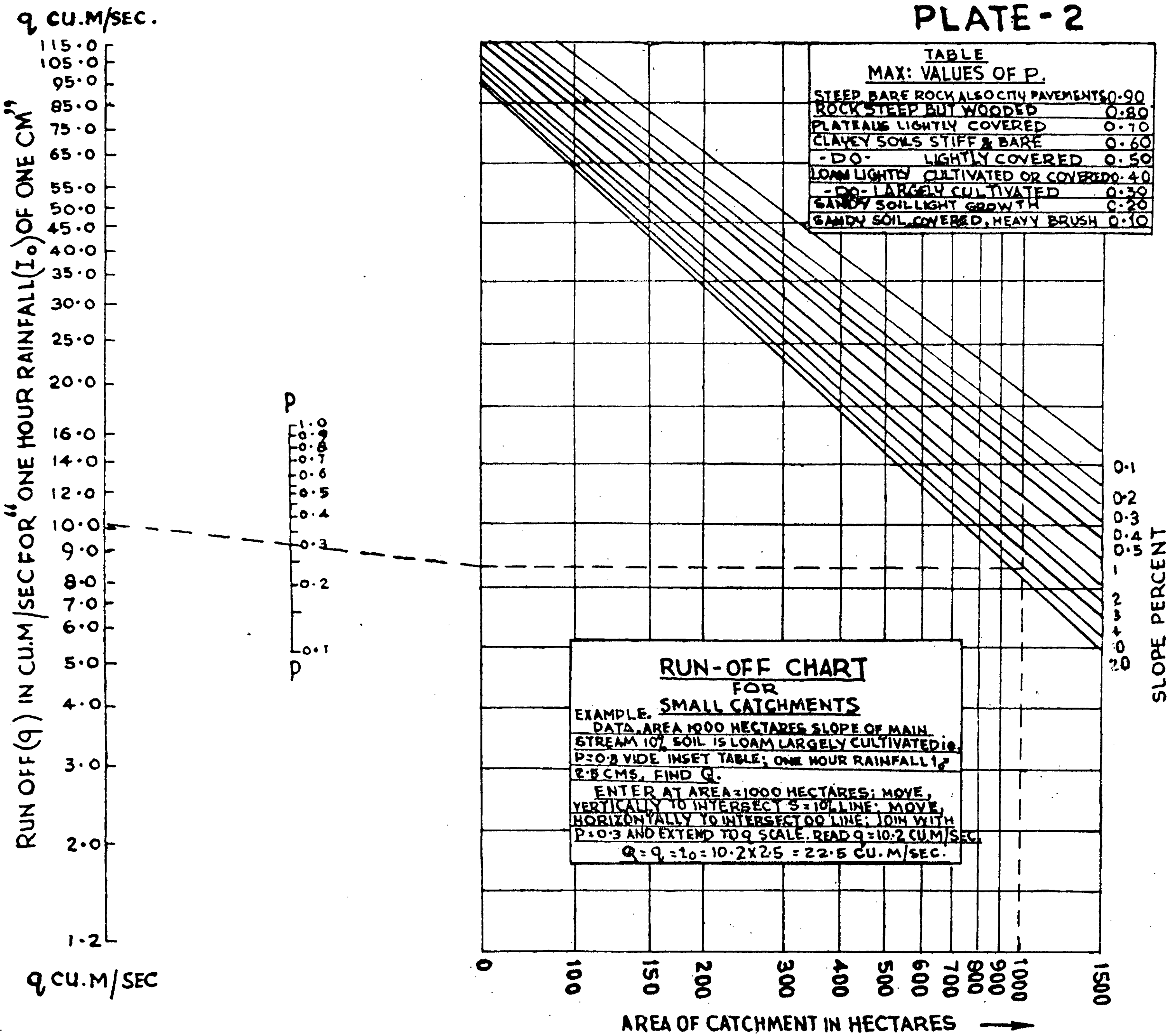
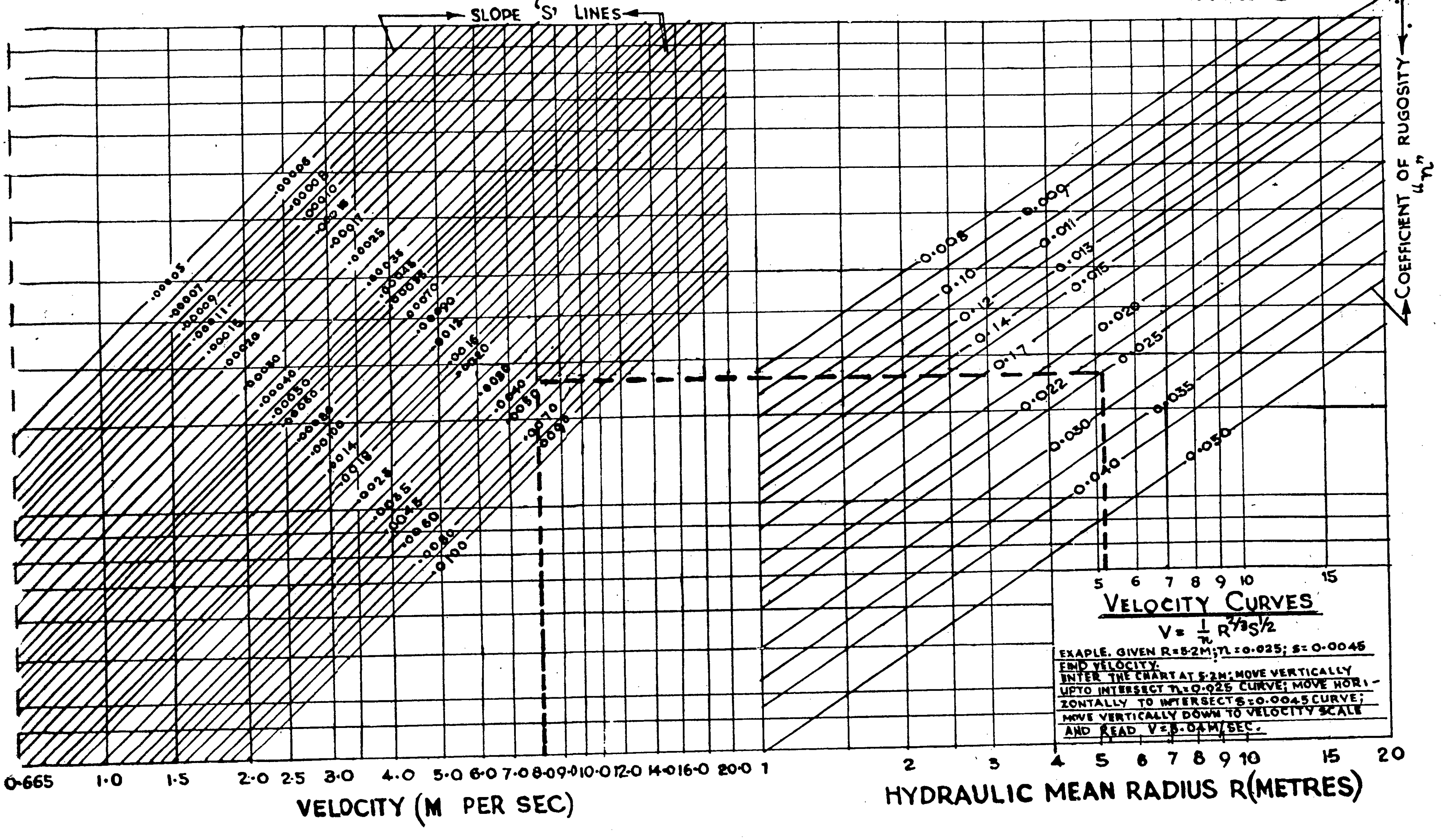


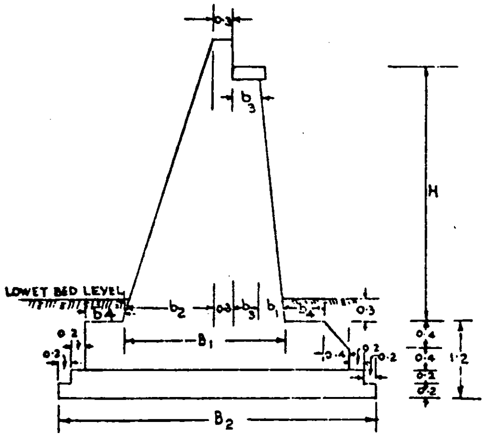
CHART FOR TIME OF CONCENTRATION



RUN-OFF CHART FOR SMALL CATCHMENTS

PLATE-3

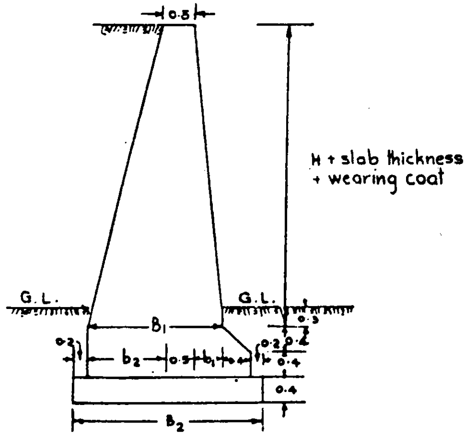

SLOPE 'S' LINES
COEFFICIENT OF RUGOSITY "n"
VELOCITY CURVES
$V = \frac{1}{n} R^{2/3} S^{1/2}$
EXAPLE. GIVEN R=5·2M; n=0·025; S=0·0045
FIND VELOCITY.
ENTER THE CHART AT 5·2M; MOVE VERTICALLY
UPTO INTERSECT n=0·025 CURVE; MOVE HORI-
ZONTALLY TO INTERSECT S=0·0045 CURVE;
MOVE VERTICALLY DOWN TO VELOCITY SCALE
AND READ V=8·0AM/SEC.
0·665 1·0 1·5 2·0 2·5 3·0 4·0 5·0 6·0 7·0 8·0 9·0 10·0 12·0 14·0 16·0 20·0
VELOCITY (M PER SEC)
1 2 3 4 5 6 7 8 9 10 15 20
HYDRAULIC MEAN RADIUS R(METRES)



SECTION OF ABUTMENT

TABLE OF DIMENSIONS FOR ABUTMENT

SPAN	6M AND 5M					4M, 3M, 2M, 1·5 AND 1M					
H	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
b_2	0·6	0·85	1·0	1·2	1·4	0·5	0·7	0·95	1·1	1·25	1·4
b_3	0·4	0·4	0·4	0·4	0·4	0·3	0·3	0·3	0·3	0·3	0·3
b_4	-	0·1	0·2	0·4	0·6	-	-	0·1	0·2	0·3	0·5
B_1	1·5	1·8	2·0	2·25	2·5	1·25	1·5	1·8	2·0	2·2	2·4
B_2	2·7	3·2	3·6	4·25	4·9	2·45	2·7	3·2	3·6	4·0	4·6



WING WALL SECTION AT HIGH END

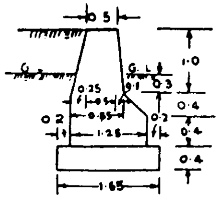
SPAN	UPTO 2 METRES						3 METRES					
H	1·50	2·00	2·50	3·00	3·50	4·00	1·50	2·00	2·50	3·00	3·50	4·00
b_1	0·18	0·23	0·28	0·33	0·38	0·43	0·19	0·24	0·29	0·34	0·39	0·44
b_2	0·45	0·57	0·70	0·82	0·95	1·07	0·46	0·59	0·71	0·84	0·96	1·09
B_1	1·13	1·30	1·48	1·65	1·83	2·00	1·15	1·33	1·50	1·68	1·85	2·03
B_2	1·93	2·10	2·28	2·45	2·63	2·80	1·95	2·13	2·30	2·48	2·65	2·83

