IRC:SP:13-2004

GUIDELINES FOR THE DESIGN OF SMALL BRIDGES AND CULVERTS

INDIAN ROADS CONGRESS

GUIDELINES FOR THE DESIGN OF SMALL BRIDGES AND CULVERTS

Published by

INDIAN ROADS CONGRESS
Jamnagar House, Shahjahan Road,
New Delhi 110011
2004

Price Rs.400/-
(Plus packing & postage)

First Published	:	August, 1973
Reprinted	:	April, 1978
Reprinted	:	April, 1982
Reprinted	:	March, 1986
Reprinted	:	June, 1990
Reprinted	:	January, 1994
Reprinted	:	June, 1995
Reprinted	:	January, 1998
Reprinted	:	September, 2000 (Incorporates the amendments)
Reprinted	:	April, 2002
Reprinted	:	December, 2003
First Revision	:	June, 2004
Reprinted	:	November, 2004
Reprinted	:	December, 2005
Reprinted	:	February, 2007
Reprinted	:	February, 2008 (Incorporates the Errata) No. 1 dated 31.10.2007)
Reprinted	:	October, 2009
Reprinted	:	July, 2011
Reprinted	:	December, 2012

Printed at : India Offset Press, New Delhi - 110064
(1000 copies)

PREFACE

The Paper entitled "Guidelines for the Design of Small Bridges and Culverts" by Shri Goverdhan Lal who retired as Additional Director General (Roads), Government of India, was published as Paper No.167 in Volume XVIII–Part 2 of the Journal of the Indian Roads Congress which was presented and discussed at its Annual Session held at Bhubaneshwar in 1954. 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 small bridges and culverts for road projects correctly and expeditiously.

While the Paper, in its original form, still has great value, IRC during 1970's felt that this Paper needed revision since its first publication in light of the feedback obtained from members of the profession and also to reflect the changes in revised IRC Codes of Practice for design of bridges. Shri Goverdhan Lal, the author of the original Paper graciously agreed for its revision and for bringing it out as a Special Publication of the Indian Roads Congress. Accordingly, Special Publication No.13 "Guidelines for the Design of Small Bridges and Culverts" was published in 1973. and this was highly appreciated by the members of the profession.

Since 1973, the IRC Codes of Practice for design of bridges have undergone further major changes. Considering various changes and new codal provisions made since 1973, a Committee (B-11) for revision of IRC:SP:13-1973 was constituted under the Chairmanship of Shri C.R. Alimchandani in 1994. The work of the Committee was continued by the reconstituted Committee of General Design Features Committee (B-2) from Jan., 2000 (Personnel given below) :

G. Sharan	Convenor
Sudip Kumar De	Co-Convenor
B.N. Singh	Member-Secretary

Members

Dr. B.P. Bagish	M.V.B. Rao
C.S. Balramamurthy	N.C. Saxena
Sunil Dayal	V.K. Sinha
Suprio Ghosh	Mahesh Tandon
G.P. Joshi	G.S. Taunk
Anil Kumar	P.S. Tyagi
Vijay Kumar	N. Venkataraman
A.D. Narain	V. Velayutham
	Sanjay Kumar Nirmal

Corresponding Members

S. Ahmed	C.V. Kand
C.R. Alimchandani	S.A. Reddi
	Ashok Basa

Ex-Officio Members

President, IRC (S.S. Rathore)	DG(RD) & SS (N.K. Sinha)

Secretary, IRC
(G. Sharan)

The revised draft of SP:13 was prepared by Shri C.V. Kand who was ably helped by S/Shri K.S. Jangde, S.M. Sabnis, A.S. Khaire & S.P. Badhe of the earlier committee.

The revised draft was considered and discussed by the newly constituted General Design Features (B-2) Committee in its meeting held on the 25th August, 2000. The draft was given a final shape by a Sub-Committee comprising of Dr. B.P. Bagish, S/Shri A.D. Narain, G.S. Taunk, Ashok Basa and S.K. Nirmal.

The finalised draft of the above Sub-Committee was considered by the B-2 Committee in its meeting held on 23rd and 24th March, 2002 and the Committee approved the draft during its meeting held on 24th August, 2002. The final draft received from the Convenor, B-2 Committee was considered and approved by the Bridges Specifications and Standards (BSS) Committee in its meeting held on the 7th December, 2002 and by the Executive Committee in its meeting held on 7th December, 2002.

The Council during its meeting held on 3rd January, 2003 approved the draft subject to modification as per suggestions given by the members and authorised the Convenor, BSS Committee to approve the document after getting the same modified by the Convenor of B-2 Committee. In April, 2003 the draft modified by the Convenor, B-2 Committee incorporating the comments of the members of the Council was sent to Convenor, B.S.S. Committee for approval. Since the Convenor, B.S.S. Committee was not in position, the Council during the Mid-term Council meeting held in Pondicherry in June, 2003 decided that the document may be got approved by President, IRC after incorporating the modifications suggested by the Ministry of Road Transport & Highways and agreed upon by Convenor, B-2 Committee. The draft modified on the above lines was put up to the then President, IRC, Shri R.R. Sheoran for approval. He suggested certain modifications and also held discussions with Shri G. Sharan, Convenor, B-2 Committee and Secretary, IRC. He desired that the document may be further reviewed. Detailed review was carried out by Secretary, IRC with the help of Shri R.H. Sarma, Technical Consultant, IRC. The modified draft was circulated to Shri Indu Prakash, Convenor, BSS Committee, Shri V.K. Sinha, Co-Convenor, BSS Committee, Shri G. Sharan, Convenor, B-2 Committee, Shri R.R. Sheoran, Past President, IRC, Shri S.S. Momin, President, IRC and Dr. B.P. Bagish. Based on their comments the document was approved by President, IRC and is published in its present form.

CONTENTS

LIST OF PLATES

Plate No.

PERSONNEL OF THE BRIDGES SPECIFICATIONS AND STANDARDS COMMITTEE
(As on 7.12.2002)

1.	N.K. Sinha* (Convenor)	Director General (Road Dev.) & Spl. Secretary to the Govt. of India, Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001
2.	The Member (Technical), (Co-Convenor)	National Highways Authority of India, Plot No. G-5/6, Sector, Dwarka, New Delhi-110045
3.	The Chief Engineer (B) S&R (Member-Secretary)	(R.S. Ninan), Ministry of Road Transport & Highways, Transport Bhawan, New Delhi-110001

Members

4.	K.N. Agrawal	Addl. Director General (W), CPWD, Nirman Bhavan, New Delhi-110011
5.	C.R. Alimchandani	Chairman & Managing Director, STUP Consultants Ltd., 1004-5, Raheja Chambers, 213, Nariman Point, Mumbai-400021
6.	Ashok Basa	Director (T), B. Engineers and Builders Ltd., 72/A, Macheshwar Indl. Estate, Bhubaneswar-751017
7.	D.S. Batra	Consulting Engineer, Sir Owen Williams Innovestment Ltd., Innovestment House, 1072, Sector-37, Noida-201303
8.	S.S. Chakraborty	Managing Director, Consulting Engg. Services (I) Ltd., 57, Nehru Place, New Delhi-110019
9.	C. V. Kand	Consultant, E-2/136, Mahavir Nagar, Bhopal-462016
10.	D.K. Kanhere	Chief Engineer, Block No.A-8, Building No. 12, Haji Ali Officer's Qtrs., Mahalaxmi, Mumbai-400034
11.	Krishan Kant	Chief General Manager, National Highways Authority of India, Plot No.G-5/6, Sector-10, Dwarka, New Delhi-110045
12.	Ninan Koshi	DG(RD) & Addl. Secy., MOST (Retd.), 56, Nalanda Apartments, Vikashpuri, New Delhi-110018
13.	Dr. R. Kapoor	Director, Unitech India Ltd., Gurgaon
14.	Vijay Kumar	CE (Building), UP PWD, Lucknow-226001
15.	N.V. Merani	Principal Secy., Maharashtra PWD (Retd.), A-47/1344, Adarsh Nagar, Worli, Mumbai-400025
16.	M.K. Mukherjee	40/182, C.R. Park, New Delhi-110019

* *ADG (B) being not in position, the meeting was presided by Shri N.K. Sinha, DG (RD) & Spl. Secretary to the Govt. of India, MORT&H.*

17.	A.D. Narain	DG(RD) & Addl. Secy., MOST (Retd.), B-186, Sector 26, NOIDA-201301
18.	M.V.B. Rao	Area Coordinator, Bridge & Instrumentation Engineering, Central Road Research Institute, P.O. CRRI, New Delhi-110020
19.	Dr. T.N. Subba Rao	Chairman, Construma Consultancy (P) Ltd., 2nd Floor, Pinky Plaza, Mumbai-400052
20.	A. Ramakrishna	President (Operations) & Dy. Managing Director, Larsen & Toubro Ltd., ECC Constn. Group, Mount Ponnamallee Road, Mannapakkam, P.O. Box No. 979, Chennai-600089
21.	S.A. Reddi	Dy. Managing Director, Gammon India Ltd., Gammon House, Prabhadevi, Mumbai-400025
22.	Ramani Sarmah	Secretary to the Govt. of Meghalaya, Public Works Department, Lower Lachumiere, Shillong-793001
23.	N.C. Saxena	Executive Director, Intercontinental Consultants & Technocrats Pvt. Ltd., A-11, Green Park, New Delhi-110016
24.	G. Sharan	Director, NITHE, A-5, Institutional Area, Sector-62, Noida-201301 (UP)
25.	S.R. Tambe	Secretary, Maharashtra PWD (Retd.),72, Pranit J. Palkar Marg, Opp. Podar Hospital, Worli, Mumbai-400025
26.	Dr. M.G. Tamhankar	BH-1/44, Kendriya Vihar, Sector-11, Kharghar, Navi Mumbai-410210
27.	Mahesh Tandon	Managing Director, Tandon Consultants (P) Ltd., 17, Link Road, Jangpura Extn., New Delhi-110014
28.	P.B. Vijay	DG (Works), CPWD (Retd.), A-39/B, DDA Flats, Munirka, New Delhi-110062
29.	The Chief Engineer (NH)	M.P. Public Works Department, 'D' Wing, 1st Floor, Satpura Bhavan, Bhopal-462004
30.	The Principal Secy. to the Govt. of Gujarat	R&B Department, Block No. 14, 2nd Floor, New Sachivalaya, Gandhinagar-382010
31.	The Chief Engineer(NH)	Public Works Deptt., Writers' Building, Block 'G' 4th Floor, Kolkata-700001
32.	The Chief Engineer(NH)	U.P. Public Works Deptt., Lucknow-226001
33.	The Chief Engineer(NH)	Punjab P.W.D., B&R Branch, Patiala-147001
34.	The Chief Engineer(R) S&R, T&T	Ministry of Road Transport & Highways, Transport Bhavan, New Delhi-110001
35.	The Engineer-in-Chief (NH)	K.R. Circle, Bangalore-560001

36.	The Director	Highways Research Station, P.B. No. 2371, 76, Sardar Patel Road, Chennai-600025
37.	The Director & Head (Civil Engg.)	Bureau of Indian Standards, Manak Bhavan, 9, Bahadurshah Zafar Marg, New Delhi-110002
38.	The Dy. Director General	(B.K. Basu, VSM, SC) Dy. Director General (Bridges), Border Roads Directorate, Seema Sadak Bhawan, Naraina, Delhi Cantt., New Delhi-110010
39.	The Director, RDSO	Director (Bridges & Structure) Research, Design & Standards Organisation, Lucknow-226001
40.	The Director General (Works)	CPWD, Central Design Orgn., Nirman Bhavan, New Delhi-110011

Ex-Officio Members

41.	President, Indian Roads Congress	S.S. Rathore Secy. to the Govt. of Gujarat, R&B Deptt., Block No. 14/2, Sardar Patel Bhawan, Sachivalaya, Gandhinagar-382010
42.	DG(RD)	N.K. Sinha, D.G.(RD) & Spl. Secy., Ministry of Road Transport & Highways, Transport Bhavan, New Delhi-110001
43.	Secretary, Indian Roads Congress	Shri G. Sharan, Director, NITHE, A-5, Institutional Area, Sector-62, Noida-201301 (UP)

Corresponding Members

1.	M.K. Agarwal	Engineer-in-Chief (Retd.), H.No.40, Sector 16, Panchkula-134113
2.	Dr. V.K. Raina	B-13, Sector-14, Noida-201301
3.	Shitala Sharan	Chief Consultant, Consulting Engg. Services (I) Ltd. 57, Nehru Place, New Delhi-110019
4.	S.P. Khedkar	Hindustan Constn. Co. Ltd., Hincon House, Lal Bahadur Shastri Marg, Vikhroli (W), Mumbai-400083
5.	The Technical Director	(H. Guha Viswas), Simplex Concrete Piles (I) Pvt. Ltd., Vaikunt, 2nd Floor, 82, Nehru Place, New Delhi-110019

INTRODUCTION

A large number of small bridges and culverts form part of most of our highways. With the massive road development plans which our country has taken up, it is necessary to look for standardization of such structures so as to reduce the time spent on project preparation. However, there is need to exercise utmost care in their design to bring about economy in the overall cost of the project. With this objective in view, the revised Special Publication No. 13 "Guidelines for the Design of Small Bridges and Culverts" taking into account a number of major changes made in the IRC Codes of Practice for design of bridges has been brought out.

The following new Articles have been added now:

Article 1 (General Aspects); Article 13 (Geometric Standards, Specifications and Quality Control); Article 20 (Protection Work and Maintenance); Article 21 (Raft Foundations), Article 22 (C.D. Works in Black Cotton Soils) and Article 23 (Box Cell Structures). Other Articles have been regrouped and modifications incorporated wherever found necessary.

This revised Publication incorporates latest standard drawings for RCC solid slab superstructure upto 10 m span, both right and skew type, supported with masonry/RCC substructure upto 4 m height, pipe culverts and RCC box culverts. This Publication does not cover RCC substructure and superstructure with bearings and also large span structures, which would need specific detailed designs. This document is also not directly applicable for higher category roads like expressways and bridges/culverts with more than two lanes which would need specific detailed designs. For rural roads, reference may be made to IRC:SP:20 "Rural Roads Manual". It is hoped that this Publication would go a long way in expediting the work of project preparation of highway projects which invariably include a number of small bridges and culverts.

ARTICLE 1

GENERAL ASPECTS

1.1. **General :** Frequency of culverts and small bridges varies depending upon the region and terrain. The location, size and other details of such structures should be decided judiciously to cater for the discharge and balancing requirements. Number of culverts in 1 km length of road in India varies from one (flat country) to three in undulating regions whereas one small bridge (upto 30 m) is found within 1 to 4 km length of the road. Number of culverts may increase in hilly/ undulating terrain.

1.2. **Definitions**

1.2.1. **Bridges :** Bridge is a structure having a total length above 6 m between the inner faces of the dirt walls for carrying traffic or other moving loads over a depression or obstruction such as channel, road or railway.

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

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

1.2.4. **Culvert :** Culvert is a cross-drainage structure having a total length of 6 m or less between the inner faces of the dirt walls or extreme ventway boundaries measured at right angles thereto.

1.2.5. The Small Bridges and Culverts can be of following types:

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

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

1.3. **Standard Designs**

1.3.1. **MORT&H standard design for slab bridges :** Ministry of Road Transport & Highways (MORT&H) in standard design of slab bridges have proposed round figures for design span (c/c of supports). With a view to avoid confusion, same nomenclature of span is considered for culverts and small bridges. The design span of 6 m will have clear span of 5.60 m. The values of clear span, effective span and end to end of deck for which standard designs of slab bridges are available in Table 1.1.

Similarly type plans of MORT&H are available for skew slab bridges for right effective spans of 4 m, 6 m, 8 m and 10 m for skew angles of 15^0, 22.5^0, 30^0 and 35^0.

Table 1.1

Clear Span m	Effective Span m	End to End of Deck m
2.6	3	3.4
3.6	4	4.4
4.6	5	5.4
5.6	6	6.4
6.6	7	7.4
7.6	8	8.4
8.6	9	9.4
9.6	10	10.4

All these RCC spans will have tar paper bearings. The type plans of MORT&H are available at the above interval and if the design span does not exactly match with the available type design, the details of next higher span length be used.

1.3.2. **H.P. culverts :** Drawings of RCC pipe culverts are available for 1000 mm diameter and 1200 mm diameter of type NP3/NP4 conforming to IS:458. PSC pipes of NP4 type conforming to IS:784 may also be used for H.P. culverts.

1.3.3. **RCC boxes :** Following RCC box section standard design of MORT&H are available with or without earth cushion.

(a) **Single Cell :**

Culvert : 2mx2m, 5mx3m, 5mx4m, 5mx5m, 2mx3m, 3mx3m, 4mx3m, 4mx4m, 4mx5m

Small bridges : 6mx3m, 6mx4m, 6mx5m, 6mx6m, 7mx5m, 7mx6m, 7mx7m, 8mx5m, 8mx6m, 8mx7m

(b) **Double Cell :**

Culvert : 2mx2m, 2mx3m

Small bridges : 3mx2m, 3mx3m

(c) **Triple Cell :**

Small bridges : 2mx2m, 3mx3m

These are designed for varying bearing capacity of foundation stratum upto 20t/m^2. If the section at site does not exactly match with the available type design, details of higher section may be adopted.

Details of segmental masonry arch bridges without footpath for span 6 m and 9 m are available at **Plate 6** of this document.

1.4. **Length Related to Catchment Area :** It is generally found that when catchment area is upto 1 sq. km a culvert is required and for catchment area more than 1 sq. km a small bridge will be necessary.

ARTICLE 2

SITE INSPECTION

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

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

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

(iii) which has well defined banks;

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

(v) which offers a square crossing.

2.2. **Existing Drainage Structures :** If, there is an existing road or railway bridge or culvert over the same stream 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 inquiry will provide very useful information, namely, marks indicating the maximum flood level, the afflux, the tendency to scour, the probable maximum discharge, the likelihood of collection of brushwood during floods, and many other particulars. It should be seen whether the existing structure is too large or too small or whether it has other defects. All these should be carefully recorded.

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

ARTICLE 3

ESSENTIAL DESIGN DATA

3.1. 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 Standard Specifications & Code of Practice for Bridges – Section I, of IRC:5 Clauses 100–102, should be followed.

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

3.3. **Cross-sections :** For a sizable stream, at least 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 m or 1/1000 and with an exaggerated vertical scale of not less than 1 cm to 1 m or 1/100. Approximate distances, upstream and downstream of the selected site of crossing at which cross-sections should be taken are given in Table 3.1.

Table 3.1

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

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

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

3.5. **Highest Flood Level :** The highest flood level should be ascertained by intelligent local observation, supplemented by local enquiry, and marked on the cross-sections.

3.6. **Longitudinal Section :** The longitudinal section should extend upstream and downstream of the proposed site for the distances indicated in Table 3.1 and should show levels of the bed, the low water level and the highest flood level.

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

3.8. **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 2 to 3 meter below the lowest bed level. The location of each trial pit should be shown in the cross-section of the proposed site. The trial pit section should be plotted to show the kind of soils passed through. However depth of trial pit in soils shall be minimum 2 m for culverts and 3 m for small bridges.

For more detailed investigation procedure given in Cl. 704 of IRC:78-2000 may be referred to.

3.9. For very small culverts, one trial pit is sufficient. The result should be inserted on the cross-section.

ARTICLE 4

EMPIRICAL AND RATIONAL FORMULAE FOR PEAK RUN-OFF FROM CATCHMENT

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

4.2. **Dickens Formula**

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

Where

Q = the peak run-off in m^3/s and M is the catchment area in sq. km
C = 11 – 14 where the annual rainfall is 60 – 120 cm
= 14 – 19 where the annual rainfall is more than 120 cm
= 22 in Western Ghats

4.3. **Ryve's Formula :** This formula was devised for erstwhile Madras Presidency.

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

Where

Q = run-off in m^3/s and M is the catchment area in sq. km
C = 6.8 for areas within 25 km of the coast
= 8.5 for areas between 25 km and 160 km of the coast
= 10.0 for limited areas near the hills

4.4. **Ingli's Formula :** This empirical formula was devised for erstwhile Bombay Presidency

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

Where

Q = maximum flood discharge in m^3/s
M = the area of the catchment in sq. km

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

4.6. A correct value of C can only be derived for a given region from an extensive analytical study of the measured flood discharges vis-à-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 judgement of the engineer. Many other similar empirical formulae are in use but none of them encompasses all possible conditions of terrain and climate.

4.7. **Rational Formulae for Peak-off from Catchment**

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

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

Rainfall

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

Nature of Catchment

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

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

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

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

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

Where c is a constant

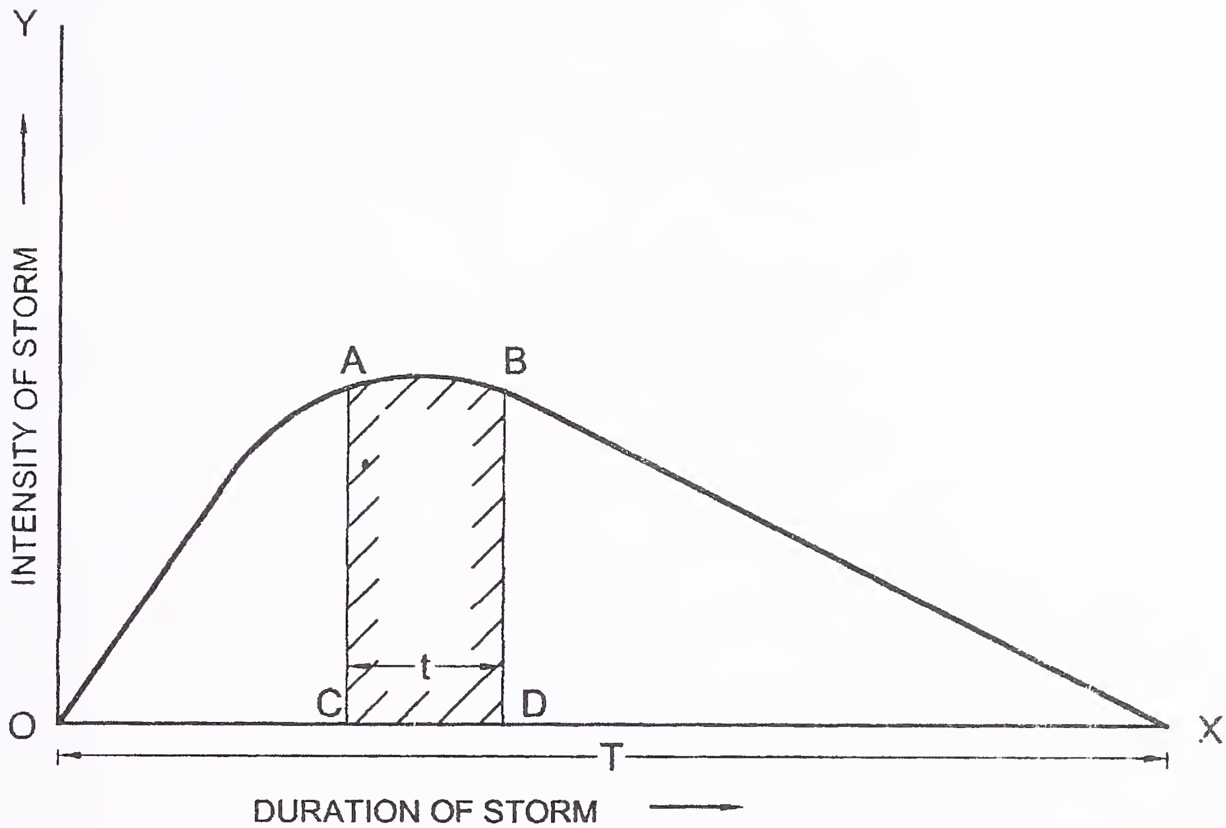


Fig. 4.1

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

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

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

Also,

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

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

---

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

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

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

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

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

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

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

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

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

Lastly, consider a very wet region of prolonged storms, where a storm drops, say, 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".

4.7.5. **"One-hour rainfall" for a region for designing waterway of bridges :** Suppose it is decided that a bridge should be designed for 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

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

or

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

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

$$I_o = \frac{F}{2}\left(1+\frac{1}{T}\right) \qquad \text{... (4.8)}$$

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

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_o of the region for the purpose of designing the waterways of bridges in that region.

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

I_o 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_o 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_o in absence of other suitable data. However, the values are upto the year 1966 and efforts are being made to obtain the current updated values.

Start with I_o 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.

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

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

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

Where

t_c	=	the concentration time in hours
L	=	the distance from the critical point to the structure in km.
H	=	the fall in level from the critical point to the structure in m.

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

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

4.7.6. **The critical or design intensity :** The critical intensity for a catchment is that maximum intensity which can occur in a time interval equal to the concentration time t_c of the catchment during the severest strom (in the region) of a given frequency I_c. Since each catchment has its own t_c it will have its own I_c.

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

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

Combinating this with Equation (4.8), we get

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

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

$$Q = 0.028\ A\ I_c\ m^3/s$$

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

Then

$$Q = 0.028\ PAI_c \qquad \text{... (4.11)}$$

Where

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

The principal factors governing P are : (i) porosity of the soil, (ii) area, shape and size of the catchment, (iii) vegetation cover, (iv) surface storage viz. existence of lakes and marshes, and (v) initial state of wetness of the soil. Catchments vary so much with regard to these characteristics that it is evidently impossible to do more than generalize on the values of P. Judgement and experience must be used in fixing P. Also see Table 4.1 for guidance.

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

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

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

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

In hydrological theories it is assumed that the spread of the storm is equal to the area of the catchment. Therefore in **Fig. 4.2** the area "*a*" is taken to be the same as the area of the catchment. The effect of this assumption can lead to errors which, on analysis have been found to be limited to about 12 per cent [5].

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

$$Q = 0.028\ PfAI_C \qquad \text{... (4.12)}$$

Also combining with Equation (4.10b)

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

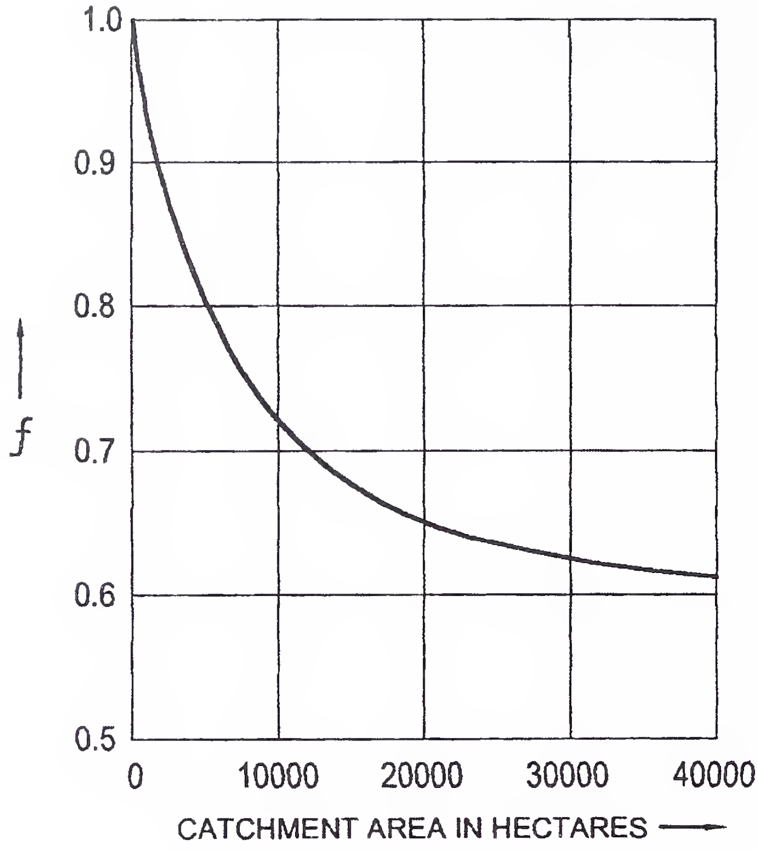


Fig. 4.2 'f' curve

$$Q = \frac{0.028 \, A I_0 \, 2 \, f \, P}{t_c + 1} \qquad \text{... (4.14)}$$

$$= A \, I_0 \, \lambda$$

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

In the equation 4.14(a), 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 discussion that the components of λ are function of A,L and H of the catchment.

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

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

Step 2 : Estimate I_0 for the region, preferably from rainfall records failing that from local knowledge.

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

Where F is rainfall in cm dropped by the 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.

Then calculate

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

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

4.7.11. **Example :** Calculate the 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 resulted in 15 cm of rain in 2.5 hours.

Solution :

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

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

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

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

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

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

$$Q = P\, I_o\, A\left(\frac{0.056}{t_c + 1}\right)$$

For $I_0 = 1$ cm, the equation becomes,

$$Q = PA \left(\frac{0.056}{t_c + 1} \right) \qquad \ldots (4.15)$$

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

Plate 2 can be used for small culverts with basins upto 1500 hectares or 15 sq. km. The value of run-off read from **Plate 2** are of "One Hour rainfall", I_O, of one cm. These values have to be multiplied by the I_0 of the region. An example will illustrate the use of this Plate.

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

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

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

$$q = 10.2 \text{ m}^3/\text{s}$$

Multiply with I_O.

$$Q = 10.2 \times 2.5 = 25.5 \text{ m}^3/\text{s}$$

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

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

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

ARTICLE 5

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

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

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

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

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

$$R = \frac{A}{P} \text{ (in m)} \qquad \text{... (5.1)}$$

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

$$V = \frac{1}{n} \left(R^{2/3} \; S^{1/2} \right) \qquad \text{... (5.2)}$$

Where

V = the velocity in m/s 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 5.1. Judgement and experience are necessary in selecting a proper value of n for a given stream.

Table 5.1 Rugosity Co-efficient, n

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

5.5. **Calculation of Discharge**

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

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

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

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

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

5.6. When the cross-section is not plotted to the natural scale (the same scale horizontally and vertically), the wetted perimeter cannot be scaled off directly from the section and has to be calculated. Divide up the wetted line into a convenient number of parts, AB, BC and CD, etc. **(Fig. 5.1)**. Consider one such part, say PQ, let PR and QR be its horizontal and vertical projections. Then $PQ = \sqrt{(PR^2 + QR^2)}$. Now, PR can be measured on the horizontal scale of the given cross-section and QR on the vertical. PQ can then be calculated. Similarly, the length of each part is calculated. Their sum gives the wetted perimeter.

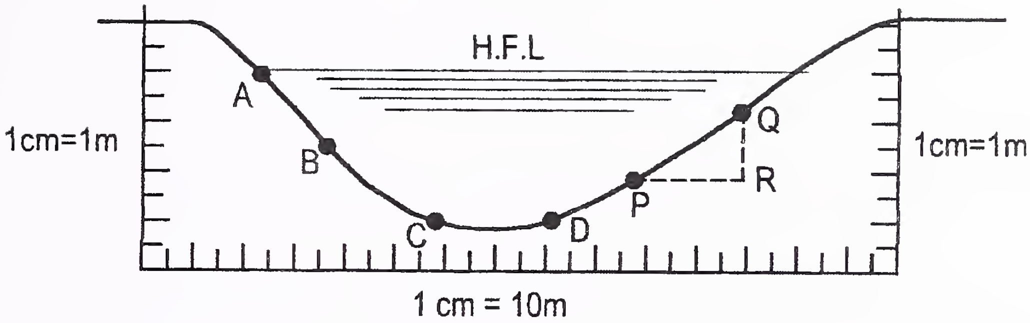


Fig. 5.1

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

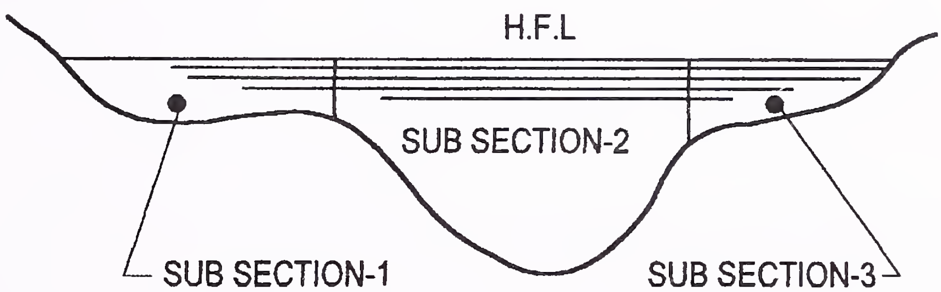


Fig. 5.2

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

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

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

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

ARTICLE 6

DESIGN DISCHARGE

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

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

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

6.2. Fixing Design Discharge

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

6.2.2. **Sound economy :** The designer is not expected to aim at designing a structure of such copious dimensions that it should pass a flood of any possible magnitude that can occur during the lifetime of the structure. Sound economy requires that the structure should be able to pass easily floods of a specified frequency (once in 50 years) and that extraordinary and rare floods should pass without causing excessive damage to the structure or the road.

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

ARTICLE 7

ALLUVIAL STREAMS LACEY'S EQUATIONS

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

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

7.3. A stream, whose bed and banks are composed of loose granular material, that has been deposited by the stream and can be picked-up and transported again by the current during flood, is said to flow through incoherent alluvium and may be briefly referred to as an alluvial stream. 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.

7.4. **Lacey's Equation :** When an alluvial stream carrying known discharge Q has come to regime, it has a regime wetted perimeter P, a regime slope S, and regime hydraulic mean depth R. In consequence, it will have a fixed area of cross-section A and a fixed velocity V.

For these regime characteristics of an alluvial channel, Lacey suggests[18] the following relationships. It should be noted that the only independent entities involved are Q and K_{sf}. The K_{sf} is called silt factor and its value depends on the size and looseness of the grains of the alluvium. Its value is given by the formula:

$$K_{sf} = 1.76 \sqrt{d_m} \qquad \text{... (7.1a)}$$

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

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

(a) Regime Cross-Section

$$P = 4.8Q^{1/2} \qquad \text{... (7.1b)}$$

(This may vary from 4.5 $Q^{1/2}$ to 6.3 $Q^{1/2}$ according to local conditions)

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

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

(a) Regime Velocity and Slope

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

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

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

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

7.5. **The Regime Width and Depth :** Provided a stream is truly 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 (7.1a).

That is

$$W = P = 4.8\,Q^{1/2} \qquad \text{... (7.2a)}$$

Also, the normal depth of scour D on a straight and unobstructed part of a wide stream may be taken as equal to the hydraulic mean radius R in Equation (7.1c). Hence,

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

ARTICLE 8

LINEAR WATERWAY

8.1. **The General Rule for Alluvial Streams:** The linear waterway of a bridge across a wholly alluvial stream should normally be kept equal to the width required for stability, viz., that given by Equation (7.2a).

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

8.3. In the ultimate analysis it may be found in some such cases, that it is cheaper to adopt a linear waterway for the bridge somewhat in excess of the regime width given by Equation (7.2a). But as far as possible, this should be avoided. When the adopted linear waterway exceeds the regime width it does not follow that the depth will become less than the regime depth D given by Equation (7.2b). Hence, such an increase in the length of the bridge 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.

8.4. **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.

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

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

8.7. As regards streams that overflow their banks and create very wide surface widths with shallow side sections, judgement 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 18.

ARTICLE 9

NORMAL SCOUR DEPTH OF STREAMS

9.1. Alluvial Streams

9.1.1. What is the significance of the Normal Scour Depth? If a constant discharge were passed through a straight stable reach of an alluvial stream for an indefinite time, the boundary of its cross-section should ultimately become 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 magnitude and duration of the flood discharge carried by it would govern the shape of the flood section of any natural stream. Some observers have found that curves representing the natural stream sections during sustained floods have sharper curvature in the middle than that of an ellipse. In consequence, it is believed that Lacey's normal depth is an under estimate when applied to natural streams subject to sustained floods. However, pending further research, Lacey's equations may be applied.

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

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

Where

W = the regime width of the stream

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

D = The normal scour depth when L = W

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

According to Clause 703 of IRC:78-2000.

$$d_{sm} = 1.34 \left(\frac{D_b^2}{k_{sf}} \right)^{1/3} \qquad (9.1)$$

Where

D_b = discharge in m³/s per m width

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

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

d_{sm} = normal scour depth in m.

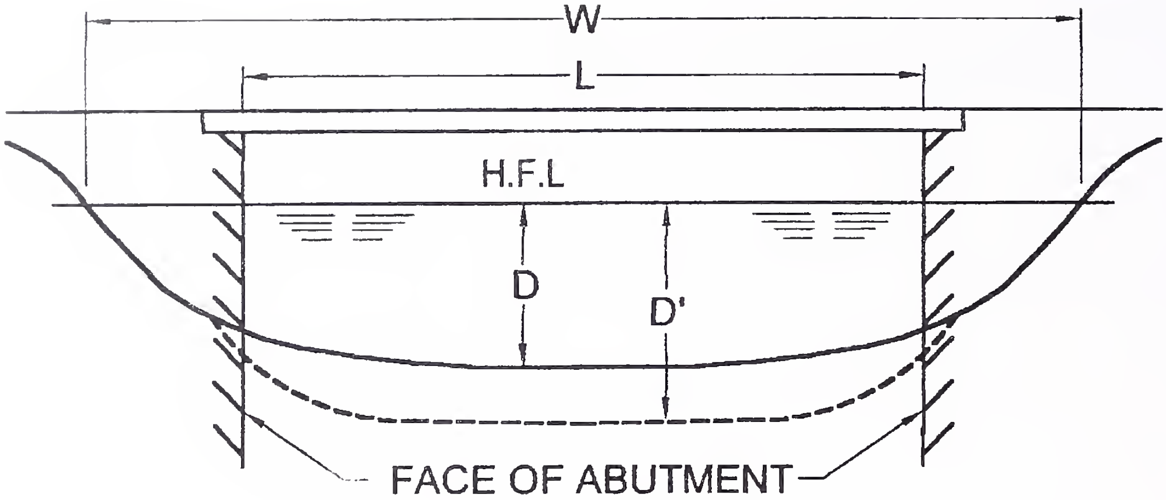


Fig. 9.1

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

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

9.2. **Quasi-Alluvial Streams**

9.2.1. **Some streams are not wholly alluvial :** A stream may flow between banks which are rigid in so far as they successfully resist erosion, but its bed may be composed of loose granular material which the current can pick-up and transport. Such a stream may be called quasi-alluvial to distinguish it, on the one hand, from a stream with wholly rigid boundaries and, on the other, from 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.

9.2.2. 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.

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

9.2.3. In any such case the width W of the section, being fixed between the rigid banks, 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 occurrence of high flood.

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

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

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

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

$$\begin{aligned} Q &= \text{area x velocity} \\ &= R\,P\,C\,R^{2/3}\,S^{1/2} \\ &= K\,R^{5/3} \qquad \text{... (9.3)} \end{aligned}$$

where K is a constant.

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

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

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

$$\begin{aligned} Q &= \text{area x velocity} \\ &= (PR)\,V = (WD)\,V, \text{ where V is the mean velocity} \qquad \text{.....(9.4)} \end{aligned}$$

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

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

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

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

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

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

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

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

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

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

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

$$= (PR)\ \left(C R^{2/3}\ S^{1/2}\right) \qquad \text{... (9.6a)}$$

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

$$Q = (P_1 R_1)\ \left(C R^{2/3}\ S^{1/2}\right) \qquad \text{... (9.6b)}$$

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

9.3. **Scour in Clay and Bouldary Strata :** There are no rational methods for assessment of scour in clay or bouldary strata. Guidelines for calculating silt factor for bed materials consisting of gravels and boulders as given in Appendix-I of IRC:78-2000 may be adopted and are reproduced in paras 9.3.1 and 9.3.2.

