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GUIDELINES FOR DESIGN OF CAUSEWAYS AND SUBMERSIBLE BRIDGES

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GUIDELINES FOR DESIGN OF CAUSEWAYS AND SUBMERSIBLE BRIDGES

1. INTRODUCTION

1.1. The Guidelines for Design of Causeways and Submersible Bridges had been under the consideration of the earlier General Design Features Committee since the year 2004. Later on this Committee was merged with the General Design Features (Bridges and Grade Separated Structures Committee (B-1) at the time of reconstitution in January, 2006. The General Design Features Committee in its meeting held on 15th May, 2004 had constituted a Sub-group consisting of Shri P.L. Bongirwar, Dr. C.V. Kand, S/Shri D.K. Rastogi, M.V.B. Rao and Late Shri N.K. Patel. Thereafter, the draft as prepared by the Sub-group and Shri S.K. Kaiastha was considered by the reconstituted General Design Features (Bridges and Grade Separated Structures Committee, B-1) in a number of meetings and finalized it in its meeting held on 12th October, 2006 subject to incorporation of certain comments by its Convenor, Shri Prafulla Kumar.

The personnel of B-1 Committee is given below:

Kumar, Prafulla	...	*Convenor*
Indoria, R.P.	...	*Co-Convenor*
Rustagi, S.K.	...	*Member-Secretary*

Members

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Jangde, K.S.	Roy, B.C.
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Ex-officio Members

President, IRC (Mina, H.L.)

DG(RD), MOSRT&H (Sharan, G.)

Secretary General, IRC (Sinha, V.K.)

1.2. Thereafter, the draft guidelines for Design of Causeways and Submersible Bridges were considered by the Bridges Specifications and Standards Committee (BSS) in its meeting held on 3rd November, 2007. The Committee formed a Sub-group comprising Shri Chaman Lal, CE(B) S&R, MOSRT&H, Shri M.V.B.Rao, Dr. C.V. Kand and Shri Sharad Varshney, Addl. Director (Technical), IRC for technical enhancement of the document. The Sub-group met thrice on 9.1.2008, 1.2.2008 and 23.5.2008 and put up the draft document again to Bridges Specifications & Standards Committee.

1.3. The valuable suggestions offered by the members of General Design Features (Bridges and Grade Separated Structures) Committee (B-1) and Bridges Specifications & Standards Committee are duly incorporated.

1.4. The draft document was approved by the Bridges Specifications and Standards Committee in its meeting held on 29.3.2008, and the Executive Committee in its meeting held on 11.4.2008 and authorized Secretary General, IRC to place the same before Council. The document was approved by the IRC Council in its 185th meeting held on 11.4.2008, at Aizwal (Mizoram) for printing subject to incorporation of some comments offered by the Council members.

2. SCOPE

This document contains guidelines for planning and design of submersible structures like fords, dips, causeways and submersible bridges on various categories of roads viz. State Highways, Major District Roads, Other District Roads and Village Roads in the country.

3. GENERAL FEATURES

3.1. Definitions

The following definitions shall be applicable for the purpose of these Guidelines.

3.1.1. Bridge

Bridge is a structure having a total length of 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. These are classified as minor and major bridges as per classification given below:

(a) Minor Bridge : A minor bridge is a bridge having a total length of upto 60 m. A minor bridge upto a total length of 30 m is sometimes classified as a small bridge.

(b) Major Bridge : A major bridge is a bridge having a total length of above 60 m.

3.1.2. High level bridge

A high level bridge is a bridge which carries the roadway above the highest flood level of the channel.

3.1.3. Submersible bridge

A submersible bridge is a bridge designed to be overtopped during floods.

3.1.4. Causeway

A causeway is a paved submersible structure with or without openings (vents) which allows flood to pass through and/or over it.

3.1.5. Ford

A ford is an unpaved shallow portion in a river or stream bed which can be used as a crossing during dry weather/normal flow.

3.1.6. Culvert

A 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 there to.

3.1.7. Channel

A channel means a natural or artificial watercourse.

3.1.8. Afflux

It is the rise in the flood level of the channel immediately on the upstream of a bridge as a result of obstruction to natural flow caused by the construction of the bridge and its approaches.

3.1.9. Highest Flood Level (HFL)

Highest flood level is the level of the highest flood ever recorded or the calculated level for the design discharge, whichever is higher.