SPAN	4 METRES						5 METRES				
H	1·50	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
b_2	0·48	0·60	0·73	0·85	0·98	1·10	0·62	0·75	0·87	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
B_2	1·97	2·14	2·32	2·49	2·67	2·84	2·17	2·35	2·52	2·70	2·88

SPAN	6 METRES				
H	2·00	2·50	3·00	3·50	4·00
b_1	0·25	0·30	0·35	0·40	0·45
b_2	0·63	0·75	0·88	1·00	1·13
B_1	1·38	1·56	1·73	1·90	2·08
B_2	2·18	2·36	2·53	2·70	2·88

NOTES:-

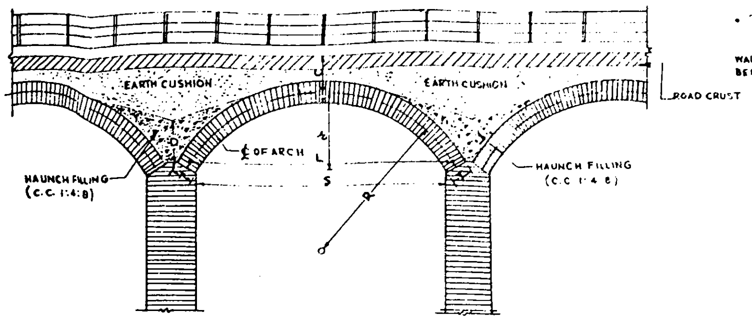
1. ABUTMENT AND WING WALL SECTIONS ARE APPLICABLE FOR A MINIMUM BEARING CAPACITY OF THE SOIL OF 16·5T/M^2. FOR SOILS HAVING LOWER BEARING CAPACITY THE SECTIONS SHOULD BE INCREASED SUITABLY.
2. ABUTMENT AND WING WALL SECTIONS FOR INTERMEDIATE HEIGHTS TO BE ADOPTED SUITABLY.
3. THE VARIOUS DIMENSIONS TO 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 SEVERER, WITHOUT PROVISION OF APPROACH SLABS.
5. THE SECTIONS SHALL BE IN CEMENT CONCRETE 1:3:6, BRICK MASONRY IN CEMENT MORTAR 1:3 OR COURSED RUBBLE MASONRY (II ND. SORT) IN CEMENT MORTAR 1:3. THE FOUNDATION CONCRETE SHALL BE IN CEMENT CONCRETE 1:3:6.



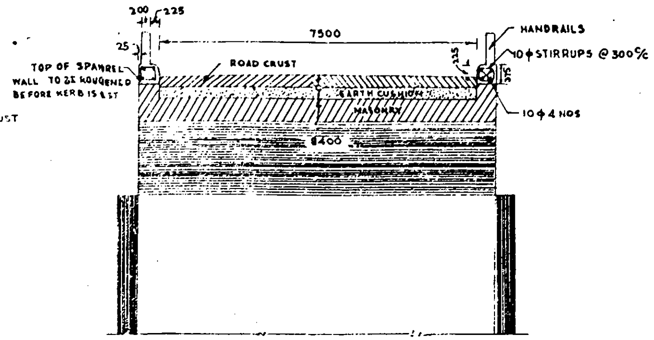
WING WALL SECTION AT LOW END
(FOR ALL SPANS)

ABUTMENT AND WING WALL SECTIONS FOR CULVERTS

PLATE 5



SECTIONAL ELEVATION



CROSS SECTION AT THE CROWN OF ARCH

SCALE 1:50

GENERAL NOTE:

1 SPECIFICATIONS — R.C STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES SECTIONS I, II AND III

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 (C.C. 1:3:6) 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/CM², 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 COMPRESSIVE STRESS, PERMISSIBLE TENSILE STRESS — AS SPECIFIED IN IRC BRIDGE CODE SECTION IV (1971)

5 RAILINGS — AS PER DETAILS APPROVED.

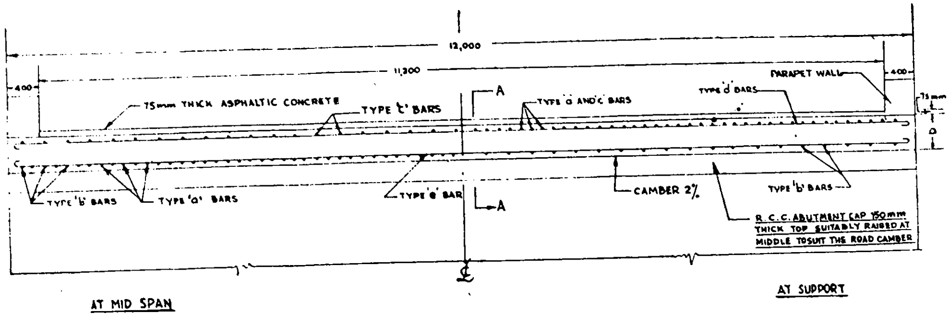
NOTES

1 THIS DRAWING IS APPLICABLE TO BRIDGES LOCATED IN NON-SEISMIC ZONES ONLY.
2 THE RATIO OF RISE : SPAN OF THE CENTRAL LINE OF ARCH RING SHALL BE 1/4.
3 SPECIFICATION FOR ROAD CRUST OVER THE ARCH BRIDGES 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.

TABLE

EFFECTIVE SPAN (L) METRES	6	9	12
CLR SPAN (S) (METRES)	5.572	8.512	11.368
RISE (h) (MILLIMETRES)	1500	2250	3000
RADIUS OF CENTRE LINE (R) (MILLIMETRES)	3750	5625	7500
CUSHION ABOVE CROWN (C) (MILLIMETRES)	610	760	760
ARCH THICKNESS (T) (MILLIMETRES) [UNIFORM SECTION FROM SPRINGING TO CROWN]	535	610	790
DEPTH OF HAUNCH FILLING AT PIER & ABUTMENT $D = \frac{4+T}{2}$ (MILLIMETRES)	1018	1430	1895

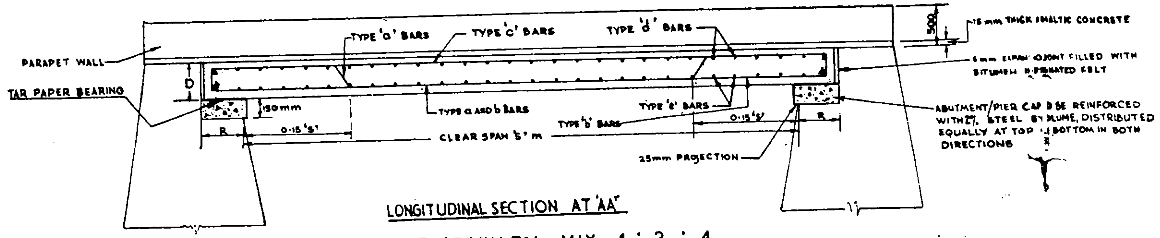
DETAILS OF SEGMENTAL MASONRY ARCH BRIDGES WITHOUT FOOTPATHS SPAN 6 M, 9 M, & 12 M



CROSS SECTION

GENL. NOTES :-

1. DESN. SPECIFICATIONS :- I.R.C Standard Specifications and Code of practice for Road Bridges Sections I, II & III.
2. DESIGN LIVE LOAD - Two lanes of IRC Class 'A' or one Lane class 70R (Tracked or Wheeled) vehicle.
3. REINFORCEMENT - Mild Steel Bars conforming to I.S 432:1966 Grade I tested steel.
4. COVER: Minimum clear cover to mild steel reinforcement shall be 25mm except where shown otherwise.
5. BEARING :- Tar paper or any suitable type as decided by the Engineer-in-charge.



LONGITUDINAL SECTION AT 'AA'

CEMENT CONCRETE ORDINARY MIX 1 : 2 : 4

STEEL IN LONGITUDINAL DIRECTION

CLEAR SPAN OF SLAB 'S' (m)	BEARING LENGTH AT EACH END 'C' (mm)	OVERALL LENGTH OF SLAB (m)	OVERALL SLAB THICKNESS 'D' (mm)	TYPE 'a' BARS: DIA (mm)	SPACING (mm)	NO. OF BARS	X (m)	Y (m)	Z (m)	2(X+Y)+Z+HOOKS	TOTAL LENGTH (m)	TYPE 'b' BARS: DIA (mm)	SPACING (mm)	NO. OF BARS	'l' (m)	LENGTH INCLUDING HOOKS	TOTAL LENGTH (m)
1·0	300	1·6	190	16	300	40	0·253	0·15	0·7	1·535	77·40	16	300	40	1·454	1·833	73·32
1·5	300	2·1	220	16	150	80	0·292	0·210	1·05	2·46	196·85	10	150	80	1·77	2·227	178·16
2·0	300	2·6	240	16	150	80	0·153	0·243	1·40	2·981	238·42	10	150	80	2·49	2·727	218·16
3·0	300	3·6	305	16	150	80	0·483	0·331	2·10	3·821	3[illegible]·48	16	150	80	3·454	3·833	306·64
4·0	300	4·6	370	18	150	80	0·519	0·427	2·8	5·139	409·52	16	150	80	4·454	4·833	386·64
5·0	400	5·8	420	20	150	80	0·715	0·495	3·5	6·394	511·52	16	150	80	5·654	6·033	482·64
6·0	400	6·8	475	20	150	80	0·81	0·572	4·2	7·438	595·04	20	150	80	6·63	7·104	568·32

CLEAR SPAN OF SLAB 'S' (m)	TYPE 'c' BARS: DIA (mm)	SPACING (mm)	NO. OF BARS	m (m)	m_1 (m)	$(m+2m_1)$ + HOOKS (m)	TOTAL LENGTH (m)	STEEL IN TRANSVERSE DIRECTION, TYPE 'e' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	J (m)	LENGTH INCLUDING HOOKS	TOTAL LENGTH (m)	TYPE 'd' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	r (m)	LENGTH INCLUDING HOOKS	TOTAL (m)	TOTAL QUANTITY OF MS. REINFORCEMENT PER SPAN IN TONNES (INCLUSIVE OF 5% FOR LAPS AND WATAGES)	TOTAL QUANTITY OF CONCRETE PER SPAN IN CUBIC METRES
1·0	10	300	40	1·55	0·10	1·987	79·48	16	150	11	11·854	12·233	134·56	10	300	6	11·89	12·127	[illegible]·76	0·582	3·65
1·5	10	300	40	2·05	0·13	2·547	101·88	16	150	14	11·854	12·233	171·262	10	300	7	11·89	12·127	[illegible]·89	0·848	5·55
2·0	10	300	40	2·55	0·15	3·087	123·48	16	125	21	11·854	12·233	256·89	10	300	9	11·89	12·127	[illegible]·14	1·117	7·48
3·0	10	300	40	3·55	0·215	4·217	168·68	16	125	29	11·854	12·233	354·75	10	300	12	11·89	12·127	[illegible]·52	1·861	13·20
4·0	10	300	40	4·55	0·28	5·347	213·88	16	125	37	11·854	12·233	452·62	10	300	15	11·89	12·121	[illegible]·91	2·506	20·41
5·0	10	300	40	5·76	0·33	6·647	265·88	16	125	47	11·854	12·233	574·95	10	300	19	11·89	12·127	[illegible]·41	3·402	29·26
6·0	10	300	40	6·75	0·385	7·757	310·28	16	125	55	11·854	12·233	672·81	10	300	23	11·89	12·127	2[illegible]·92	4·513	38·80