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

Sand - Where ϕ is equal to or more than 15° even if c (Cohesion of soil) is more than 0.2 kg/cm^2
(Silt factor K_{sf} will be calculated as per provisions of para 7.4 or Table 7.1).

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

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

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

Where

$$\begin{aligned} c &= \text{Cohesion in kg/cm}^2 \text{ and} \\ F &= 1.5\,\phi \text{ for } \phi \geq 10^0 < 15^0 \\ &= 1.75 \text{ for } \phi \geq 5^0 < 10^0 \\ &= 2.0 \text{ for } \phi < 5^0 \end{aligned} \qquad \text{... (9.9)}$$

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

D_b = discharge per unit width

9.3.2. **Bouldary strata :** There is no rational method to assess scour in bouldary strata of boulders or pebbles. In the absence of any formula K_{sf} may be determined as per Clause 703.2.2 of IRC:78 and adopted. If, say, average size of pebbles is d_b

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

for d_b = 50 mm

$$K_{sf} = 1.76\,(50)^{1/2} = 12.4 \qquad \text{... (9.10)}$$

$$\text{Scour depth} = 1.34 \left(\frac{D_b^2}{K_{sf}} \right)^{1/3} = 1.34 \left(\frac{D_b^2}{12.4} \right)^{1/3}$$

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

ARTICLE 10

MAXIMUM SCOUR DEPTH

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

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

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

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

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

10.4. Rules for finding Maximum Scour Depth. The rules for calculating the maximum scour depth from the normal scour depth are:

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

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

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

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

10.6. **Example 1.** A bridge is proposed across an alluvial stream (K_{sf}= 1.2) carrying a discharge of 50 m^3/s. Calculate the depth of maximum scour when the bridge consists of (a) 3 spans of 6 m and (b) 3 spans of 8 m

Regime surface width of the stream

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

Regime depth

$$D = 0.473 \frac{Q^{1/3}}{K_{sf}^{1/3}} = \frac{0.473 \times 50^{1/3}}{(1.2)^{1/3}} = 1.64 \text{ m}$$

Maximum scour depth

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

50/18, i.e., 2.778 cumecs

$d_{sm} = 1.34 (2.778^2/1.2)^{1/3} = 2.49$ m

(i) Maximum depth of scour for pier

$= 2\ d_{sm} = 2 \times 2.49 = 4.98$ m

(ii) Maximum depth of scour for abutment

$= 1.27\ d_{sm} = 1.27 \times 2.49 = 3.16$ m

(b) When span is 3 x 8 m , D_b the discharge per metre width is

50/24, i.e., 2.083 cumecs

$d_{sm} = 1.34 (2.083^2/1.2)^{1/3} = 2.055$ m

(i) Maximum depth of scour for pier

$= 2\ d_{sm} = 2 \times 2.055 = 4.11$ m

(ii) Maximum depth of scour for abutment

$= 1.27\ d_{sm} = 1.27 \times 2.055 = 2.61$ m

ARTICLE 11

DEPTH OF FOUNDATIONS

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

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

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

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

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

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

ARTICLE 12

SPAN AND VERTICAL CLEARANCE

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

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

For Masonry arch bridges $S = 2H$

For RCC Slab Bridges $S = 1.5H$

Where

S = Clear span length in metres

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

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

Table 12.1

Discharge in m^3/s		Minimum vertical clearance in mm
Upto 0.30	-	150
Above 0.3 and upto 3.0	-	450
Above 3 and upto 30	-	600
Above 30 and upto 300	-	900
Above 300 and upto 3000	-	1200
Above 3000	-	1500

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

Further to keep the free board of approaches not less than 1750 mm (Clause 107.1 of IRC:5) the vertical clearance in slab/box cell bridges may be increased suitably.

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

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

12.4. **The Number of Spans:**

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

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

$$L = NS$$

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

12.5. To facilitate inspection and carrying out repairs, the minimum vent height of culverts should normally be 1500 mm. The vent size of irrigation culverts may be decided considering the actual requirements and site condition. For pipe culverts minimum diameter should be 1000 mm.

ARTICLE 13

GEOMETRIC STANDARDS, SPECIFICATIONS AND QUALITY CONTROL

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

13.2. Geometric Standards

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

NH and SH	-	12 m
MDR	-	8.4 m

Fig. 13.1 gives width for 4-lane roads.

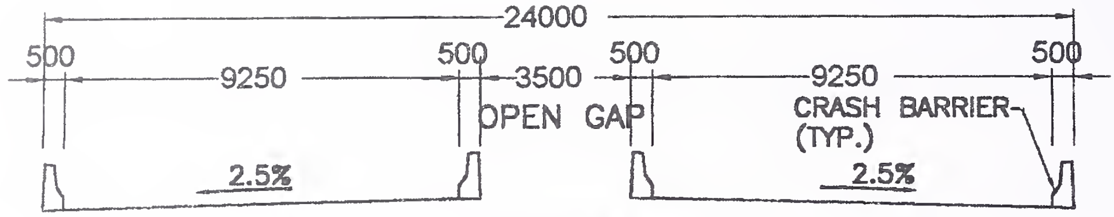
13.2.2. **Design loads for 2-lane roadway :** Design loading for culverts and small bridges should be as below:

Village Road and ODR (Rural Roads)	-	2-lanes IRC Class A
NH, SH and MDR	-	70 R or 2-lanes of Class A whichever gives worst effect

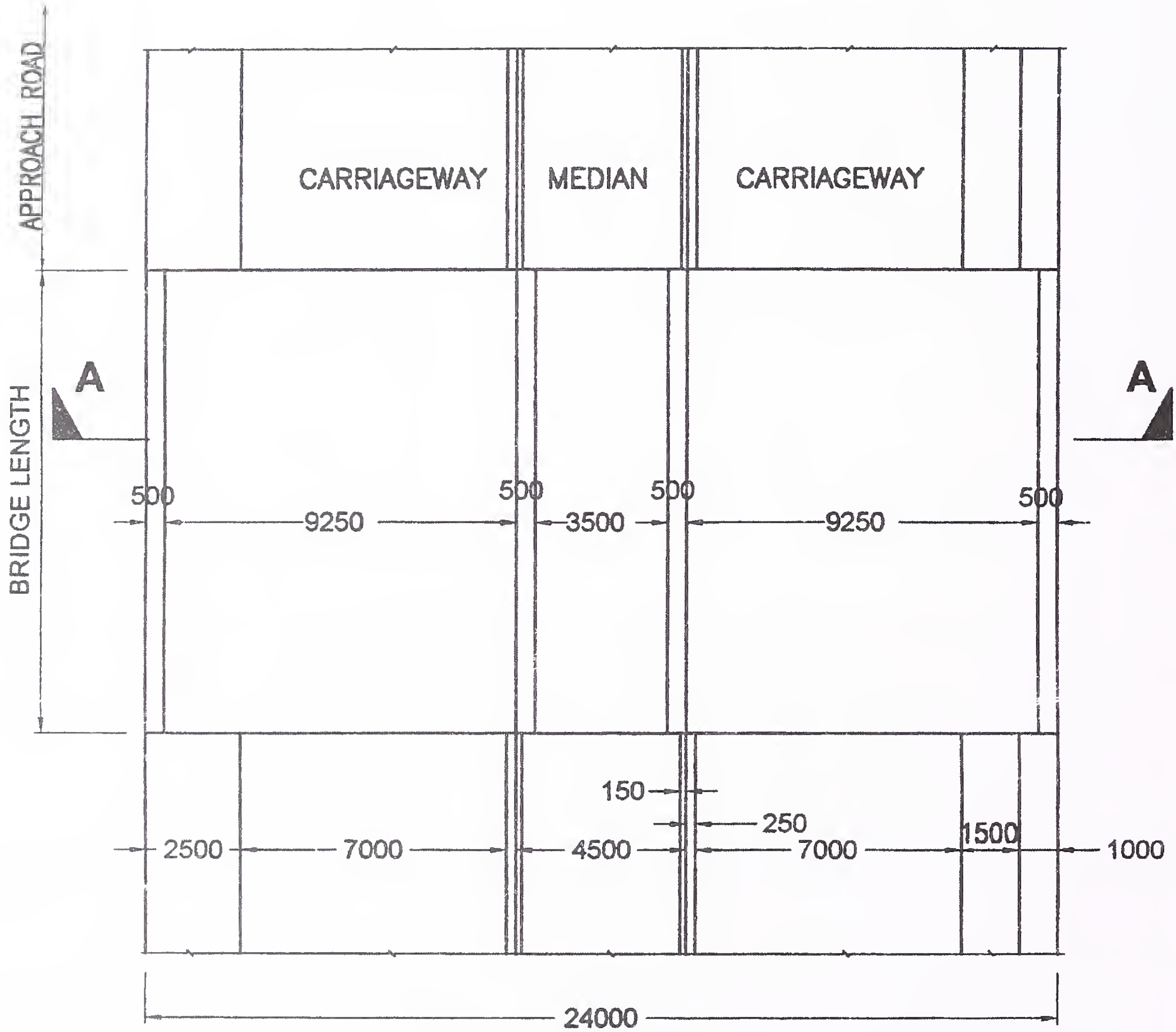
13.2.3. **Width of roadway :** The width of a culvert and small bridge (along the direction of flow) should be such that the distance between the outer faces of the parapets will equal the full designed width of the formation of the road. Any proposed widening of the road formation in the near future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

13.2.4. In small bridges, the width (parallel to the flow of the stream) should be sufficient to give a minimum clear carriageway of 4.25 m for a single-lane bridge and 7.5 m for a two-lane bridge between the inner faces of the kerbs or wheel guards. Extra provision should be made for footpaths, etc., if any are required.

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



SECTION A-A



PLAN

Fig. 13.1

13.3. **Design :**

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

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

(2) Duplicate work of consolidation of approaches giving rise to extra cost.

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

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

For small size structures, RCC pipe culverts with single row or upto six rows of R.C.C. pipes, depending upon the discharge may be used as far as possible, as they are likely to prove comparatively cheaper than slab culverts.

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

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

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

13.4 **Numbering of Culverts and Small Bridges :**

13.4.1. The number of culvert/small bridge is indicated in each km. For instance number 21/8 represents the 8th CD structure in kilometer 21.

The information regarding (1), the number of spans (2), clear span length in m and (3) the type of decking or culverts is indicated below:

Number of spans, clear span in m, type of culvert/small bridges are given, e.g., $1 \times 2 \times S$ means 1 span of 2 m with RCC solid slab. For various types of culverts and small bridges the suffix (3) will be represented by –

RCC Solid slab	-	S	Pipe culvert	-	P
Arch	-	A	Box Type	-	B
Stone Slab	-	ST			

13.4.2. The number of the structure shall be inscribed near the top of he left hand side parapet wall as seen by traffic in the end elevation when approaching the structure from each direction.

In situtations where instead of parapet walls, the structure is provided with railings, but having no end supporting pillars on which the number could be inscribed, the number of the structure shall be indicated by means of a numbering plate of the size 300 x 300 mm. There shall be two such numbering plates, one for each direction of travel. The plates shall be welded or fixed securely to the railing on the left hand side facing the carriageway as close to the entrance to the structure as possible.

In the case of buried culverts, such as pipe culverts, where there is usually no parapet walls or railings at the roadway level, two stone or RCC posts, having a cross-section of 150x150 mm and exposed height 300 mm shall be set up, one on each side to mark the position of the culvert. Care shall be taken to locate the marker posts fully outside the prescribed roadway width. The culvert number shall not be engraved on the marker posts but be either engraved or painted at their base.

13.4.3. For details reference may be made to "Recommended Practice for Numbering Bridges and Culverts", IRC:7-1971.

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

13.5.1. **Parapet wall and railing :** For culverts, where parapet walls are provided they shall be of plain concrete M15 grade or brick or store masonry with 450 mm top width. In case of pipe culverts no parapet walls are needed and guard stones would be adequate except for culverts on hill roads. Guard stones provided shall be of size 200x200x600 mm. Railings as given in Standard Drawings of MORT&H may also be provided for culverts and small bridges. Railings or parapets shall have a minimum height above the adjacent roadway or footway safety kerb surface of 1.1 m less one half the horizontal width of the top rail or top of the parapet. Crash barriers may be provided when they are found functionally required. Crash barriers when provided shall conform to provisions in IRC:5 and while adopting MORT&H standard drawings, the design of deck slab shall be checked for provision of crash barriers.

13.5.2. **Wearing coat :** Normally, the wearing surfaces of the road shall be carried over the culverts/small bridges. However for low category road which do not have bituminous surfaces, concrete wearing coat of average 75 mm need be adopted and approach profile may be suitably graded.

13.5.3. **Approach slab :** Approach slab can be dispensed with in case of culverts.

13.5.4. **Deck slab :** M 20 concrete for moderate and M 25 concrete for severe conditions of exposure and high strength deformed bars conforming to IS:1786 are specified for the deck slabs.

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

RCC slab shall rest on tar paper over abutment/pier cap.

13.5.6. **Pier/abutment cap/coping :** The minimum thickness of reinforced cap over solid PCC/RCC substructure shall be 200 mm and that in case of masonry substructure shall not be less than 500 mm. The minimum grade of concrete shall be M 20 and M 25 for moderate and severe conditions of exposure respectively. However, the coping over the returns may be of M 15 grade and thickness not less than 100 mm.

13.5.7. **Section of pier abutment and returns :** The abutment and pier sections should be so designed as to withstand safely the worst combination of loads and forces as specified in the IRC:6-2000.

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

Table 13.1

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

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

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

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

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

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

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

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

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

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

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

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

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

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

(1) Raft foundations are suitable for all types of structures other than pipe culverts.
(2) Protection needs to be provided in the form of apron.
(3) Cut-off should be done first, i.e., before the raft. Immediately, after the raft is complete, aprons on U/s and D/s should be completed.
(4) Details of raft foundation are given in Article 21.

13.6. **Quality Control**

13.6.1. Although, the work of culverts and small bridges is simple it is necessary to have quality control in the work of stone/brick masonry and concrete in deck slab, bar bending, etc. Reference may be made to ''Guidelines on Quality Systems for Road Bridges'', IRC:SP:47-1998.

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

13.7. **Setting out of culverts and small bridges :** Setting out of culverts and small bridges should be done from 4 masonry pillars, two in the direction of road and two along the stream, all placed along two center lines. The top of pillars in the direction of road should be at the proposed top level of deckslab. Two lines, one along the direction of stream and the other along the center line of road should be inscribed on one of the pillars and all distances should be measured with respect to these lines. The pillars should be placed sufficiently away from the zone of excavation.

13.8. **Masonry Work**

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

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

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

13.9. **Concrete**

13.9.1. According to IRC:21, the minimum grade of plain concrete is M 15 of concrete and that of RCC is M 20. The size of metal to be used for RCC slabs and the grading of aggregates are specified in relevant codes. It is advisable to use power driven concrete mixer. Similarly, vibrators should also be made available. Furthermore, precast concrete cover blocks must be provided to ensure bottom cover to reinforcement. Water cement ratio must be limited to 0.45 maximum. In case of use of Plasticiser w/c ratio can be restricted to 0.4. Size of coarse aggregate will be 20 mm for RCC and upto 40 mm for plain concrete.

13.10. **Bar bending :** Lengths of bars and numbers are given in standard drawings. Cutting of bars from available stock must be done carefully. Generally, tendency of cutting bars of required lengths and discarding pieces of shorter lengths give rise to greater wastages. Normally staggered overlaps to the extent of 25 per cent may be provided. Calculated quantities of steel are increased suitably to account for overlaps, its length conforming to IRC:21. Steel chairs should be provided for maintaining correct position of top bars.

ARTICLE 14

STRUCTURAL DETAILS OF SMALL BRIDGES AND CULVERTS

14.1. **Abutment and Wing Wall Sections :** For RCC slab culverts designed for IRC single lane of class 70 R loading or 2-lanes of IRC class A loading, the abutment and wing wall sections upto 4 m height for a minimum bearing capacity of the soil of 16.5 t/m^2 are given in **Plate 5.** These sections are not applicable for seismic zones IV and V.

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.

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

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

14.3. **Unreinforced Masonry Arches : Plate 6** shows the details of arch ring of segmental masonry arch bridges without footpaths for spans 6 m and 9 m.

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

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

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

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

Table 14.2 Influence Line Ordinates for Horizontal Reaction (H) Vertical Reaction at Support (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 (Fig. 14.1)

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

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

14.4. RCC Slabs

14.4.1. The details of RCC slabs to be used for culverts and bridges at right crossings and skew crossings (with and without footpaths) based on MORT&H's standard drawings are given in **Plates 7 to 12** as brought out below:

Right crossings (with and without footpaths)

Plate 7 – General Notes
Plate 8 – General arrangement details
Plate 9 – Depth of slab and quantities per span

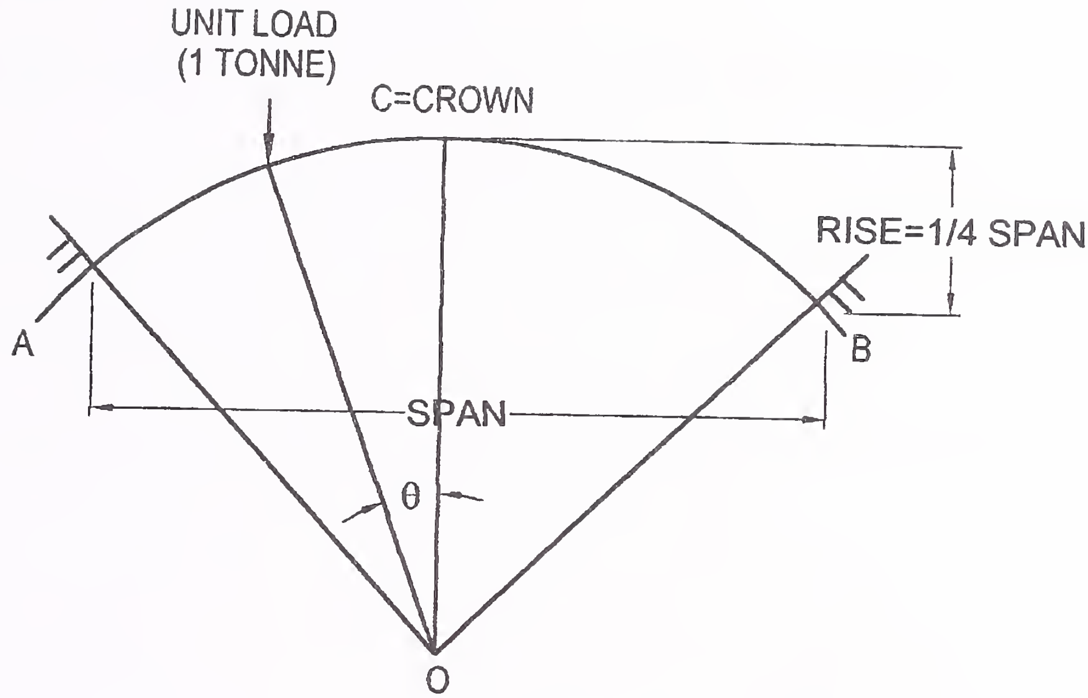


Fig. 14.1

Skew crossings (with and without footpaths)

Plate 10 – General Notes
Plate 11 – General arrangement details
Plate 12 – Depth of slab and quantities per span

14.4.2. For carriageway widths less or more than that prescribed in the **Plates 8 and 11** quantities can be worked out proportionately based on the actual carriageway widths.

14.5. **Box Cell Structures :** The details for single cell box upto 8 m opening, for double cell upto 3 m opening of each cell and triple cell upto 3 m opening of each cell with and without earth cushion for varying bearing capacity upto 20 t/m^2 based on MORT&H's standard drawings are given in **Plates 13 to 22** as brought out below:

Plate 13 – General Notes
Plate 14 – Index Sheet
Plate 15 to 20 – General arrangement
Plate 21 – Quantities of steel and concrete
Plate 22 – Floor Protection works

14.6. **RCC Pipe Culverts :** The details of pipe culverts of 1 m dia, with single or double pipes having cement concrete or granular materials in bed are given in **Plates 23 to 26.**

14.7. In this document the drawing of abutments and wing walls in plain cement concrete upto 4 m height has been included. For other sub-structure and foundations in R.C.C. and P.C.C./ masonry, the design details may be worked out as per relevant IRC Codes depending upon the type of superstructure and foundation conditions.

ARTICLE 15

ELEMENTS OF THE HYDRAULICS OF FLOW THROUGH BRIDGES

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

15.2. **Broad Crested Weir Formulae applied to Bridge Openings : In Fig. 15.1,** X-X is the water surface profile, and Z-Z the total energy line. At Section 1, the total energy.

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

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

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

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

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

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

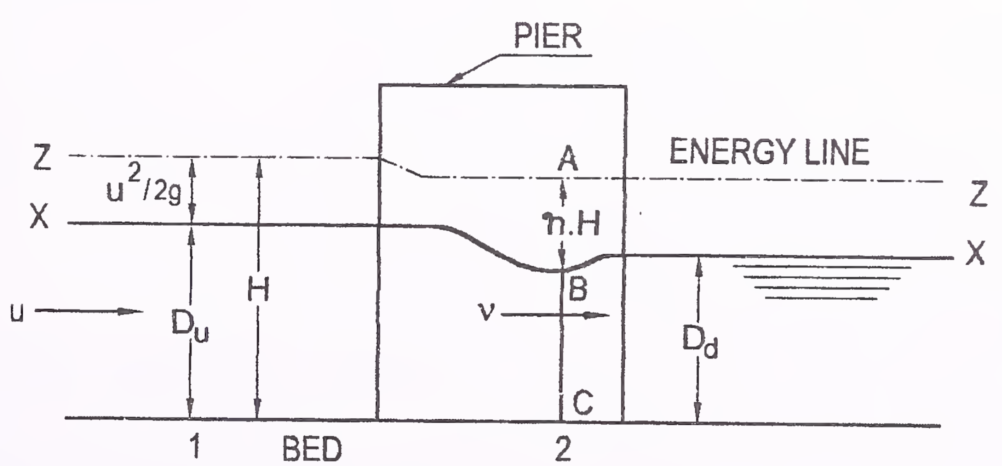


Fig. 15.1

The area of flow at Section 2,

$$a = BC \times \text{linear waterway}$$

$$= (1-n)\,HL$$

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

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

Therefore, the discharge through the bridge

$$Q = av$$
$$= (1\text{-}n)\ HL\ (2gn\ H)^{1/2}$$

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

$$Q = C_w\,(1\text{-}n)\ HL\ (2gn\ H)^{1/2}$$
$$= C_w\ \sqrt{2g}\ LH^{3/2}\left(n^{1/2} - n^{3/2}\right) \qquad \ldots (15.4)$$

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

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

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

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

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

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

C[illegible] with Eq. (15.1)

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

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

On exit from the bridge, some of the velocity head is reconverted into potential head due to the expansion of the section and the water surface is raised, so that D_d is somewhat greater than BC, i.e. greater than 66.7 per cent of H. In fact, observations have proved that, in the limiting condition, D_d can be 80 per cent of D_u. Hence, the following rule :

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

The fact that the downstream depth D_d has no effect on the discharge Q, nor on the upstream depth D_u when the afflux is not less than $^1/_4$ D_d is due to the formation of the "Standing Wave"

The coefficient Cw 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

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

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

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

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

or

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

Then

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

Put $D_u - D = h^1$

Then,

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

The discharge through the Section 2,

$Q = a\ v$

Substituting

$$Q = LD^1\sqrt{2g}\left(h^1 + \frac{u^2}{2g}\right)^{1/2} \qquad \dots (15.6)$$

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

$$Q = LD_d\sqrt{2g}\left(h' + \frac{u^2}{2g}\right)^{1/2} \qquad \dots (15.7)$$

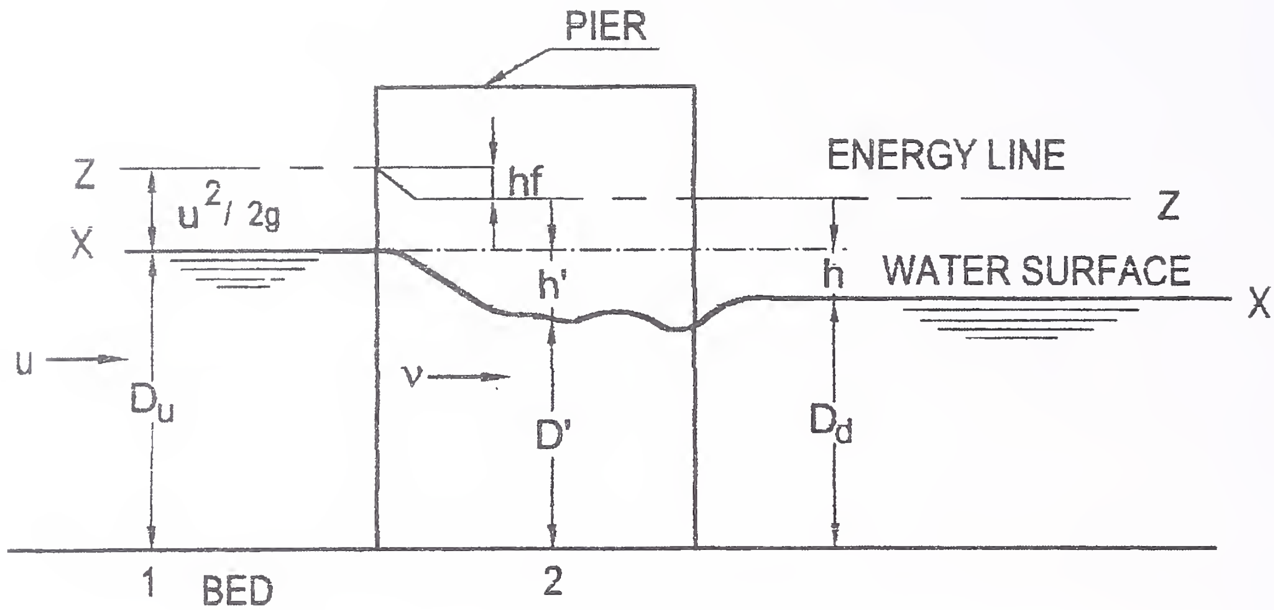


Fig. 15.2

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 $eu^2/2g$ represents the velocity energy that is converted into potential head.

Then

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

Substituting in equation (15.7)

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

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

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

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

15.4. **In Conclusion :** Let us get clear on some important points

(1) In all these formulae D_d is not affected in any way by the existence of the bridge. It depends only on the conveyance factor and slope of tail race. D_d has, therefore, got to be actually measured or calculated from area – slope data of the channel as explained already in Article 7.

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

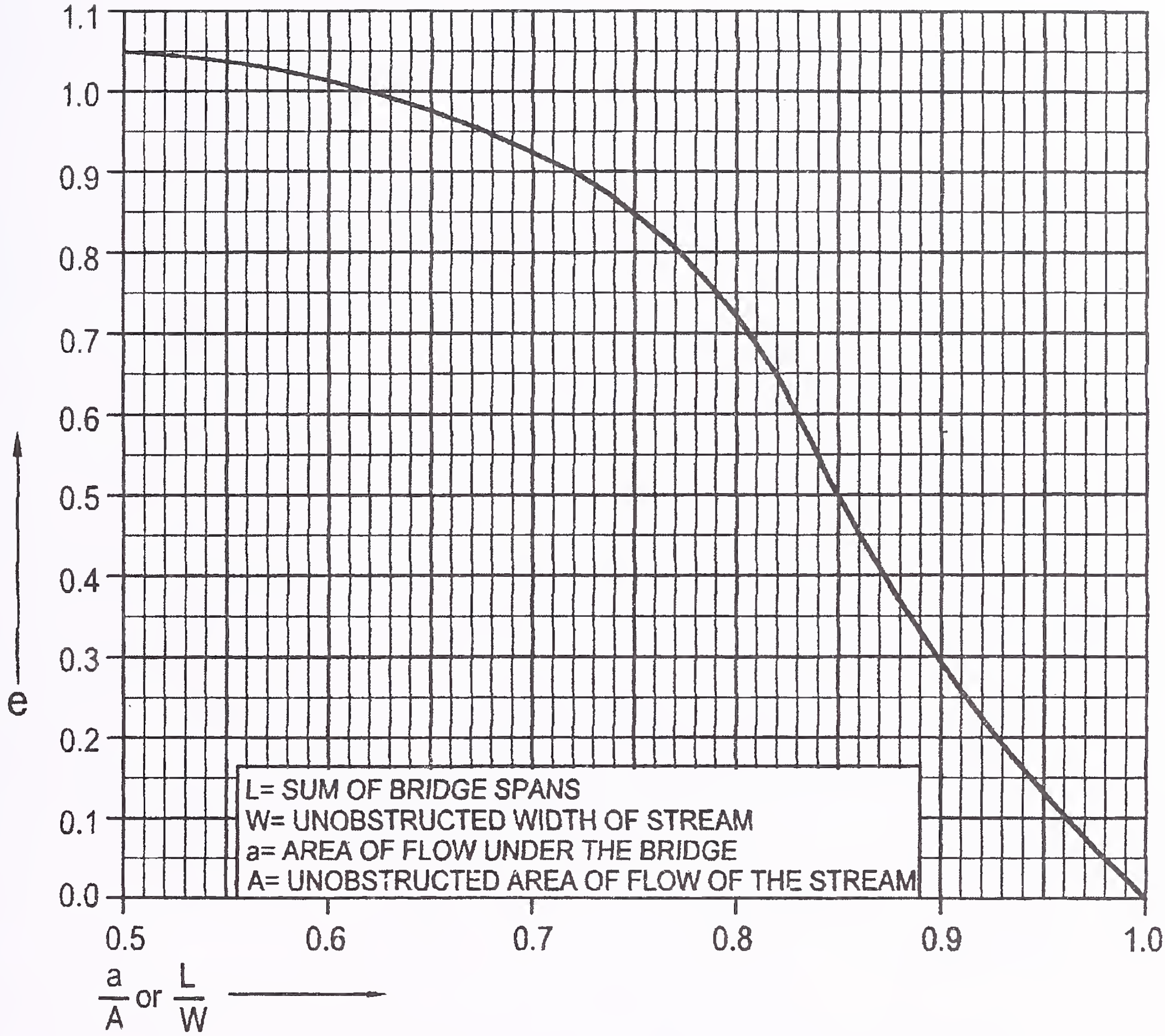


Fig. 15.3 Elements of the hydraulics of flow through bridges The orifice formula coefficient "e"

(3) The Orifice Formulae with the suggested values of C_0 and e should be applied when the afflux is less than 1/4 D_d.

15.5. Examples have been worked out in Articles 16 and 17 to show how these formulae can be used to calculate aflux and discharge under bridges.

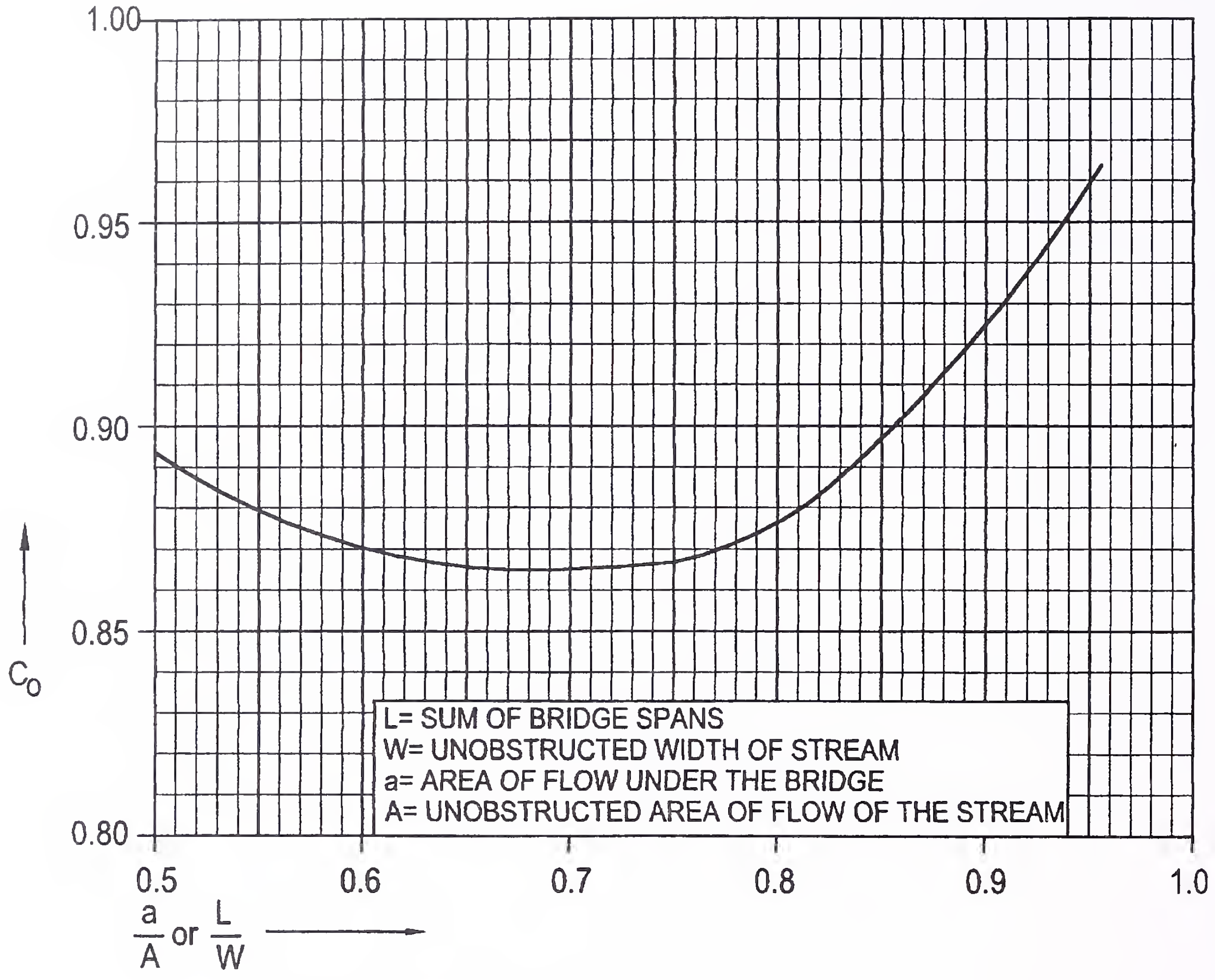


Fig. 15.4 The Orifice Formula Coefficient "C_o"

ARTICLE 16

AFFLUX

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

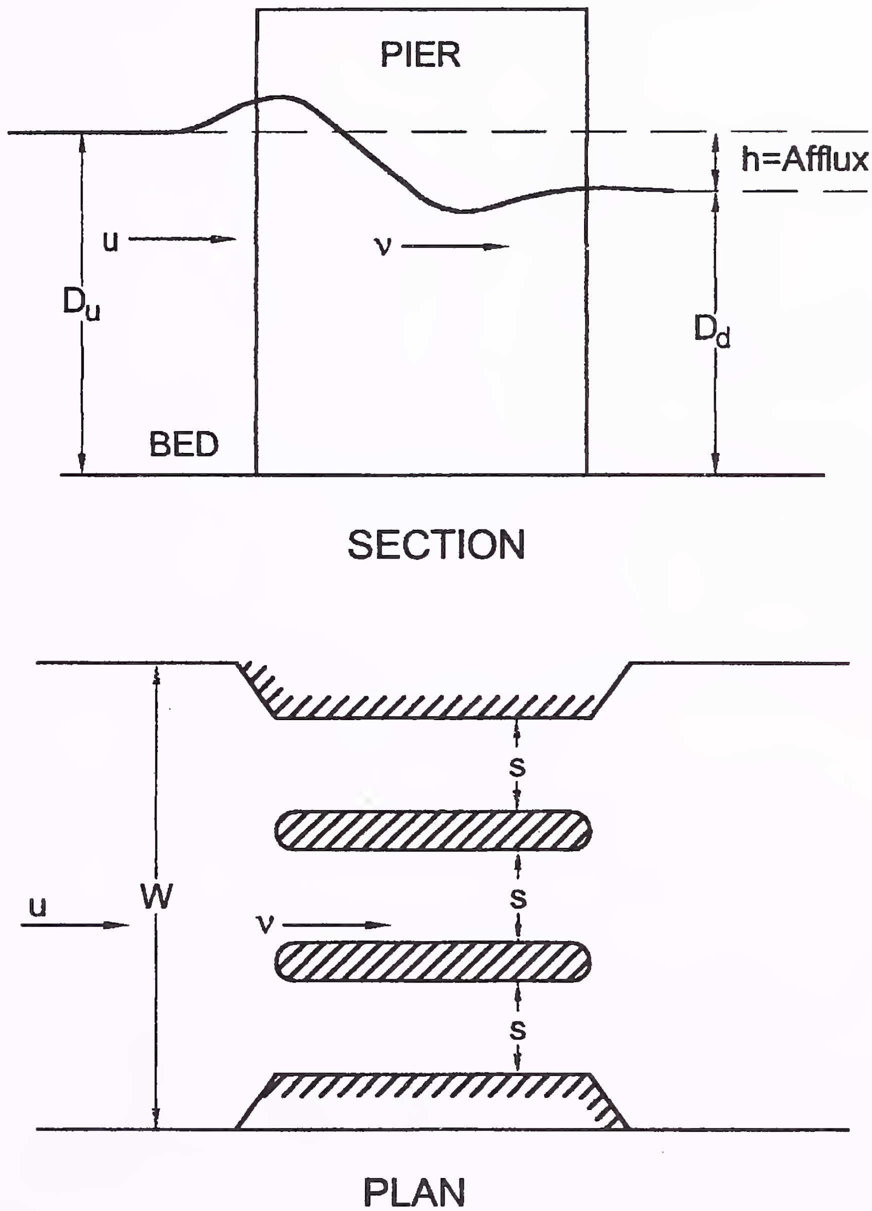


Fig. 16.1

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

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

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

16.5. The downstream depth D_d is not affected by the bridge : it is controlled by the conveyance factor and slope of the channel below the bridge. Also, the depth, that prevails at the bridge site before the construction of the bridge, can be assumed to continue to prevail just 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.

16.6. **Example :** A bridge, having a linear waterway of 25 m, spans a channel 33 m wide carrying a discharge of 70 m³/s. Estimate the afflux when the downstream depth is 1 m.

$D_d = 1\text{m}$; $W = 33$ m ; $L = 25$ m

Discharge through the bridge by the Orifice Formula.

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

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

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

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

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

Also, just upstream of the bridge

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

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

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

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

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

Substituting for u in (16.1)

$h = 0.263$ m

Alternatively, assume that h is more than ¼ D_d

and apply the Weir Formula

$Q = 1.706\ CwLH^{3/2}$

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

$H = 1.45$ m

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

Or ; $D_u = 1.45$ m (approx.)

Now,

$Q = W\ D_u\ u$

$\therefore\ 70 = 33 \times 1.45\ u$

$$\therefore\ u = 1.46; \quad \frac{u^2}{2g} = 0.1086 \text{ m}$$

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

i.e.