3.1.10. Ordinary Flood level (OFL)

Ordinary flood level is the level of flood expected to occur every year. It can be determined by averaging the highest flood levels of seven consecutive years.

3.1.11. Low Water Level (LWL)

Low water level is the level of the water surface attained generally in the dry season. It can also be determined by averaging the low water levels recorded in seven consecutive years.

3.1.12. Design Flood Level (DFL)

It is the highest flood level for which the structure must be designed. It corresponds to level of highest flood of 50 years or 100 years return period (whichever is chosen for design) or the highest known flood level if the same happens to be higher.

3.1.13. Defined Cross-section

It is the undisturbed natural cross-section of river which does not exhibit signs of erosion or silting of bed.

3.1.14. Protected Bed Level (PBL)

It is the level at which the bed surface is protected against erosion due to flow of water.

3.2. Types of Submersible Structures

3.2.1. Fords

Fords are unpaved structures and are suitable only for roads having very low volume of traffic. These are the simplest form of crossings where the stream is wide and shallow, velocity of flowing water is low and bed surface is relatively firm.

In case the bed surface is not firm enough and not capable of carrying the vehicular traffic, the bed can be strengthened and made more even with buried stones just below the bed surface. If the stones are likely to be carried away in flow, this is prevented by construction of barriers made of suitable size of boulders or wooden piles. Boulders (neither too large which may result in scouring of bed nor too small likely to be carried away by flow) are placed across the river bed at downstream side of the ford to filter the flow of water and retain small size particles of bed material like sand, gravels etc. resulting in a more even surface for vehicular traffic. **Fig. 3.1** shows a typical cross-section of such type of ford.