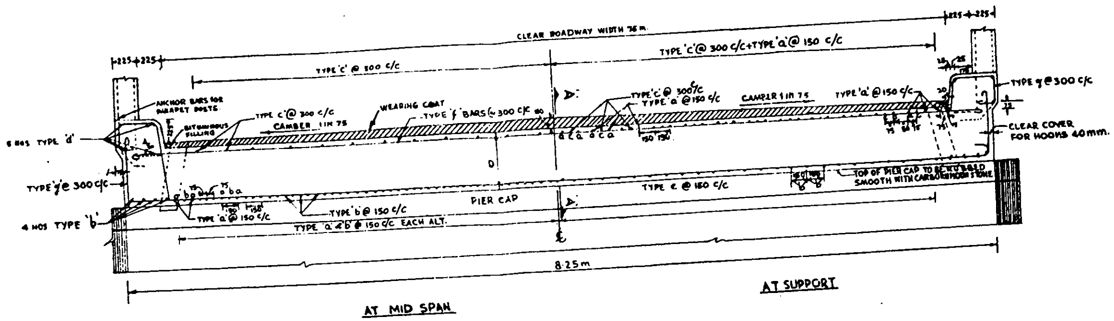
CONTROLLED CONCRETE M 200

STEEL IN LONGITUDINAL DIRECTION

CLEAR SPAN OF SLAB 'S' (m)	BEARING LENGTH AT EACH END 'C' (mm)	OVERALL LENGTH OF SLAB (m)	OVERALL THICKNESS OF SLAB 'D' (mm)	TYPE 'a' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	X (m)	Y (m)	Z (m)	2(X+Y)+Z+HOOKS (m)	TOTAL LENGTH (m)	TYPE 'b' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	'l' (m)	LENGTH INCLUDING HOOKS (m)	TOTAL LENGTH (m)
1·0	300	1·6	150	16	180	86	0·293	0·119	0·7	1·903	125·6	12	180	66	1·478	1·762	116·29
1·5	300	2·1	180	16	180	66	0·228	0·161	1·05	2·427	160·18	16	180	66	1·954	2·333	153·99
2·0	300	2·6	215	12	150	80	0·384	0·214	1·4	2·888	231·04	16	150	80	2·454	2·833	226·66
3·0	300	3·6	275	16	150	80	0·468	0·296	2·1	4·008	320·64	18	150	80	3·442	3·870	309·6
4·0	300	4·6	345	16	150	80	0·548	0·395	2·8	5·066	405·28	20	150	80	4·43	4·904	392·32
5·0	400	5·8	395	20	150	80	0·74	0·46	3·5	6·374	509·92	20	150	90	4·63	5·104	[illegible]8·32
6·0	400	6·8	445	16	150	80	0·848	0·537	4·2	7·35	588·0	25	150	80	6·6	7·192	575·4

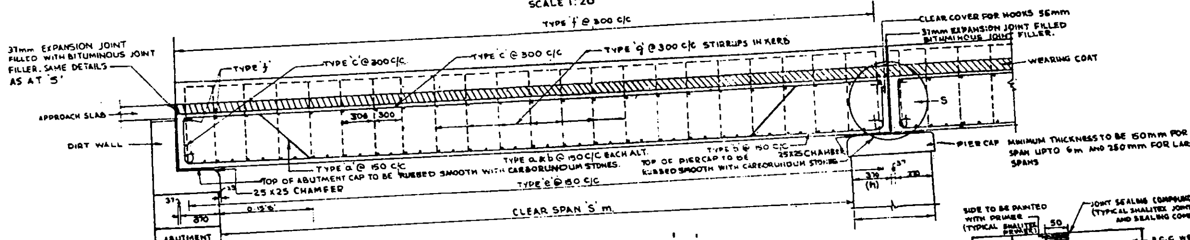
CLEAR SPAN OF SLAB 'S' (m)	TYPE 'c' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	m (m)	m_1 (m)	$(m+2m_1)$ + HOOKS (m)	TOTAL LENGTH (m)	STEEL IN TRANSVERSE DIRECTION, TYPE 'e' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	J (m)	LENGTH INCLUDING HOOKS (m)	TOTAL LENGTH (m)	TYPE 'd' BARS: DIA (mm)	SPACING (mm)	NO OF BARS	r (m)	LENGTH INCLUDING HOOKS (m)	LENGTH (m)	TOTAL QUANTITY OF M.S. REINFORCEMENT PER SPAN IN TONNES (INCLUSIVE OF 5% FOR LAPS AND WASTAGES)	TOTAL QUANTITY OF CONCRETE PER SPAN IN CUBIC METRES
1·0	10	360	33	1·55	0·06	1·907	62·93	16	125	13	11·854	12·233	159·03	10	300	5	11·89	12·127	[illegible]·63	0·662	2·88
1·5	10	360	33	2·05	0·09	2·467	81·41	16	100	21	11·854	12·233	256·89	10	300	7	11·89	12·127	[illegible]·88	1·054	4·54
2·0	10	300	40	2·55	0·125	3·037	121·48	16	100	26	11·854	12·233	318·06	10	300	9	11·89	12·127	[illegible]·14	1·268	6·71
3·0	10	300	40	3·55	0·185	4·157	166·28	18	150	24	11·842	12·268	294·45	10	300	12	11·89	12·127	[illegible]524	2·000	11·88
4·0	10	300	40	4·58	0·255	5·297	211·88	16	150	30	11·854	12·233	366·99	10	300	15	11·89	12·127	[illegible]905	2·556	19·05
5·0	10	300	40	5·75	0·005	6·597	263·88	16	150	39	11·854	12·233	477·09	10	300	19	11·89	12·127	[illegible]·41	3·703	27·50
6·0	10	300	40	6·75	0·355	7·697	307·88	16	150	44	11·854	12·233	538·25	10	300	23	11·89	12·127	[illegible]·92	4·581	36·30

REINFORCEMENT DETAILS OF R.C. SLAB FOR CULVERTS



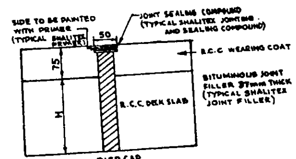
CROSS SECTION

SCALE 1:20



LONGITUDINAL SECTION AT 'AA'

SCALE 1:20



PIER CAP

DETAIL AT 'S'

SCALE 1:5

SCHEDULE OF REINFORCEMENT (NOT TO SCALE)

(PER SPAN)

STEEL IN LONGITUDINAL DIRECTION

CLEAR SPAN OF SLAB S (m)	BEARING LENGTH AT EACH END (mm)	OVERALL LENGTH OF SLAB (m)	OVERALL DEPTH OF SLAB 'D' (mm)	Type 'a' DIA (mm)	Type 'a' SPACING (mm)	Type 'a' NO. OF BARS	Type 'a' 'x' (m)	Type 'a' 'y' (m)	Type 'a' 'z' (m)	Type 'a' 2(x+y)+z+HOOKS (m)	Type 'a' TOTAL LENGTH (m)	Type 'b' DIA (mm)	Type 'b' SPACING (mm)	Type 'b' NO. OF BARS	Type 'b' 'l' (m)	Type 'b' LENGTH INCLUDING HOOKS (m)	Type 'b' TOTAL LENGTH (m)	Type 'c' DIA (mm)	Type 'c' SPACING (mm)	Type 'c' NO. OF BARS	Type 'c' 'm' (m)	Type 'c' 'n' (m)	Type 'c' (m+2n)+HOOKS (m)	Type 'c' TOTAL LENGTH (m)	Type 'd' DIA (mm)	Type 'd' NO. OF BARS	Type 'd' 'j' (m)	Type 'd' LENGTH INCLUDING HOOKS (m)	Type 'd' TOTAL LENGTH (m)
3	370	3.74	275	16	150	50	.553	.296	2.10	4.036	200.30	18	150	57	3.622	[illegible]	224.82	10	300	24	3.63	215	4.24	101.76	10	10	3.63	3.81	38.10
4	370	4.74	345	16	150	50	.833	.395	2.80	5.144	257.20	20	150	57	4.69	[illegible]	238.86	10	300	24	4.63	265	5.38	129.12	10	10	4.63	4.51	40.10
5	370	5.74	395	20	150	50	.735	.46	3.50	6.258	312.50	20	150	57	5.629	[illegible]	340.867	10	300	24	5.63	335	6.48	155.52	10	10	5.63	5.51	58.10
6	370	6.74	445	16	150	50	.833	.557	4.20	7.288	361.40	25	150	57	6.615	[illegible]	402.705	10	300	24	6.63	385	7.58	181.97	10	10	6.63	6.51	68.10
8	370	8.74	555	22	150	50	1.024	.683	5.60	9.414	470.70	25	150	57	8.615	[illegible]	516.708	10	300	24	8.63	4.95	9.80	235.20	10	10	8.62	8.51	88.10

STEEL IN TRANSVERSE DIRECTION

CLEAR SPAN OF SLAB S (m)	Type 'e' DIA (mm)	Type 'e' SPACING (mm)	Type 'e' NO. OF BARS	Type 'e' 'r' (mm)	Type 'e' LENGTH INCLUDING HOOKS (m)	Type 'e' TOTAL LENGTH (m)	Type 'f' DIA (mm)	Type 'f' SPACING (mm)	Type 'f' NO. OF BARS	Type 'f' 'h' (m)	Type 'f' LENGTH INCLUDING HOOKS (m)	Type 'f' TOTAL LENGTH (m)	Type 'g' DIA (mm)	Type 'g' SPACING (mm)	Type 'g' NO. OF BARS	Type 'g' 'x' (mm)	Type 'g' 'y' (mm)	Type 'g' LENGTH INCLUDING HOOKS (mm)	Type 'g' TOTAL LENGTH (m)	* TOTAL QUANTITY OF M.S. REINFORCEMENT PER SPAN IN TONNES	** TOTAL QUANTITY OF CONCRETE PER SPAN IN CUBIC METRES
3	18	150	25	8.154	8.476	211.80	10	300	13	8.00	8.18	106.34	10	300	26	255	344	2.171	56.446	1.454	9.46
4	16	150	32	8.154	8.442	270.15	10	300	17	8.00	8.18	139.06	10	300	34	325	352	2.320	78.68	1.878	14.69
5	16	150	30	8.154	8.442	320.79	10	300	20	8.00	8.18	163.60	10	300	40	375	358	2.436	94.44	2.357	20.10
6	16	150	45	8.154	8.442	379.89	10	300	23	8.00	8.18	188.14	10	300	46	425	368	2.532	116.42	3.217	26.40
8	16	150	58	8.154	8.442	489.64	10	300	30	8.00	8.18	245.40	10	300	60	530	375	2.754	165.24	4.817	44.20

MAXIMUM REACTIONS IN TONNES

Span	At Abutment D.L.	At Abutment D.L.+L.L.	At Abutment D.L.+L.L.+I	At Pier D.L.	At Pier D.L.+L.L.	At Pier D.L.+L.L.+I
3 m	14.15	52.05	70.26	28.30	75.75	87.61
4 m	21.60	61.36	78.80	43.20	94.80	107.70
5 m	29.15	70.92	87.55	58.30	114.10	127.30
6 m	38.10	82.13	98.21	76.20	132.90	144.90
8 m	57.60	107.40	119.80	115.20	174.60	189.70

GENERAL NOTES

Design Specification : I.R.C. Standard specifications and codes of practice for road bridges sections I, II & III (1966)

Design Live Load : Two lanes of IRC Class 'A' or one lane of IRC Class 70-R (tracked or wheeled)

Concrete : Concrete with a minimum 28 days works cube strength of 200 kg/cm² on 150 mm cubes

Reinforcement : Mild steel conforming to I.S. 432/latest grade I tested steel.

Cover : Clear cover to mild steel reinforcement shall be 25 mm or Dia. of bar whichever is greater except where shown otherwise

Hand Rail & Wearing Coat : Shall be of any approved type.

Drainage : As per I.R.C. Clause 117 Sec. I.