$1.45 = D_u + 0.1086$

$D_u = 1.3414$ m

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

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

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

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

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

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

By the Orifice Formula the discharge through the bridge

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

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

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

Now, the discharge just upstream of the bridge

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

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

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

$u = 2.81$ m/sec

Putting for u in (16.4)

$h = 0.32$ m $\quad < {}^1/_4 . D_d$

16.8. **Example :** A bridge of 3 spans of 8 m each is proposed across a stream, whose unobstructed width is 36 m, slope 1/2000 and discharge 400 m³/sec. Calculate the afflux (n=0.03) **(Fig. 16.2).**

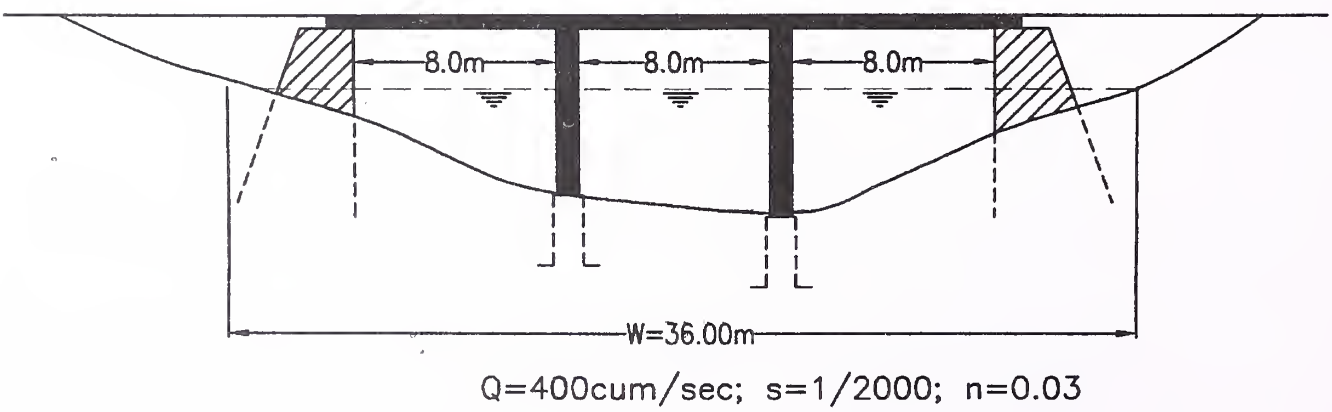


Fig. 16.2

We have first to find D_d

$$Q = AV = (RP)\, V = RWV$$

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

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

$R = 5.1$

$V = 2.18$

Take $D_d = R \quad = 5.1$ m

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

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

By the Orifice Formula, the discharge through the bridge

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

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

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

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

The discharge just upstream of the bridge

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

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

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

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

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

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

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

ARTICLE 17

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

17.1. Calculating Discharge by the Weir Formulae

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

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

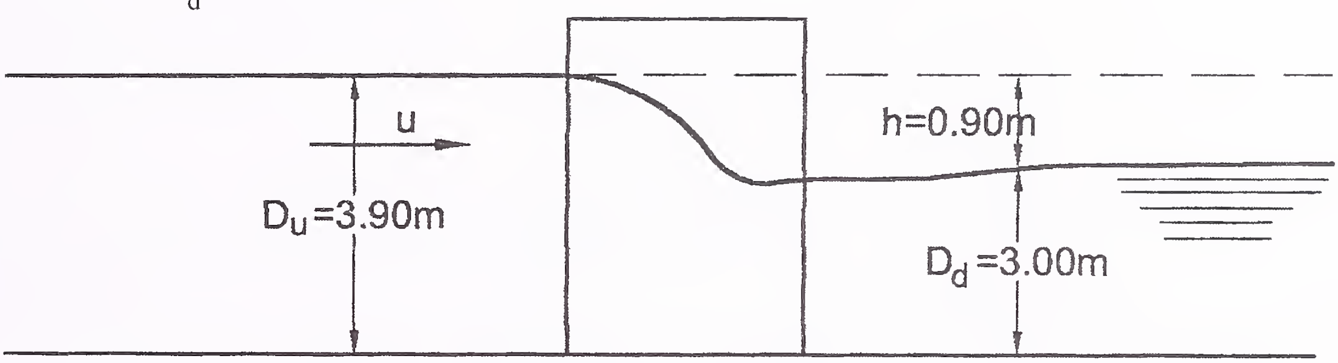


Fig. 17.1

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

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

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

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

The discharge through the bridge by the Weir formula

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

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

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

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

Rearranging

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

or $\quad u = 2.45$ m/sec

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

$$Q = 156 \times 2.45$$
$$= 382 \text{ m}^3\text{/sec}$$

Try the Orifice Formula

$$\frac{L}{W} = \frac{27}{40} = 0.675$$

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

Discharge through the bridge by the Orifice Formula

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

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

Discharge just upstream of the bridge

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

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

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

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

Simplifying

$$u = 2.36$$

Substituting for u in (17.1c) and (17.1d) we get Q

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

This result is about the same as given by the first method. In fact, the Orifice Formula, with the recommended value 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.

17.2. Calculating Discharge by the Orifice Formula

Example : The unobstructed width of a stream is 30 m and the linear waterway of the bridge across is 22 m. During a flood the average depth of flow down stream of the bridge was 1.6 m and the afflux 0.10 m. Calculate the discharge **(Fig. 17.2)**.

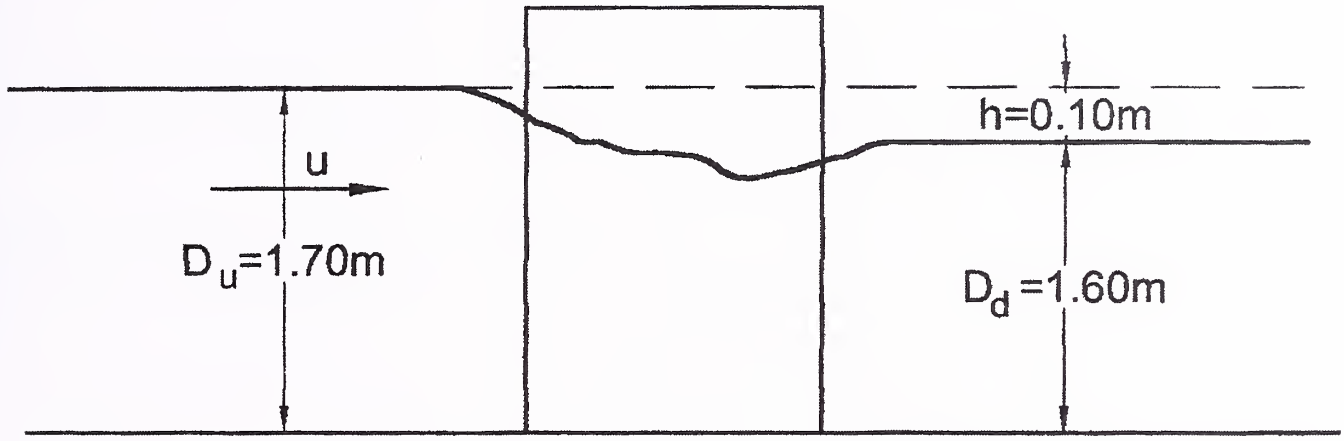


Fig. 17.2

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

$$Q = u \times 1.7 \times 30 \qquad \text{... (17.2a)}$$

$$\text{Contraction} = \frac{a}{A} = \frac{L}{W} = \frac{22}{30} = 0.73$$

Corresponding to this $C_0 = 0.87$ and e = 0.90

The discharge under the bridge, by the Orifice Formula

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

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

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

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

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

$$u = 1.51 \text{ m/s}$$

Substituting for u in (17.2a) and (17.2b) to get Q

$$Q = 1.51 \times 1.7 \times 30$$

$$= 77.01 \text{ cu. m/sec}$$

17.3. **The Border Line Cases :** An example will now follow to illustrate what results are obtained by applying the Weir Formula and Orifice Formulae to cases which are on the border line, i.e., where the afflux is just ¼ D_d.

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

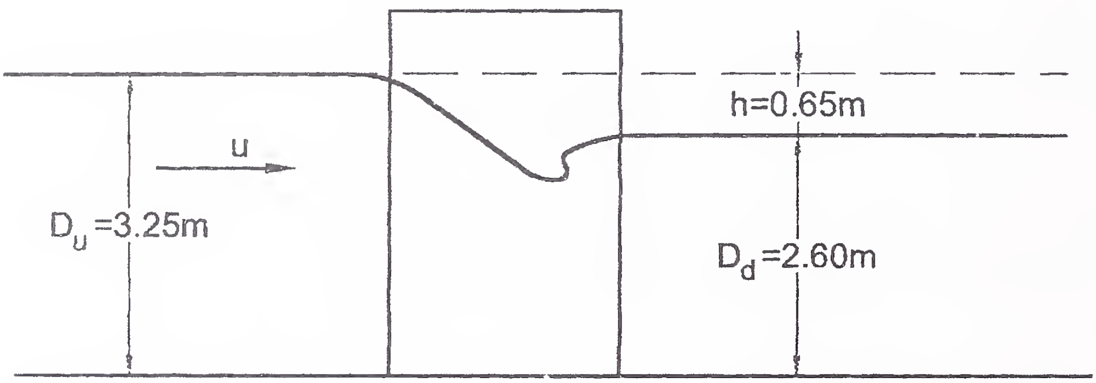


Fig. 17.3

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

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

By the Weir Formula

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

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

The discharge through the bridge

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

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

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

$u = 3.27$ m/s

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

$Q = 113.75 \times 3.2 = 371.96 \ m^3/s$

By the Orifice Formula

$$\frac{a}{A} = \frac{L}{W} = \frac{30}{35} = 0.85$$

$C_e = 0.90 \qquad e = 0.44$

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

$Q = 35 \times 3.25u = 113.75u$ ... (17.3c)

The discharge under the bridge by the Orifice Formula

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

$= 310.98 \ (0.65 + 0.0735 \ u^2)^{1/2}$ ... (17.3d)

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

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

$\therefore \ u = 3.27$ m/s

Substituting for u in ((17.3c) and (17.3d), we get Q

$Q = 113.75 \times 3.27 = 371.96 \ m^3/s$

ARTICLE 18

OVERTOPPING OF THE BANKS

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

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

18.3. **Subsidiary or Relief Culverts :** Sometimes, however, the overflow spreads far and away from the banks. This is often the case in alluvial plains, where the ground level falls continuously away from the banks of the stream. In 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.

18.4. **Dips and Breaching Sections in Approach Roads :** It is sometimes feasible as well as economical to provide permanent dips (or alternatively breaching sections) in the bridge approaches to take excessive overflows in emergencies. The dips or breaching sections have to be sited and designed so that the velocity of flow through them does not become erosive, cutting deep channels and ultimately leading to the shifting of the main current.

18.5. **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 fall. Under such conditions deep erosion can take place. A deep channel is formed, beginning at the outfall in the mains stream and retrogressing towards the culvert. This endangers the culvert. To provide against this, protection has to be designed downstream of the culvert so as to dissipate the energy of the falling water on the same lines as is done on irrigation falls. That is a suitable cistern and baffle wall should be added for dissipating the energy and the issuing current should be stilled through a properly designed expanding flume.

ARTICLE 19

PIPES AND BOX CULVERTS

19.1. Feasibility of Pipe and Box Culverts Flowing Full

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

19.1.2. In such flat regions the road runs across wide but shallow dips and, therefore, the most 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.

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

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

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

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

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

19.1.6. In the past, use was extensively made of empirical formulae which gave the ventway area required for a culvert to drain a given catchment area. Dun's Drainage Table is one of the class and is purely empirical. This table is still widely used, as it saves the trouble of hydraulic calculations. But it is unfortunate that recourse is often taken rather indiscriminately to such short cuts, even where other more accurate and rational procedure is possible and warranted by the expense involved. Dun's Table or other in that class, should NOT be used until suitable correction factors have been carefully evolved from extensive observations (in each particular region with its own singularities of terrain and climate) of the adequacy or otherwise of the existing culverts vis-à-vis their catchment area.

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

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

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

19.2. Hydraulics of the Pipe and Box Culverts Flowing Full

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

For a given design discharge the extent of upstream heading up depends on the 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.

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

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

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

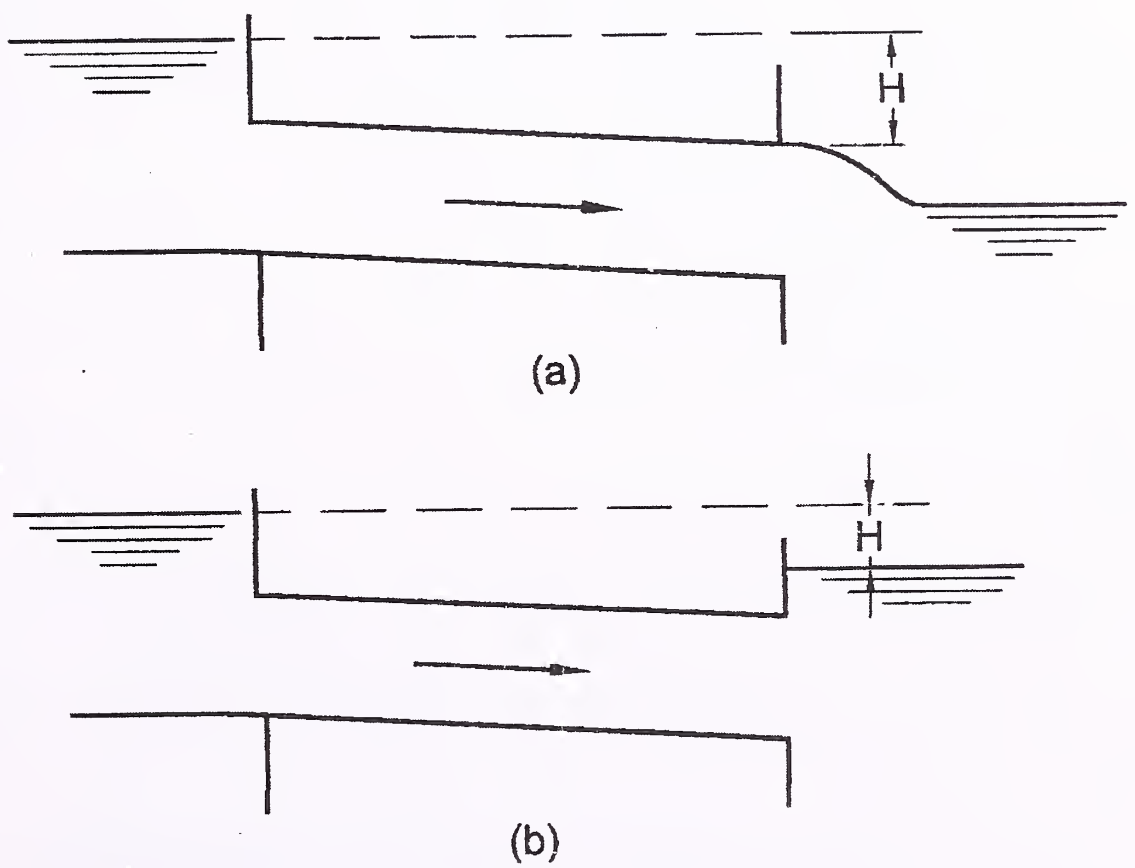


Fig. 19.1

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

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

$$H = [1 + K_e + K_f] \frac{v^2}{2g} \qquad \text{... (19.1)}$$

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

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

$$K_e = 0.08 \text{ for bevelled or Bell – mouthed entry}$$

$$= 0.505 \text{ for sharp edged entry} \qquad \text{... (19.2)}$$

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:

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

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

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

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

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

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

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

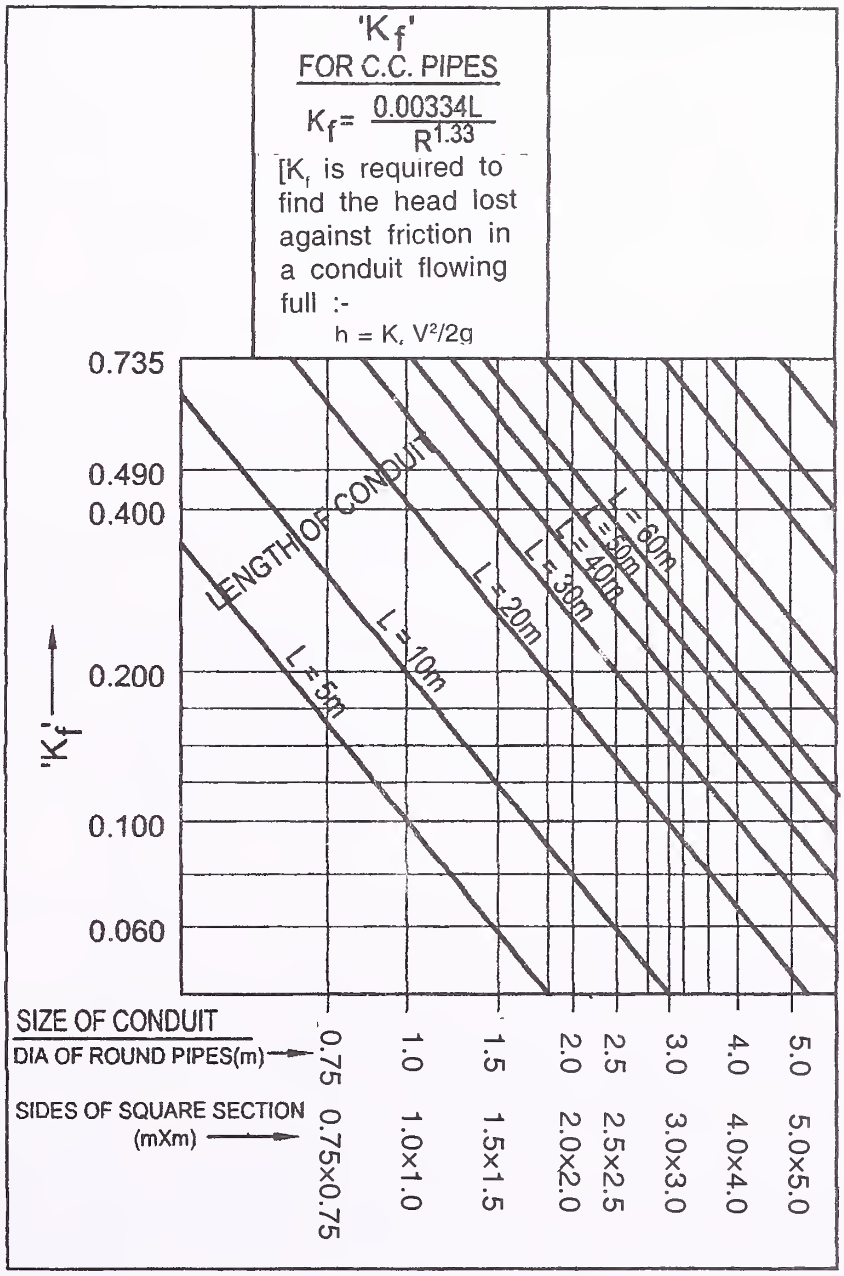


Fig. 19.2

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

Table 19.1 Values of K_e and K_f [9]

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

19.2.7. **Design calculations**: We have said that

$$H = (1 + K_e + K_f)\,\frac{v^2}{2g}$$

$$\text{i.e.}\quad v = 4.43\left(\frac{H}{1 + K_e + K_f}\right)^{1/2} \qquad \text{... (19.5)}$$

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

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

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

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

19.2.8. **Design chart (Plate 27) :** Equation (19.5) may be written as

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

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

It is obvious that all components of λ in Equation (19.7) are functions of the cross-section, length, roughness, and the shape of the inlet of the pipe. Therefore, λ represents the conveying capacity of the pipe and may be called the 'Conveyance Factor'. The discharge, then depends on the conveyance factor of the pipe and the operating head. In **Plate 26**, curves have been constructed from equation (19.7) from which Q can be directly read for any known values of λ and H.

Also, in the same Plate, Tables are included from which λ can be taken for any known values of (i) length, (ii) diameter in case of circular pipes or sides in case of rectangular pipes, and

(iii) conditions of entry, viz., sharp-edged or round. The material assumed is cement, concrete and values of K_e and K_f used in the computation are based on functions in Table 19.1.

The use of **Plate 27** renders the design procedure very simple and quick. Examples will now follow to illustrate.

19.2.9. **Example data :**

(1) Circular cement concrete pipe flowing full with bevelled entry
(2) Operating head = 1 m
(3) Length of the pipe = 25 m
(4) Diameter = 1 m

Find the discharge.

See, in **Plate 27**, the Table for circular pipes with rounded entry.

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

λ=0.618

Now refer to the curves in the same Plate. For λ = 0.618 and H=1 m

Q=2.72 m^3/sec

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

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

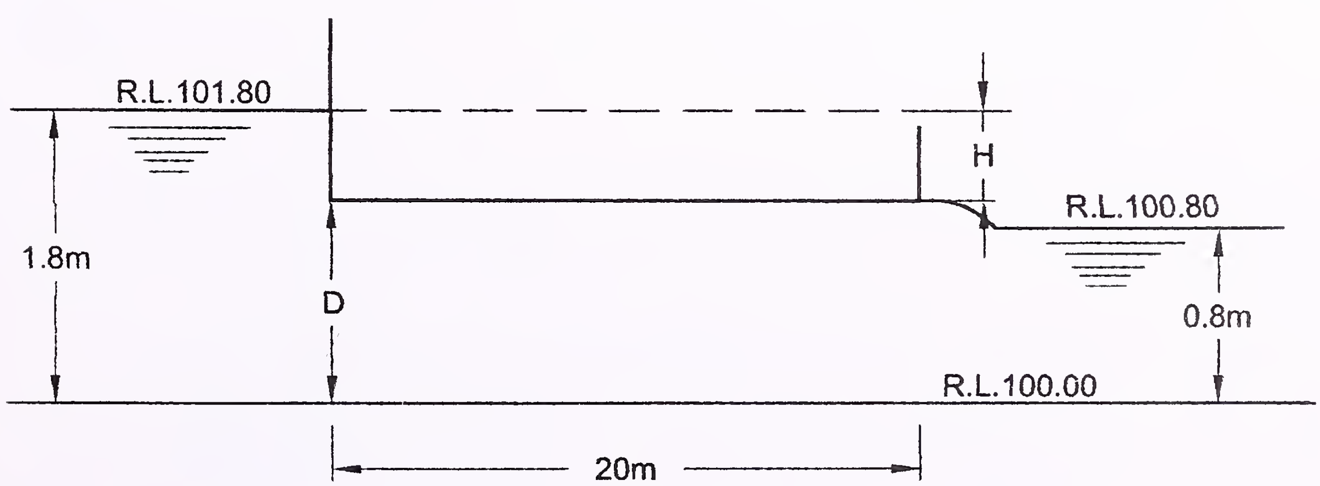


Fig. 19.3

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

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

Corresponding

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

Discharge per pipe

From **Plate 27**, Q = (1) 2.54 m^3/s; (2) 3.5 m^3/s

Number of pipes

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

Say 4 Say 3

Hence, 4 pipes of 1 metre diameter will suit.

ARTICLE 20

PROTECTION WORK AND MAINTENANCE

20.1. Floor Protection Works:

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

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

20.1.2. Suggested Specifications:

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

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

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

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

20.1.2.2. **Rigid flooring :** The rigid flooring should be provided under the bridge and it should extend for a distance of at least 3 m on upstream side and 5 m on down stream side of the bridge. However, in case the splayed wing walls of the structure are likely to be longer, the flooring should extend upto the line connecting the end of wing walls on either side of the bridge.

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

Flooring should consist of 150 mm thick flat stone/bricks on edge in cement mortar 1:3 laid over 300 mm thick cement concrete M15 grade laid over a layer of 150 mm thick cement concrete M10 grade. Joints at suitable spacings (say 20 m) may be provided.

20.1.2.3. **Curtain walls :** The rigid flooring should be enclosed by curtain walls (tied to the wing walls) with a minimum depth below floor level of 2 m on upstream side and 2.5 m on downstream side. The curtain wall should be in cement concrete M15 grade or brick/stone masonry in cement mortar 1:3. The rigid flooring should be continued over the top width of curtain walls. In this context, relevant provision in ''Guidelines for design and construction of river training and control works for road bridges'', IRC: 89-1997 is also referred.

20.1.2.4. **Flexible apron :** Flexible apron 1 m thick comprising of loose stone boulders (weighing not less than 40 kg) should be provided beyond the curtain walls for a minimum distance of 3 m on upstream side and 6 m on downstream side. Where required size stones are not economically available, cement concrete blocks or stones in wire crates may be used in place of isolated stones. In this context, relevant provision in IRC:89-1997 is also referred.

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

20.2. **Maintenance :**

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

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

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

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

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

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

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

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

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

ARTICLE 21

RAFT FOUNDATIONS

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

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

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

21.1.3. The depth of cut-off wall should be 30 cm below the scour level. The normal scour depth is worked out by the formula $d_{sm} = 1.34 \times \left(\frac{D_b^2}{K_{sf}} \right)^{1/3}$ (Refer Equation 9.1).

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

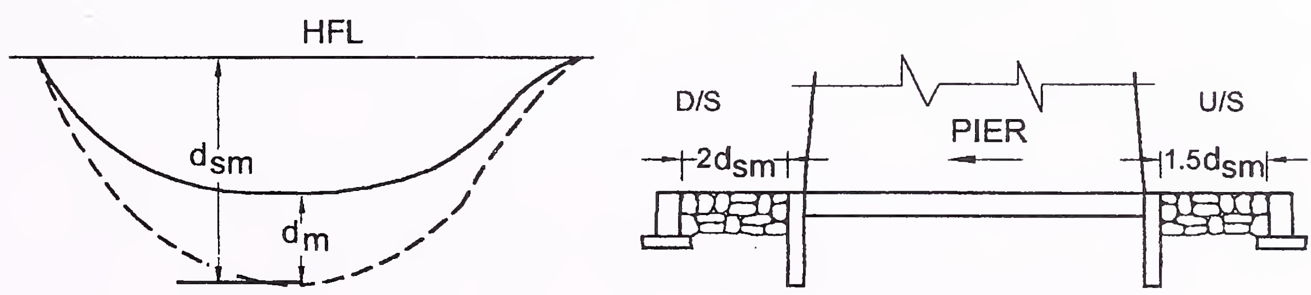


Fig. 21.1 Scour Depth and Apron Width for Raft

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

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

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

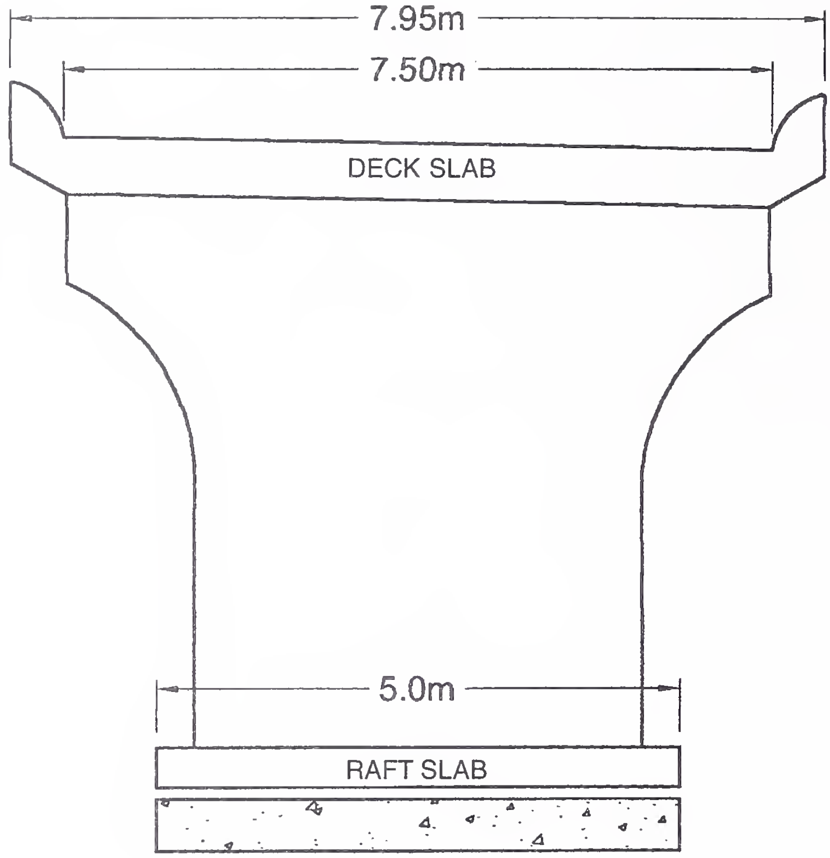


Fig. 21.2 Raft Slab Narrower than Deck Width

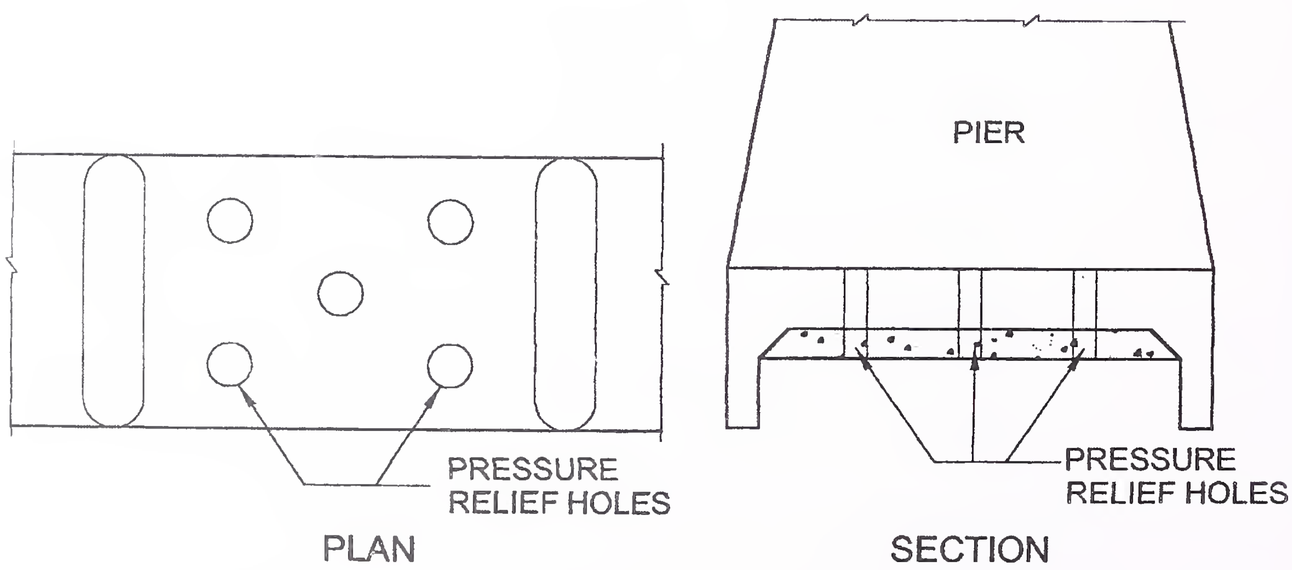


Fig. 21.3 Pressure Relief Holes in Raft Slab

ARTICLE 22

C.D. WORKS IN BLACK COTTON SOILS

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

22.2. The expansive soils have very poor bearing capacity. The same needs improvement, which can be done by providing layer of metal/boulder with sand having thickness of about 450 to 600 mm. Such layer, improves Safe Bearing Capacity (SBC) of the strata to a considerable extent and safeguards the foundation from the adverse effects of the expansive soil also **(Fig. 22.1)**.

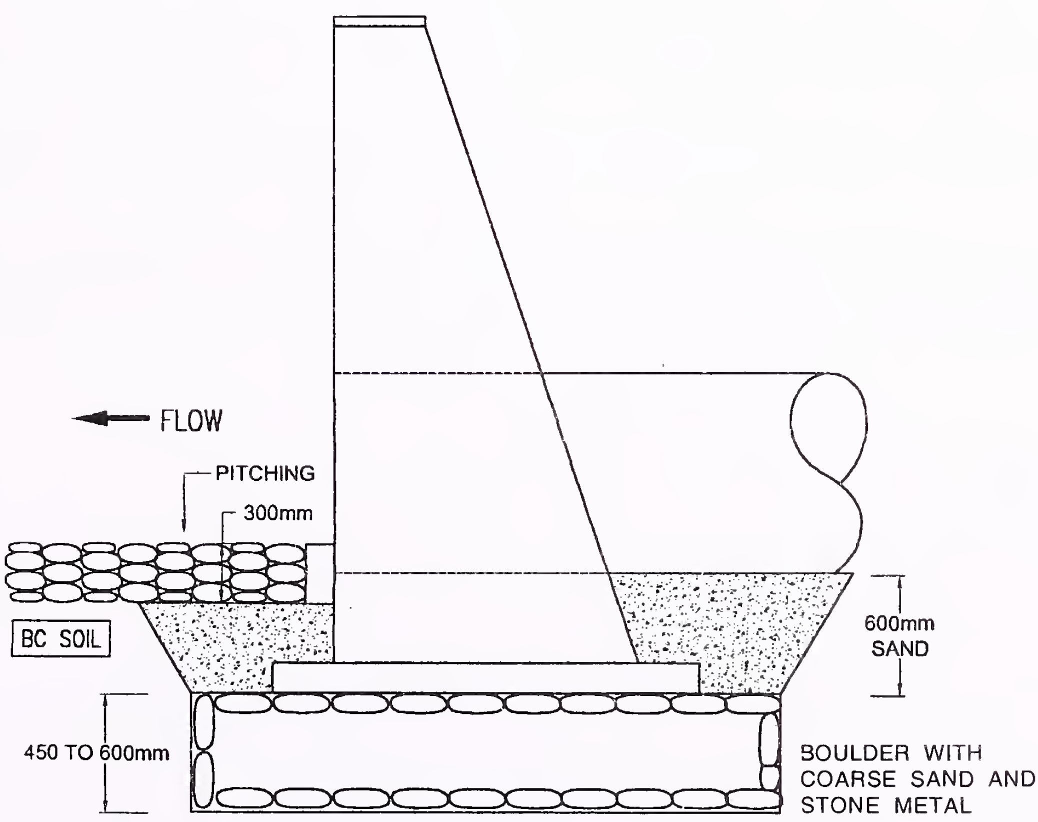


Fig. 22.1 Hume Pipe Culvert in BC Soil

ARTICLE 23

BOX CELL STRUCTURES

23.1. **Where to Provide Box Structures :** Box structures are hydraulically efficient structures where thickness of walls and slab are small and there is least obstruction to flow.

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

23.2. **Type of Boxes :** Box cell structures with and without earth cushion based on MORT&H Standard Drawings are given in 10 plates as brought out in para 14.5.

23.3. **Foundation :** Where there is purely clayey strata top 900 mm below box should have granular material, like, sandy murum or stone dust.

Where there is murum and mixed soil having ϕ more than 15°, there is no need of providing sandy layer.

The box cell structures are of concrete of M 20 grade for moderate and M 25 grade for severe conditions of exposure with HYSD steel bars.

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

BIBLIOGRAPHY

(1) "Highway Practice in the United States of America", Public Roads Administration, Federal Works Agency, Washington 25, D.C. ,1949.

(2) "Engineering Hydraulics, Proceedings of the Fourth Hydraulic Conference", Iowa Institute of Hydraulics Research, June 12-15, 1949, Edited by Hunter Rouse, John Wiley & Sons, Inc., New York, 1950.

(3) "Elements of Applied Hydrology", Don Johnstone and William P. Cross, The Ronald Press Company, New York, 1949.

(4) "Rainfall and Run-off", Edgar E. Foster, The Macmillian Company, New York, 1949.

(5) "The Flood Estimation and Control", B.D. Richards, Chapman and Hall Ltd., London, 1944.

(6) "A Treatise on Applied Hydraulics", Herbert Addison, 3rd Edition, Chapman and Hall Ltd., London, 1944.

(7) "Irrigation Pocket Book", Compiled by Robert Burton, Bockley, E & F N Spoon Ltd., London, 1913.

(8) "Canals and Related Structures, Design", Supplement No. 3, Part 2, Engineering Design of Volume X, Design and Construction Reclamation Manual", United States Department of Interior, Bureau of Reclamation, 1952.

(9) University of Iowa Studies, Bulletin No. 1, "Flow of Water Trough Culvert", David L. Yarid, Floyd A. Nagler and Sherman M. Woodward, The University of Iowa, Iowa, June 1926.

(10) United States Department of Agriculture, Technical Bulletin No. 442, "Bridge Piers as Channel Obstructions", David L. Yarnell, United States Department of Agriculture, Washington D.C., 1934.

(11) "California Culvert Practice", State of California, Department of Public Works, Division of Highways, Sacramento, 1944.

(12) "Highway Design and Construction", Arthur G. Bruce, and John Clarkeson, 3rd Edition, International Text-book Company, Scranton, Pennsylvania, 1951.

(13) "Hydraulics for Engineers and Engineering Students", F.C. Lea, 4th Edition, Edward Arnold & Co., 1926.

(14) "Military Engineer Services Handbook, Roads, Volume III", Sixth Edition, Government of India, Central Publication Branch, Calcutta, 1930.

(15) "Standard Specification and Code of Practice for Roads Bridges – Section I – General Features of Design (in Metric Units) (Seventh Revision)", IRC:5 – 1998.

(16) "Standard Specifications and Code of Practice for Road Bridges – Section II – Loads and Stresses (Fourth Revision)", IRC:6-2000.

(17) "Standard Specifications and Code of Practice for Roads Bridges", Section III – Cement Concrete (Plain and Reinforced) (Third Revision), IRC:21-2000.

(18) "Regime Flow in Incoherent Alluvium", G. Lacey, Central Board of Irrigation and Power, Paper No. 20.

(19) "The Hydraulic of Culverts", by F.T. Mavis, The Pennsylvania State College Engineering Experimental Research Station Series Bulletin No. 56.

(20) "Standard Specifications and Code of Practice for Road Bridges, Section VII (Second Revision) Foundation & Substructure", IRC:78-2000.

(21) "Standard Specifications and Code of Practice for Road Bridges, Section IV (Brick, Stone and Cement Concrete Block Masonry)'', IRC:40-2002.