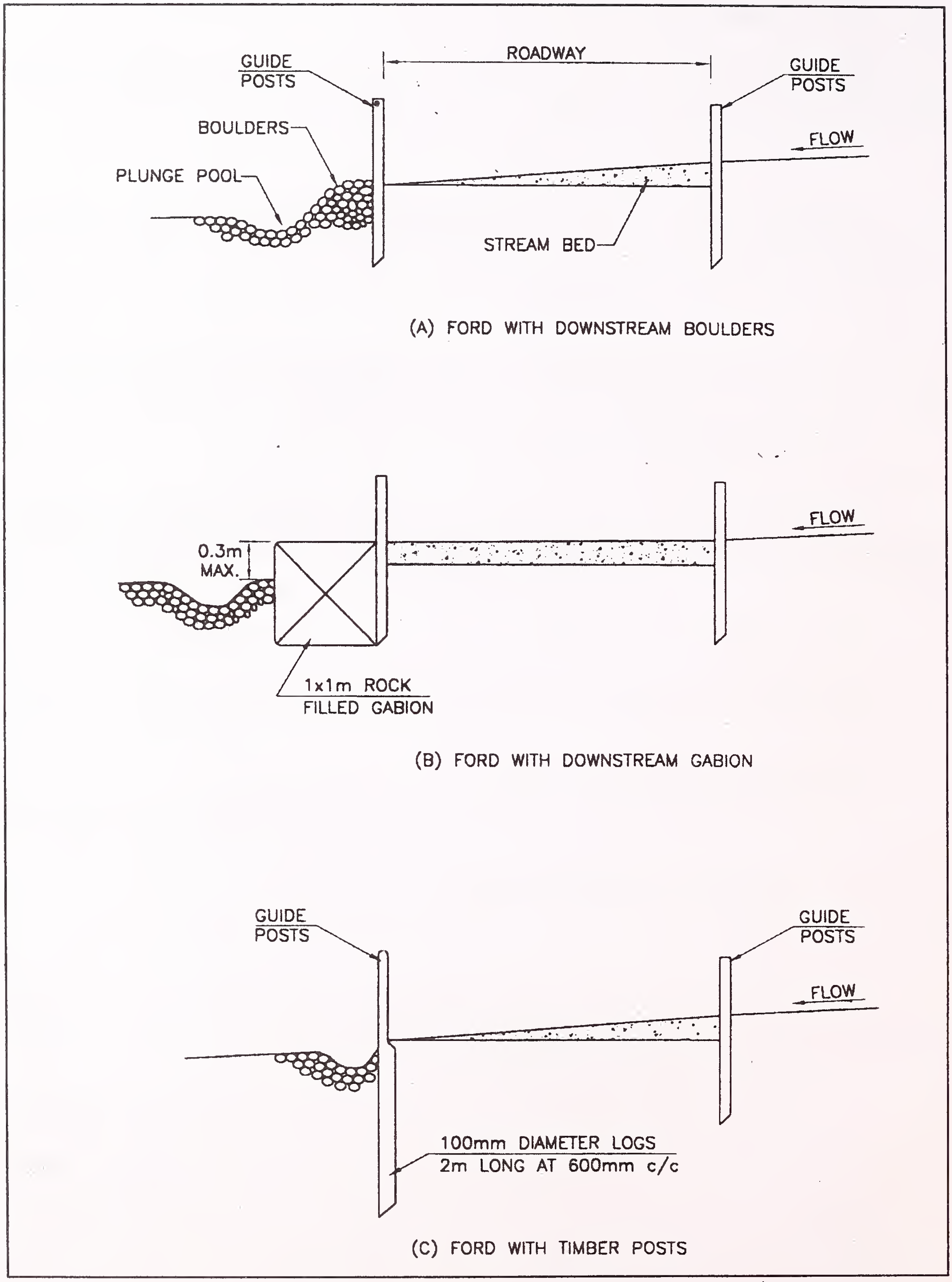


Fig. 3.1. Typical Details of Fords

3.2.2. Causeways

There are mainly three types of causeways:

(a) Flush causeway

In this type of causeway which is also called paved dip or road dam, the top level of road is kept same as that of bed level of the channel. It is suitable where the crossing remains dry for most of part of year i.e. the stream is not perennial. Flush causeways are not suitable for crossing the streams with steep bed slopes causing high velocity even in low floods. The causeway covers the full width of the channel **Fig. 3.2.**