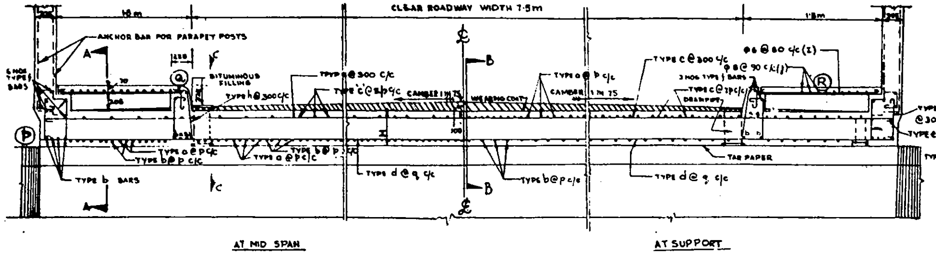
NOTES :

1. The dimensions and the diameter of reinforcement bars indicated are in mms unless otherwise specifically mentioned.
2. The width of bearing 'K' shall be 370 mm. However 'K' may be suitably modified to suit every individual case at the discretion of engineer incharge. In such an event the schedule of reinforcement shall be suitably modified.
3. The details of slab given in this drawing apply for square crossings only and shall not be followed where the slab spans are skew.
4. The dimensions of bars as appearing in the schedule are exact. For the purpose of estimating steel quantities shall be taken as approximate only.
5. Full scale elevation for the bars shall lined on plane plastered floor to the exact dimensions so as to get exact clearance between different bar & then the bars bent exactly to these shapes
6. If full length bars are not available, lap joints are permissible provided no joints come at mid span and all joints are well staggered.
7. Precast mortar blocks of strength not less than that of bar concrete should be used to ensure the requisite clear cover for the reinforcement.
8. 12 mm Dia. steel chair should be used to maintain the required distances between the top and bottom layers of steel.
9. This drawing is meant for bridges (having a length more than 6 metres) with a roadway width of 7.5 metres.
10. If any of these spans is going to be used as a floating span in a balanced cantilever bridge suitable provision of steel should be made over bearings in such spans for ensuring bear action and also against concentrated contact stresses in concrete.
11. The top of pier cap and abutment cap shall be finished smooth by rubbing with carborundom stones. The deck slab shall be cast after applying shuttering oil on top of cap (typical filmo-35 manufactured by M/s. Esso Standard Eastern Inc).
12. The joint filler shall be placed in position butting the dirt wall over abutments and butting the vertical face of the slab of 1 span over piers prior to the concreting of the adjacent spans. The sealing compound shall be poured in the joint between the slabs on the top of the filler after cleaning the joints by the help of compressed air.

* Including 5% extra for laps and wastage, but excluding steel in parapets & wearing co

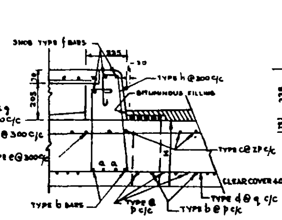
**Excluding wearing coat and parapets.

R.C.C. SLAB BRIDGES WITHOUT FOOTPATHS, SIMPLY SUPPORTED CLEAR SPANS 3, 4, 5, 6 & 8 M FOR SQUARE CROSSINGS



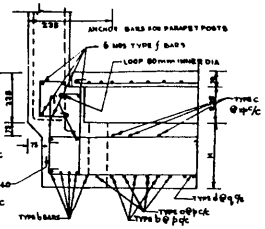
CROSS SECTION

SCALE 1:20



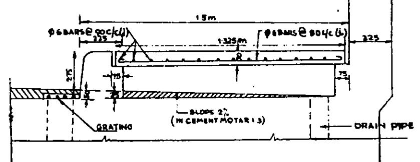
DETAILS AT Q

SCALE 1:10



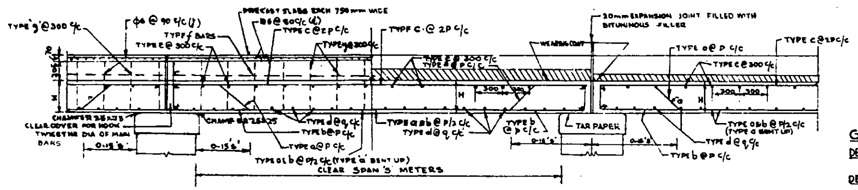
DETAILS AT P

SCALE 1:10



DETAILS AT R

SCALE 1:10



L-SECTION OF FOOTPATH AT AA (ALSO SHOWING THE ARRANGEMENT OF REINFORCEMENT AT THE SLAB EDGE)

L-SECTION AT BB (NEAR CENTRE)

L - SECTION AT CC

LONGITUDINAL SECTION

SCALE 1:20

SCHEDULE OF REINFORCEMENT (PER SPAN) (NOT TO SCALE)

MAIN SLAB

Clear span S (m)	Bearing length at end (mm)	Overall length of slab (m)	Overall thickness of slab H (mm)	Steel in longitudinal direction: Type 'a' Dia (mm)	Spacing P (mm)	No. of bars	x (mm)	y (mm)	z (m)	Length incl. hooks (m)	Total length (m)	Type 'b' Dia (mm)	Spacing P (mm)	No. of bars	l (m)	Length incl. hooks (m)	Total length (m)	Type 'c' Dia (mm)	Spacing 2P (mm)	No. of bars	l (m)	h (mm)	l+2h (m)	Total length (m)	Steel in transverse direction: Type 'd' Dia (mm)	Spacing q (mm)	No. of bars	l (m)	l + hooks (m)	Total length (m)	Type 'e' Dia (mm)	Spacing (mm)	No. of bars	l (m)	l + hooks (m)	Total length (m)
4	370	4.74	330	16	150	68	674	374	2.800	5.184	[illegible]	20	160	77	4.440	5.020	[illegible]	10	300	34	4.440	270	5.380	[illegible]	20	220	22	10.720	11.080	[illegible]	10	300	16	10.720	[illegible]	[illegible]
6	370	6.74	425	20	150	68	875	505	4.200	[illegible]	[illegible]	22	150	77	[illegible]	7.048	[illegible]	10	300	34	[illegible]	360	7.582	[illegible]	20	220	31	10.720	11.080	[illegible]	10	300	23	10.720	[illegible]	[illegible]
8	370	8.74	520	22	150	68	1068	649	[illegible]	[illegible]	[illegible]	25	150	77	[illegible]	[illegible]	[illegible]	10	300	34	[illegible]	470	[illegible]	[illegible]	20	220	40	10.720	11.080	[illegible]	10	320	30	10.720	[illegible]	[illegible]

INNER & OUTER KERBS / FOOTPATH SLABS

Clear span S (m)	Inner & outer kerbs, steel in longitudinal direction: Type f Dia (mm)	Spacing (mm)	No. of bars	l (m)	Length incl. hooks (m)	Total length (m)	Steel in cross direction: Type g Dia (mm)	Spacing (mm)	No. of stirrups	g_1 (mm)	g_2 (mm)	Length incl. hooks (m)	Total length (m)	Steel in cross direction: Type h Dia (mm)	Spacing (mm)	No. of stirrups	h_1 (mm)	h_2 (mm)	h_3 (mm)	Length incl. hooks (m)	Total length (m)	Footpath slabs, steel in cross direction: Type i Dia (mm)	Spacing (mm)	No. of bars	l (m)	l + hooks (m)	Total length (m)	Steel in longitudinal direction: Type j Dia (mm)	Spacing (mm)	No. of bars	l (m)	l + hooks (m)	Total length (m)
4	10	AS SHOWN	18	4.440	4.840	[illegible]	8	300	32	452	292	[illegible]	[illegible]	8	300	33	511	432	206	1.618	[illegible]	6	80	120	1.275	[illegible]	[illegible]	6	90	180	0.740	0.848	152.64
6	10	AS SHOWN	18	[illegible]	[illegible]	[illegible]	8	300	46	545	365	2.369	[illegible]	8	300	46	604	525	217	[illegible]	[illegible]	6	80	180	1.275	1.383	[illegible]	6	90	270	0.740	0.848	228.96
8	10	AS SHOWN	18	[illegible]	[illegible]	[illegible]	8	300	60	647	487	[illegible]	[illegible]	8	300	60	706	627	238	[illegible]	[illegible]	6	80	220	1.275	[illegible]	[illegible]	6	90	330	0.740	0.848	[illegible]

GENERAL NOTES:-

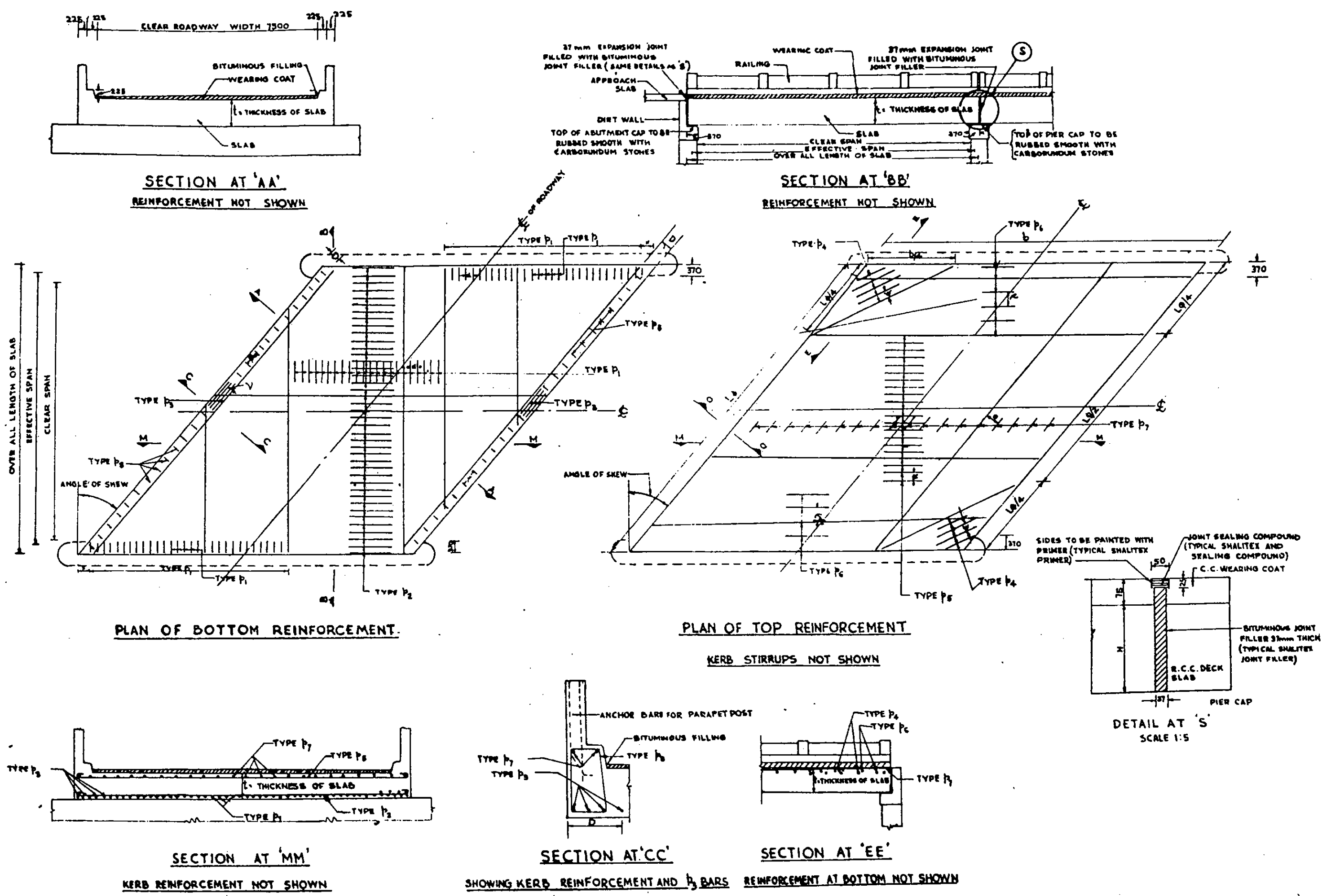
DESIGN SPECIFICATIONS	IRC STANDARD SPECIFICATIONS AND CODES OF PRACTICE FOR ROAD BRIDGE SECTIONS I, II & III (1966)
DESIGN LIVE LOAD	TWO LANES OF IRC CLASS 'A' OR ONE LANE OF CLASS 70R (TRACKED OR WHEELED) VEHICLE
CONCRETE	CONCRETE TO HAVE 28 DAYS WORK. CUBE STRENGTH OF 200Kg/cm² ON 150mm CUBES
PERMISSIBLE STRESSES	fc = 67 Kg/cm² AND fs = 1250 Kg/cm², m = 14
REINFORCEMENT	MILD STEEL BARS CONFORMING TO IS:432/1966 GRADE I. TESTED STEEL
COVER	MINIMUM CLEAR COVER MILD STEEL REINFORCEMENT SHALL BE 25mm EXCEPT WHERE SHOWN OTHERWISE
CAMBER	CAMBER IN THE TRANSVERSE DIRECTION SHALL BE PROVIDED IN THE WEARING COAT
DRAINAGE	SHALL CONFORM TO CLAUSE 117 OF BRIDGE CODE SECTION I
EXPANSION JOINT	SHALL BE OF ANY APPROVED TYPE
WEARING COAT	SHALL BE OF ANY APPROVED TYPE
RAILINGS	SHALL OF ANY APPROVED TYPE
BEARING	TAR PAPER OR ANY SUITABLE TYPE AS DIRECTED BY THE ENGINEER INCHARGE