Appendix–A

HEAVIEST RAINFALL IN ONE HOUR (mm)
(Time in Indian Standard Time)

	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	60.0	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 (1964-1966)												
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 (1951-1966)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
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
8. **Asansol** (1953-65)												
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) (1952-66)												
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** (1962-65)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
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
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 (1957-66)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
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 (1964-66)												
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
23. Bhuj (1964-66)												
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	6	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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
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	1967
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 (1952-61)												
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	–	–

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
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)												
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)												
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	15,22	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)												
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)												
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. Ganavaram (1963-66)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
43. Gauhati (1955-66)												
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)												
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)												
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)												
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)												
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)												
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)												
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)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
51. Indore (1963-66)												
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)												
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)												
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)												
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)												
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)												
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. Jammu (1955-65)												
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)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
59. Jharsuguda (1954-66)												
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)												
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)												
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)												
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, 1957	1956	1956	1962	1956	1965	1952	1961	1963	1961	1952	1961
63. Khalari (1963-66)												
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 (1958-61)												
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 (1948-66)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
66. Konar (1960-64)												
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 (1963-66)												
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) (1953-66)												
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) (1948-66)												
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
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 (1948-66)												
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 (1957-66)												
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 (1953-66)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
74. Marmagao (1964-66)												
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 (1960-66)												
mm	10.0	12.6	45.6	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 (1963-66)												
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 (1956-66)												
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
78. Nagpur (1948-66)												
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 (1962-66)												
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	–	24	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 (1948-66)												
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 (1957-66)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
82. Okha (1963-66)												
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 (1952-66)												
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
87. Panchat Hill (1953-66)												
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 (1957-61)												
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 (1962-65)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
90. Pokhara (1956-66)												
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)												
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)												
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)												
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)												
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
95. Putki (1960-66)												
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)												
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)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
98. Sagar Island (1948-66)												
mm	20.3	34.5	24.5	48.8	51.4	64.9	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)												
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)												
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)												
mm	10.1	13.2	11.5	10.2	[illegible]	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 (1953-66)												
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	1956	1959	1953
103. Shanti Niketan (1960-66)												
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 (1954-56)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
105. Tehri (1956-66)												
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 (1957-64)												
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	1957	1963	1963	1963	1961	1963	1959	1964
107. Thikri (1952-66)												
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 (1956-66)												
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 (1954-66)												
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)												
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)												
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

	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sept.	Oct.	Nov.	Dec.
112. Veraval (1952-66)												
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)												
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

Appendix-B

FILLING BEHIND ABUTMENTS, WING AND RETURN WALLS

1. FILLING MATERIALS

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

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

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

2. LAYING AND COMPACTION

2.1. Laying of Filter Media for Drainage

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

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

2.2. Density of Compaction

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

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

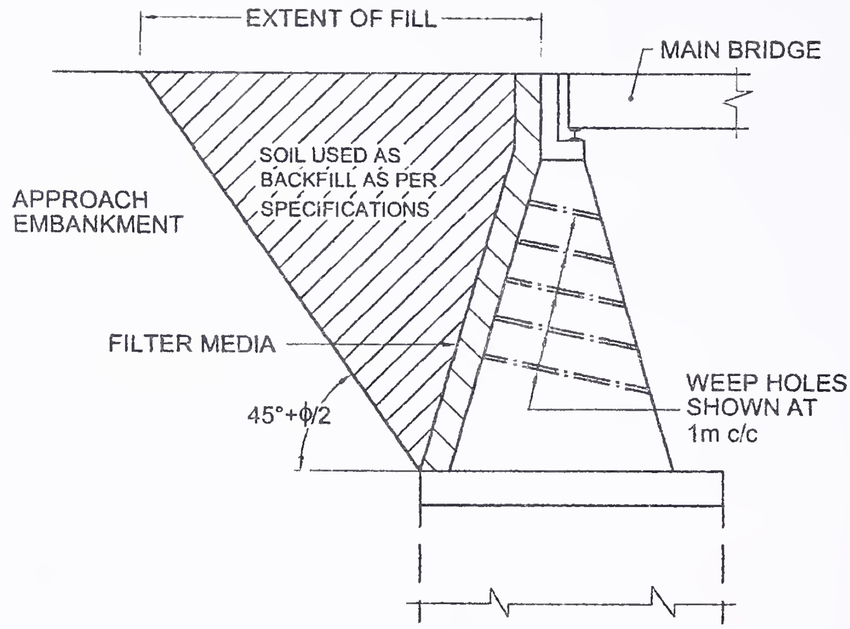


Fig. 1

3. EXTENT OF BACKFILL

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

4. PRECAUTIONS TO BE TAKEN DURING CONSTRUCTION

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

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

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

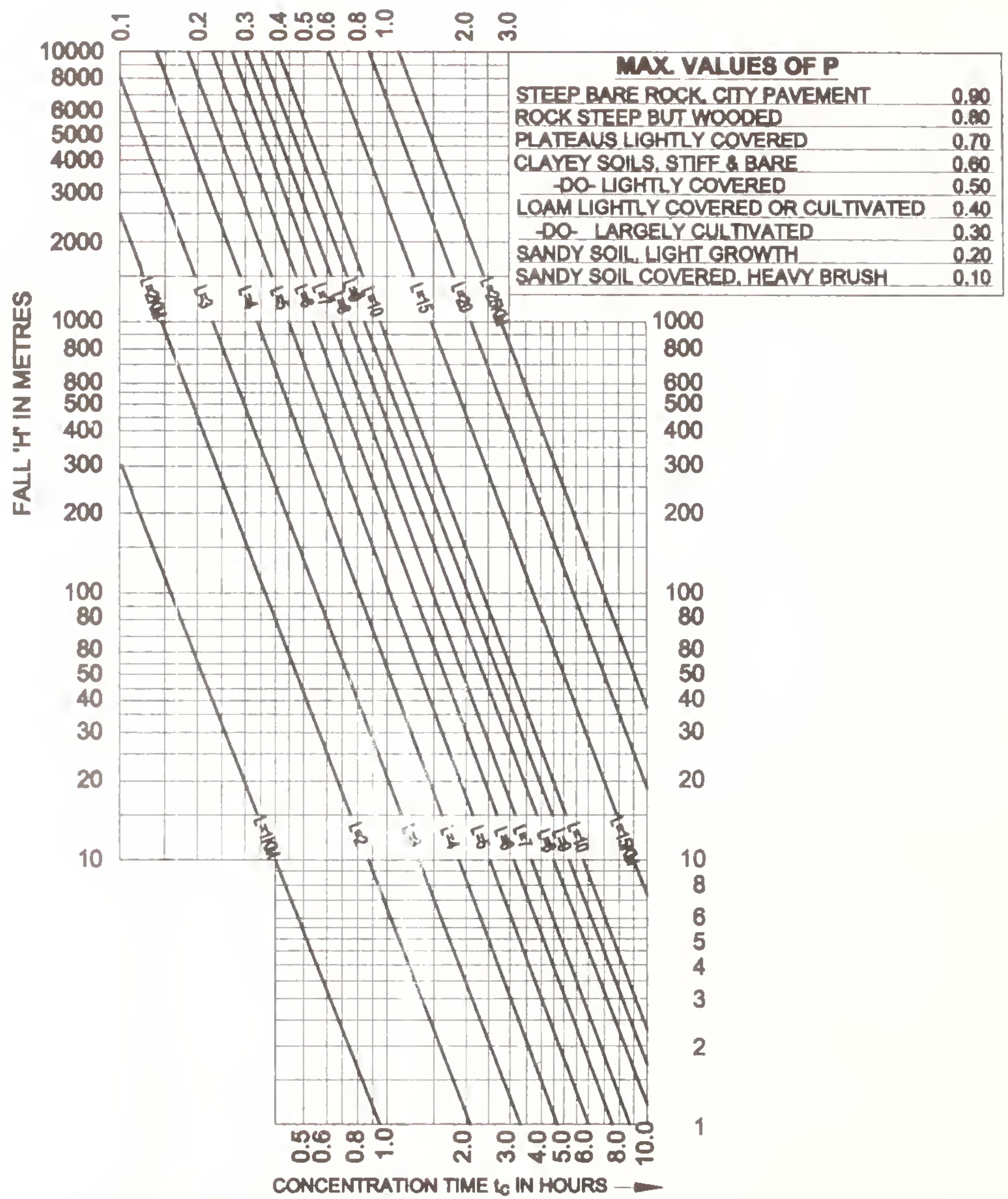
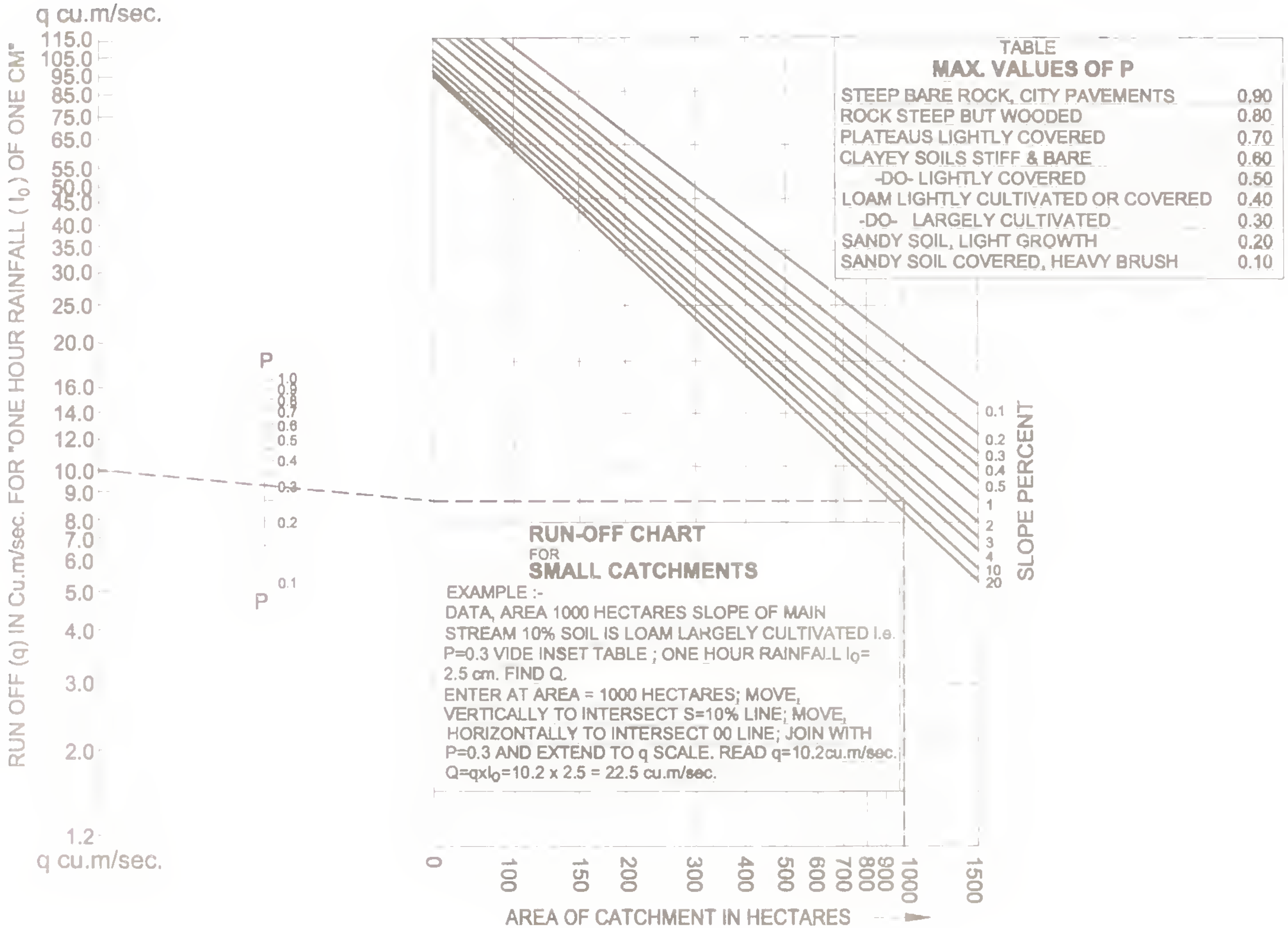
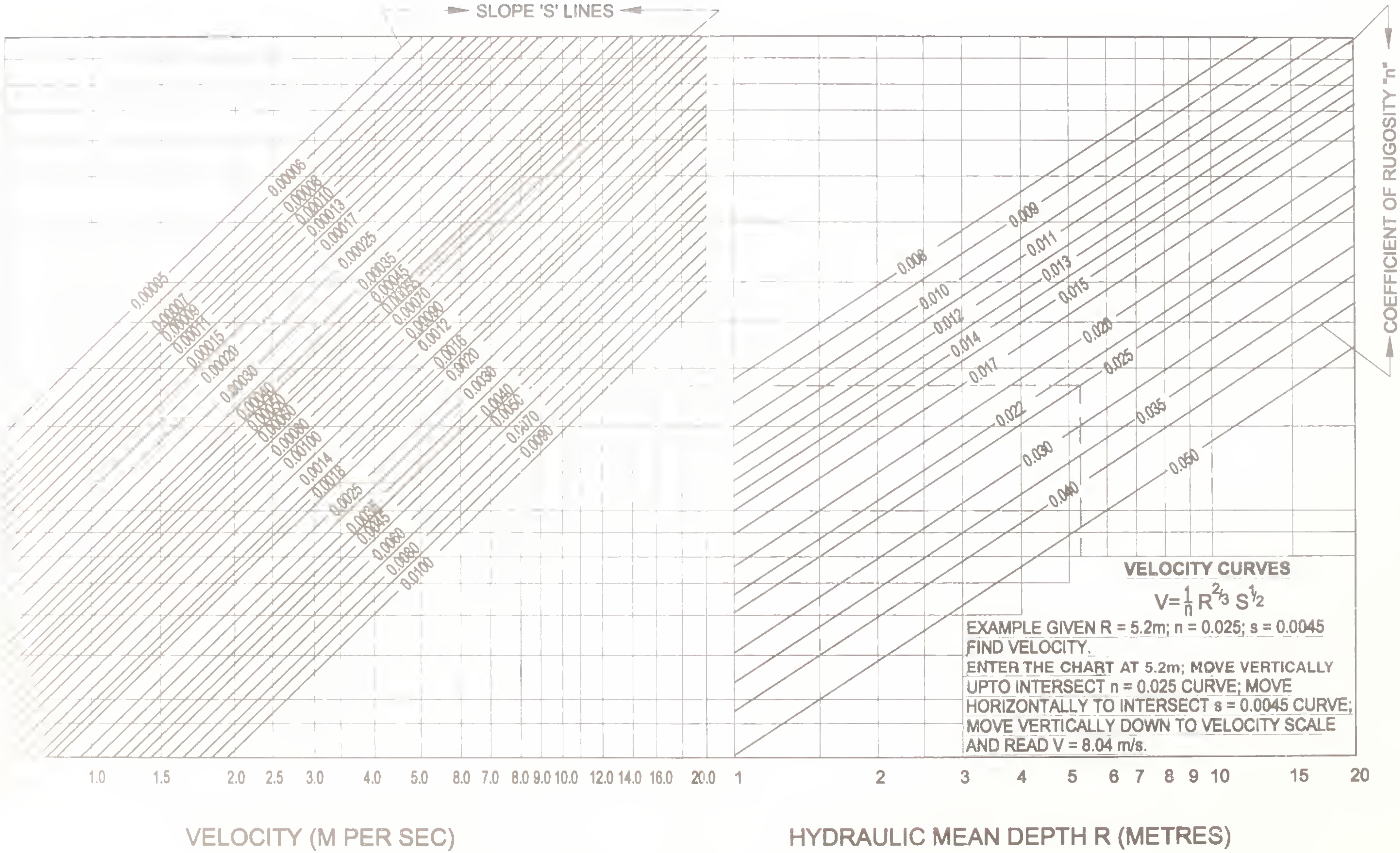


Chart for Time of Concentration



Run-off Chart for Small Catchments


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

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

TABLE A

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

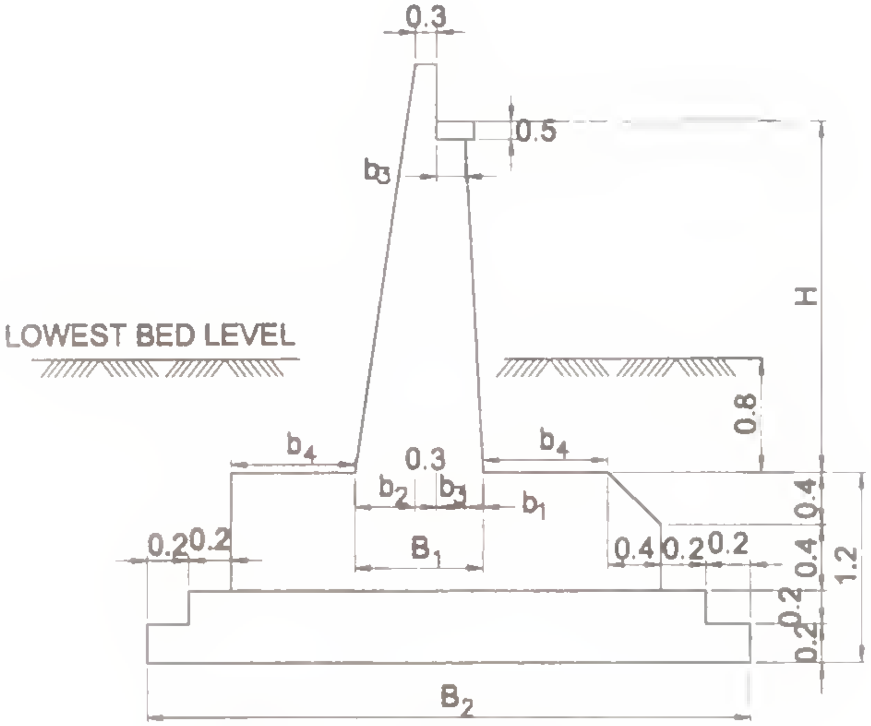
TABLE B

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

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

$= 0.74365$ Say 0.74

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



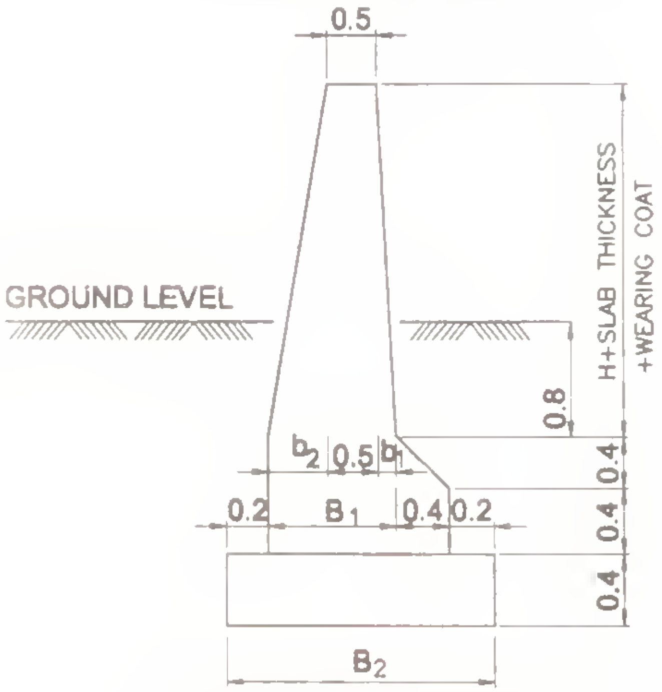
SECTION OF ABUTMENT

TABLE OF DIMENSIONS FOR ABUTMENT

Effective Span	6m and 5m					3m, 2m, 1.5m and 1m						3.5m and 4m					
H	2.0m	2.5m	3.0m	3.5m	4.0m	1.5m	2.0m	2.5m	3.0m	3.5m	4.0m	1.5m	2.0m	2.5m	3.0m	3.5m	4.0m
b_1	0.2	0.25	0.3	0.35	0.4	0.15	0.2	0.25	0.3	0.35	0.4	0.15	0.2	0.25	0.3	0.35	0.4
b_2	0.6	0.85	1.0	1.2	1.4	0.5	0.7	0.95	1.1	1.25	1.4	0.5	0.7	0.95	1.1	1.25	1.4
b_3	1.0	1.0	1.0	1.0	1.0	0.5	0.5	0.5	0.5	0.5	0.5	1.0	1.0	1.0	1.0	1.0	1.0
b_4	–	0.1	0.2	0.4	0.5	–	–	0.1	[illegible]	[illegible]	0.5	–	–	0.1	0.2	0.3	0.5
B_1	2.1	2.4	2.8	2.85	3.1	1.45	1.7	2.0	2.20	2.4	2.8	1.95	2.2	2.5	2.7	2.9	3.1
B_2	3.3	3.8	4.2	4.85	5.3	2.65	2.9	3.4	3.8	4.2	4.8	3.15	3.4	3.9	4.3	4.7	5.3

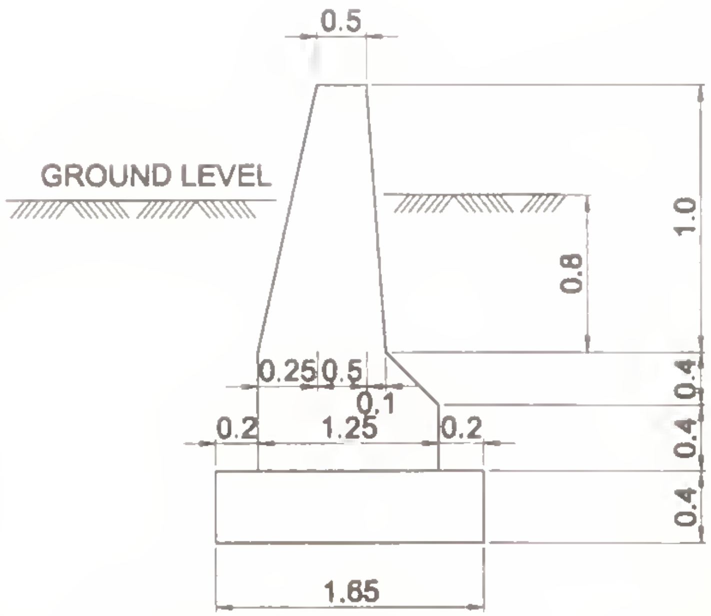
TABLE OF DIMENSIONS FOR WING WALL (HIGH END)

SPAN	UPTO 2 METRES						3 METRES						4 METRES						5 METRES					6 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	1.50	2.00	2.50	3.00	3.50	4.00	2.00	2.50	3.00	3.50	4.00	2.00	2.50	3.00	3.50	4.00
b_1	0.18	0.23	0.28	0.33	0.38	0.43	0.19	0.24	0.29	0.34	0.39	0.44	0.19	0.24	0.29	0.34	0.39	0.44	0.25	0.30	0.35	0.40	0.45	0.25	0.30	0.35	0.40	0.45
b_2	0.45	0.57	0.70	0.82	0.95	1.07	0.46	0.59	0.71	0.84	0.96	1.09	0.48	0.60	0.73	0.85	0.98	1.10	0.62	0.75	0.87	1.00	1.13	0.63	0.75	0.88	1.00	1.13
B_1	1.13	1.30	1.48	1.85	1.83	2.00	1.15	1.33	1.50	1.88	1.85	2.03	1.17	1.34	1.52	1.69	1.87	2.04	1.37	1.55	1.72	1.90	2.08	1.38	1.55	1.73	1.90	2.08
B_2	1.93	2.10	2.28	2.45	2.85	2.80	1.95	2.13	2.30	2.48	2.65	2.83	1.97	2.14	2.32	2.49	2.67	2.84	2.17	2.35	2.52	2.70	2.88	2.18	2.35	2.53	2.70	2.88



WING WALL SECTION

AT HIGH END



WING WALL SECTION

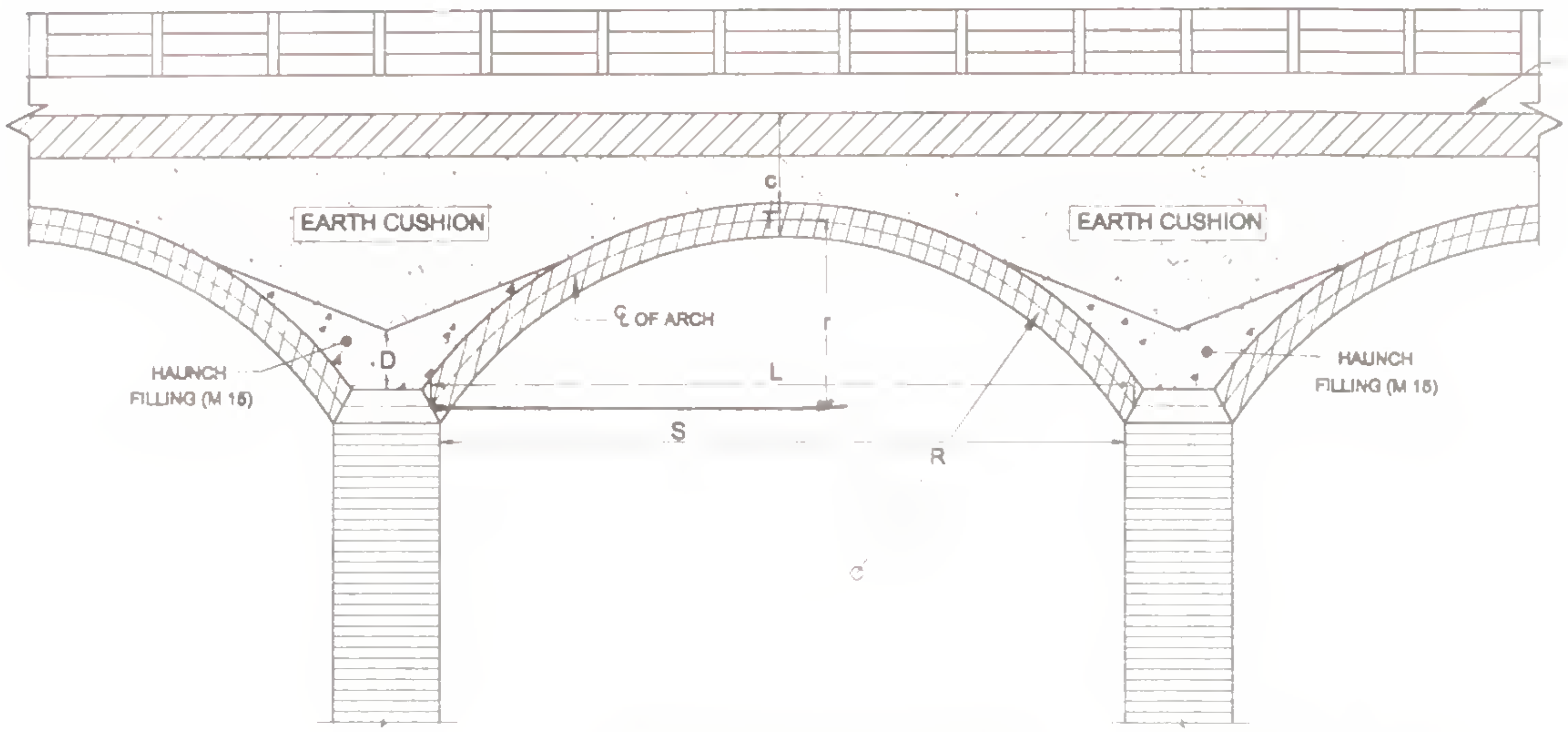
AT LOW END

(FOR ALL SPANS)

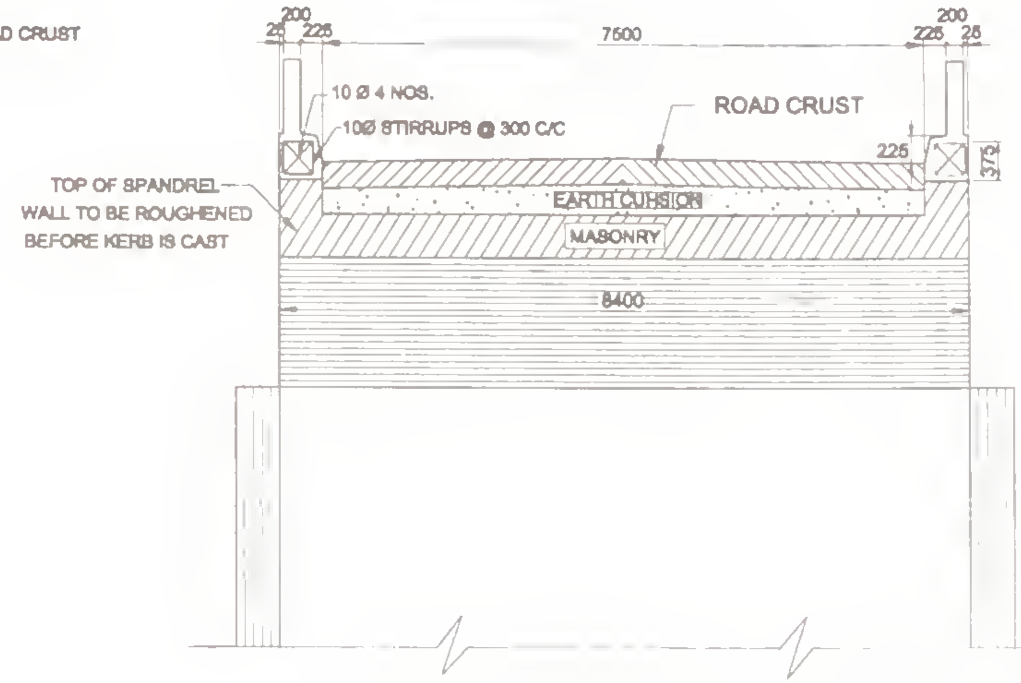
NOTES :-

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

ABUTMENT AND WING WALL SECTIONS FOR CULVERTS



SECTIONAL ELEVATION



CROSS SECTION AT THE CROWN OF ARCH

TABLE

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

GENERAL NOTES :-

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

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

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

(A) GENERAL

[illegible] notes are applicable for the Standard Drawings [illegible] R.C.C. solid slab superstructure with and without footpaths.

[illegible] Drawings are applicable only for right bridges [illegible] width of 12 m.

[illegible] footpaths shall be provided on the bridges [illegible] length less than 30 m unless the same are [illegible] existing on the approaches.

[illegible] dimensions are in millimetres unless otherwise [illegible]. Only written dimensions are to be followed [illegible] shall be scaled.

[illegible] criteria

[illegible] design is according to the following codes

[illegible] IRC [illegible] 1985

[illegible] IRC [illegible] 1966 [illegible] 1985 reprint)

[illegible] 1987

[illegible] loads have been considered in the [illegible]

[illegible] of IRC class 70R or two lanes of IRC class A on carriageway, whichever governs.

[illegible] Footpath load of 5 kN/sq.m for superstructure [illegible] footpaths.

[illegible] Wearing coat load of 2 kN/sq.m.

[illegible] are applicable for 'MODERATE' AND [illegible] conditions of exposure.

[illegible] services (except water supply and [illegible]) shall be carried over the bridge [illegible] diameter ducts provided in the [illegible]. Total load of such services shall not be [illegible] kN per metre on each footpath. Water pipeline shall not be carried over any part of superstructure. Inspection chambers in footpaths [illegible] provided as shown in the drawing. The location [illegible] of chambers along the footpath will be [illegible] the Engineer-in-charge in consultation with [illegible]

[illegible] shall consist of the following:

[illegible] mastic asphalt 6 mm thick with a [illegible] coat over the top of the deck before the [illegible] coat [illegible]. The prime coat of mastic [illegible] shall be 30 per cent straight run 30 [illegible] penetration grade bitumen and 50 per cent [illegible] solvent (Benzol) to be laid over the deck [illegible] of 6 mm thick mastic asphalt [illegible] 75 per cent lime stone dust filler [illegible] per cent of [illegible] penetration grade bitumen shall be laid at 375 F with broom over prime coat.

(b) 50 mm thick asphaltic concrete wearing coat in two layers of 25 mm each as per Clause 512 of MoRT&H's Specifications for Road and Bridge Works (Second Revision, 1988).

II In case of isolated bridge construction or bridges located in remote areas where provision of mastic and asphaltic concrete wearing coat is not practicable, the Engineer-in-charge may permit provision of 75 mm thick cement concrete wearing coat in M 30 grade concrete with maximum water cement ratio as 0.40. The reinforcement shall consist of 8 mm High Yield Strength Deformed bars @ 200 mm centres reducing to 100 centres in both the directions over a strip of 300 mm near the expansion joint. Reinforcement shall be placed at the centre of the wearing coat. Wearing coat shall be discontinued at expansion joint locations. Joint fillers shall extend upto the top of wearing coat.

8 20 mm expansion joint does not cater for any allowance for possible tilting of abutment.

9 Support for the deck slab shall provide a bearing widt of 400 mm.

10 In urban areas, chequered tiles may be provided in the footpath portion by suitably adjusting the thickness of the footpath slab.

11 Type position of return walls, railings, guard posts, ramp etc. in approach portion shall be decided by the Engineer-in-charge.

(B) MATERIALS SPECIFICATIONS

Concrete

1 Concrete shall be of design mix and shall have minimum 28 days characteristic strength on 150 mm cubes for all elements of superstructure as indicated below

Conditions of exposure	Concrete grade	Characteristic Strength
MODERATE	M 25	25 MPa (for 3 m to 9 m span)
MODERATE	M 30	30 MPa (for 10 m span)
SEVERE	M 30	30 MPa (for 3 m to 10 m span)

2 High strength ordinary portland cement conforming to IS 8112 or ordinary portland cement conforming to IS 269 capable of achieving the required design concrete strength shall only be used.

3 The minimum cement content and maximum water cement ratio in the concrete design mix shall be 310 kg/cu.m and 0.45 respectively for 'MODERATE' conditions of exposure. The minimum cement content and maximum water cement ratio in the concrete design mix shall be 400 kg/cu.m and 0.40 respectively for 'SEVERE' conditions of exposure.

Reinforcement

All reinforcing bars shall be High Yield Strength Deformed bars (Grade designation S 415) conforming to IS 1786.

Water

Water to be used in concreting and curing shall conform to Clause 302.4 of IRC 21-1987.

(C) WORKMANSHIP/DETAILING

1 Minimum clear cover to any reinforcement including stirrups shall be 50 mm unless shown otherwise in the drawings.

2 For ensuring proper cover of concrete to reinforcement bars specially made polymer cover blocks shall only be used.

3. **Construction Joints**

I The location and provision of construction joints shall be approved by Engineer-in-charge. The concreting operation shall be carried out continuously upto the construction joint.

II The concrete surface at the joint shall be brushed with a stiff brush after casting while the concrete is still fresh and it has only slightly hardened.

III Before new concrete is poured the surface of old concrete shall be prepared as under:

a) For hardened concrete, the surface shall be thoroughly cleaned to remove debris/laitance and made rough so that 1/4 of the size of the aggregate or structurally damaging the concrete.

b) For partially hardened concrete, the surface shall be treated by wire brush followed by an air jet.

c) The old surface shall be soaked with water without leaving puddles immediately before starting concreting to prevent the absorption of water from new concrete.

IV New concrete shall be thoroughly compacted in the region of the joint.

4 Welding of reinforcement bars shall not be permitted.

5 Laps in reinforcement

I Minimum lap length of reinforcement shall be kept as 83 d where 'd' is the diameter of bar.

II Not more than 50 per cent of reinforcement shall be lapped at any one location.

6 Bending of reinforcement bars shall be as per IS:2502.

7. Supporting chairs of 12 mm diameter shall be provided at suitable intervals as per IS:2502.

8 Concrete shall be produced in a mechanical mixer of capacity not less than 200 litres having integral weight-batching facility and automatic water measuring and dispensing device.

9. Proper compaction of concrete shall be ensured by use of full width screed vibrators for concrete in deck slab.

10. Properly braced steel plates shall be used as shuttering.

11 Sharp edges of concrete shall be chamfered.

(D) GENERAL SPECIFICATIONS

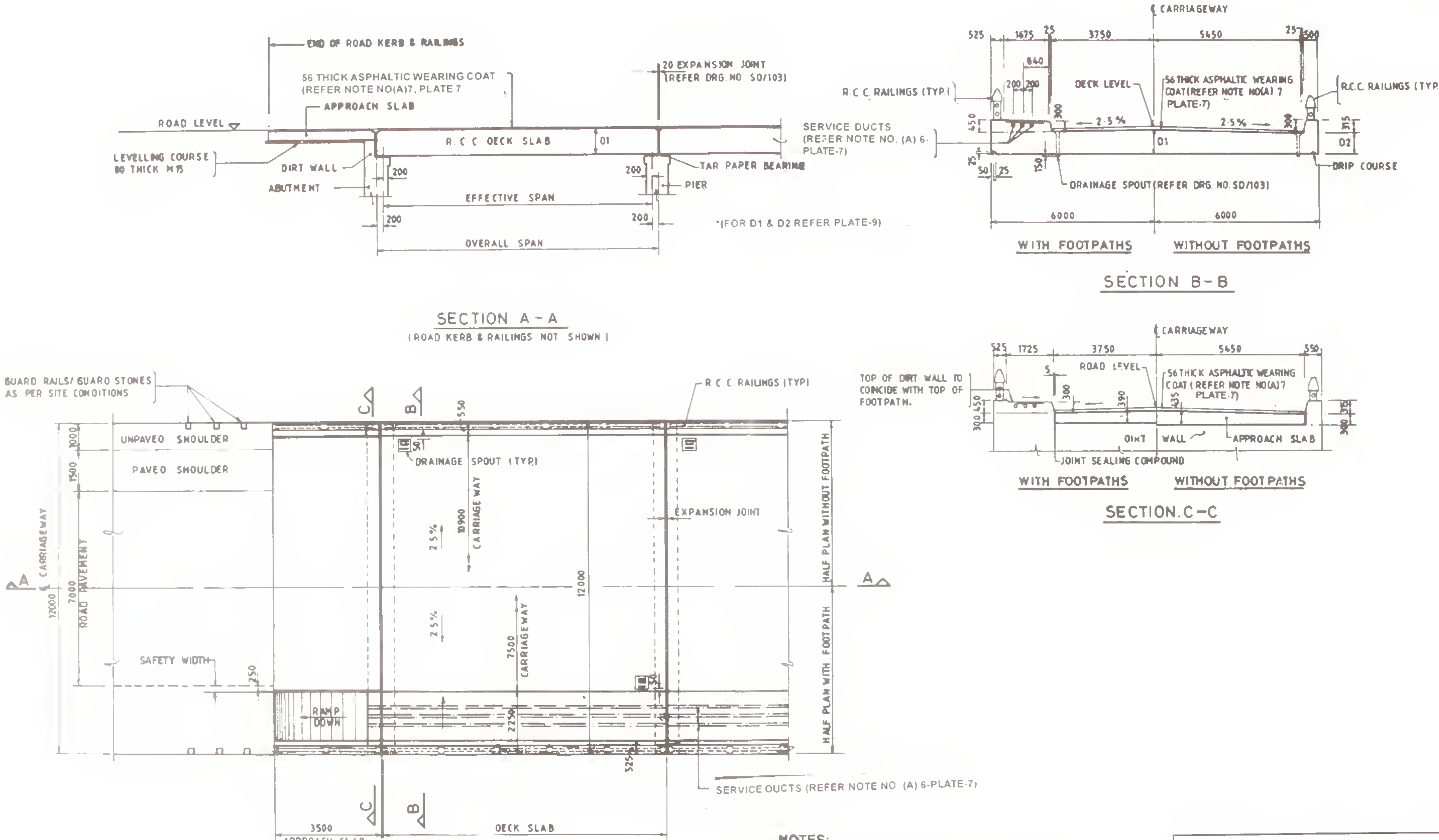
The work shall be executed in accordance with MoRT&H's Specification for Road and Bridge Works (Fourth Revision, 2001) except wherever otherwise mentioned.

(E) For details refer MORT&H's Standard Drawings as listed below:

Drawing No.	Title
SD 102*	General Arrangement (*Extracts are also reproduced at Plate-8 for reference only)
SD 103 & SD 104	Miscellaneous Details
SD/105	Details of R.C.C. Railings (without footpaths)
SD/106	Details of R.C.C. Railings (with foot-paths)
SD 107 through SD/114	R.C.C. solid slab superstructure (right) Spans 3 m to 10 m (without footpaths)
SD 115 through SD 122	R.C.C. Solid slab superstructure (right) Spans 3 m to 10 m (with footpaths)

R.C.C. SOLID SLAB
SUPERSTRUCTURE (RIGHT)
EFFECTIVE SPAN
3.0 m TO 10.0 m
(WITH AND WITHOUT FOOTPATHS)

GENERAL NOTES



PLAN.