TOTAL QUANTITIES (PER SPAN)

SPAN	4m	6m	8m
* CONCRETE	19.09m³	34.032m³	54.041m³
** STEEL	2698 Kgs	4500 Kgs	6703 Kgs

* EXCLUDING PARAPETS & WEARING COATS
** INCLUDING 5% EXTRA FOR LAPS & WASTAGE

R.C.C. SLAB BRIDGES WITH FOOTPATHS SIMPLY SUPPORTED CLEAR SPANS 4, 6 & 8 METRES FOR SQUARE CROSSINGS



GENERAL ARRANGEMENT OF R.C.C. SKEW SLAB BRIDGE—WITHOUT FOOTPATHS—SIMPLY SUPPORTED CLFAR RIGHT SPANS 5,6 & 8 M

SCHEDULE OF REINFORCEMENT (PER SPAN)

PLATE 10

CLEAR SPAN = 8.00M. EFFECTIVE SPAN = 8.37M. OVERALL LENGTH OF SLAB = 8.74M.

Angle of skew	Thickness of slab (mm)	X (mm)	Type P1 Dia. of bar (mm)	P1 Spacing (mm)	P1 Length of bar incl. hooks (mm)	P1 No. of bars	P1 Total length (m)	P1 Total weight (kg)	Type P2 Dia. of bar (mm)	P2 Spacing (mm)	P2 Length of bar incl. hooks (mm)	P2 No. of bars	P2 Total length (m)	P2 Total weight (kg)	Type P3 D (mm)	P3 Dia. of bar (mm)	P3 Spacing (mm)	P3 Length of bar incl. hooks (mm)	P3 No. of bars	P3 Total length (m)	P3 Total weight (kg)	Type P4 Dia. of bar (mm)	P4 Spacing (mm)	P4 Length of bar incl. hooks (mm)	P4 No. of bars	P4 Total length (m)	P4 Total weight (kg)
15°	615	270	28	105	VARYING [illegible] TO 9133	101	757	3640	16	180	8922	67	598	955	3875	25	90	9394	8	75	285	25	300	VARYING [illegible]	10	27	88
30°	615	[illegible]	28	105	VARYING [illegible] TO 9133	132	851	4095	16	160	9914	55	548	870	4375	25	80	10434	12	125	475	25	260	VARYING [illegible]	10	28	107
45°	615	[illegible]	28	95	VARYING 1140 TO 9133	202	1165	5390	16	280	12098	32	386	618	5725	25	80	12660	16	203	770	25	150	VARYING [illegible]	16	52	198
60°	615	[illegible]	28	70	VARYING [illegible] TO 9133	415	2240	10750	16	300	16960	29	492	831	697	25	88	17730	22	390	1485	25	80	VARYING [illegible]	28	125	475

Angle of skew	Type P5 Dia. of bar (mm)	P5 Spacing (mm)	P5 Length of bar incl. hooks (mm)	P5 No. of bars	P5 Total length (m)	P5 Total weight (kg)	Type P6 Dia. of bar (mm)	P6 Spacing (mm)	P6 Length of bar incl. hooks (mm)	P6 No. of bars	P6 Total length (m)	P6 Total weight (kg)	Type P7 Dia. of bar (mm)	P7 Spacing (mm)	P7 Length of bar incl. hooks (mm)	P7 No. of bars	P7 Total length (m)	P7 Total weight (kg)	Type P8 Dia. of bar (mm)	P8 Spacing (mm)	P8 Length of bar incl. hooks (mm)	P8 No. of bars	P8 Total length (m)	P8 Total weight (kg)	Total quantity of steel including 5% extra for laps and wastage (kg)	Total quantity of concrete excluding railings and wearing coat (cu.m)
15°	12	300	8864	16	142	128	10	300	8828	14	124	75	10	300	9174	32	294	176	10	300	2650	62	164	99	5719	49.19
30°	12	290	9858	16	158	142	10	300	9822	14	138	83	10	300	10224	32	328	197	10	300	2650	66	175	105	6378	54.90
45°	12	150	12046	30	361	325	10	300	12010	14	168	101	10	300	12460	32	398	239	10	300	2650	78	207	124	8151	68.70
60°	12	70	16966	64	1085	976	10	300	16930	14	237	142	10	300	17610	32	565	389	10	300	2650	108	286	172	15929	94.62

CLEAR SPAN = 6.00M EFFECTIVE SPAN = 6.3714. OVERALL LENGTH OF SLAB = 6.74M.

Angle of skew	Thickness of slab (mm)	X (mm)	Type P1 Dia. of bar (mm)	P1 Spacing (mm)	P1 Length of bar incl. hooks (mm)	P1 No. of bars	P1 Total length (m)	P1 Total weight (kg)	Type P2 Dia. of bar (mm)	P2 Spacing (mm)	P2 Length of bar incl. hooks (mm)	P2 No. of bars	P2 Total length (m)	P2 Total weight (kg)	Type P3 D (mm)	P3 Dia. of bar (mm)	P3 Spacing (mm)	P3 Length of bar incl. hooks (mm)	P3 No. of bars	P3 Total length (m)	P3 Total weight (kg)	Type P4 Dia. of bar (mm)	P4 Spacing (mm)	P4 Length of bar incl. hooks (mm)	P4 No. of bars	P4 Total length (m)	P4 Total weight (kg)
15°	485	210	25	105	VARYING [illegible] TO 7090	97	586	2230	16	140	8922	48	430	689	256	22	70	7266	8	59	177	22	280	VARYING [illegible]	10	19	57
30°	485	[illegible]	25	105	VARYING [illegible] TO 7090	122	659	2500	16	140	9914	48	476	763	396	22	70	8069	12	97	290	22	260	VARYING [illegible]	10	22	66
45°	485	[illegible]	25	95	VARYING [illegible] TO 7090	182	898	3415	16	140	12098	45	542	866	536	22	70	9780	16	157	470	22	160	VARYING [illegible]	14	39	116
60°	485	1305	25	80	VARYING [illegible] TO 7090	324	[illegible]	5750	16	260	17024	26	443	709	675	22	70	13700	20	274	816	22	95	VARYING [illegible]	20	83	248

Angle of skew	Type P5 Dia. of bar (mm)	P5 Spacing (mm)	P5 Length of bar incl. hooks (mm)	P5 No. of bars	P5 Total length (m)	P5 Total weight (kg)	Type P6 Dia. of bar (mm)	P6 Spacing (mm)	P6 Length of bar incl. hooks (mm)	P6 No. of bars	P6 Total length (m)	P6 Total weight (kg)	Type P7 Dia. of bar (mm)	P7 Spacing (mm)	P7 Length of bar incl. hooks (mm)	P7 No. of bars	P7 Total length (m)	P7 Total weight (kg)	Type P8 Dia. of bar (mm)	P8 Spacing (mm)	P8 Length of bar incl. hooks (mm)	P8 No. of bars	P8 Total length (m)	P8 Total weight (kg)	Total quantity of steel including 5% extra for laps and wastage (kg)	Total quantity of concrete excluding railings and wearing coat (cu.m)
15°	12	300	8864	13	115	104	10	300	8828	10	89	54	10	300	7088	32	226	136	10	300	2351	46	108	65	3688	30.23
30°	12	300	9858	13	129	124	10	300	9822	10	99	60	10	300	7897	32	252	151	10	300	2351	50	118	71	4226	33.73
45°	12	240	12046	15	181	163	10	300	12010	12	144	87	10	300	9618	32	308	185	10	300	2351	60	141	85	5656	40.96
60°	12	145	16916	24	406	245	10	300	16880	12	202	122	10	300	13560	32	434	260	10	300	2351	82	193	116	8479	56.34

CLEAR SPAN = 5.00M EFFECTIVE SPAN = 5.37M OVERALL LENGTH OF SLAB = 5.74M

Angle of skew	Thickness of slab (mm)	X (mm)	Type P1 Dia. of bar (mm)	P1 Spacing (mm)	P1 Length of bar incl. hooks (mm)	P1 No. of bars	P1 Total length (m)	P1 Total weight (kg)	Type P2 Dia. of bar (mm)	P2 Spacing (mm)	P2 Length of bar incl. hooks (mm)	P2 No. of bars	P2 Total length (m)	P2 Total weight (kg)	Type P3 D (mm)	P3 Dia. of bar (mm)	P3 Spacing (mm)	P3 Length of bar incl. hooks (mm)	P3 No. of bars	P3 Total length (m)	P3 Total weight (kg)	Type P4 Dia. of bar (mm)	P4 Spacing (mm)	P4 Length of bar incl. hooks (mm)	P4 No. of bars	P4 Total length (m)	P4 Total weight (kg)
15°	420	170	22	90	VARYING [illegible] 6048	111	562	[illegible]	16	125	8922	46	410	655	205	20	80	6226	6	38	95	20	300	VARYING [illegible]	8	16	40
30°	420	385	22	[illegible]	VARYING [illegible]	137	654	950	6	125	9914	46	456	730	345	20	75	6886	10	69	173	20	235	VARYING [illegible]	10	[illegible]	53
45°	420	630	22	90	VARYING [illegible]	183	806	2420	16	125	12098	46	555	887	465	20	70	8356	16	117	292	20	150	VARYING [illegible]	14	40	100
60°	420	[illegible]	22	85	VARYING 760 TO [illegible]	287	1252	3582	16	160	16960	36	610	975	605	20	70	11880	18	210	525	20	90	VARYING [illegible]	20	75	188

Angle of skew	Type P5 Dia. of bar (mm)	P5 Spacing (mm)	P5 Length of bar incl. hooks (mm)	P5 No. of bars	P5 Total length (m)	P5 Total weight (kg)	Type P6 Dia. of bar (mm)	P6 Spacing (mm)	P6 Length of bar incl. hooks (mm)	P6 No. of bars	P6 Total length (m)	P6 Total weight (kg)	Type P7 Dia. of bar (mm)	P7 Spacing (mm)	P7 Length of bar incl. hooks (mm)	P7 No. of bars	P7 Total length (m)	P7 Total weight (kg)	Type P8 Dia. of bar (mm)	P8 Spacing (mm)	P8 Length of bar incl. hooks (mm)	P8 No. of bars	P8 Total length (m)	P8 Total weight (kg)	Total quantity of steel including 5% extra for laps and wastage (kg)	Total quantity of concrete excluding railings and wearing coat (cu.m)
15°	12	300	8664	11	98	89	10	300	8826	10	89	54	10	300	6078	32	195	117	10	300	2222	40	89	54	2981	2852
30°	12	300	9855	11	109	98	10	300	9819	10	99	60	10	300	6742	32	216	130	10	300	2222	44	98	59	3418	25.04
45°	12	210	12046	15	181	163	10	300	12010	10	121	73	10	300	8218	32	263	158	10	300	2222	50	111	67	4368	3061
60°	12	205	16916	15	254	228	10	300	16860	10	169	102	10	300	11560	32	370	222	10	300	2222	68	152	92	8210	4394

GENERAL NOTES:-

SPECIFICATIONS — I.R.C. STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES SECTIONS I, II & III (1966)

DESIGN LIVE LOAD — TWO LANES OF IRC CLASS A OR ONE LANE OF CLASS 70-R (TRACKED OR WHEELED) VEHICLE

CONCRETE — CONCRETE TO HAVE 28 DAYS WORKS CUBE STRENGTH 200 Kg/cm² ON 150mm CUBES.