NOTES:

1. All dimensions are in millimetres unless otherwise mentioned. Only written dimensions are to be followed.
2. Typical arrangement of drainage spouts has been shown in plan. Suitable modifications may be made by the Engineer-in-charge as per site conditions and intensity of rainfall

R.C.C. SOLID SLAB

SUPERSTRUCTURE (RIGHT)
EFFECTIVE SPAN
3.0 m to 10.0 m

(WITH AND WITHOUT FOOTPAHTS)

GENERAL ARRANGEMENT

Type	Effective Span (m)	Depth of Slab D1 (mm)	D2 (mm)	Concrete (m^3)	Steel (kg)	Asphaltic wearing coat (m^2)	Type	Effective Span (m)	Depth of Slab D1 (mm)	D2 (mm)	Concrete (m^3)	Steel (kg)	Asphaltic wearing coat (m^2)
Without Foot Paths	3.0	450	300	16.42	835	37.06	With Foot Paths	3.0	450	300	20.44	917	25.50
	4.0	500	350	23.89	1296	47.96		4.0	500	350	29.09	1391	33.00
	5.0	550	400	32.57	2061	58.86		5.0	550	400	38.95	2170	40.50
	6.0	600	450	42.43	2825	69.76		6.0	600	450	50.00	2955	48.00
	7.0	650	500	53.51	3657	80.66		7.0	650	500	62.25	3794	55.50
	8.0	750	600	70.81	4727	91.56		8.0	750	600	80.74	4823	63.00
	9.0	820	670	87.14	6076	102.46		9.0	820	670	98.26	6166	70.50
	10.0	900	750	106.40	7047	113.36		10.0	900	750	118.70	7124	78.00

Note:

1. Quantity of steel includes 5 per cent extra for laps and wastage
2. Quantities pertaining to approach slab and railings not included

R.C.C. Solid Slab Superstructure (Right)
Effective Span 3.0 m to 10.0 m (With and Without Footpaths) Depth of Slab and Quantities Per Span

(A) GENERAL

[illegible] notes are applicable for the Standard Drawings [illegible] solid slab superstructure with and without [illegible].

[illegible] drawings are applicable only for skew bridges [illegible] right width of 12 m

[illegible] footpaths shall be provided on the bridges [illegible] length less than 30 m measured along skew [illegible] unless the same are otherwise existing on the [illegible].

[illegible] are in millimetres unless otherwise [illegible] only written dimensions are to be followed [illegible] shall be scaled

[illegible] teria

The design is according to the following codes:

(a) IRC:5-1985

(b) IRC:6-1966 (1985 reprint)

(c) IRC:21-1987

[illegible] following loads have been considered in the [illegible]:

(a) One-lane of IRC class 70R or two lanes of IRC class A on carriageway, whichever governs

(b) Footpath load of 5 kN/sq m for superstructure having footpaths

(c) Wearing coat load of 2 kN/sq m

The designs are applicable for 'MODERATE' AND 'SEVERE' conditions of exposure

[illegible] services (except water supply and sewerage), [illegible] shall be carried over the bridge through 150 [illegible] ducts provided in the footpaths. Total load [illegible] shall not be more than 1.0 kN per metre [illegible] path. Water sewerage pipeline shall not be [illegible] over any part of the superstructure. Inspection [illegible] footpaths may be provided as shown in the [illegible] location and spacing of chambers along [illegible] will be decided by the Engineer-in-charge [illegible] with the users.

[illegible] shall consist of the following:

(a) A coat of mastic asphalt 6 mm thick with a prime coat over the top of the deck before the wearing coat is laid. The prime coat of [illegible] asphalt shall be 30 per cent straight [illegible] 80/40 penetration grade bitumen and 50 [illegible] cent light solvent (Benzol) to be laid over the deck slab. The insulating layer of [illegible] thick mastic asphalt with 75 per cent [illegible] stone dust filler and 25 per cent of 30/40 penetration grade bitumen shall be laid at 375°F with broom over prime coat

(b) 50 mm thick Bituminous Concrete wearing coat in two layers of 25 mm each as per Clause 509 of MoRT&H's Specifications for Road and Bridge Works (Fourth Revision 2001)

II In case of isolated bridge construction or bridges located in remote areas where provision of mastic and asphaltic concrete wearing coat is not practicable, the Engineer-in-charge may permit provision of 75 mm thick reinforced cement concrete wearing coat in M30 grade concrete with maximum water cement ratio as 0.40. The reinforcement shall consist of 8 mm High Yield Strength Deformed bars @ 200 mm centres. However, in the 300 mm width adjacent to expansion joint the reinforcement shall comprise 8 mm dia HYSD bars spaced at 100 mm centre-to-centre in both directions

8 Wearing coat shall be discontinued at expansion joint locations. Joint fillers shall extend upto the top of wearing coat

9 Width of expansion joint has been kept 20 mm only which does not cater for any allowance for possible tilting of abutment.

10 Support for the deck slab shall provide a bearing width of 400 mm measured in a direction perpendicular to support

11 In urban areas, chequered tiles may be provided in the footpath portion by suitably adjusting the thickness of the footpath slab

12 Type position of return walls, railings, guard posts, ramp etc. in approach portion shall be decided by the Engineer-in-charge

(B) MATERIALS SPECIFICATIONS

Concrete

1 Concrete shall be design mix shall have minimum 28 days characteristic strength of 30 MPa on 150 mm cubes for all elements of superstructure for both 'SEVERE' and 'MODERATE' conditions of exposure

2 Ordinary portland cement conforming to IS 269 or high strength ordinary portland cement conforming to IS 8112 capable of achieving the required design concrete strength shall only be used

3 The minimum cement content and maximum water cement ratio in the concrete design mix shall be 310 kg/cu m and 0.45 respectively for 'MODERATE' conditions of exposure. The minimum cement content and maximum water cement ratio in the concrete design mix shall be 400 kg/cu m and 0.40 respectively for 'SEVERE' conditions of exposure

Reinforcement

[illegible] reinforcement bars shall be High Yield Strength Deformed bars (Grade designation S 415) conforming to IS 1786

Water

Water to be used in concreting and curing shall conform to clause 302.4 of IRC 21-1987

(C) WORKMANSHIP/DETAILING

1 Minimum clear cover to any reinforcement including stirrups shall be 50 mm unless shown otherwise in the drawings

2 For ensuring proper cover of concrete to reinforcement bars specially made polymer cover blocks shall only be used

3 Construction joints

I The location and provision of construction joints shall be approved by the Engineer-in-charge. The concreting operation shall be carried out continuously upto the construction joint

II The concrete surface at the joint shall be brushed with a stiff brush after casting while the concrete is still fresh and it has only slightly hardened.

III Before new concrete is poured the surface of old concrete shall be prepared as under

(a) For hardened concrete, the surface shall be thoroughly cleaned to remove debris laitance and made rough so that 1/4 of the size of the aggregate is exposed but without dislodging the aggregate or structurally damaging the concrete

(b) For partially hardened concrete, the surface shall be treated by wire brush followed by an air jet

(c) The old surface shall be soaked with water without leaving puddles immediately before starting concreting to prevent the absorption of water from new concrete

(d) New concrete shall be thoroughly compacted in the region of the joint

4 Welding of reinforcement bars shall not be permitted

5 Laps in reinforcement

I Minimum lap length of reinforcement shall be kept as 83 d where 'd' is the diameter of bar.

II Not more than 50 per cent of reinforcement shall be lapped at any one location

III For closely spaced bars lapping may be avoided by providing suitable type of mechanical splices.

6 Bending of reinforcement bars shall be as per IS 2502

7 Supporting chairs of 1. mm diameter shall be provided at suitable intervals as per IS:2502.

8 Concrete shall be produced in a mechanical mixer of capacity not less than 200 litres having integral weigh-batching facility and automatic water measuring and dispensing device

9 Proper compaction of concrete shall be ensured by use of full width screed vibrators for concrete in deck slab

10 Properly braced steel plates shall be used as shuttering

11 Sharp edges of concrete shall be chamfered.

(D) GENERAL SPECIFICATIONS

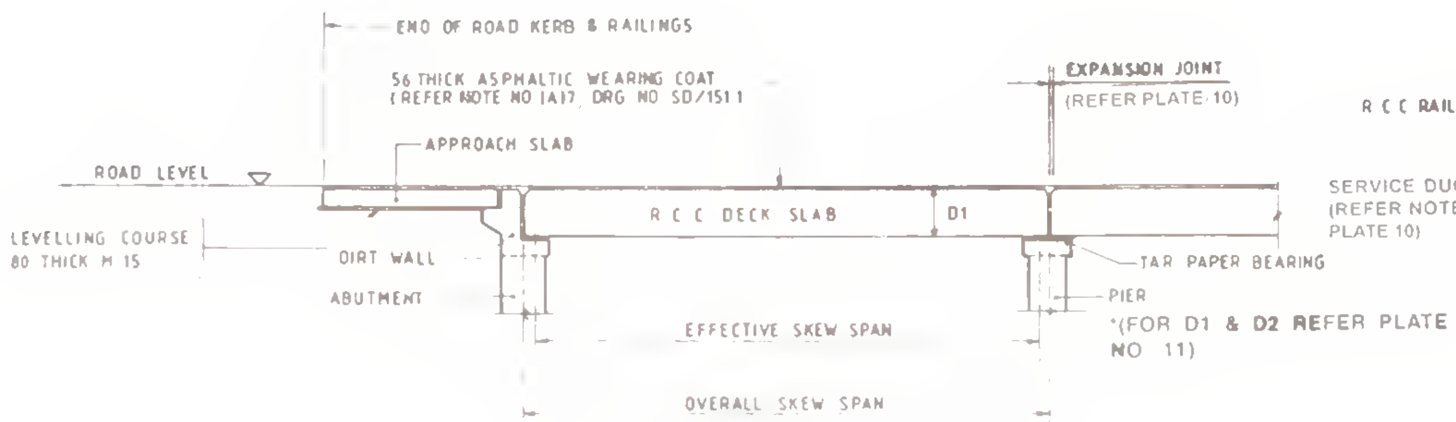
The work shall be executed in accordance with MoRT&H's Specifications for Road and Bridge Works (Fourth Revision, 2001) except wherever otherwise mentioned

(E) For details refer MoRT&H's Standard Drawings as listed below

Drawing No.	Title
SD 152, SD 153 SD 154, SD 155*	General Arrangement for various skew angles (*Extracts are also reproduced at Plate-12 for reference only)
SD 156, SD 157 & SD 158	Miscellaneous details
SD 159	Details of R C C Railings (without footpaths)
SD 160	Details of R C C Railings (with foot-paths)
SD 161-SD 164 SD 169-SD 172 SD 177-SD 180 SD 185-SD 188	R C C solid slab Superstructure (without footpaths) for various skew angles
SD 165-SD 168 SD 173-SD 176 SD 181-SD 184 SD 189-SD 192	R C C solid slab superstructure (with footpaths) for various skew angles

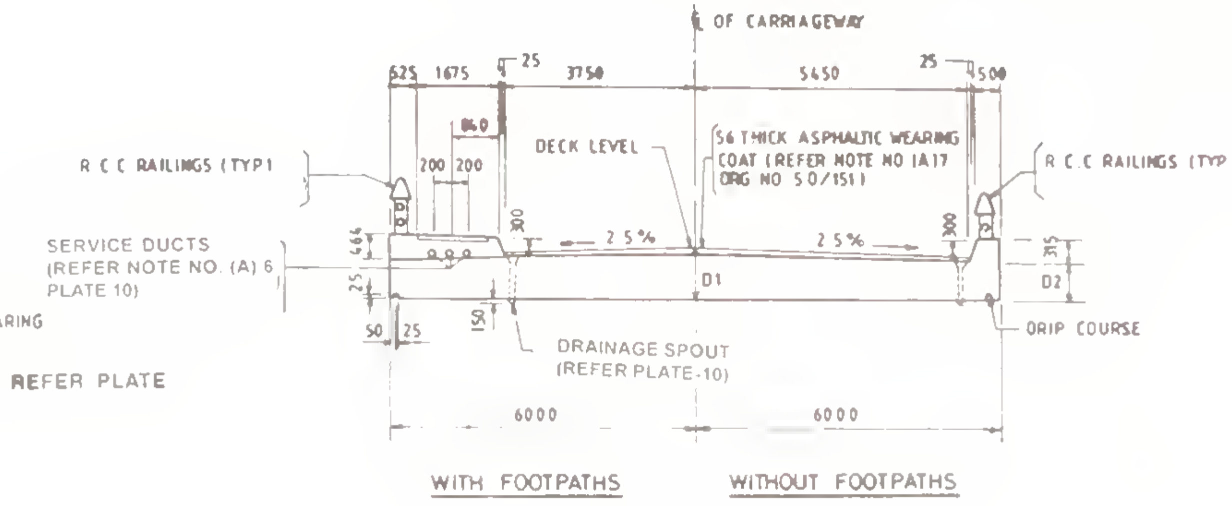
R.C.C. SOLID SLAB SUPERSTRUCTURE (SKEW) RIGHT EFFECTIVE SPAN 4.0 m TO 10.0 m (WITH AND WITHOUT FOOTPATHS)

GENERAL NOTES

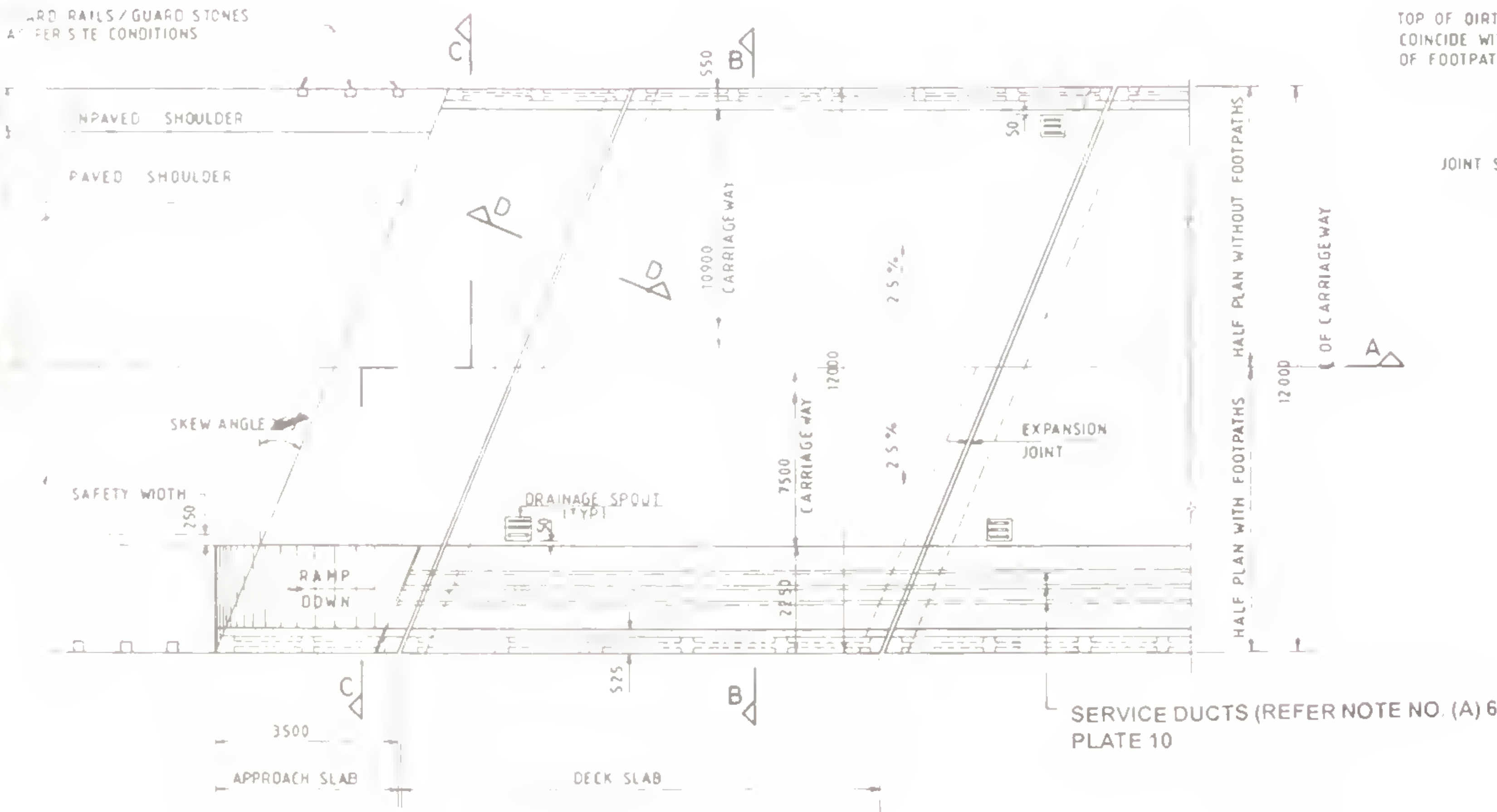


SECTION A - A

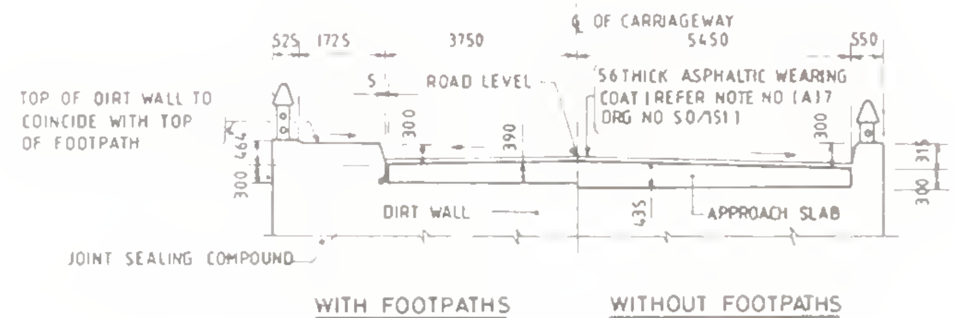
(ROAD KERB & RAILINGS NOT SHOWN)



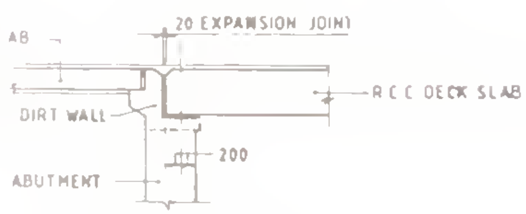
SECTION B - B



PLAN



SECTION C-C



SECTION D-D

NOTES:

1. All dimensions are in millimetres unless otherwise mentioned. Only written dimensions are to be followed.
2. Typical arrangement of drainage spouts has been shown in plan. Suitable modifications may be made by the Engineer-in-charge as per site conditions and intensity of rainfall

R.C.C. SOLID SLAB

SUPERSTRUCTURE (SKEW)
RIGHT EFFECTIVE SPAN

4.0; 6.0; 8.0; 10.0 m

(WITH AND WITHOUT FOOTPAHTS)

GENERAL ARRANGEMENT

Type	Skew Angle	Right Effective Span (m)	Depth of Slab D$_1$ (mm)	Depth of Slab D$_2$ (mm)	Concrete (m^3)	Steel (kg)	Asphaltic wearing Coat (m^2)	Type	Skew Angle	Right Effective Span (m)	Depth of Slab D$_1$ (mm)	Depth of Slab D$_2$ (mm)	Concrete (m^3)	Steel (kg)	Asphaltic wearing Coat (m^2)
↑	15°	4	525	375	26.10	1768	49.65	↑	15°	4	525	375	31.47	1846	34.16
	15°	6	625	475	45.90	3956	72.23		15°	6	625	475	53.73	3773	49.7
	15	8	775	625	75.920	7064	94.79		15°	8	775	625	86.17	7094	65.22
	15°	10	925	775	113.40	8785	117.36		15°	10	925	775	126.07	8781	80.75
	22.5°	4	525	375	27.33	1882	51.92		22.5°	4	525	375	32.91	2003	35.72
	22.5°	6	625	475	48.06	3851	75.50		22.5°	6	625	475	56.17	4030	51.95
	22.5°	8	775	625	79.45	5867	99.10		22.5°	8	775	625	90.10	6108	68.19
Without	22.5°	10	925	775	118.6	9704	122.70	With	22.5°	10	925	775	131.81	9983	84.43
Foot	30°	4	525	375	29.12	2063	55.37	Foot	30°	4	525	375	35.10	2193	38.10
Paths	30°	6	625	475	51.22	3886	80.55	Paths	30°	6	625	475	59.92	4059	55.43
	30°	8	775	625	84.69	6292	105.73		30°	8	775	625	96.12	6548	72.75
	30°	10	925	775	126.47	10147	130.90		30°	10	925	775	140.61	10474	90.07
	35°	4	525	375	30.82	2279	58.54		35°	4	525	375	37.10	2415	40.28
	35°	6	625	475	54.21	4329	85.16		35°	6	625	475	63.35	4529	58.60
	35°	8	775	625	89.61	7804	111.78		35°	8	775	625	101.61	8100	76.91
↓	35°	10	925	775	133.78	11311	138.38	↓	35°	10	925	775	148.63	11654	95.21

Note:

1 Quantity of steel includes 5 per cent extra for laps and wastage

2 Quantities pertaining to approach slab and railings not individed

R.C.C. Solid Slab Superstructure (Skew)

Right Effective Span
4.0; 6.0; 8.0; 10.0 m
(With and Without Footpaths)

DEPTH OF SLAB AND QUANTITIES PER SPAN

(A) GENERAL

1. These notes are applicable for the Standard Drawings for R.C.C. Box Cell Structures with earth cushion (3 m, 4 m & 5 m) and without earth cushions. For intermediate heights immediately higher value of earth cushion can be taken for standard drgs.

2. These drawings are applicable for right crossings with overall width of 12 m for the roadway on top.

3. All dimensions are in millimeters unless otherwise mentioned. Only written dimensions are to be followed. No drawing shall be scaled.

4. Box cell designation i.e. Nc ab Ec stands for No. of cells Clear width-Clear height Height of earth cushion.

5. Design criteria

I. The design is according to the following codes

(a) IRC 5-1985

(b) IRC 6-1966 (1985 reprint)

(c) IRC 21-1987 (1997 reprint)

(d) IRC 78-1985

II. The following loads have been considered in the design

(a) One-lane of IRC class 70R or two lanes of class A on carriageway, whichever governs

(b) Wearing coat load of 3 kN sq m

III. The designs are applicable for 'MODERATE' AND 'SEVERE' conditions of exposure

6. Wearing coat shall consist of the following for Box Cell Structures without earth cushions

I (a) A coat of mastic asphalt 12 mm thick with a prime coat over the top of the deck is to be provided before the wearing coat is laid

(b) 50 mm thick asphaltic concrete wearing coat as per Clause 512 of MOST's Specifications for Road and Bridge Works (Third Revision-1995)

II. In case of isolated construction of Box Cell Structures located in remote areas where provision of mastic and asphaltic concrete wearing coat is not practicable, Engineer-in-Change may permit provision of 75 mm thick cement concrete wearing coat in M30 grade concrete with maximum water cement ratio as 0.40. The reinforcement shall be consist of 8 mmϕ High Yield Strength Deformed bars @ 200 mm centers reducing to 100 centres in both direction over a strip of 300 mm near the expansion joint. Reinforcement shall be placed at the centre of the centre of the wearing coat, wearing coat shall be discontinued at expansion joint locations. Joint fiters shall extend upto the top of wearing coat

III. For Box Cell Structures with earth cushion, no wearing coat shall be provided

7. Type position of return walls, railings, guards, posts, ramp, etc. in approach portion shall be decided by the Engineer-in-charge

8. Lowest point in the proposed Box Cell plan area is assumed as *Natural Ground level*

9. Invert level of Top surface of Bottom Slab is assumed as *Bed Level*

(B) MATERIALS SPECIFICATIONS

Concrete

1. Concrete shall be design mix and shall have minimum 28 days characteristic strength on 150 mm cubes for all elements of structure as indicated below

Element	Concrete grade		Characteristic Strength (MPa)	
	Moderate condition of exposure	Severe condition of exposure	Moderate condition of exposure	Severe condition exposure
(a) Box Cell Structure	M20	M25	20	25
(b) Wing Walls	M20	M20	20	20
(c) Curtain Wall	M15	M20	15	20
(d) Levelling Coarse	M15	M15	15	15

2. High strength ordinary Portland cement conforming to IS 8112 or ordinary Portland cement conforming to IS 269 capable of achiving the required design concrete strength shall only be used

3. The minimum cement concrete and water cement ratio in the concrete design mix shall be 310 kg per cu m and 0.45 respectively for 'MODERATE' conditions of exposure. The minimum cement content and maximum water cement ratio in the concrete design mix shall be 400 kg cu m and 0.40 respectively for 'SEVERE' conditions of exposure

4. The total chloride contents and Sulfuric anhydride (SO3) of all concrete as a percentage of mass of cement in mix shall be limited to 0.3 per cent and 4 per cent respectively

5. The slump of concrete shall be checked as per IS 516. Concrete should have the slump of 50-75 mm

6. Use of admixtures such as super plasticisers for concrete may be made with the approval of the Engineer-in-charge

7. Aggregate shall confirm to CL 302.3 of IRC 21-1987 (1997 reprint) and maximum aggregate size should not exceed 40 mm

Reinforcement

1. All reinforcement shall be High Yield Strength Deformed bars (Grade designation S 415) conforming to IS 1786

2. Unless otherwise shown on the drawing, bars are marked in numerical numbers [as (1), (2) or (3)] and corresponding information is provided in bar bending schedule. Bars configuration is shown as–

TOR BAR
10 Φ @ 200 c/c
DIA OF BAR — SPACING OF BARS

3. Spacing given for all reinforcement is perpendicular to bar unless otherwise shown on drawings.

Earth Fill/Embankment

Backfilling material should confirm to CL 305.2 of MOST Specification and earth cushion embankment should be constructed in accordance to section 300 of MOST Specification (Third Revision 1995)

Water

1. Water to be used in concreting and curing shall be conforming to Clause 302.4 of IRC:21–1987.

Expansion Joint

1. The asphalt plug expansion joint shall be provided in accordance with **MOST** Specification and shall be procured from manufacturers as approved by **MOST**

(C) Workmanship/Detailing

1. Minimum clear cover to any reinforcement including stirrups shall be 50 mm unless otherwise shown in the drawings

2. Construction Joints.

I. The location and provision of construction joints shall be approved by Engineer-in-charge suggested location of construction joints in the direction parallel to the direction of water flow is shown in the General Arrangement drawings of Box Cell Structures. The concreting operation shall be carried out continuous upto the construction joints

II. The concrete surface at the joint shall be brushed with a stiff brush after casting while the concrete is still fresh and it has only slightly hardened.

III. Before new concrete is poured the surface of old concrete shall be prepared as under

(a) For hardened concrete, the surface shall be thoroughly cleaned to remove debris/laitance and made rough so that 1/4 of the size of the aggregate is exposed

(b) For partially hardened concrete, the surface shall be treated by wire brush followed by an air jet.

(c) The old surface shall be soaked with water without leaving puddles immediately, before starting concreting to prevent the absorption of water from new concrete.

IV. New concrete shall be thoroughly compacted in the region of the joint.

3. Welding of reinforcement bars shall not be permitted

4. Laps in reinforcement:

I. Minimum lap length of reinforcement shall be decided as per the reinforcement arrangement based on the Clause–304.6.6 of IRC:21–1987.

II. Not more than 50 per cent of reinforcement shall be lapped at any one location.

5. Bending of reinforcement bars shall be as per IS:2502.

6. Supporting chairs of 12 mm diameter shall be provided at suitable intervals as per IS:2502.

7. Concrete shall be produced in a mechanical mixer of capacity not less than 200 lts. having integral weigh-batching facility and automatic water measuring and dispensing device.

8. Proper compaction of concrete shall be ensured by use of full width screed vibrators for concrete.

9. Properly braced steel plates shall be used as shuttering

10. sharp edges of concrete shall be chamfered.

11. Filter media should be provided in accordance to Clause 2504.2.2 of MOST Specifications (Third Revision 1995)

12. In presence of soil with aggressive soil condition, the concrete faces in contact with earth shall be protected with approved bituminous point or coating as decided by the Engineer-in-Change.

(D) General Specifications

1. The work shall be executed in accordance with MOST's Specification for Road and Bridge Works (Fourth Revision, 2001) except wherever otherwise mentioned.

(E) For reinforcement and all other details refer MORT&H's Standard Drawing (SD/102-SD-117).

DRAWINGS FOR BOX CELL STRUCTURES
GENERAL NOTES

WITH EARTH CUSHION CASES

1.1 SINGLE CELL BOX

S.NO.	Nc/ab/Ec
1	1/22/3
2	1/22/4
3.	1/22/5
4	1/23/3
5	1/23/4
6	1/23/5
7	1/33/3
8	1/33/4
9	1/33/5
10	1/34/3
11	1/34/4
12	1/34/5
13	1/43/3
14.	1/43/4
15	1/43/5
16	1/44/3
17	1/44/4
18	1/44/5
19	1/45/3
20	1/45/4

S.NO.	Nc/ab/Ec
21	1/45/5
22.	1/53/3
23.	1/53/4
24	1/53/5
25	1/54/3
26	1/54/4
27.	1/54/5
28.	1/55/3
29	1/55/4
30	1/55/5
31	1/63/3
32.	1/63/4
33.	1/63/5
34	1/64/3
35	1/64/4
36	1/64/5
37	1/65/3
38	1/65/4
39	1/65/5
40	1/66/3

S.NO.	Nc/ab/Ec
41	1/66/4
42	1/66/5
43	1/75/3
44	1/75/4
45	1/75/5
46	1/76/3
47.	1/76/4
48	1/76/5
49	1/77/3
50	1/77/4
51	1/77/5
52	1/85/3
53	1/85/4
54	1/85/5
55	1/86/3
56	1/86/4
57	1/86/5
58	1/87/3
59	1/87/4
60	1/87/5

1.2 DOUBLE CELL BOX

S.NO.	Nc/ab/Ec
1	2/22/3
2	2/22/4
3.	2/22/5
4	2/23/3
5.	2/23/4
6	2/23/5
7	2/32/3
8	2/32/4
9	2/32/5
10	2/33/3
11	2/33/4
12	2/33/5

1.3 TRIPLE CELL BOX

S No	Nc/ab/Ec
1	3/22/3
2	3/22/4
3	3/22/5
4	3/33/3
5	3/33/4
6	3/33/5

WITHOUT EARTH CUSHION CASES

1.4 SINGLE CELL BOX

S No.	Nc/ab/Ec
1	1/22/0
2.	1/23/0
3	1/33/0
4	1/43/0
5	1/44/0
6	1/45/0
7	1/53/0
8	1/54/0
9	1/55/0
10.	1/63/0
11	1/64/0
12	1/65/0
13	1/66/0
14	1/75/0
15	1/76/0
16	1/77/0
17	1/85/0
18	1/86/0
19	1/87/0

1.5 DOUBLE CELL BOX

S.NO.	Nc/ab/Ec
1	2/22/0
2.	2/23/0
3	2/32/0
4	2/33/0

1.6 TRIPLE CELL BOX

S.NO.	Nc/ab/Ec
1.	3/22/0
2.	3/33/0

NOTES :

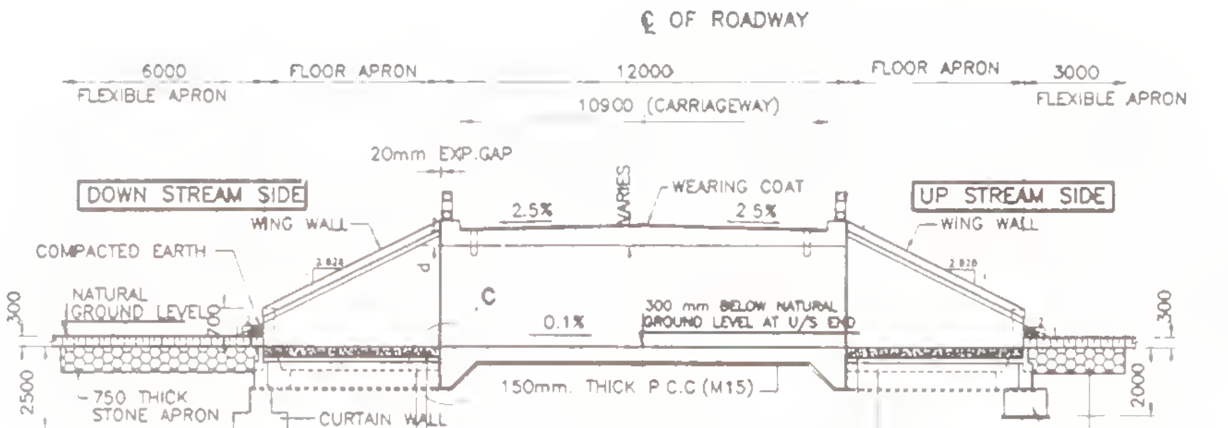
1. THIS INDEX SHEET DEFINES THE VARIOUS SIZE OPTIONS OF BOX CELL STRUCTURES INCLUDED IN THESE STANDARD PLANS.
2. Nc / ab / Ec STANDS FOR NO. OF CELLS/ CLEAR WIDTH - CLEAR HEIGHT/HEIGHT OF EARTH CUSION.

DRAWINGS FOR BOX CELL STRUCTURES

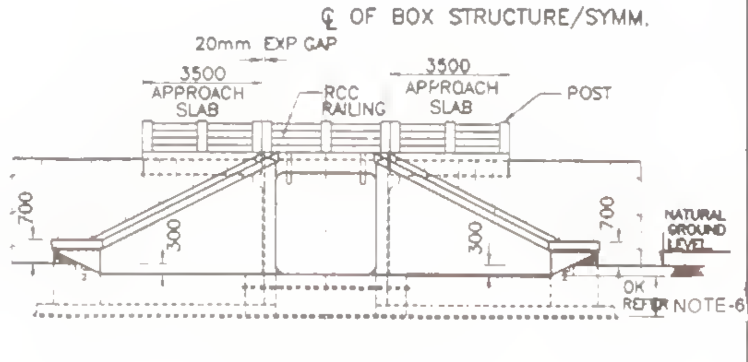
INDEX SHEET

TABLE SHOWING SALIENT DIMENSIONS

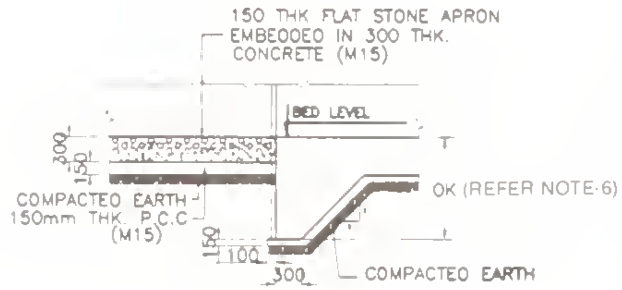
BOX CELL DESIGNATION No/ab/Ec	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f (mm)	REQUIRED BEARING CAPACITY CATEGORY	DESIGN BASE PRESSURE (T/M²)	ALLOWABLE DRAWDOWN AT A TIME (M)
1/22/0	2000	2000	500	350	380	300	C	11.62	0.00
1/23/0	2000	3000	1300	370	420	380	C	11.01	2.98
1/33/0	3000	5000	900	420	420	420	C	10.50	2.15
1/34/0	3000	4000	1700	460	480	480	C	11.37	4.61
1/43/0	4000	3000	400	450	300	500	C	10.53	1.29
1/44/0	4000	4000	1200	480	550	550	C	11.07	3.99
1/45/0	4000	3000	1900	520	620	850	C	12.48	6.14
1/53/0	6000	3000	300	600	670	670	B	9.83	1.73
1/54/0	3000	4000	600	520	600	620	C	10.66	3.40
1/55/0	5000	5000	1500	580	680	700	C	12.40	5.61
1/63/0	6000	5000	300	680	720	720	B	9.88	2.96
1/64/0	6000	4000	300	680	720	750	C	10.93	3.18
1/65/0	6000	5000	1100	680	750	750	C	11.77	5.35
1/66/0	6000	6000	1600	700	600	850	C	13.21	7.50
1/75/0	7000	5000	600	750	850	850	C	11.59	4.86
1/76/0	7000	6000	1200	780	920	950	C	13.00	6.93
1/77/0	7000	7000	1300	780	950	950	C	13.10	7.18
1/85/0	6000	5000	300	600	950	950	C	11.37	4.83
1/86/0	6000	6000	900	820	970	1000	C	12.62	6.86
1/87/0	6000	7000	1300	850	1100	1200	C	14.36	8.62



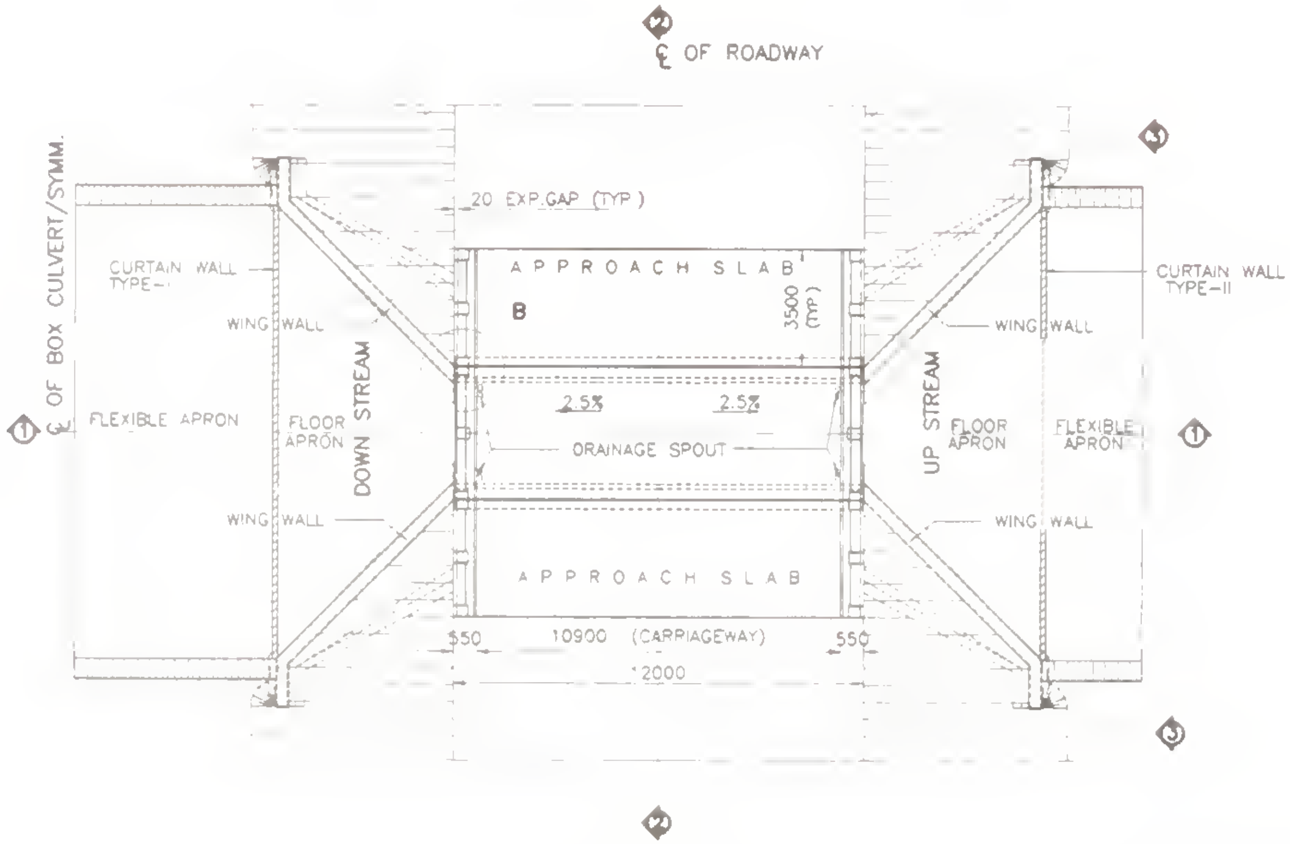
SECTION 1-1



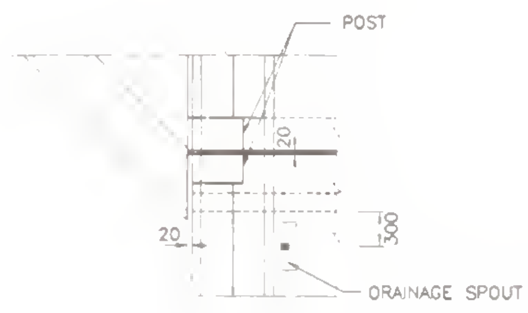
VIEW 3-3



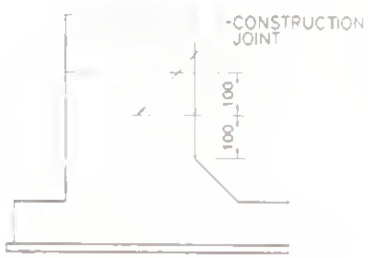
DETAIL - C



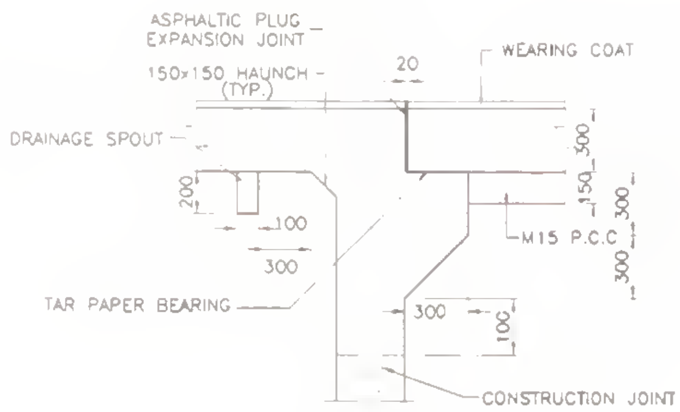
PLAN



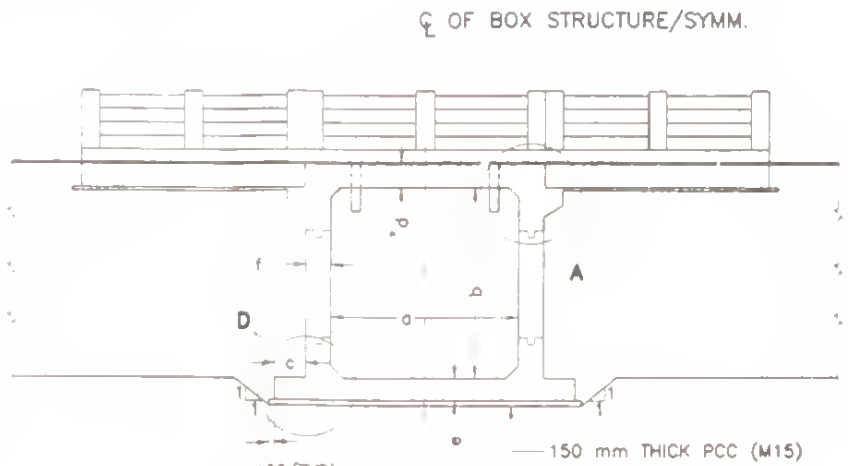
DETAIL - B



DETAIL D



DETAIL - A



SECTION 2-2

* d DECK SLAB THICKNESS AT INNER EDGE OF KERB AS SHOWN IN SECTION 1-1

NOTES:

1. ALL DIMENSIDNS ARE IN MILLIMETRES UNLESS DTHERWISE MENTIDNED. DNLY WRITTEN DIMENSIDNS ARE TD BE FDLLDWED.
2. SDIL DENSITY DN THE BACKFILL SHDULD CDNFDRM TD CLAUSE 305.2.1.5 OF MDRT&H SPECIFICATIDN FDR RDAD & BRIDGE WORKS (FDURTH REVISION 2001)
3. NET BEARING CAPACITY REDUIRED FDR SOIL IS DIVIDED IN FOUR CATEGORIES AS SHDWN BELOW

CATEGDRY	NET BEARING CAPACITY REDUIRED FOR SDIL
A	5 T/M²
B	10 T/M²
C	15 T/M²
D	20 T/M²

4. SOFT AND LOOSE PATCHES IN THE BEARING AREA ARE TO BE REPLACED BY COMPACTED GRANULAR FILLS WITH LAYERS NOT EXCEEDING 300mm
5. DESIGNS ARE GIVEN FOR THE BOX CELL STRUCTURES ONLY THESE HAVE NO BEARING WITH DESIGN OF EMBANKMENT WHICH WILL BE TAKEN UP BY ENGINEER-IN-CHARGE SEPARATELY
6. 'DK' IS DEPTH OF KEY AT BASE SLAB

FOR BASE SLAB THICKNESS	VALUE OF 'DK'
UPTO 900mm	1200mm
GREATER THAN 900mm	e+300mm

e = BASE SLAB THICKNESS

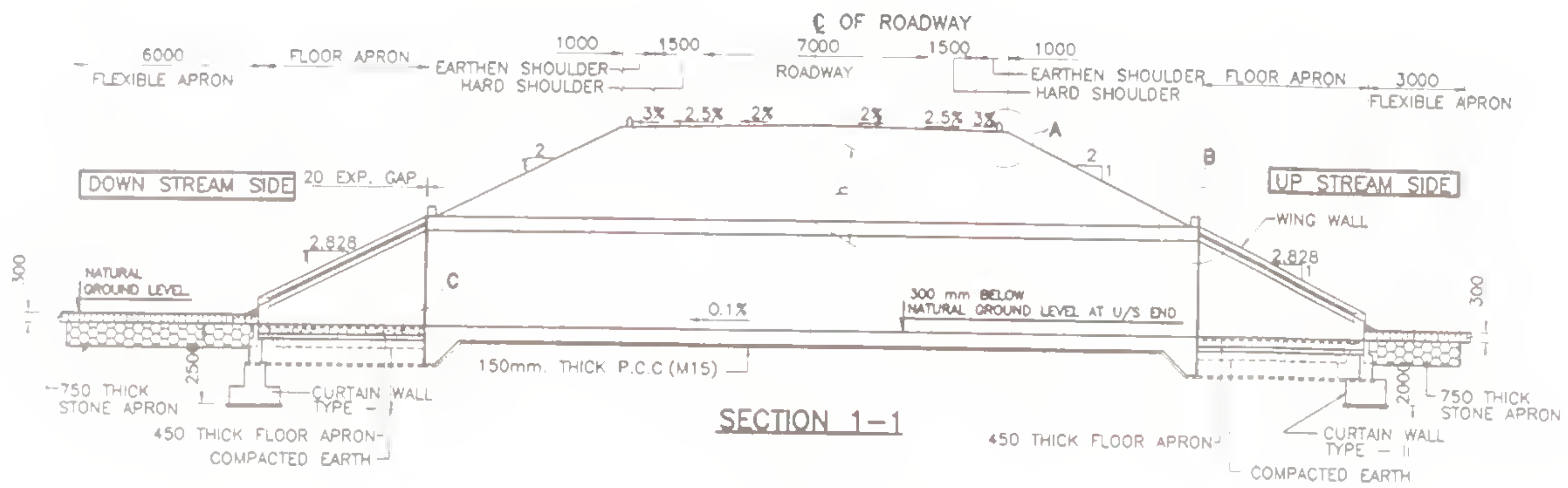
DRAWINGS FOR BOX CELL STRUCTURES

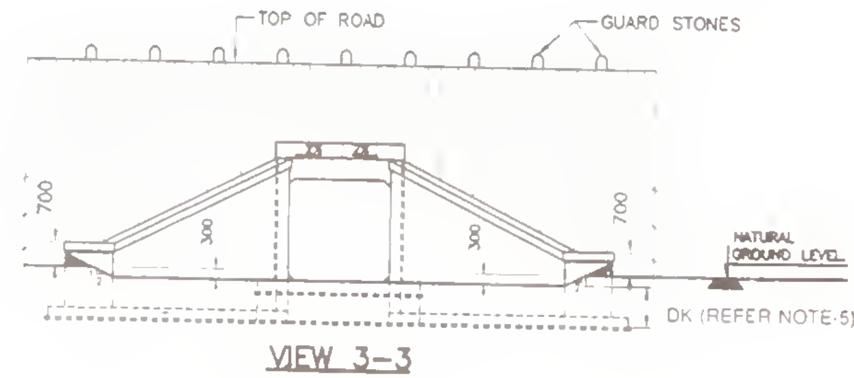
SINGLE CELL R.C.C. BOX STRUCTURES

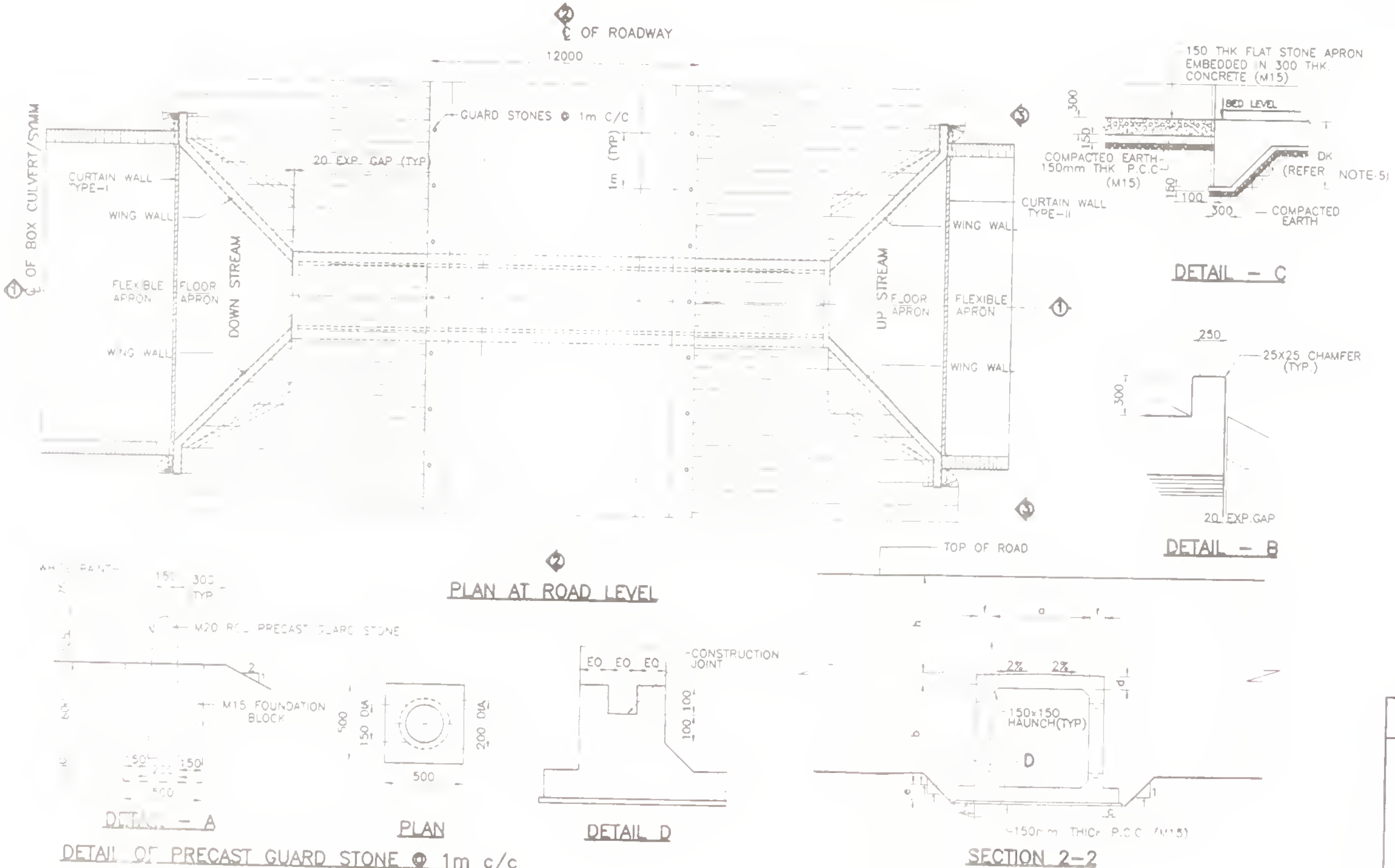
2m x 2m TO 8m x 7m

(WITHOUT EARTH CUSION)

GENERAL ARRANGEMENT







NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONED. ONLY WRITTEN DIMENSIONS ARE TO BE FOLLOWED.
2. SOIL DENSITY ON THE BACKFILL AND EARTH CUSHION SHOULD CONFORM TO CLAUSE 305.2.1.5 OF MORT&H SPECIFICATION FOR ROAD & BRIDGE WORKS (FOURTH REVISION-2001)
3. SOFT AND LOOSE PATCHES IN THE BEARING AREA ARE TO BE REPLACED BY COMPACTED GRANULAR FILLS WITH LAYERS NOT EXCEEDING 300mm.
4. DESIGNS ARE GIVEN FOR THE BOX CELL STRUCTURES ONLY. THESE HAVE NO BEARING WITH DESIGN OF EMBANKMENT WHICH WILL BE TAKEN UP BY ENGINEER-IN-CHARGE SEPARATELY.
5. 'DK' IS DEPTH OF KEY AT BASE SLAB

FOR BASE SLAB THICKNESS	VALUE OF 'DK'
UPTO 900MM	1200MM
GREATER THAN 900MM	e+300mm

e= BASE SLAB THICKNESS

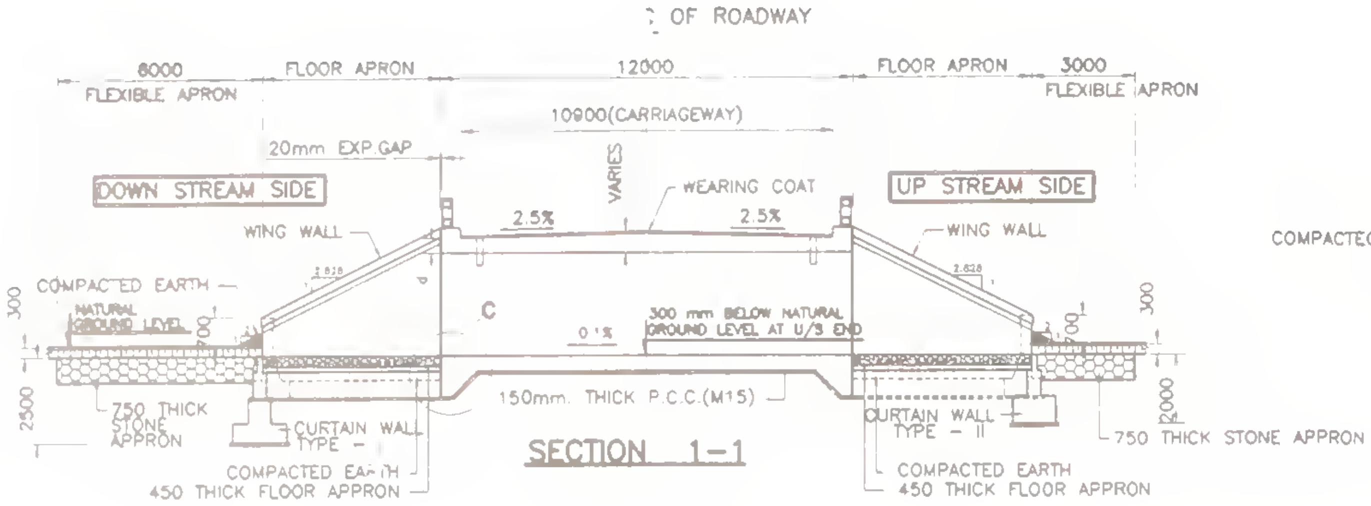
DRAWINGS FOR BOX CELL STRUCTURES

SINGLE CELL R.C.C. BOX STRUCTURES

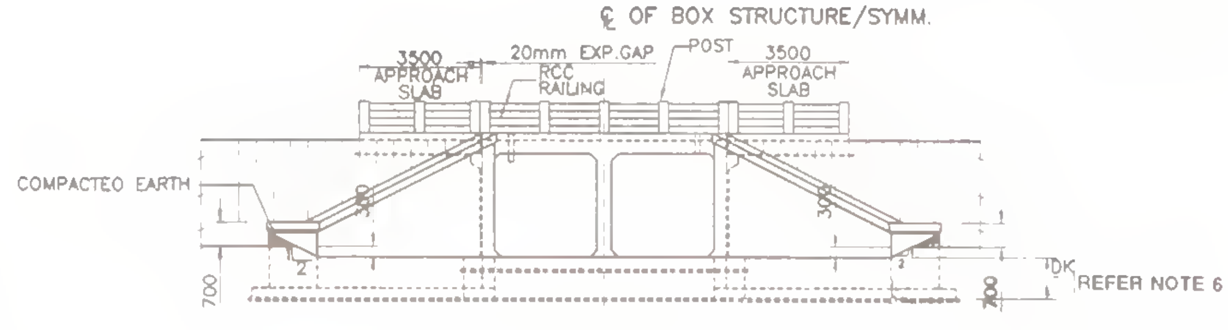
2m x 2m TO 8m x 7m

(WITH EARTH CUSION)

GENERAL ARRANGEMENT (SHEET 1)



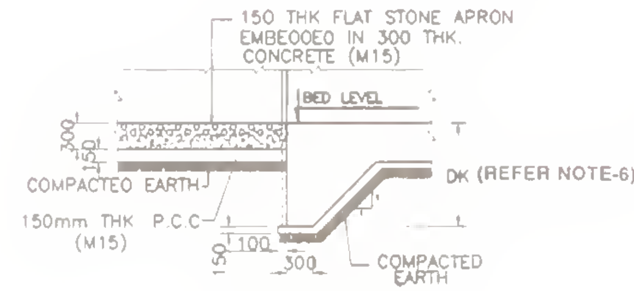
SECTION 1-1



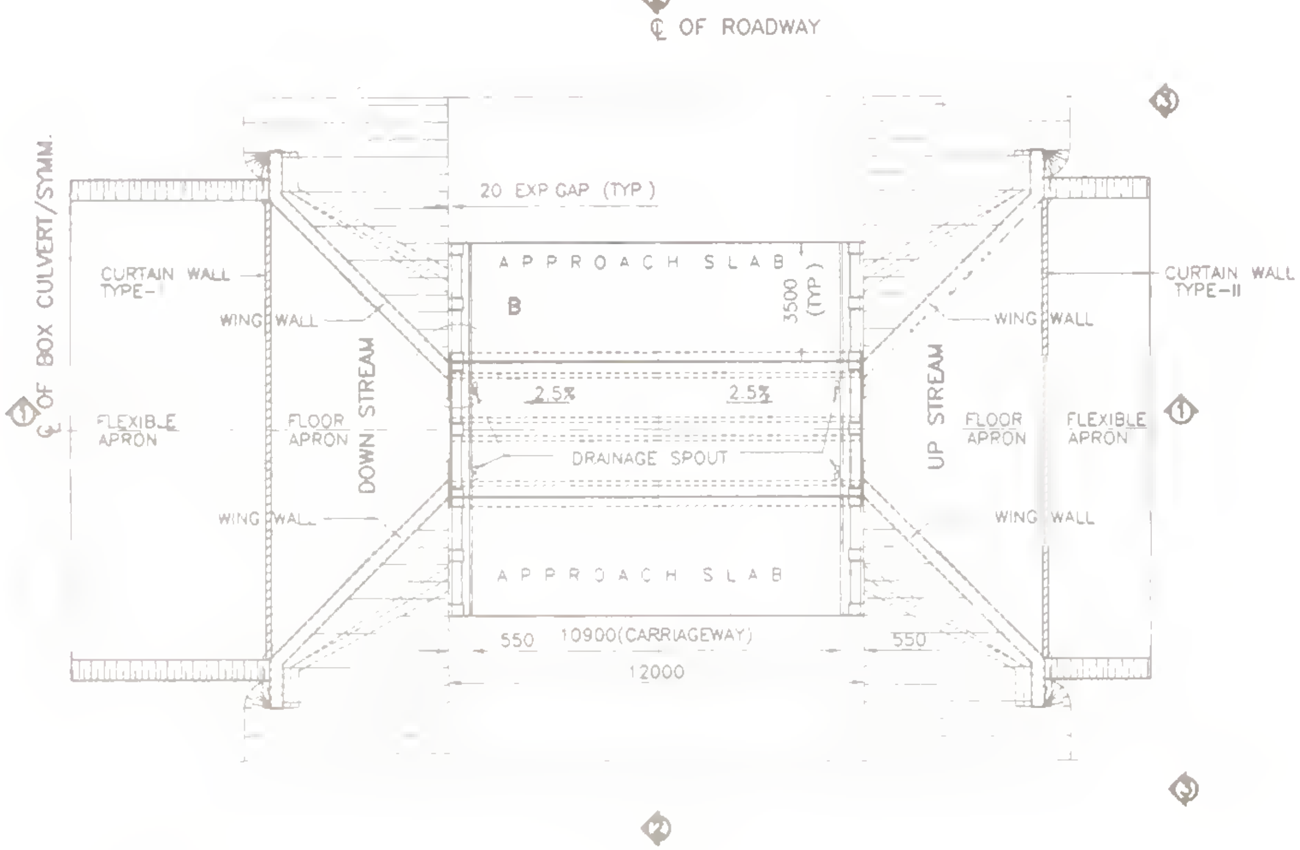
VIEW 3-3

TABLE SHOWING SALIENT DIMENSIONS

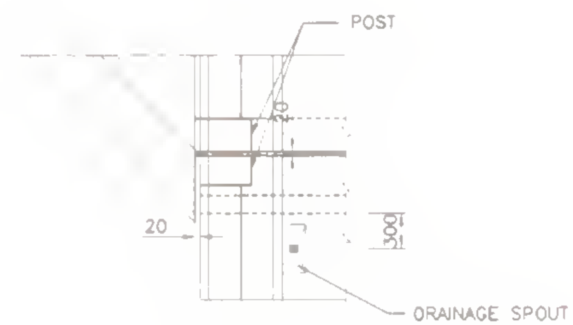
Box Cell Designation Nc/ab/Ec	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f (mm)	g (mm)	Required Bearing Capacity Category	Design Base Pressure (T/M²)	Allowable drawdown at a time (M)
2/22/0	2000	2000	300	400	400	370	300	B	8 97	0 76
2/23/0	2000	3000	300	400	420	400	300	C	10 35	0.91
2/32/0	3000	2000	300	450	450	400	300	B	7.58	1 47
2/33/0	3000	3000	300	470	450	420	350	B	8.52	1 75



DETAIL - C



PLAN



DETAIL - B

NOTES:

1 ALL OIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONEO. ONLY WRITTEN OIMENSIONS ARE TO BE FOLLOWEO.

2 SOIL DENSITY ON THE BACKFILL AND EARTH CUSHION SHOULO CONFORM TO CLAUSE 305.2.1.5 OF MORT&H SPECIFICATION FOR ROAO & BRIOGE WORKS (FOURTH REVISION-2001).

3 NET BEARING CAPACITY REOUIREO FOR SOIL IS DIVIDEO IN FOUR CATEGORIES AS SHOWN BELOW:

CATEGORY	NET BEARING CAPACITY REQUIRED FOR SOIL
A	5 T/M²
B	10 T/M²
C	15 T/M²
D	20 T/M²

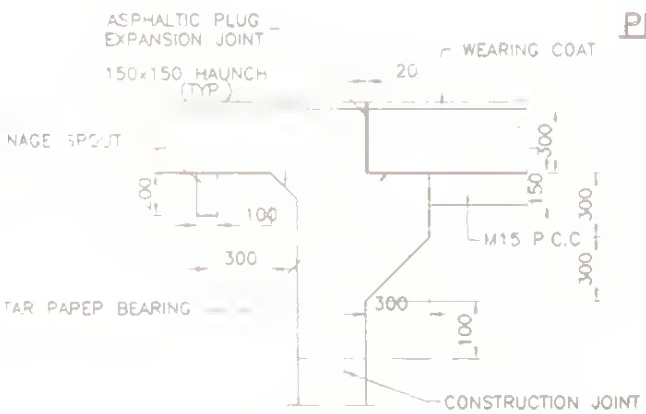
4 SOFT AND LOOSE PATCHES IN THE BEARING AREA ARE TO BE REPLACED BY COMPACTED GRANULAR FILLS WITH LAYERS NOT EXCEEDING 300mm

5 DESIGNS ARE GIVEN FOR THE BOX CELL STRUCTURES ONLY THESE HAVE NO BEARING WITH DESIGN OF EMBANKMENT WHICH WILL BE TAKEN UP BY ENGINEER-IN-CHARGE SEPARATELY

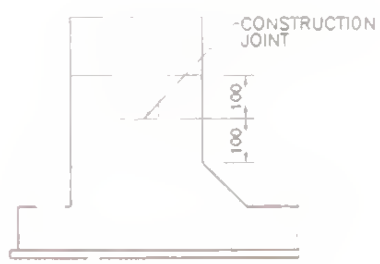
6 'DK' IS DEPTH OF KEY AT BASE SLAB

FOR BASE SLAB THICKNESS	VALUE OF 'DK'
UPTO 900mm	1200mm
GREATER THAN 900mm	e+300mm

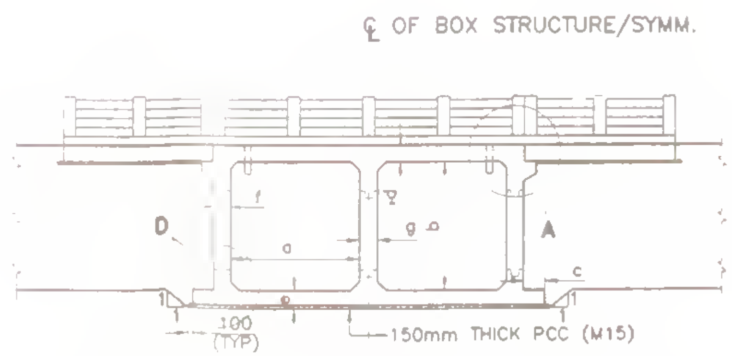
e = BASE SLAB THICKNESS



DETAIL - A



DETAIL D



SECTION 2-2

DRAWINGS FOR BOX CELL STRUCTURES
DOUBLE CELL R.C.C. BOX STRUCTURES 2m x 2m TO 3m x 3m (WITHOUT EARTH CUSION) GENERAL ARRANGEMENT

* d DECK SLAB THICKNESS AT INNER EDGE OF KERBS AS SHOWN IN SECTION 1-1

TABLE SHOWING SALIENT DIMENSIONS OF SINGLE CELL BOX CULVERT

BOX CELL DESIGNATION Nc/ab/Ec	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f (mm)	h (mm)	REQUIRED BEARING CAPACITY CATEGORY	DESIGN BASE PRESSURE (T/M^2)
1/22/3	2000	2000	600	250	300	250	5000	B	8.80
1/22/4	2000	2000	500	280	350	280	4000	[illegible]	10.17
1/22/5	2000	2000	500	300	350	300	5000	[illegible]	11.31
1/23/3	2000	3000	1100	280	300	350	5000	[illegible]	10.59
1/23/4	2000	5000	1000	320	370	400	4000	C	12.09
1/23/5	2000	5000	1100	320	350	380	5000	[illegible]	13.[illegible]4
1/33/3	5000	3000	500	370	450	400	5000	[illegible]	10.3[illegible]
1/33/4	3000	3000	750	380	470	420	4000	C	11.57
1/33/5	3000	3000	750	420	300	420	5000	C	12.66
1/34/3	3000	4000	1400	370	470	480	3000	C	12.17
1/34/4	3000	4000	1400	400	550	500	4000	C	13.68
1/34/5	3000	4000	1300	450	550	520	5000	C	14.85
1/43/3	4000	5000	500	470	550	420	3000	C	10.12
1/43/4	4000	3000	400	300	600	450	4000	C	11.42
1/43/5	4000	5000	300	570	670	300	5000	C	12.88
1/44/3	4000	4000	1000	470	600	550	5000	C	11.87
1/44/4	4000	4000	1000	500	600	550	4000	C	13.0[illegible]
1/44/5	4000	4000	1000	300	650	550	5000	C	14.31
1/45/3	4000	5000	1600	450	600	630	3000	C	13.85
1/45/4	4000	5000	1600	300	650	650	4000	C	15.[illegible]1
1/45/5	4000	5000	1300	550	700	700	5000	[illegible]	16.38
1/53/3	5000	3000	300	550	650	500	5000	C	10.11
1/53/4	5000	3000	300	620	700	520	4000	C	11.54
1/53/5	5000	3000	300	670	770	550	5000	C	12.98
1/54/3	5000	4000	600	550	700	530	3000	C	11.56
1/54/4	5000	4000	700	600	750	570	4000	C	12.90
1/54/5	5000	4000	600	650	800	600	5000	C	14.23
1/55/3	5000	5000	1400	550	750	650	5000	C	13.40
1/55/4	5000	5000	1200	600	780	700	4000	C	14.86
1/55/5	5000	5000	1200	650	820	730	5000	D	16.04

BOX CELL DESIGNATION Nc/ab/Ec	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f (mm)	h (mm)	REQUIRED BEARING CAPACITY CATEGORY	DESIGN BASE PRESSURE (T/M^2)
1/63/3	6000	3000	300	650	750	580	5000	C	10.39
1/63/4	6000	5000	300	750	850	630	4000	C	12.06
1/63/5	6000	3000	300	820	900	670	5000	C	13.51
1/64/3	6000	4000	400	700	800	580	5000	C	11.47
1/64/4	6000	4000	300	750	850	630	4000	C	12.60
1/64/5	6000	4000	300	800	950	700	5000	C	14.40
1/65/3	6000	5000	1000	650	850	680	5000	C	13.05
1/65/4	6000	5000	900	650	900	720	4000	C	14.39
1/65/5	6000	5000	850	770	1000	750	5000	D	15.93
1/66/3	6000	6000	1500	650	900	850	3000	C	14.87
1/66/4	6000	6000	1400	700	1000	880	4000	D	16.29
1/66/5	6000	6000	1250	750	1050	950	5000	D	17.58
1/75/3	7000	5000	700	750	1000	700	3000	C	12.97
1/75/4	7000	5000	300	850	1050	770	4000	C	14.35
1/75/5	7000	5000	400	920	1150	800	5000	D	15.62
1/76/3	7000	6000	1300	750	1100	800	5000	C	14.86
1/76/4	7000	6000	1100	820	1100	880	4000	D	16.06
1/76/5	7000	8000	1100	870	1150	900	5000	D	17.46
1/77/3	7000	7000	1700	750	1100	1000	5000	D	16.58
[illegible]4	7000	7000	1600	820	1150	1050	4000	D	17.95
1/[illegible]5	7000	7000	1500	900	1200	1100	5000	D	19.37
1/85/3	6000	5000	300	870	1100	750	5000	C	12.83
1/85/4	8000	5000	00	950	1200	850	4000	C	14.18
1/8[illegible]	8000	5000	00	1100	1500	900	5000	D	16.00
1/86/3	8000	6000	800	850	1200	900	5000	C	14.55
1/86/4	8000	6000	700	850	1250	950	4000	D	15.99
1/86/5	8000	6000	600	1050	1300	1000	5000	D	17.44
1/87/3	6000	7000	1300	850	1200	1050	3000	D	16.19
1/87/4	8000	7000	1200	950	1350	1100	4000	D	17.90
[illegible]	8000	7000	1100	1050	1400	1150	5000	D	19.35

NOTES:

1. FOR GENERAL ARRANGEMENT, REFER TO PLATE 16 (SHEET 1 OF 2)
2. SAFE BEARING CAPACITY REQUIRED FOR SOIL IS DIVIDED IN FOUR CATEGORIES AS SHOWN BELOW

CATEGORY	SAFE BEARING CAPACITY REQUIRED FOR SOIL
A	$5\ T/M^2$
B	$10\ T/M^2$
C	$15\ T/M^2$
D	$20\ T/M^2$

3. ALL CASES WITH EARTH CUSION HEIGHTS OF 3m TO 5m ARE SAFE FOR DRAW DOWN CONDITION
4. FOR EARTH CUSION BETWEEN 3m AND 4m BOX WITH 4m EARTH CUSION IS TO BE USED AND FOR EARTH CUSHION BETWEEN 4m TO 5m, BOX WITH 5m EARTH CUSION IS TO BE USED. NO RECOMMENDATION IS GIVEN FOR BOX CELL STRUCTURE WITH EARTH CUSHION HEIGHT IN BETWEEN 0m TO 3m
5. IN CASE ALLOWABLE SOIL BEARING CAPACITY IS LESS THAN ACTUAL BEARING PRESSURE, THE SOIL SHOULD BE STABILISHED TO ACHIEVE SAME
6. FOR DIMENSIONAL PARAMETERS REFER SECTION (1-2 OF SHEET 1 OF 2)

DRAWINGS FOR BOX CELL STRUCTURES

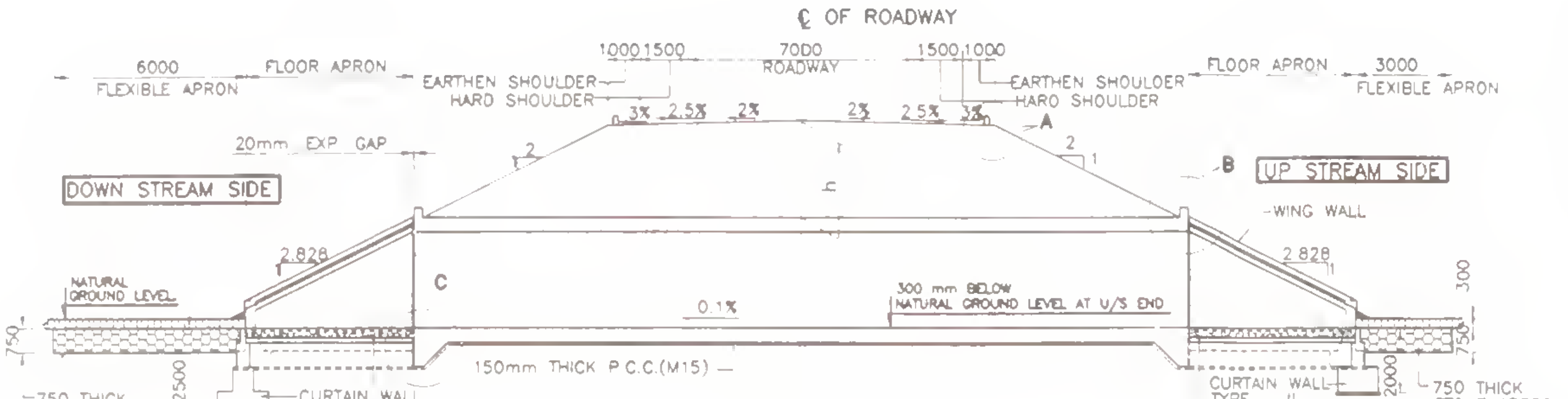
SINGLE CELL R.C.C. BOX STRUCTURES

2m x 2m TO 8m x 7m

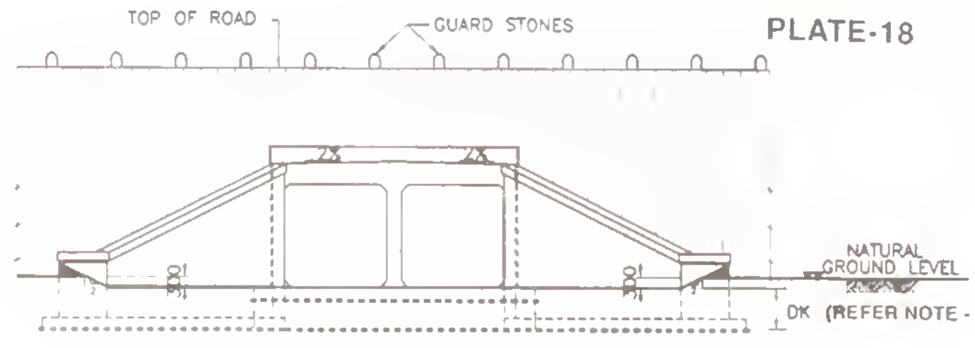
(WITH EARTH CUSION)

GENERAL ARRANGEMENT (SHEET 2)