PERMISSIBLE STRESSES — f_c = 67 Kg/cm² AND f_s = 1250 Kg/cm², m = 14.

REINFORCEMENT — MILD STEEL BARS CONFORMING TO I.S. 432/1966 GRADE I TESTED STEEL.

COVER — MINIMUM CLEAR COVER TO MILD STEEL REINFORCEMENT SHALL BE 25mm EXCEPT WHERE SHOWN OTHERWISE.

CAMBER — CAMBER IN THE TRANSVERSE DIRECTION SHALL BE PROVIDED IN THE WEARING COAT.

DRAINAGE — SHALL CONFORM TO CAUSE 117 OF I.R.C. BRIDGE CODE SECTION I.

WEARING COAT — SHALL BE AS PER DETAILS APPROVED.

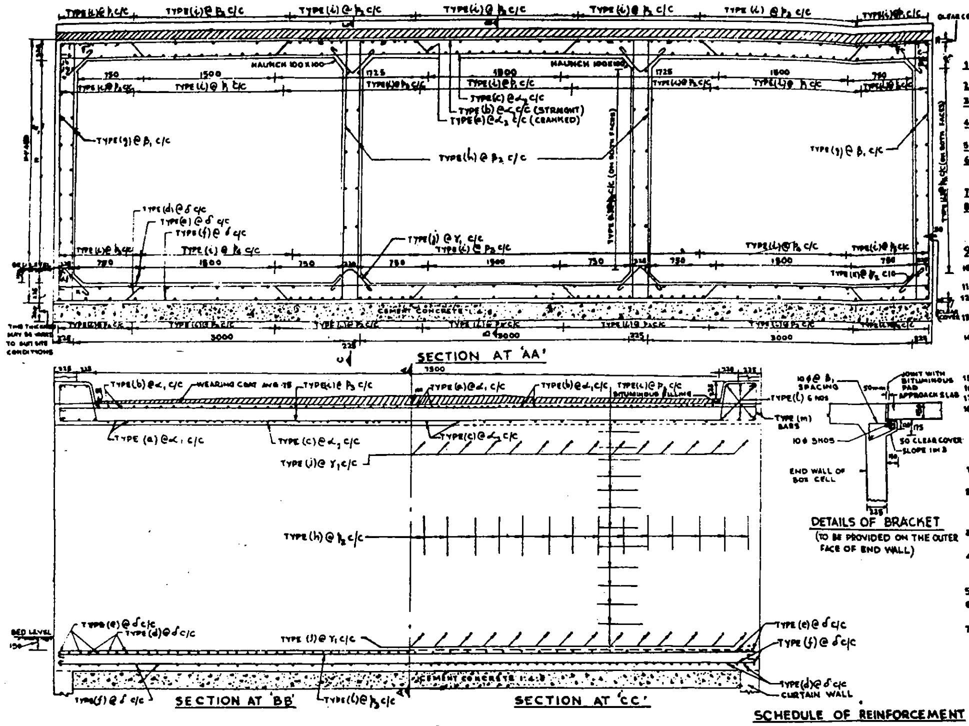
RAILINGS — SHALL BE AS PER DETAILS APPROVED.

NOTES

1. ALL DIMENSIONS ARE IN mm EXCEPT WHERE SHOWN OTHERWISE.
2. THE DIMENSIONS OF BARS AS APPEARING IN THE SCHEDULE ARE FOR THE PURPOSE OF ESTIMATING STEEL QUANTITIES
3. FULL SCALE ELEVATIONS FOR THE BARS SHALL FIRST BE LINEDOUT ON A PLANE PLASTERED FLOOR TO THE EXACT DIMENSIONS SO AS TO GET EXACT CLEARANCE BETWEEN DIFFERENT BARS AND THEN THE BARS BENT EXACTLY TO THESE SHAPES.
4. IF FULL LENGTH BARS ARE NOT AVAILABLE JOINTS HAVE TO BE PROVIDED; WELDED JOINTS ARE PERMISSIBLE IF THEY ARE OF THE SAME STRENGTH AS THE BARS JOINED AND ARE WELL STAGGERED. THE ARRANGEMENT OF JOINTS OR LAPPING OF BARS SHALL BE DECIDED BY THE ENGINEER-IN-CHARGE WELL IN ADVANCE
5. SUPPORT CHAIRS OF 12Ø SHALL BE PROVIDED AT SUITABLE INTERVALS.
6. PRECAST MORTAR BLOCK OF STRENGTH NOT LESS THAN THAT OF SLAB CONCRETE SHOULD BE USED TO ENSURE THE REQUISITE CLEAR COVER FOR THE REINFORCEMENT.
7. THE WIDTH OF BEARING SHALL BE 370mm, HOWEVER, THE WIDTH MAY BE SUITABLY MODIFIED TO SUIT EVERY INDIVIDUAL CASE AT THE DISCRETION OF THE ENGINEER-IN-CHARGE, IN SUCH AN EVENT THE SCHEDULE OF REINFORCEMENT IS TO BE SUITABLY MODIFIED.
8. THIS DRAWING IS APPLICABLE FOR SKEW BRIGES WITH ANGLE OF SKEW 15° TO 60° FOR ANY INTERMEDIATE ANGLE OF SKEW BETWEEN THOSE GIVEN IN THE REINFORCEMENT TABLE, SPACING OF BARS CAN BE LINEARLY INTERPOLATED.
9. THIS DRAWING IS APPLICABLE FOR BRIDGES WITH A CLEAR ROADWAY WIDTH OF 7.5 METRES ONLY.
10. FOR SKEW ANGLES LESS THAN 15°, THE USUAL METHOD OF CALCULATING MOMENT FOR A RIGHT BRIDGE OF SPAN CENTRE TO CENTRE OF SUPPORTS MEASURED PARALLEL TO CENTRE LINE OF ROADWAY AND PROVIDING MAIN REINFORCEMENT IN A DIRECTION PARALLEL TO TRAFFIC WITH TRANSVERSE REINFORCEMENT PARALLEL TO ABUTMENT MAY BE ADOPTED.
11. THESE SKEW SLAB BRIDGES ARE DESIGNED IN ACCORDANCE WITH THE INFLUENCE SURFACE CONTOURS PRESENTED IN THE BOOK BY RUSCH AND HERGENRODER PUBLISHED BY C & CA (LONDON)
12. THE TOP OF PIER CAP AND ABUTMENT CAP SHALL BE FINISHED SMOOTH BY RUBBING WITH CARBORUNDUM STONES. THE DECK SLAB SHALL BE CAST AFTER APPLYING SHUTTERING OIL ON TOP OF CAPS (TYPICAL:- FILMO 35 MANUFACTURED BY M/S ESSO STANDARD EASTERN INC.)
13. THE JOINT FILLER SHALL BE PLACED IN POSITION BUTTING THE DIRT WALL OVER ABUTMENTS AND BUTTING THE VERTICAL FACE OF THE SLAB OF ONE SPAN OVER PIERS PRIOR TO THE CONCRETING OF THE ADJACENT SPANS THE SEALING COMPOUND SHALL BE POURED IN THE JOINT BETWEEN THE SLABS ON THE TOP OF THE FILLER AFTER CLEANING THE JOINT BY THE HELP OF COMPRESSED AIR.

REINFORCEMENT DETAILS OF R.C.C. SKEW SLAB BRIDGE WITHOUT FOOTPATHS SIMPLY SUPPORTED CLEAR RIGHT SPANS 5, 6 & 8 M

PLATE 11



GENERAL NOTES:-

1. SPECIFICATIONS — I.R.C. STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES SECTIONS I, II & III (1966)
2. DESIGN STRESSES — $f_c = 67\ Kg/Cm^2$, $f_s = 1250\ Kg/Cm^2$ & $m = 14$
3. DESIGN LIVE LOAD — I.R.C. CLASS 'A' TWO LANES OR 70-R LOADING TRACKED OR WHEELED ONE LANE.
4. CONCRETE — CONCRETE WITH MINIMUM 28 DAYS WORKS STRENGTH 200 Kg/Cm^2 ON 150 mm CUBES
5. REINFORCEMENT — MILD STEEL BARS CONFORMING TO I.S. 432 PART I/1966
6. COVER — CLEAR COVER TO ANY REINFORCEMENT SHALL BE 25mm OR DIA OF BAR WHICHEVER IS GREATER EXCEPT WHERE SHOWN OTHERWISE.
7. RAILINGS — SHALL BE OF ANY APPROVED TYPE.
8. EXPANSION JOINT, WEARING COAT, DRAINAGE OF ROADWAY — SHALL BE OF ANY APPROVED TYPE
9. DRAINAGE OF BACKFILL — SHALL CONFORM TO CAUSE 217.4 OF I.R.C. BRIDGE CODE SECTION II
10. THE DETAILS OF BOX CELL AS SHOWN IN THIS DRAWING APPLY FOR SQUARE CROSSINGS ONLY AND SHALL NOT BE FOLLOWED WHERE THE CROSSINGS ARE SKEW.
11. BACK FILL SHALL NOT BE PLACED UNTIL THE TOP SLAB IS IN PLACE.
12. SUITABLE WING WALLS [NOT CANTILEVERED OUT FROM THE END WALLS OF THE BOX CELL BRIDGE] ARE TO BE PROVIDED AS PER SITE CONDITIONS.
13. ADEQUATELY DESIGNED CURTAIN WALLS SHALL BE PROVIDED BOTH ON THE UPSTREAM SIDE AND ON THE DOWN STREAM SIDE DEPENDING ON ANTICIPATED SCOUR.
14. ADEQUATELY DESIGNED REINFORCED CONCRETE APPROACH SLAB CONFORMING TO CLAUSE 217.3 OF THE I.R.C. BRIDGE CODE SECTION II COVERING THE ENTIRE WIDTH OF ROADWAY WITH ONE END RESTING ON THE BRACKET PROVIDED ON THE OUTER FACE OF THE END WALL OF BOX CELL BRIDGE SHALL BE PROVIDED.
15. THE MAXIMUM PRESSURE ON FOUNDATION SOIL IS 5 TONNES PER SQUARE METRE.
16. DIFFERENTIAL SETTLEMENT OF THE FOUNDATION IS NOT COSIDERED IN THE DESIGN
17. FOR LARGER CROSSINGS MULTIPLE UNITS OF THIS DESIGN CAN BE ADOPTED
18. THE DESIGN CALCULATIONS ARE BASED ON THE ASSUMPTION THAT THE ANGLE OF INTERNAL FRICTION OF THE FILL IS EQUAL TO 30° AND ITS WEIGHT IS 1.8 T/m^3

NOTES:-

1. THE DIMENSIONS AND DIAMETRES OF REINFORCEMENT BARS INDICATED ARE IN M.M. EXCEPT WHERE SHOWN OTHERWISE.
2. IF FULL LENGTH BARS ARE NOT AVAILABLE JOINTS HAVE TO BE PROVIDED WELDED JOINTS ARE PERMISSIBLE IF THEY ARE OF THE SAME STRENGTH AS THE BARS JOINED. THE ARRANGEMENTS OF THESE JOINTS OR/LAPPING OF BARS SHALL BE SUITABLY STAGGERED AS DECIDED BY THE ENGINEER-IN-CHARGE WELL IN ADVANCE.
3. THE DIMENSIONS OF BARS APPEARING IN THE SCHEDULE ARE FOR THE PURPOSE OF ESTIMATING STEEL QUANTITIES.
4. FULL SCALE ELEVATION FOR THE BARS SHALL BE LINED OUT ON A PLANE PLASTERED FLOOR TO THE EXACT DIMENSIONS SO AS TO GET EXACT CLEARANCE BETWEEN DIFFERENT BARS AND THEN THE BARS BENT EXACTLY TO THESE SHAPES.
5. SUPPORT CHAIRS OF 12Φ BARS SHALL BE PROVIDED AT SUITABLE INTERVALS.
6. PRECAST MORTAR BLOCKS OF THE SAME STRENGTH AS THE BOX CELL CONCRETE SHALL BE USED UNDER THE REINFORCEMENT TO ENSURE REQUIRED COVER.
7. REINFORCEMENTS FOR THE BRACKET ARE NOT INCLUDED IN THE SCHEDULE.