PLATE-18

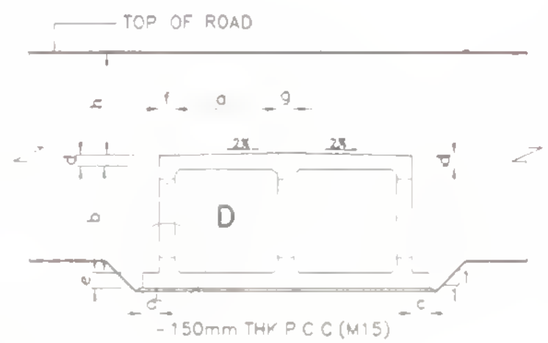


SECTION 1-1

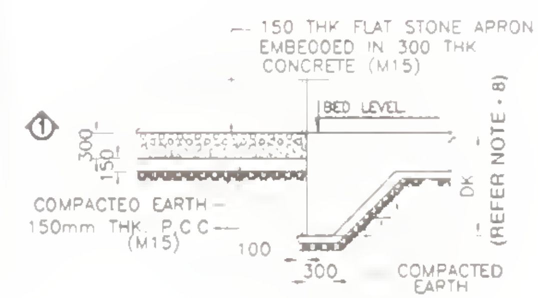


VIEW - 3-3

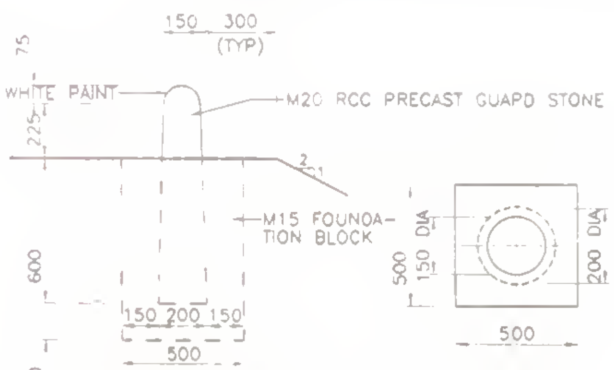
PLAN AT ROAD LEVEL



SECTION 2-2



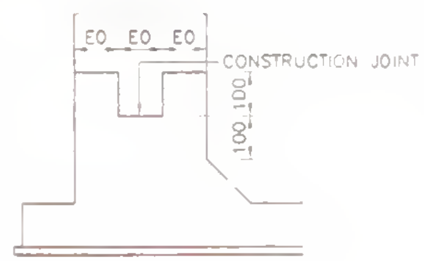
DETAIL - C



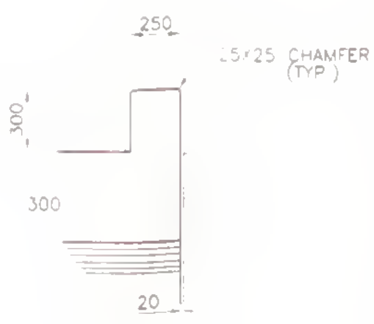
DETAIL - A

PLAN

DETAIL OF PRECAST GUARD



DETAIL - D



DETAIL - B

TABLE SHOWING SALIENT DIMENSIONS

BOX CELL DESIGNATION Nc/ab/Ec	a (mm)	b (mm)	c (mm)	d (mm)	e (mm)	f (mm)	g (mm)	h (mm)	CATEGORY OF BEARING CAPACITY (T/m2)	DESIGN BASE PRESSURE (T/m2)
2/22/3	2000	2000	300	270	300	250	250	3000	B	7.70
2/22/4	2000	2000	300	300	350	250	250	4000	B	9.00
2/22/5	2000	2000	300	320	350	270	250	5000	C	10.16
2/23/3	2000	3000	300	300	350	350	250	3000	B	[illegible]
2/23/4	2000	3000	300	320	350	350	250	4000	C	10.41
2/23/5	2000	3000	400	350	350	380	250	5000	C	11.54
2/32/3	3000	2000	300	400	420	300	250	3000	B	7.97
2/32/4	3000	2000	300	450	470	300	250	4000	B	9.30
2/32/5	3000	2000	300	500	500	300	250	5000	C	10.46
2/33/3	3000	3000	300	400	420	350	250	3000	B	8.52
2/33/4	3000	3000	300	450	450	380	250	4000	B	9.80
2/33/5	3000	3000	300	480	480	400	250	5000	C	11.01

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONED. ONLY WRITTEN DIMENSIONS ARE TO BE FOLLOWED.
2. SOIL DENSITY ON THE BACKFILL AND EARTH CUSHION SHOULD CONFORM TO CLAUSE 305.2.1.5 OF MORT&H SPECIFICATION FOR ROAD & BRIDGE WORKS (FOURTH REVISION-2001).
3. NET BEARING CAPACITY REQUIRED FOR SOIL IS DIVIDED IN FOUR CATEGORIES AS SHOWN BELOW:

CATEGORY		NET BEARING CAPACITY REQUIRED FOR SOIL
A	-	5 T/M²
B		10 T/M²
C		15 T/M²
D	=	20 T/M²

4. SOFT AND LOOSE PATCHES IN THE BEARING AREA ARE TO BE REPLACED BY COMPACTED GRANULAR FILLS WITH LAYERS NOT EXCEEDING 300mm
5. DESIGNS ARE GIVEN FOR THE BOX CELL STRUCTURES ONLY THESE HAVE NO BEARING WITH DESIGN OF EMBANKMENT WHICH WILL BE TAKEN UP BY ENGINEER-IN-CHARGE SEPARATELY
6. ALL CASES WITH EARTH CUSION OF 3m TO 5m HEIGHT ARE SAFE FOR SUDDEN DRAW DOWN CONDITION
7. FOR EARTH CUSHION BETWEEN 3m AND 4m, BOX WITH 4m EARTH CUSHION IS TO BE USED AND FOR EARTH CUSHION BETWEEN 4m TO 5m BOX WITH 5m EARTH CUSHION IS TO BE USED. NO RECOMMENDATION IS GIVEN FOR BOX CELL STRUCTURE WITH EARTH CUSHION BELOW 3m HEIGHT
8. DK IS DEPTH OF KEY AT BASE SLAB

FOR BASE SLAB THICKNESS	VALUE OF DK
UPTO 900mm	1200mm
GREATER THAN 900mm	e+300mm

e = BASE SLAB THICKNESS

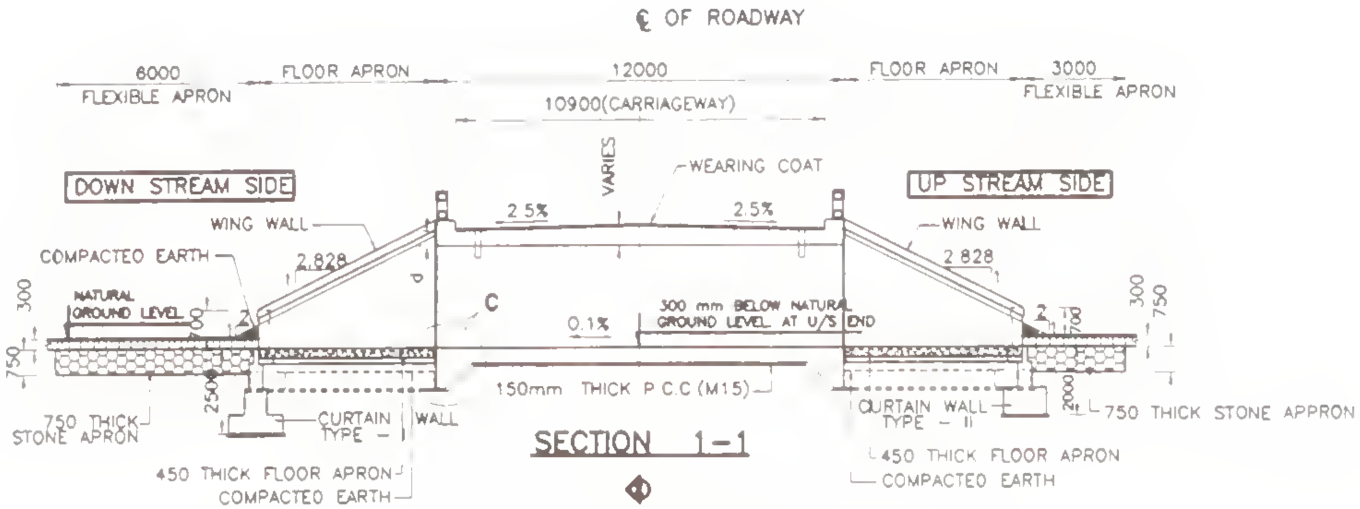
DRAWINGS FOR BOX CELL STRUCTURES

DOUBLE CELL R.C.C. BOX STRUCTURES

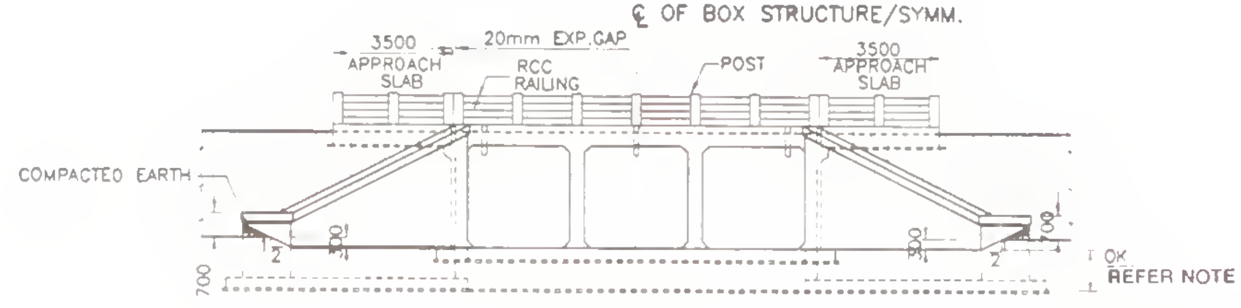
2m x 2m TO 3m x 3m

(WITH EARTH CUSION)

GENERAL ARRANGEMENT



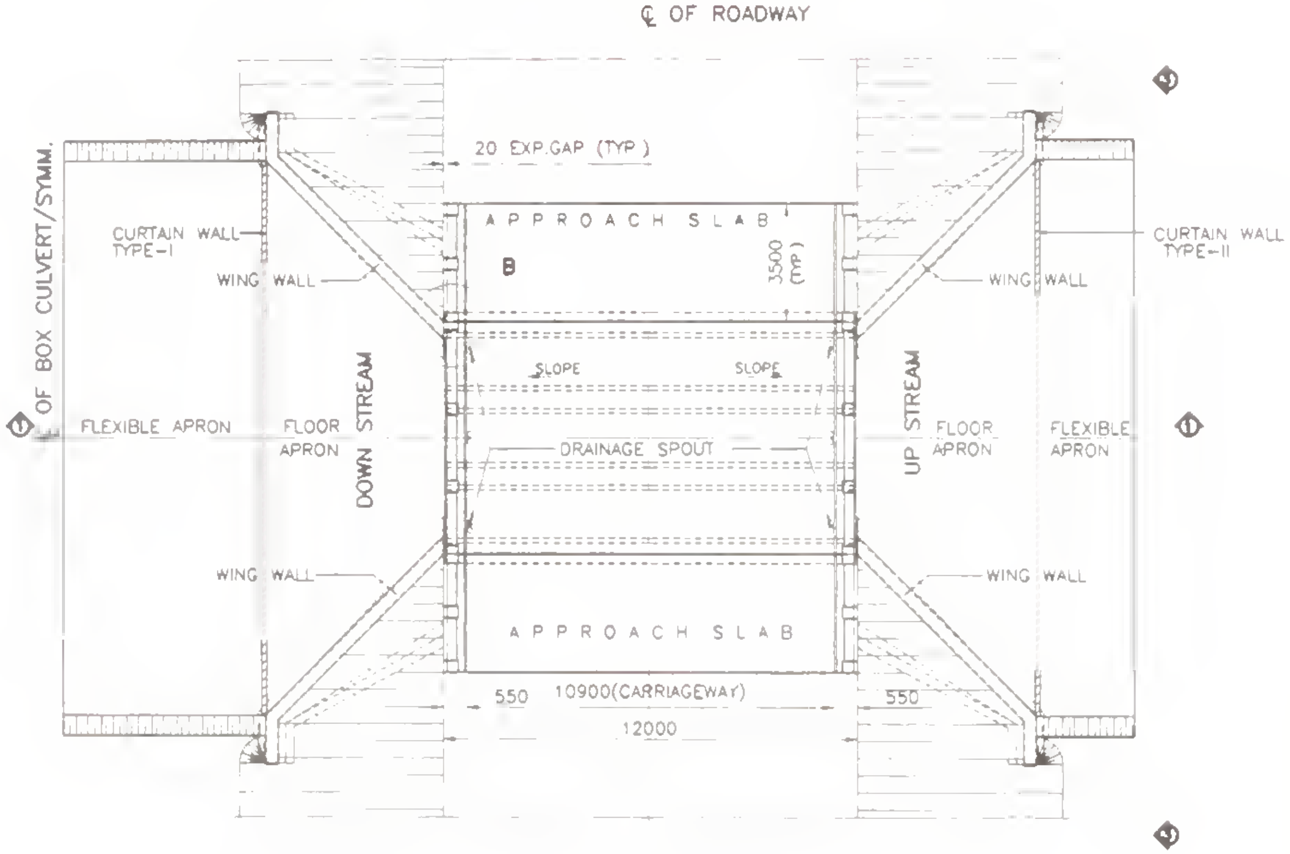
SECTION 1-1



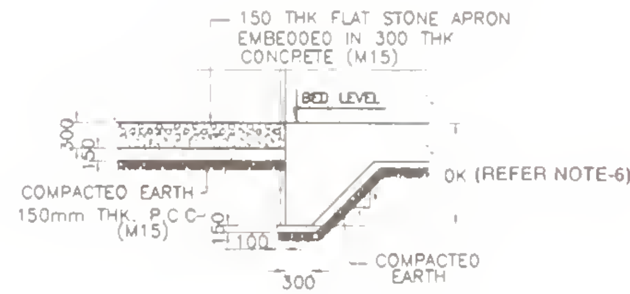
VIEW 3-3

TABLE SHOWING SALIENT DIMENSIONS

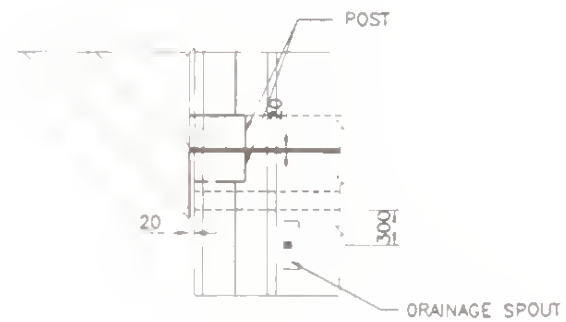
BOX CELL DESIGNATION Nc/ob/Ec	a (mm)	b (mm)	d (mm)	e (mm)	f (mm)	g (mm)	REQUIRED BEARING CAPACITY CATEGORY	DESIGN BASE PRESSURE (T/M²)	ALLOWABLE DRAWDOWN AT A TIME (M)
3/22/0	2000	2000	370	370	370	300	B	7.50	0.85
3/33/0	3000	3000	450	500	470	400	B	7.18	2.18



PLAN



DETAIL - C



DETAIL - B

NOTES:

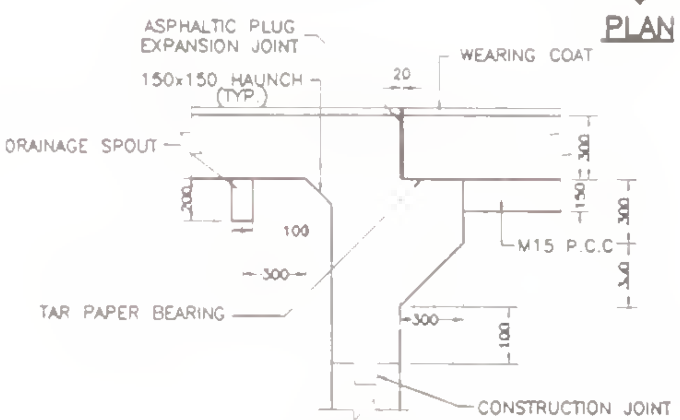
1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONED ONLY WRITTEN DIMENSIONS ARE TO BE FOLLOWED
2. SOIL DENSITY ON THE BACKFILL AND EARTH CUSHION SHOULD CONFORM TO CLAUSE 305 2 1 5 OF MORT&H SPECIFICATION FOR ROAD & BRIDGE WORKS (FOURTH REVISION-2001)
3. NET BEARING CAPACITY REQUIRED FOR SOIL IS DIVIDED IN FOUR CATEGORIES AS SHOWN BELOW

CATEGORY	NET BEARING CAPACITY REQUIRED FOR SOIL
A	5 T/M²
B	10 T/M²
C	15 T/M²
D	20 T/M²

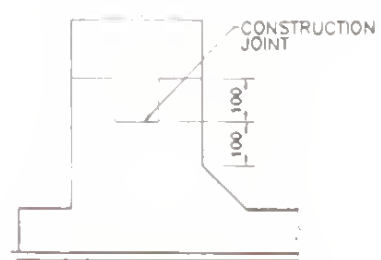
4. SOFT AND LOOSE PATCHES IN THE BEARING AREA ARE TO BE REPLACED BY COMPACTED GRANULAR FILLS WITH LAYERS NOT EXCEEDING 300mm
5. DESIGNS ARE GIVEN FOR THE BOX CELL STRUCTURES ONLY THESE HAVE NO BEARING WITH DESIGN OF EMBANKMENT WHICH WILL BE TAKEN UP BY ENGINEER-IN-CHARGE SEPARATELY
6. 'DK' IS DEPTH OF KEY AT BASE SLAB

FOR BASE SLAB THICKNESS	VALUE OF 'DK'
UPTO 900mm	1200mm
GREATER THAN 900mm	e+300mm

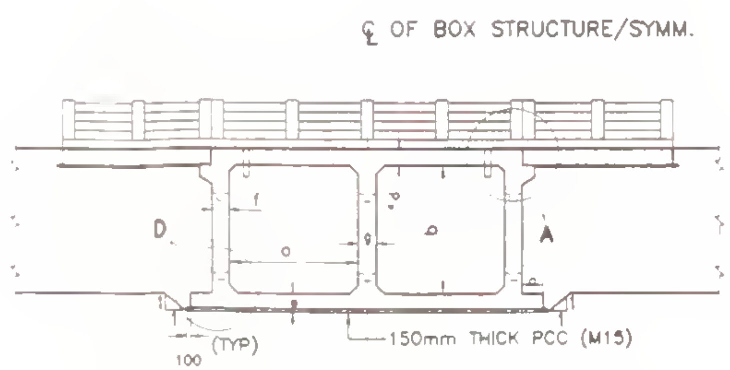
e = BASE SLAB THICKNESS



DETAIL - A



DETAIL D



SECTION 2-2

• d DECK SLAB THICKNESS AT INNER EDGE OF KERB AS SHOWN IN SECTION 1-1

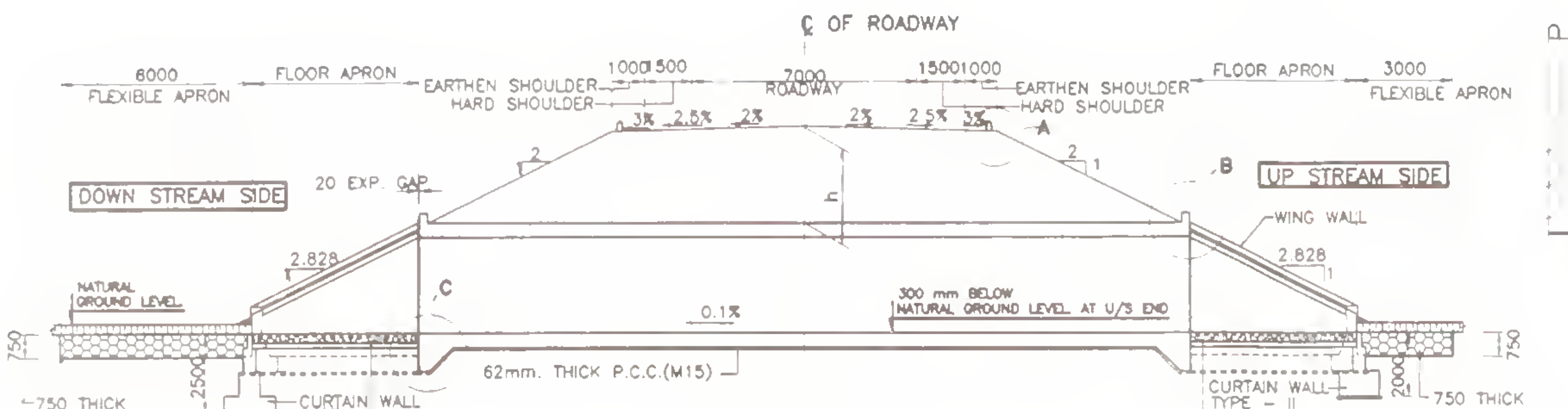
DRAWINGS FOR BOX CELL STRUCTURES

TRIPLE CELL R.C.C. BOX STRUCTURES

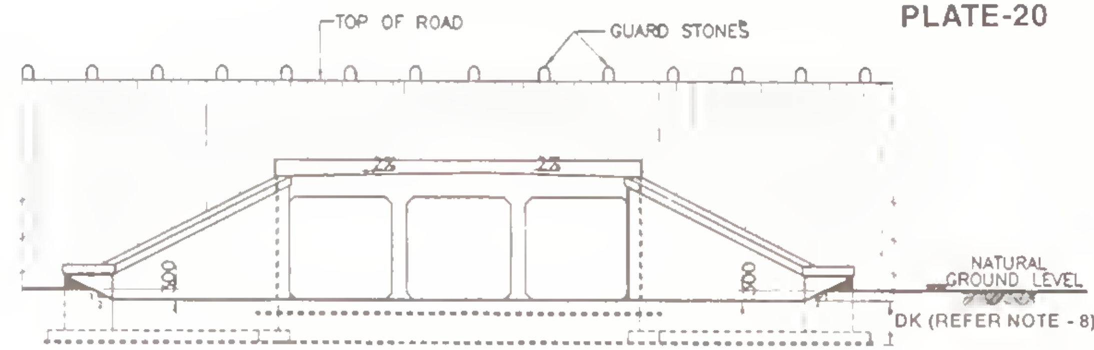
2m x 2m TO 3m x 3m

(WITHOUT EARTH CUSION)

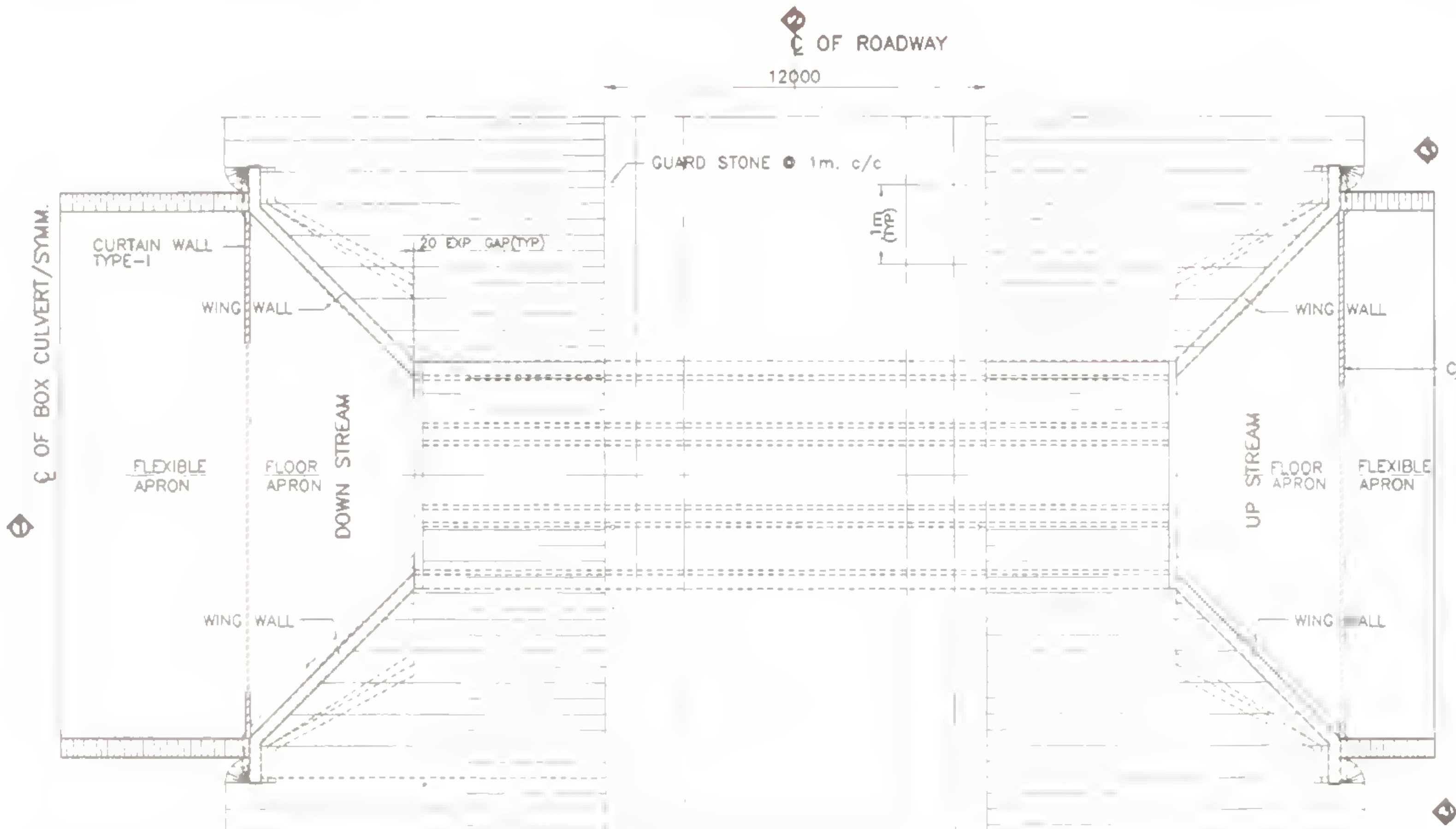
GENERAL ARRANGEMENT



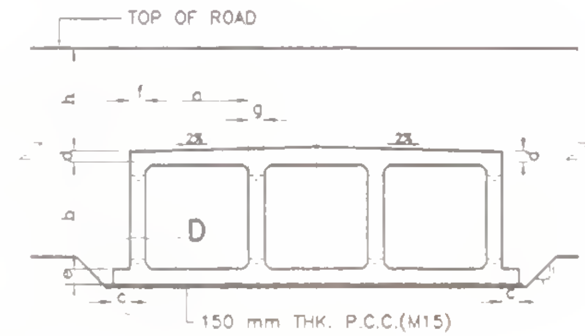
SECTION 1-1



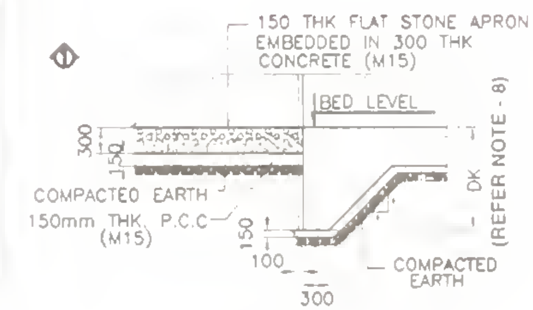
VIEW 3-3



PLAN AT ROAD LEVEL



SECTION 2-2



DETAIL - C

NOTES:

1 ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONED ONLY WRITTEN DIMENSIONS ARE TO BE FOLLOWED.

2 SOIL DENSITY ON THE BACKFILL AND EARTH CUSHION SHOULD CONFORM TO CLAUSE 305 2 1 5 OF MORT&H SPECIFICATION FOR ROAD & BRIDGE WORKS (FOURTH REVISION-2001).

3 NET BEARING CAPACITY REQUIRED FOR SOIL IS DIVIDED IN FOUR CATEGORIES AS SHOWN BELOW

CATEGORY	NET BEARING CAPACITY REQUIRED FOR SOIL
A	5 T/M²
B	10 T/M²
C	15 T/M²
D	20 T/M²

4 SOFT AND LOOSE PATCHES IN THE BEARING AREA ARE TO BE REPLACED BY COMPACTED GRANULAR FILLS WITH LAYERS NOT EXCEEDING 300mm

5 DESIGNS ARE GIVEN FOR THE BOX CELL STRUCTURES ONLY THESE HAVE NO BEARING WITH DESIGN OF EMBANKMENT WHICH WILL BE TAKEN UP BY ENGINEER-IN-CHARGE SEPARATELY

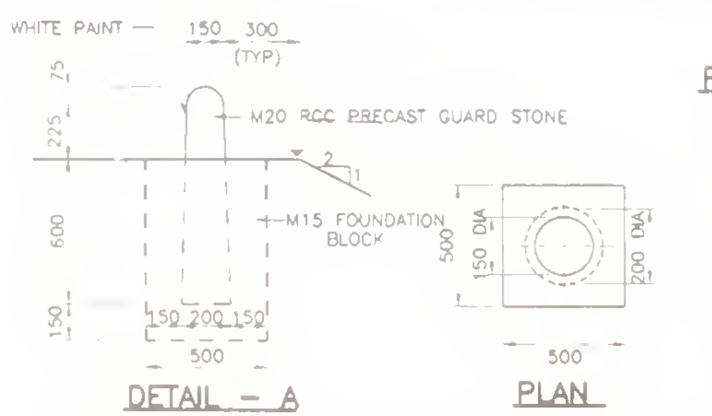
6 ALL CASES WITH EARTH CUSION OF 3m TO 5m HEIGHT ARE SAFE FOR SUDDEN DRAW DOWN CONDITION

7 FOR EARTH CUSHION BETWEEN 3m AND 4m, BOX WITH 4m EARTH CUSHION IS TO BE USED AND FOR EARTH CUSHION BETWEEN 4m TO 5m, BOX WITH 5m EARTH CUSHION IS TO BE USED NO RECOMMENDATION IS GIVEN FOR BOX CELL STRUCTURE WITH EARTH CUSHION BELOW 3m HEIGHT

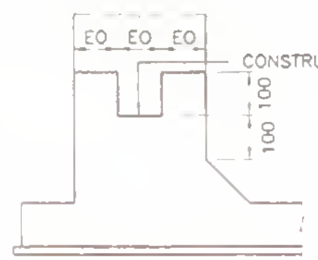
8 DK IS DEPTH OF KEY AT BASE SLAB

FOR BASE SLAB THICKNESS	VALUE OF DK
UPTO 900mm	1200mm
GREATER THAN 900mm	e+300mm

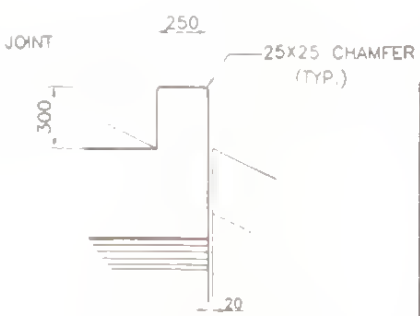
e = BASE SLAB THICKNESS



DETAIL OF PRECAST GUARD STONE



DETAIL - D



DETAIL - B

TABLE SHOWING SALIENT DIMENSIONS

BOX CELL DESIGNATION Nc/ab/Ec	a (mm)	b (mm)	d (mm)	e (mm)	f (mm)	g (mm)	h (mm)	CATEGORY OF BEARING CAPACITY (T/m2)	DESIGN BASE PRESSURE (T/m2)
3/22/3	2000	2000	270	300	250	250	3000	B	7 43
3/22/4	2000	2000	300	320	300	250	4000	B	8.68
3/22/5	2000	2000	300	350	280	250	5000	B	9 82
3/33/3	3000	3000	350	400	350	250	3000	B	8 06
3/33/4	3000	3000	400	450	400	250	4000	B	9 40
3/33/5	3000	3000	450	480	420	250	5000	C	10.70

DRAWINGS FOR BOX CELL STRUCTURES

TRIPLE CELL R.C.C. BOX STRUCTURES

2m x 2m TO 3m x 3m

(WITH EARTH CUSION)

GENERAL ARRANGEMENT

Without Earth Cushion			With Earth Cushion			With Earth Cushion			With Earth Cushion		
Box Cell Designation Nc/ab/Ec	Total Concrete (m^3)	Total Steel (kg)	Box Cell Designation Nc/ab/Ec	Total Concrete (m^3)	Total Steel (kg)	Box Cell Designation Nc/ab/Ec	Total Concrete (m^3)	Total Steel (kg)	Box Cell Designation Nc/ab/Ec	Total Concrete (m^3)	Total Steel (kg)
1/22/0	54	3458	1/22/3	72.69	4682.5	1/65/4	591.80	34436.1	1/76/5	1011.87	62177.7
1/23/0	77	5642	1/23/3	112.57	6451.1	1/66/4	755.76	42767.3	1/77/5	1243.88	72126.2
1/33/0	94	5688	1/33/3	158.18	8340.4	1/75/4	713.34	43425.1	1/85/5	1084.93	66569.3
1/34/0	124	7541	1/34/3	213.25	11091.3	1/76/4	849.08	48812.5	1/86/5	1205.93	73861.2
1/43/0	114	7120	1/43/3	201.10	11385.7	1/77/4	1032.22	60820.8	1/87/5	1442.27	82807.1
1/44/0	149	9164	1/44/3	274.86	14146.1	1/85/4	859.18	53000	2/22/3	117.86	6458.3
1/45/0	194	10607	1/45/3	340.39	18338.0	1/86/4	995.71	57521.4	2/22/4	147.06	7898.6
1/53/0	142	8941	1/53/3	264.07	15156.0	1/87/4	1197.34	64698	2/22/5	173.62	9100.8
1/54/0	174	10695	1/54/3	320.36	18582.1	1/22/5	110.86	6732.9	2/23/3	166.85	7991.9
1/55/0	224	14339	1/55/3	413.16	22355.3	1/23/5	165.70	9182.1	2/23/4	196.14	9338.9
1/63/0	201	11238	1/63/3	346.13	21164.2	1/33/5	226.56	12537.5	2/23/5	232.59	11187.9
1/64/0	222	13848	1/64/3	395.12	22527.6	1/34/5	317.71	16131.9	2/32/3	198.08	9677.8
1/65/0	258	14479	1/65/3	479.72	26581.4	1/43/5	316.30	17777.7	2/32/4	248.85	12507.2
1/66/0	315	18100	1/66/3	306.97	34677.4	1/44/5	380.33	22494.1	2/32/5	300.92	15728.5
1/75/0	301	17607	1/75/3	566.63	34498.9	1/45/5	517.9	26751.5	2/33/3	227.87	11404.9
1/76/0	365	20915	1/76/3	693.65	35823.5	1/53/5	412.08	25685.6	2/33/4	282.77	14818.6
1/77/0	394	22120	1/77/3	839.05	47098.2	1/54/5	482.70	29935.6	2/33/5	345.20	17142.8
1/85/0	350	20650	1/85/3	659.64	40603.8	1/55/5	611.85	35055.0	3/22/3	163.05	9226.9
1/86/0	478	23539	1/86/3	802.86	45004.9	1/63/5	562.31	34418.8	3/22/4	205.64	11267.6
1/87/0	502	27371	1/87/3	940.83	54298.4	1/64/5	624.30	37597.7	3/22/5	236.37	13486.7
2/22/0	91	5650	1/22/4	93.10	5557.8	1/65/5	732.06	43135.5	3/33/3	298.06	16341.3
2/23/0	108	7377	1/23/4	150.22	7130.8	1/66/5	917.77	53263.2	3/33/4	384.25	19704.3
2/32/0	125	8103	1/33/4	190.43	10302.3	1/75/5	870.79	54626.4	3/33/5	468.42	25104.6
2/33/0	144	9895	1/34/4	270.65	12978.8						
3/22/0	116	9243	1/43/4	248.97	14472.6						
3/33/0	207	16027	1/44/4	324.07	17445.2						
			1/45/4	421.49	21973.4						
			1/53/4	332.67	19818.1						
			1/54/4	399.46	24235.0						
			1/55/4	506.50	28928.8						
			1/63/4	456.87	27382.8						
			1/64/4	492.08	30403.8						

Notes:

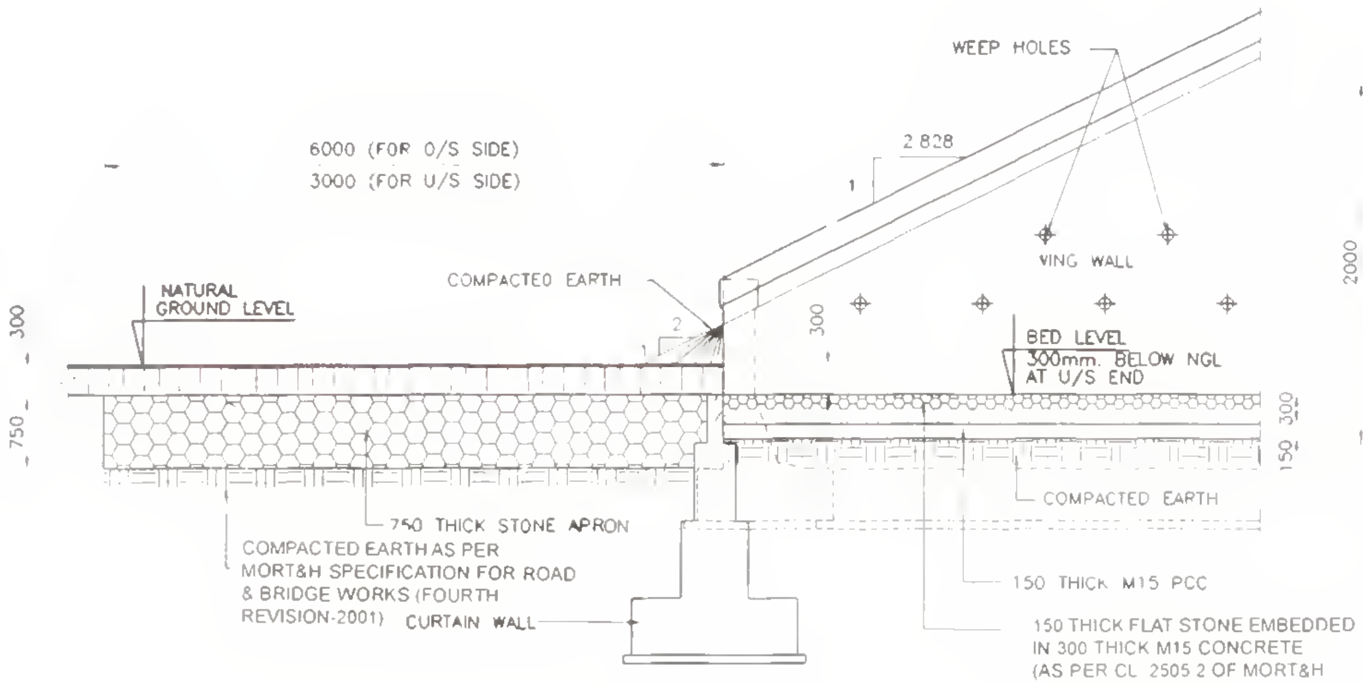
1. Quantity of concrete does not include Guard Stone/Railings
2. Quantity of steel does not include 5 per cent extra for wastage and lap