SCHEDULE OF REINFORCEMENT (FOR ONE UNIT)

[illegible]	TYPE (a) Dia (mm)	Spacing (mm)	No of bars	(mm)	(mm)	(mm)	(mm)	(mm)	Length including hooks (mm)	Total length (m)
1500	16	155	46	370	766	225	1500	1407	11,192	587
2000	16	148	50	370	766	225	1500	1407	11,192	560
2500	16	140	52	370	766	225	1500	1407	11,192	582

[illegible]	TYPE (b) Dia (mm)	Spacing (mm)	No of bars	O (mm)	O_1 (mm)	Length including hooks (mm)	Total length (m)	TYPE (c) Dia (mm)	Spacing (mm)	No of bars	P (mm)	Length including hooks (mm)	Total length (m)
1500	12	155	49	[illegible]	[illegible]	10,516	515	8	310	25	9800	9944	249
2000	12	148	51	[illegible]	[illegible]	10,516	536	8	290	26	9800	9944	259
2500	12	140	53	[illegible]	[illegible]	10,516	557	8	280	27	9800	9944	268

[illegible]	TYPE (d) Dia (mm)	Spacing (mm) (δ)	No of bars	q (mm)	q_1 (mm)	q_2 (mm)	q_3 (mm)	q_4 (mm)	Length including hooks (mm)	Total length (m)
1500	16	250	33	370	766	225	1500	1407	11,192	370
2000	16	230	36	370	766	225	1500	1407	11,192	403
2500	16	210	39	370	766	225	1500	1407	11,192	436

[illegible]	TYPE (e) Dia (mm)	Spacing (mm) (δ)	No of bars	r (mm)	Length including hooks (mm)	Total length (m)	TYPE (f) Dia (mm)	Spacing (mm)	No of bars	s (mm)	s_1 (mm)	Length including hooks (mm)	Total length (m)
1500	8	250	34	9600	9944	338	16	250	34	[illegible]	[illegible]	10,736	368
2000	8	230	37	9600	9944	368	16	230	37	[illegible]	[illegible]	10,748	399
2500	8	210	40	9800	9944	390	16	210	40	[illegible]	[illegible]	10,798	432

[illegible]	TYPE (g) Dia (mm)	Spacing (mm)	No of bars	t (mm)	t_1 (mm)	t_2 (mm)	Length including hooks (mm)	Total length (m)
1500	16	[illegible]	[illegible]	1868	118	345	4535	490
2000	16	[illegible]	[illegible]	2368	118	345	5535	576
2500	16	[illegible]	[illegible]	2868	118	345	6535	706

[illegible]	TYPE (h) Dia (mm)	Spacing (mm)	No of bars	U (mm)	U_1 (mm)	U_2 (mm)	Length including hooks (mm)	Total length (m)
1500	16	180	60	1640	1415	345	4889	295
2000	16	260	66	2140	1415	345	5889	388
2500	16	230	74	2640	1415	345	6889	510

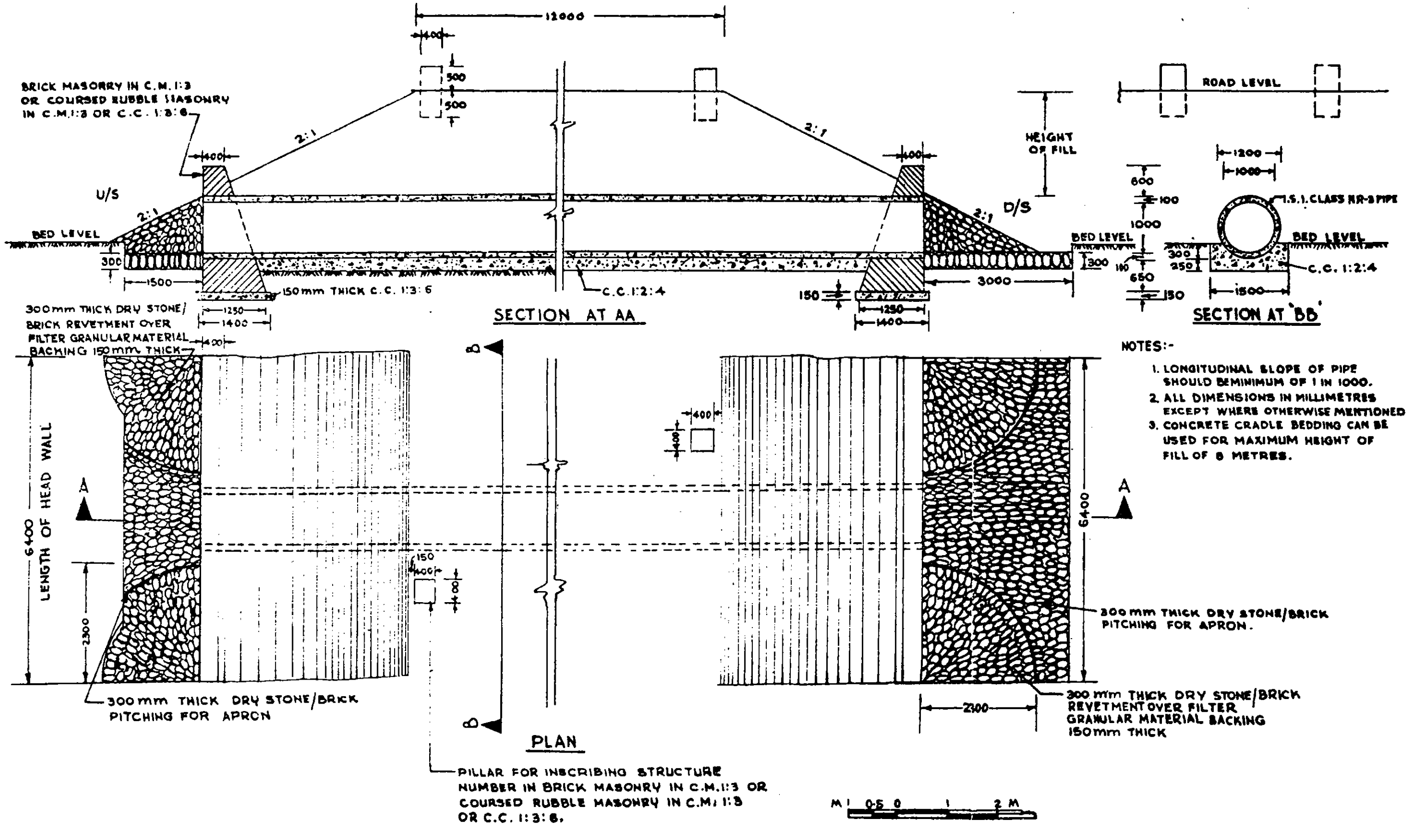
[illegible]	TYPE (i) Dia (mm)	Spacing (mm)	No of bars	Spacing (mm) (β_2)	No of bars	Spacing (mm)	No of bars	Spacing (mm) (β_4)	No of bars	Total no	V (mm)	Length including hooks (mm)	Total length (m)
1500	10	170	42	125	48	450	57	280	44	191	[illegible]	[illegible]	1623
2000	10	160	42	120	48	450	61	270	52	209	[illegible]	8530	1732
2500	10	155	42	120	48	450	65	260	62	217	[illegible]	8530	1851

[illegible]	TYPE (j) Dia (mm)	Spacing (mm)	No of bars	x (mm)	Length including hooks (mm)	Total length (m)	TYPE (k) Dia (mm)	Spacing (mm)	No of bars	s_1 (mm)	Length including hooks (mm)	Total length (m)
1500	8	280	120	320	776	93	8	310	108	320	464	50
2000	8	260	132	320	776	102	8	330	104	320	464	48
2500	8	250	148	320	776	115	8	310	108	320	464	50

[illegible]	TYPE (l) Dia (mm)	No of bars	y (mm)	Length including hooks (mm)	Total length (m)	TYPE (m) Dia (mm)	Spacing (mm)	No of bars	h (mm)	h_1 (mm)	h_2 (mm)	h_3 (mm)	Length including hooks (mm)	Total length (m)	Total quantity of steel including 2% extra lapping & wastage (kg)	Total quantity of concrete excluding parapet cum
1500	10	12	9850	10030	120	10	[illegible]	66	414	403	416	305	1828	121	5430	51.66
2000	10	12	9850	10030	120	10	[illegible]	66	414	403	416	305	1828	121	5970	55.44
2500	10	12	9850	10030	120	10	[illegible]	66	414	403	416	305	1828	121	6672	59.18