DRAWINGS FOR BOX CELL STRUCTURE

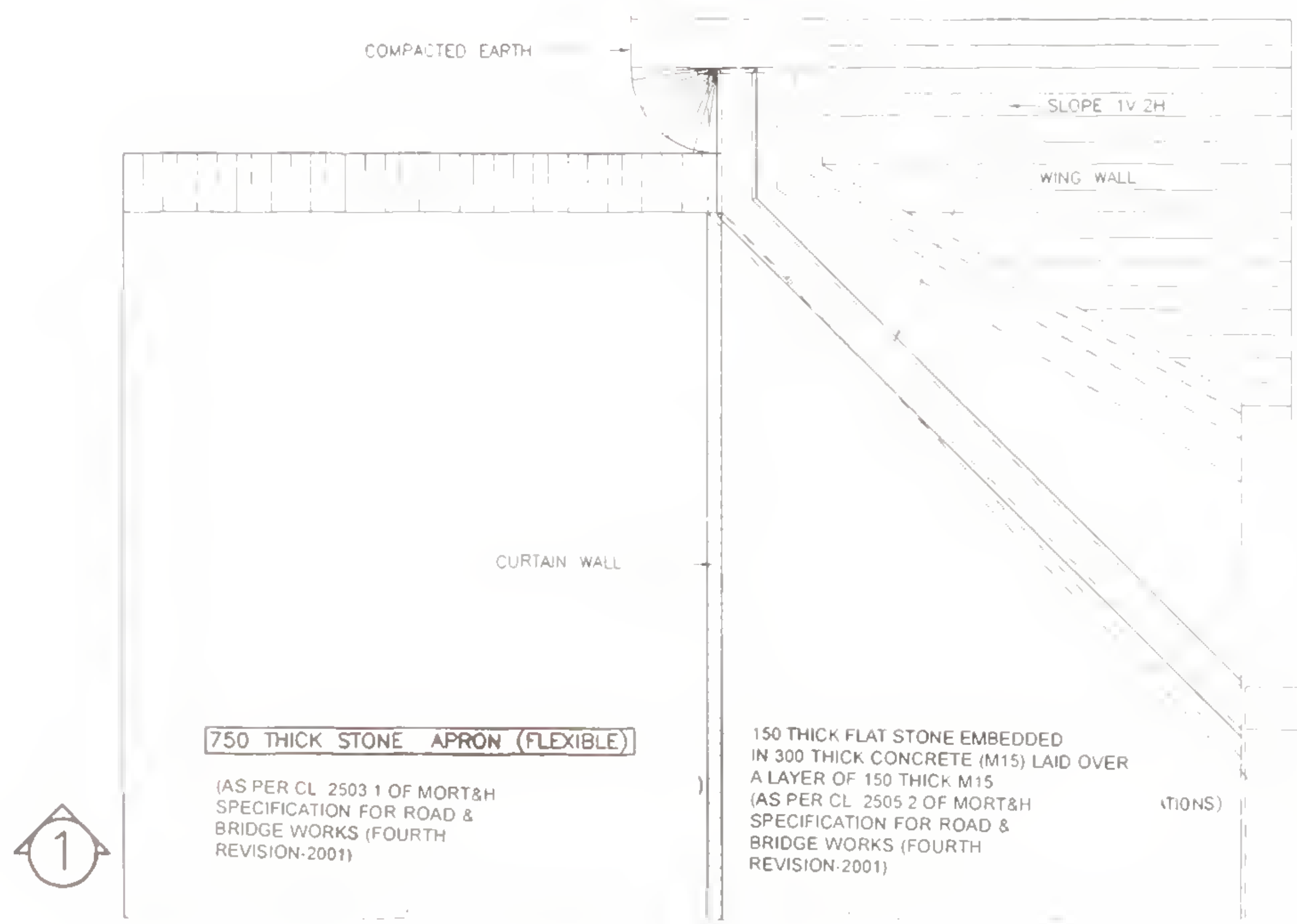
SINGLE DOUBLE AND TRIPLE CELL R.C.C. BOX STRUCTURES

(WITH AND WITHOUT EARTH CUSHION)

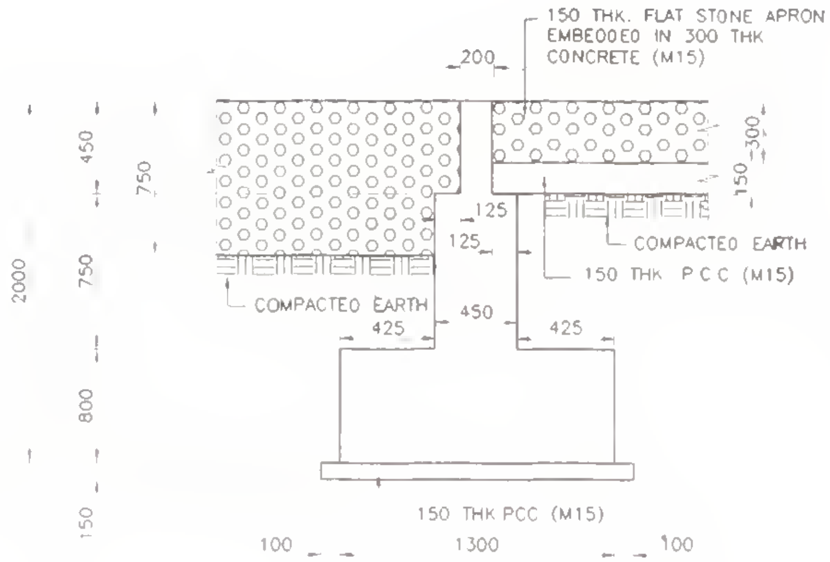
QUANTITIES OF STEEL AND CONCRETE



SECTION 1-1



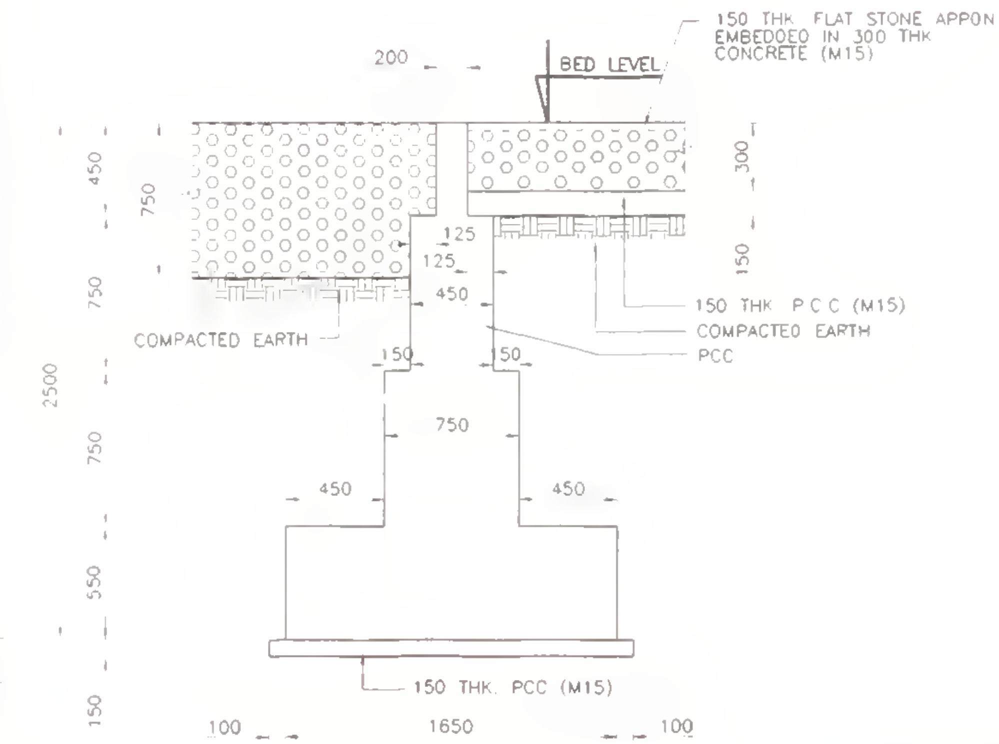
PLAN



TYPICAL CROSS SECTION OF CURTAIN WALL TYPE-II
(UP STREAM SIDE)

NOTES:

1. ALL DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE MENTIONED ONLY WRITTEN DIMENSIONS ARE TO BE FOLLOWED
2. COMPACTED EARTH SHOULD CONFORM TO CLAUSE, 30S 2 1 S OF MOST SPECIFICATIONS

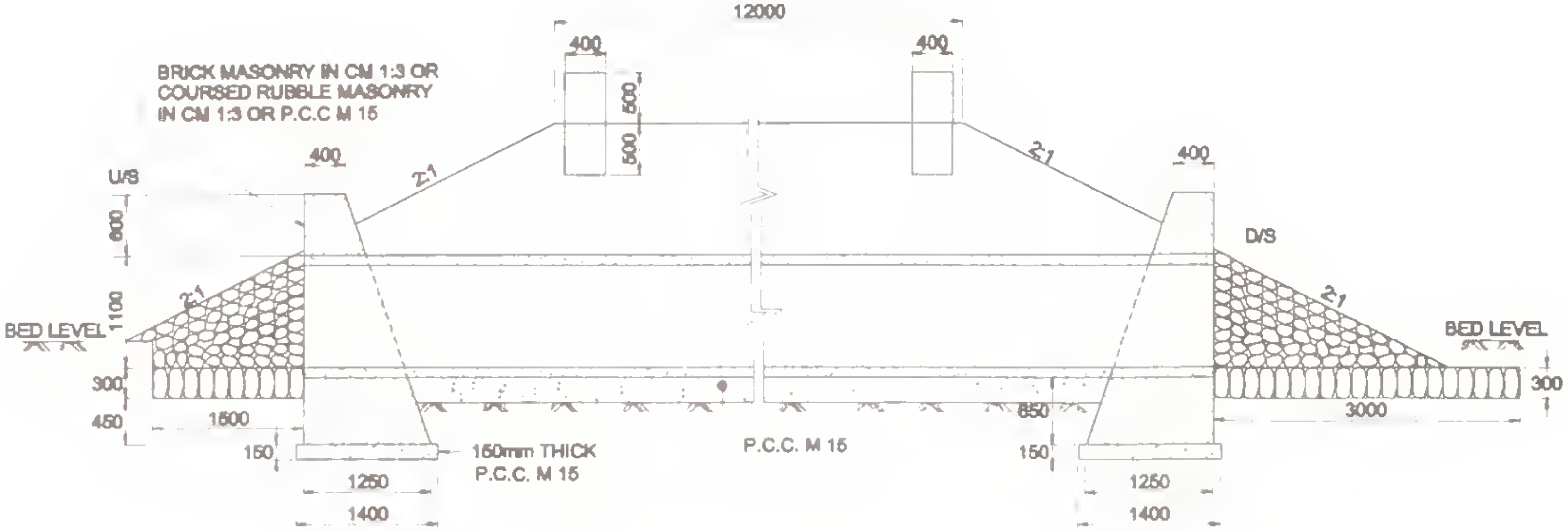


TYPICAL CROSS SECTION OF CURTAIN WALL TYPE-I
(DOWN STREAM SIDE)

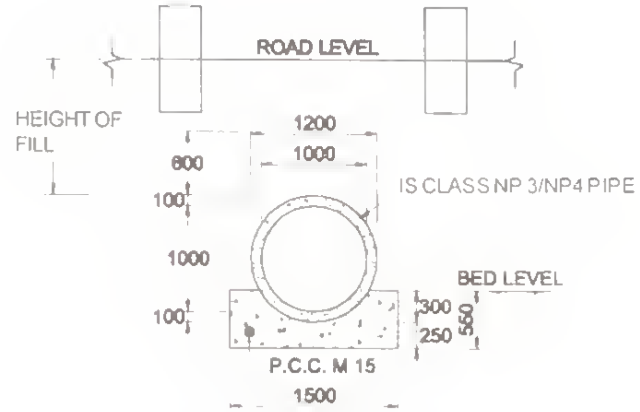
DRAWINGS FOR BOX CELL STRUCTURES

TYPICAL DETAILS OF FLOOR PROTECTION WORKS

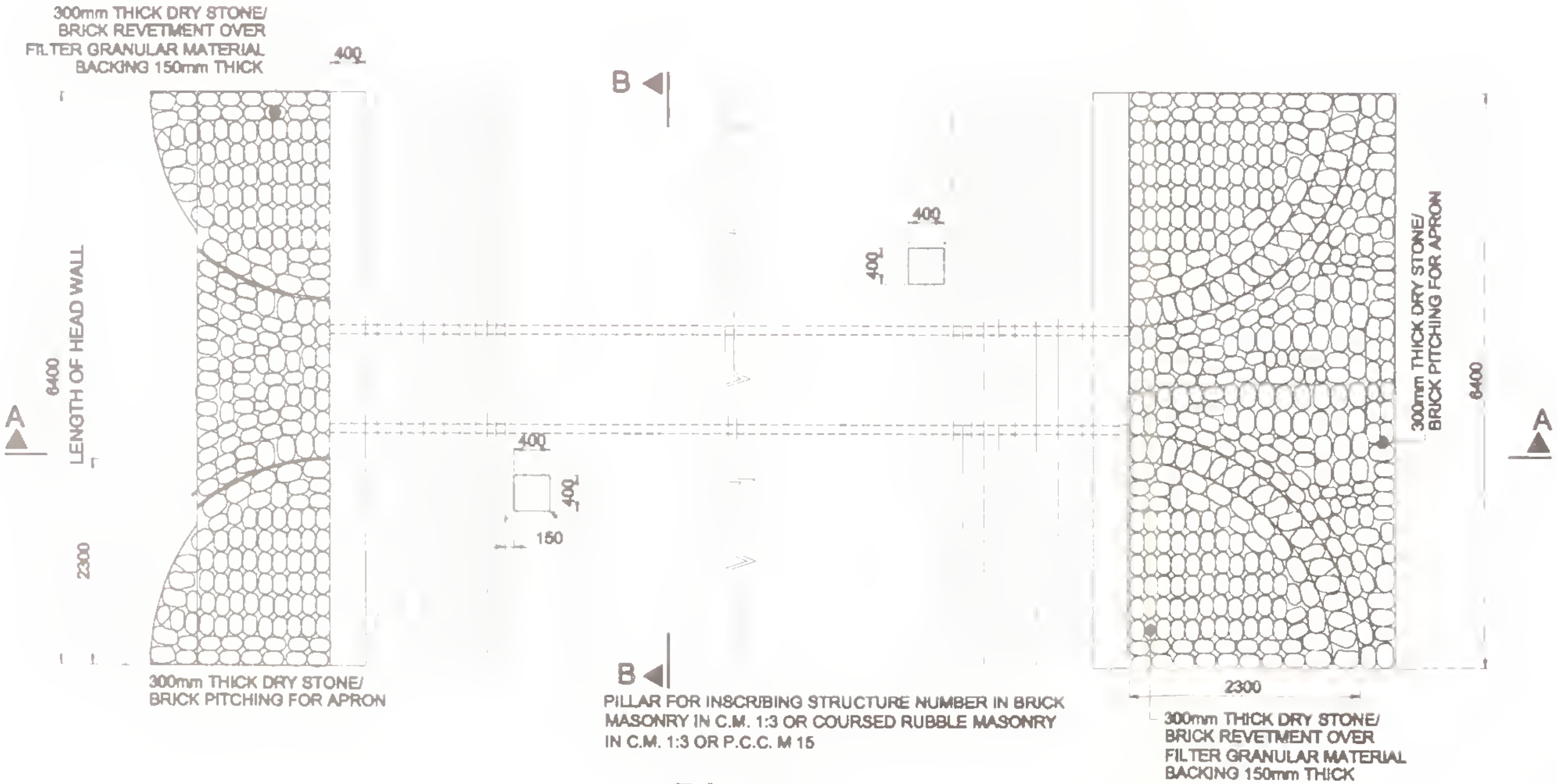
GENERAL ARRANGEMENT



SECTION AT 'AA'



SECTION AT 'BB'

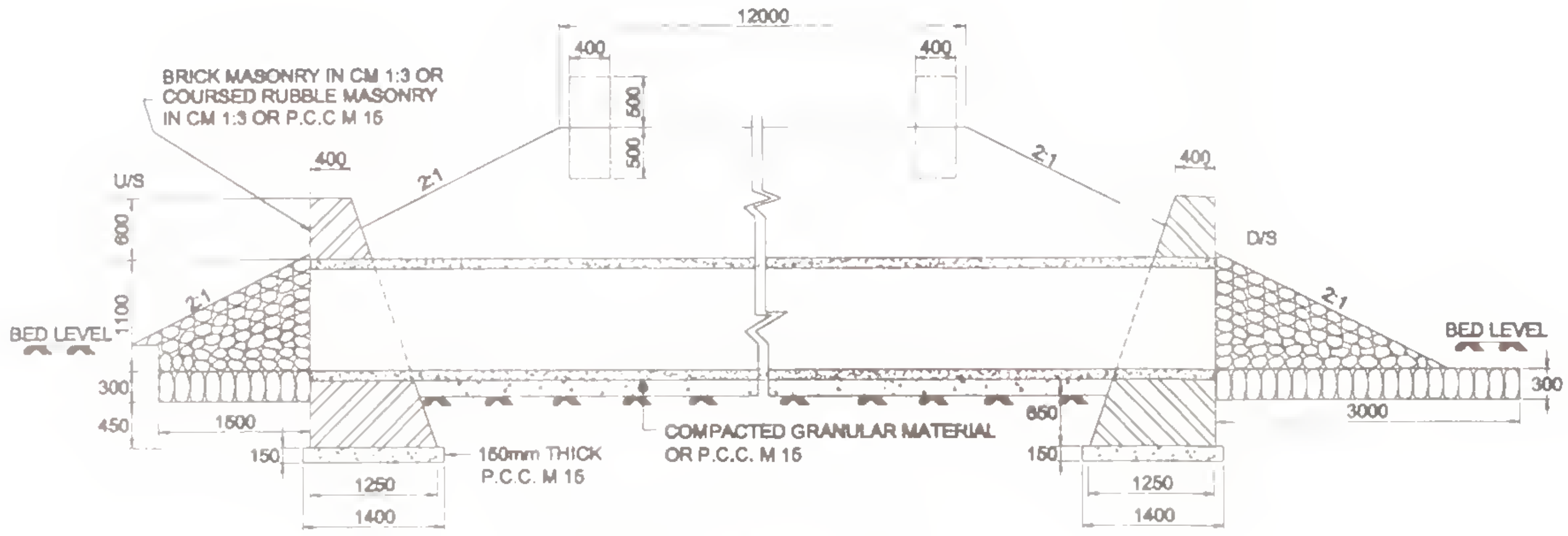


PLAN

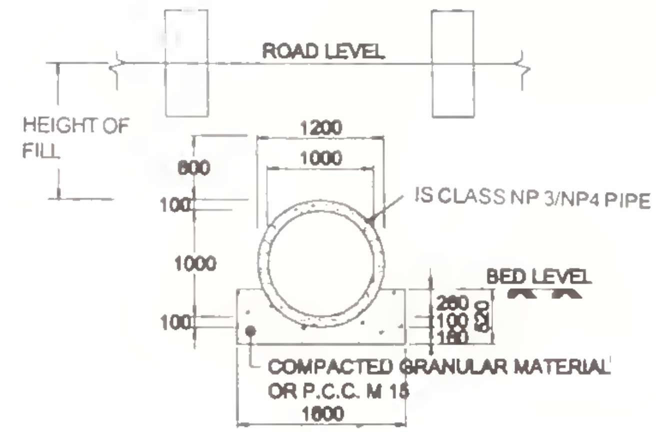
NOTES :-

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

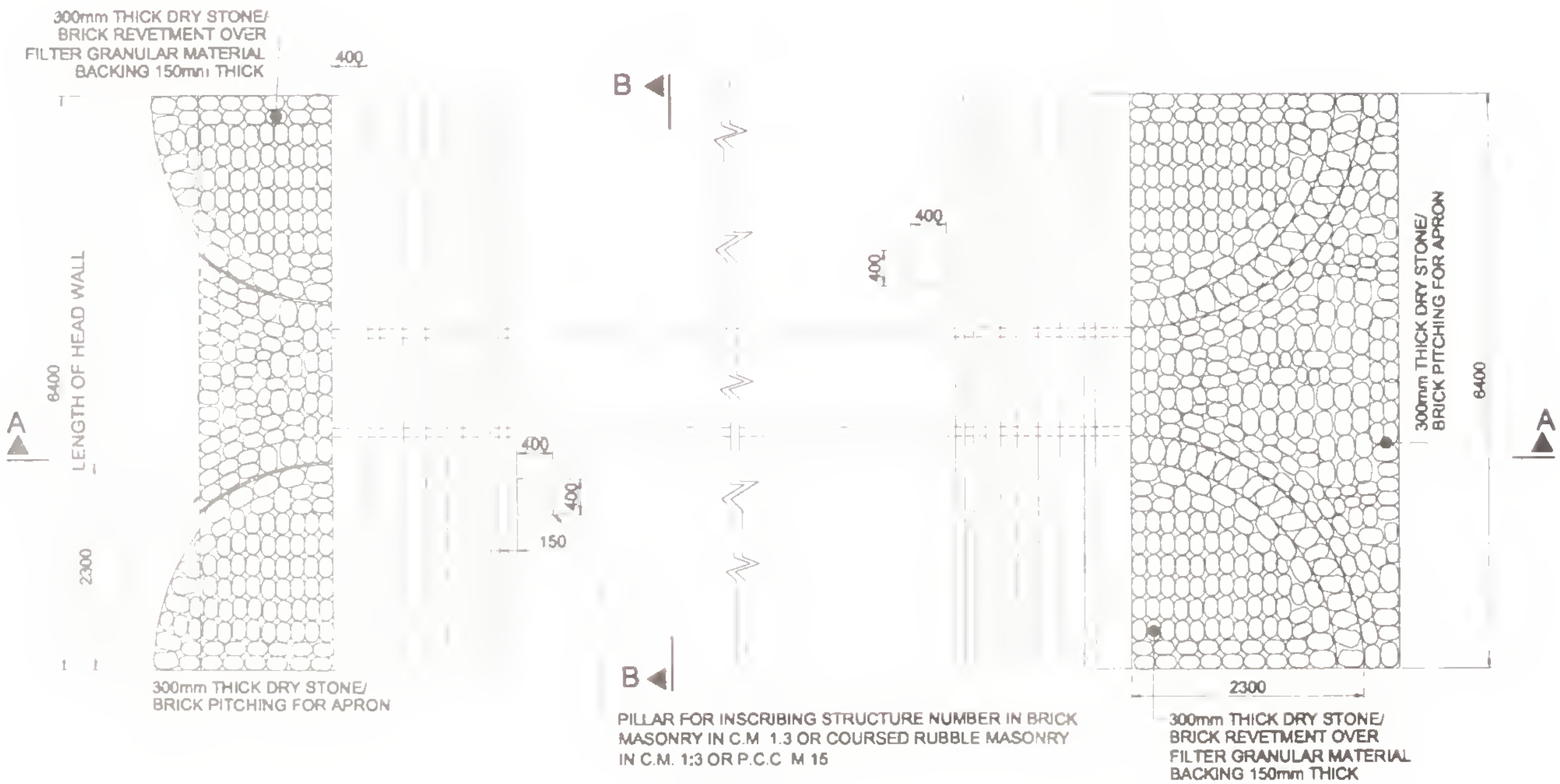
R.C.C. PIPE CULVERT WITH SINGLE PIPE OF 1 METRE DIA AND CONCRETE CRADLE BEDDING FOR HEIGHTS OF FILL VARYING FROM 4.0 m - 8.0 m



SECTION AT 'AA'



SECTION AT 'BB'

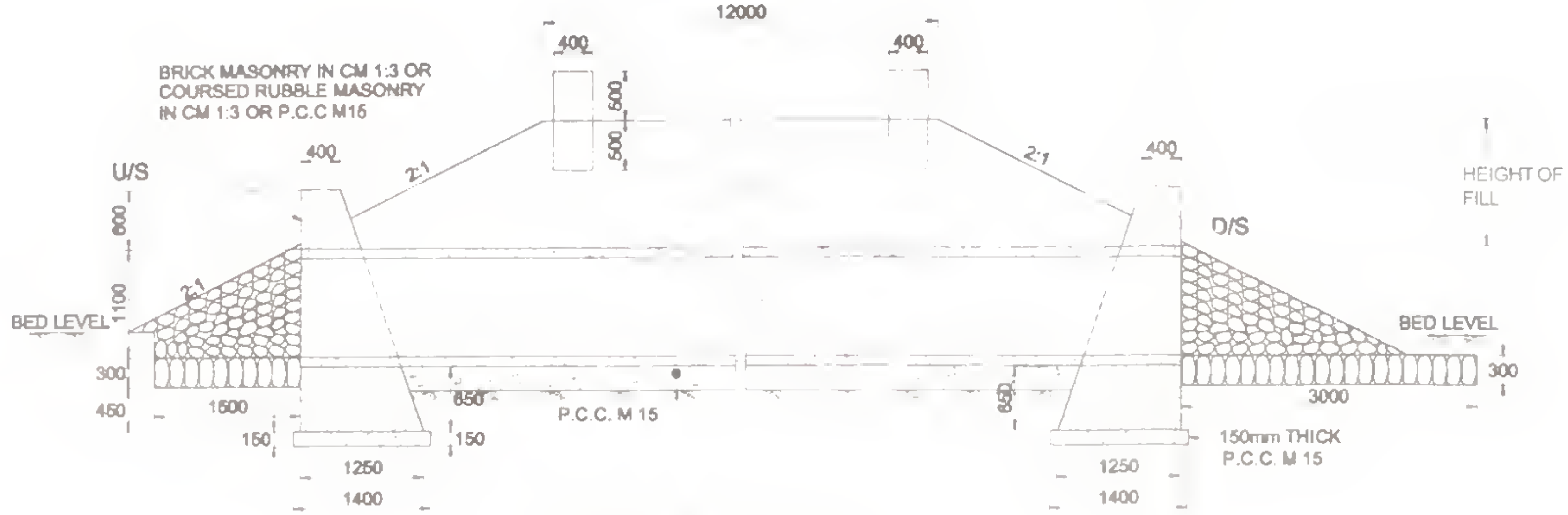


PLAN

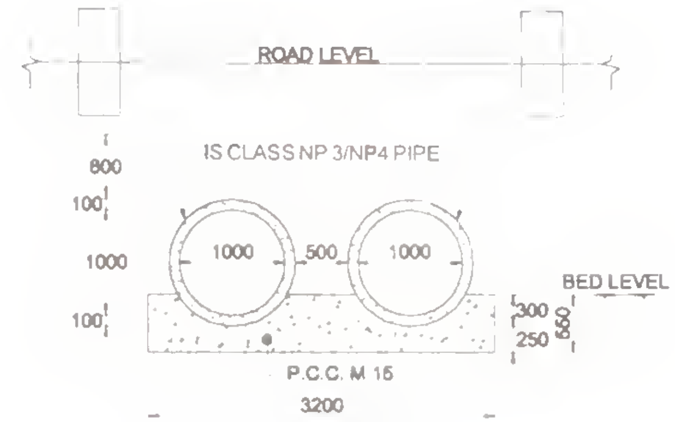
NOTES :-

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

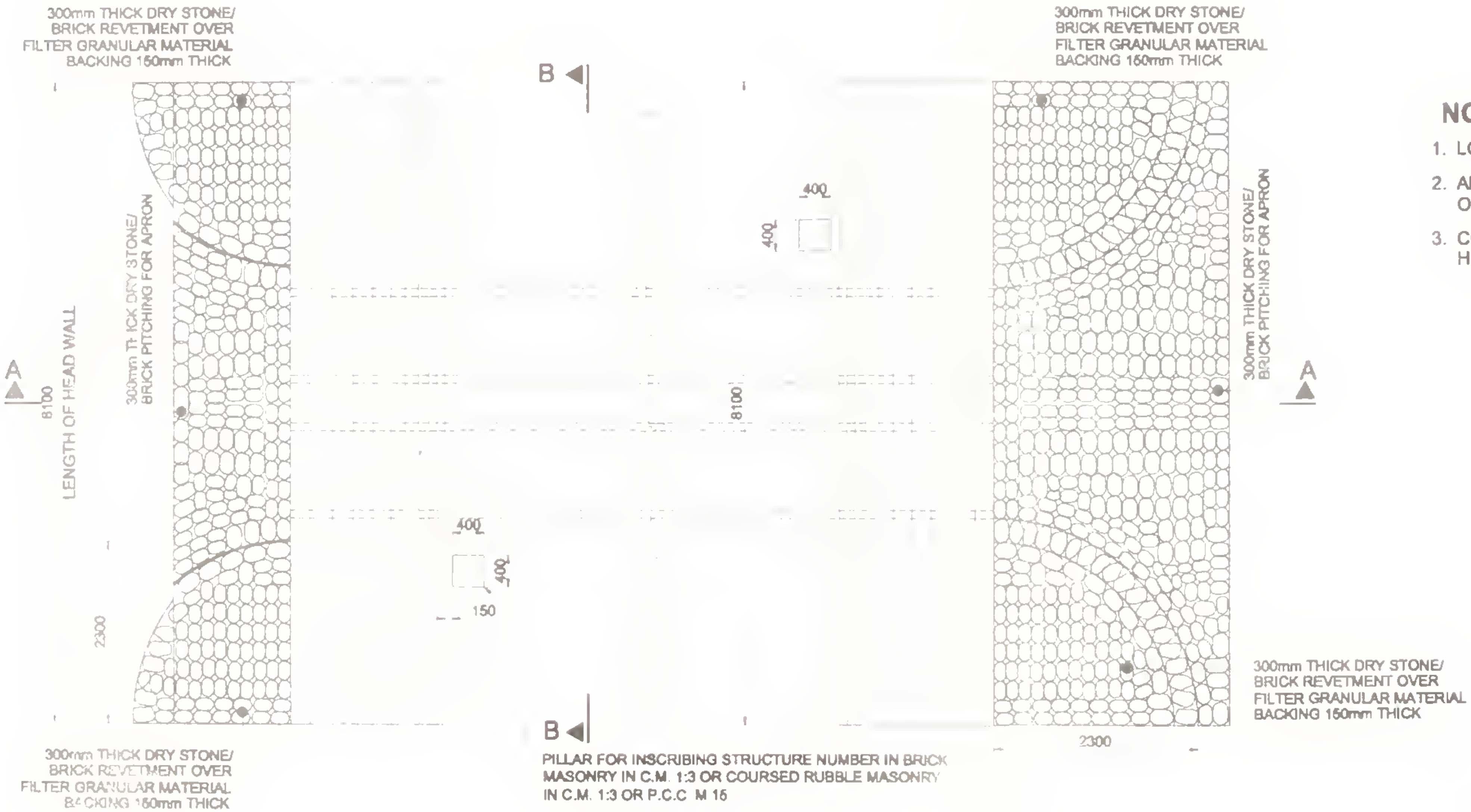
R.C.C. PIPE CULVERT WITH SINGLE PIPE OF 1 METRE DIA AND FIRST CLASS BEDDING FOR HEIGHTS OF FILL VARYING FROM 0.6 m-4.0 m



SECTION AT 'AA'



SECTION AT 'BB'

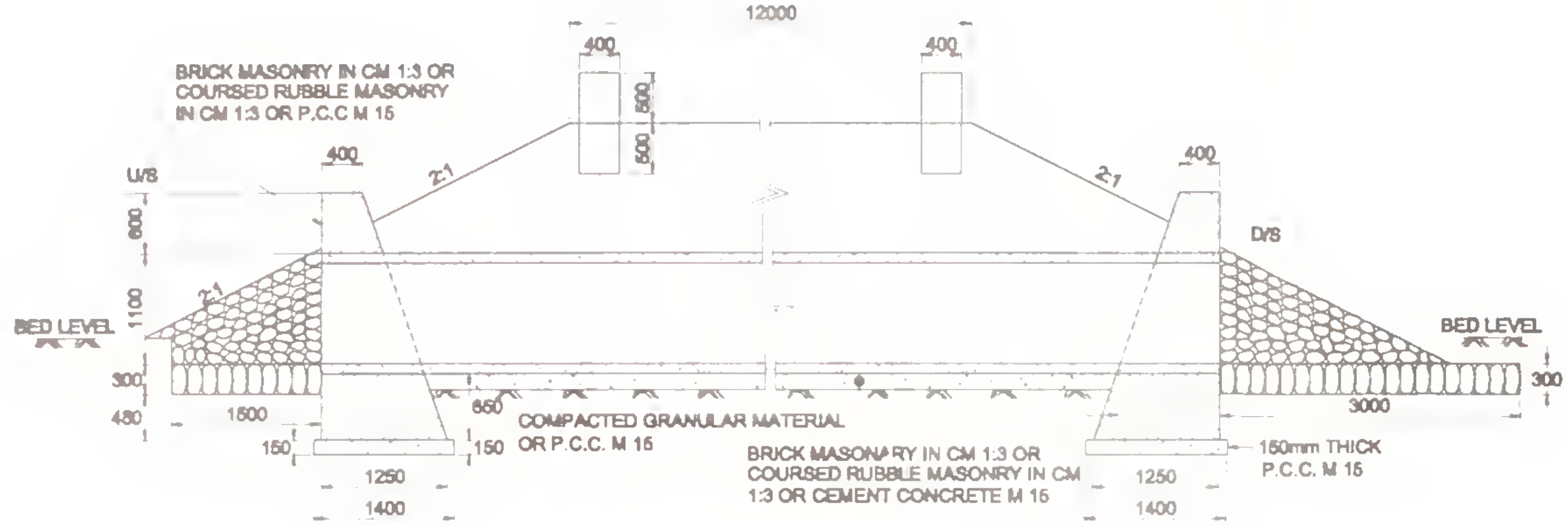


PLAN

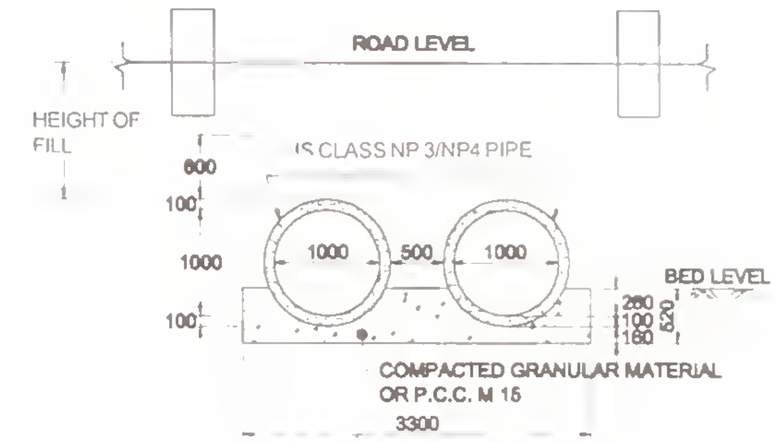
NOTES :-

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

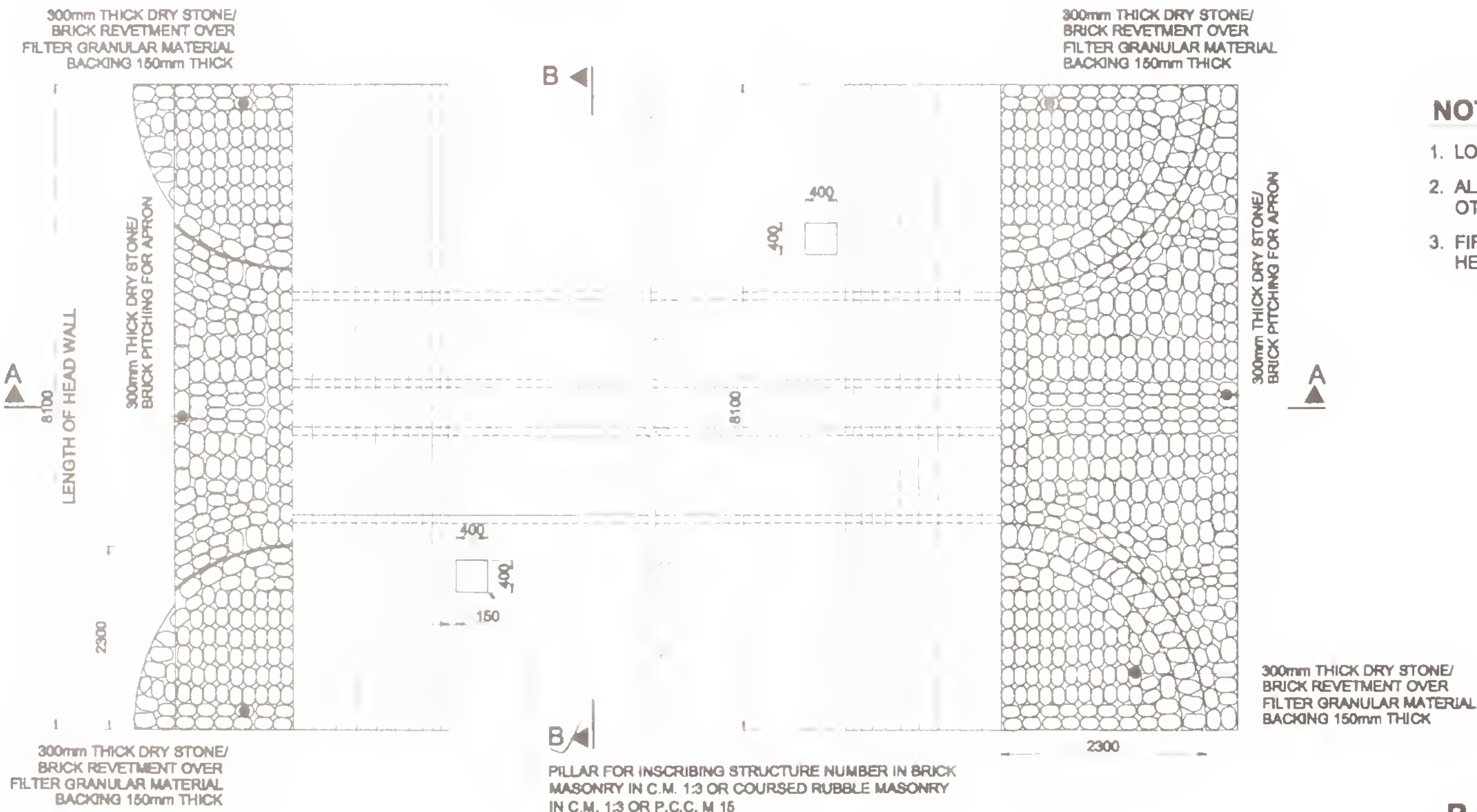
R.C.C. PIPE CULVERT WITH 2 PIPES OF 1 METRE DIA AND CONCRETE CRADLE BEDDING FOR HEIGHTS OF FILL VARYING FROM 4.0 m - 8 0 m



SECTION AT 'AA'



SECTION AT 'BB'



PLAN

NOTES :-

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

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

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

	Length "M" → / ↓ Diameter "M"	5	10	15	20	25	30	35	40	45	50	55	60
Entry Round Edged	0.75	0.394	0.373	0.356	0.339	0.325	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	0.536	0.522	0.512
	1.5	1.637	1.60	1.56	1.52	1.485	1.45	1.42	1.395	1.365	1.34	1.315	1.295
	2.0	2.93	2.86	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	1.296	1.275	1.256	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 x 0.75	0.516	0.488	0.466	0.445	0.427	0.412	0.397	0.384	0.373	0.362	0.352	0.393
	1.0 x 0.75	0.693	0.68	0.632	0.607	0.584	0.562	0.545	0.528	0.513	0.497	0.485	0.473
	1.0 x 1.0	0.935	0.90	0.867	3.837	0.81	0.788	0.765	0.745	0.727	0.71	0.693	0.877
	1.25 x 1.0	1.175	1.136	1.1	1.068	1.037	1.01	0.985	0.96	0.937	0.917	0.898	0.88
	1.25 x 1.25	1.47	1.43	1.385	1.35	1.315	1.285	1.252	1.225	1.2	1.175	1.15	1.13
	1.5 x 1.25	1.78	1.73	1.68	1.64	1.6	1.568	1.532	1.5	1.47	1.44	1.415	1.39
	1.5 x 1.5	2.14	2.08	2.03	1.99	1.95	1.91	1.87	1.835	1.8	1.765	1.74	1.71
Entry Sharp Edged	0.75 x 0.75	0.48	0.442	0.425	0.41	0.398	0.383	0.371	0.361	0.35	0.343	0.334	0.326
	1.0 x 0.75	0.615	0.593	0.572	0.553	0.535	0.52	0.505	0.492	0.48	0.468	0.457	0.447
	1.0 x 1.0	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 x 1.0	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 x 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 x 1.25	1.545	1.51	1.48	1.45	1.425	1.4	1.37	1.35	1.33	1.31	1.29	1.27
	1.5 x 1.5	1.85	1.81	1.78	1.75	1.72	1.69	1.665	1.64	1.615	1.59	1.57	1.55