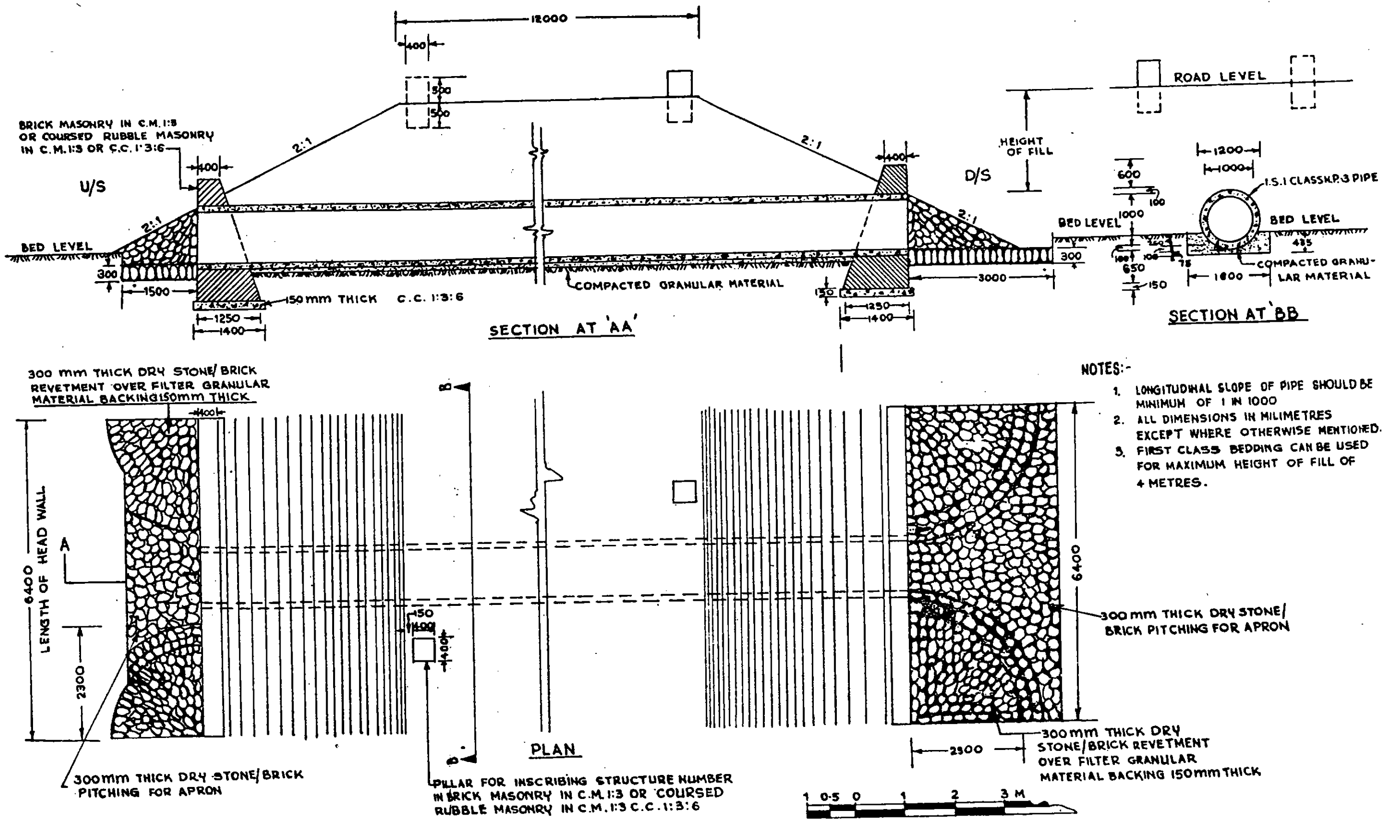
R.C.C. BOX CELL BRIDGE WITHOUT FOOTPATHS 3 CELLS EACH OF 3 M CLEAR SPAN

PLATE 12

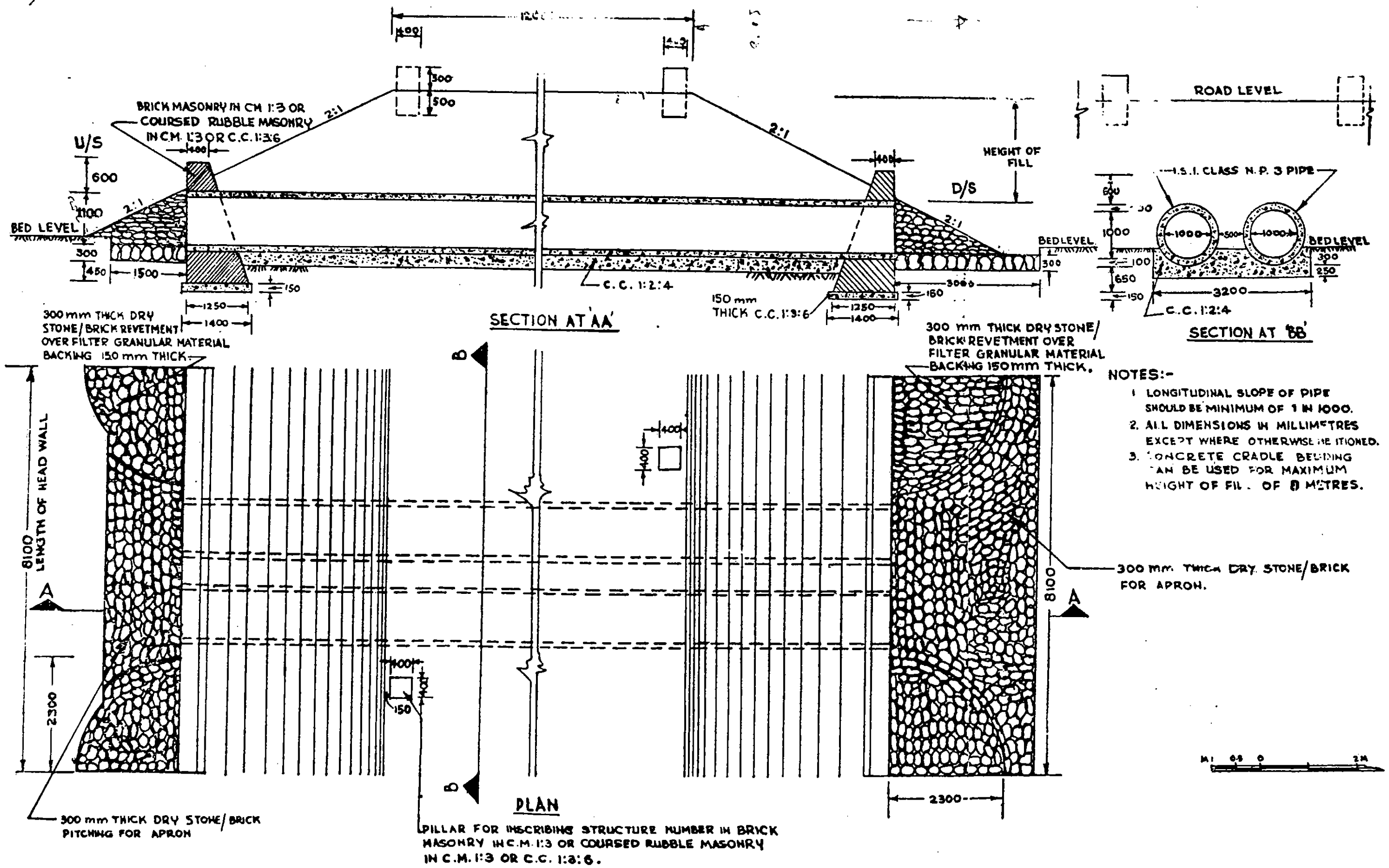


R.C.C. PIPE CULVERTS WITH SINGLE PIPE OF 1 METRE DIA AND CONCRETE CRADLE BEDDING FOR HEIGHTS OF FILL FROM 4.0 M TO 8.0 M

PLATE 13

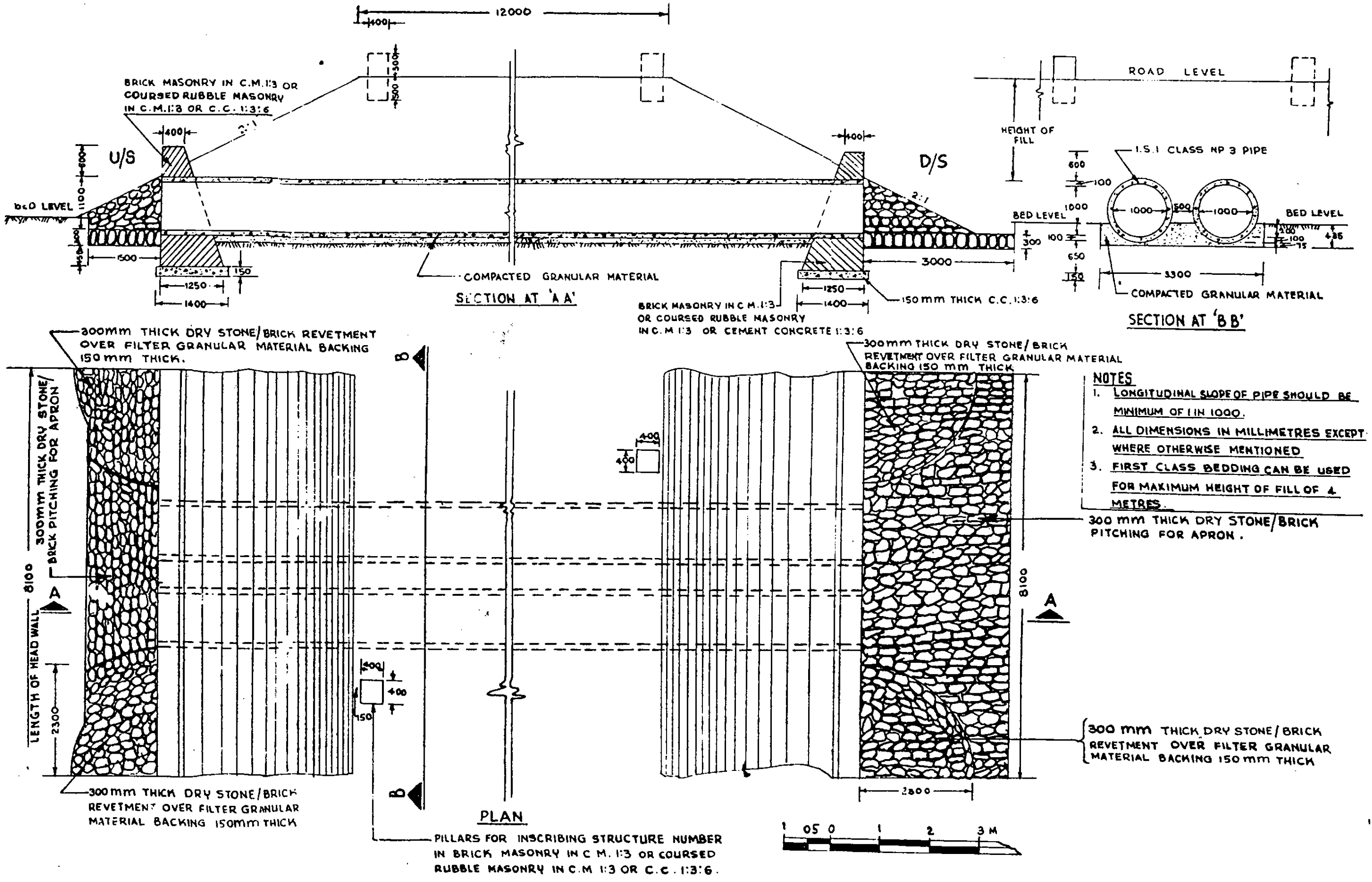


R.C.C. PIPE CULVERT WITH SINGLE PIPE OF 1 METRE DIA AND 1ST CLASS BEDDING FOR HEIGHTS OF FILL VARYING FROM 0.6 M-4.0 M



R.C.C. PIPE CULVERTS WITH 2 PIPE OF 1 METRE DIA AND CONCRETE CRADLE BEDDING FOR HEIGHT OF FILL FROM 4.0 M TO 8.0 M

PLATE 15



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

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	0.355	0.339	0.325	0.313	0.302	0.292	0.283	0.225	0.267	0.26
	1.0	0.714	0.686	0.66	0.638	0.618	0.597	0.582	0.565	0.55	0.535	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	2.88	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	1.315	1.295	1.275	1.255	1.235	1.215	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 × 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 × 0.75	0.693	0.66	0.632	0.607	0.584	0.562	0.545	0.528	0.513	0.497	0.485	0.473
	1 × 1	0.935	0.90	0.867	3.837	0.81	0.788	0.765	0.745	0.727	0.71	0.693	0.677
	1.25 × 1	1.175	1.135	1.1	1.068	1.037	1.01	0.985	0.96	0.937	0.917	0.898	0.88
	1.25 × 1.25	1.47	1.43	1.385	1.35	1.315	1.285	1.252	1.225	1.2	1.175	1.15	1.13
	1.5 × 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 × 1.5	2.14	2.08	2.03	1.99	1.95	1.91	1.87	1.835	1.8	1.765	1.74	1.71
Entry Sharp Edged	0.75 × 0.75	0.46	0.442	0.425	0.41	0.396	0.383	0.371	0.361	0.35	0.343	0.334	0.326
	1.0 × 0.75	0.615	0.593	0.572	0.553	0.535	0.52	0.505	0.492	0.48	0.468	0.457	0.447
	1 × 1	0.824	0.797	0.775	0.755	0.735	0.717	0.7	0.685	0.67	0.655	0.642	0.632
	1.25 × 1	1.03	1.005	0.98	0.955	0.933	0.914	0.895	0.877	0.86	0.844	0.828	0.815
	1.25 × 1.25	1.285	1.255	1.225	1.2	1.175	1.15	1.13	1.11	1.09	1.07	1.05	1.035
	1.5 × 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 × 1.5	1.85	1.81	1.78	1.75	1.72	1.69	1.665	1.64	1.615	1.59	1.57	1.55