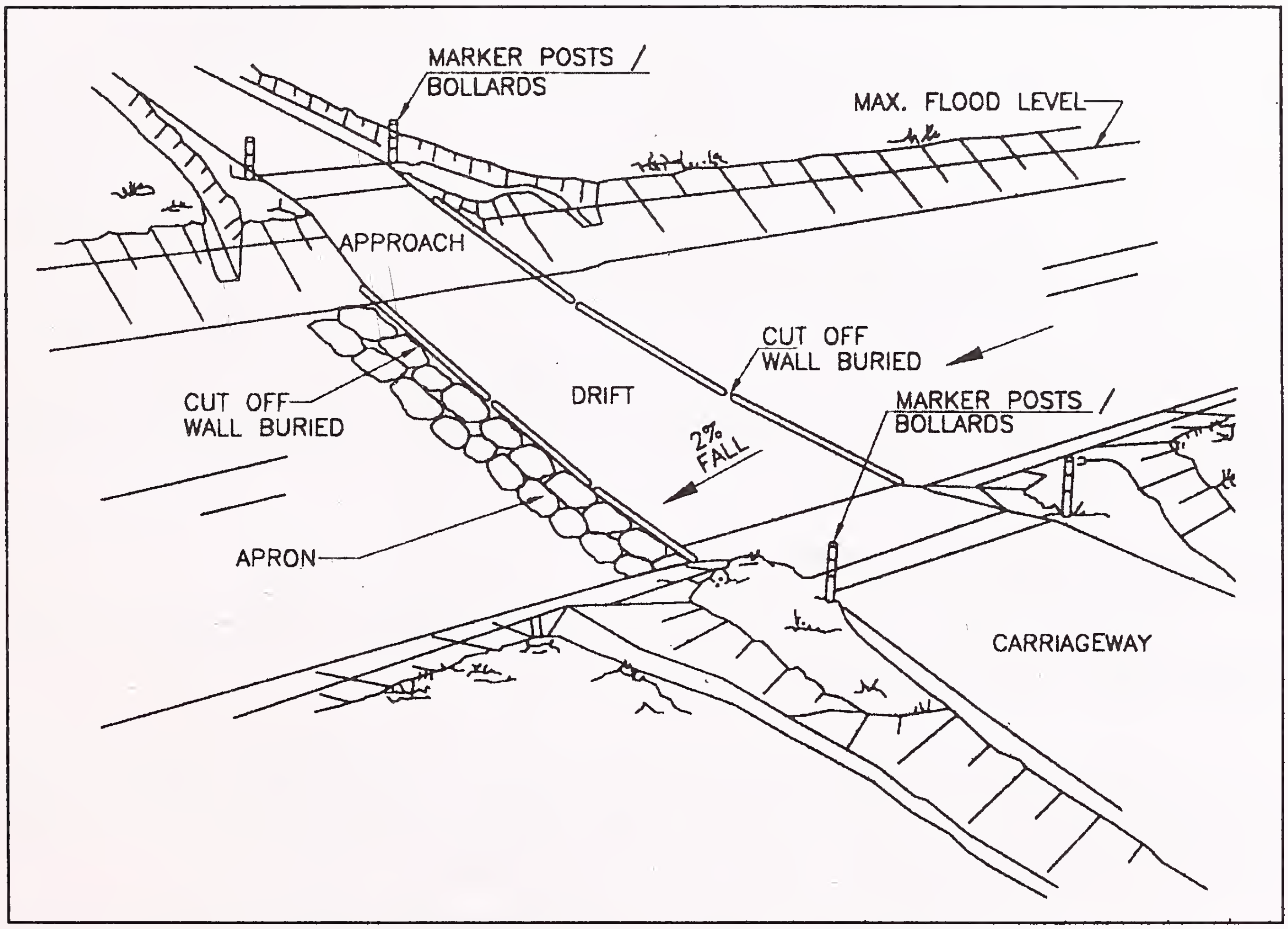


Fig. 3.2. Typical Features of Paved dip/Flush Causeway

(b) Vented causeway

A causeway provided with vents to permit normal flow of the stream to pass under the causeway is known as vented causeway. Vented causeways are classified as low vented causeways and high vented causeways.

(i) Low vented causeway

Low vented causeways are provided to cross quasi-perennial streams having sandy beds in areas with annual rainfall less than 1000 mm and where the carriageway of a flush causeway would be liable to get slushy due to post monsoon flow in the stream. The height is generally less than 1.20 m above the bed of the watercourse. In exceptional cases, the height may be 1.50 m above the bed level. Small size of vents in the form of hume pipes, short span slabs/R.C.C. Box cells are provided in the width of stream. The sill level of vents is kept about 150 mm – 300 mm below the average bed level of the stream.

(ii) High vented causeway

High vented causeway is provided when a road crosses a stream having one or more of the following characteristics:

(i) Sizeable catchment area with annual rainfall more than 1000 mm

(ii) Depth of post monsoon flow is more than 900 mm

(iii) Flow is perennial but not large

(iv) Banks are low necessitating construction of high embankment in the stream bed from considerations of the free board in non-submersible portion as well as geometric standards of approach roads

The height of the causeway above the bed is generally kept between 1.5 m to 3.0 m and larger size of vents comprising of hume pipes or simply supported/continuous R.C.C. slab superstructure over a series of short masonry piers or series of arches or boxes with individual spans less than 3 m are provided.

3.2.3. Submersible bridge

Submersible bridge is normally sub-classified as high submersible bridge or low submersible bridge depending upon deck level with reference to OFL.

The deck level of high submersible bridge is fixed with reference to OFL and vertical clearance, and as such the structure serves as high level bridge during OFL but gets submerged under higher floods with permissible number and duration of interruptions. This type of bridge is suitable for streams having large variation between HFL and OFL.

The deck level of low submersible bridge is fixed above the OFL so as to ensure that the interruptions caused to traffic remain within permissible limits.

3.3. Selection of Type of Submersible Bridge/Causeway

3.3.1. General

The type of structure (i.e. high level or submersible) across a watercourse (channel) has to be judiciously selected on the basis of reconnaissance inspection report and available data. The choice mainly depends on the classification of the project road, requirements of the user authority, hydrology of the watercourse and availability of funds for the project.

3.3.2. Considerations in the selection of type of submersible structures

Selection of type of submersible structures (i.e. ford or causeway or submersible bridge) inter-alia depends on:

(a) Requirements of user authority and availability of funds

(b) Category, importance of road and traffic intensity

(c) Population to be served

(d) Nature of stream i.e. flashy/perennial/seasonal etc. and velocity of water during floods

(e) Duration, magnitude of floods and interruption to traffic

(f) Spread and depth of water during floods and post monsoon period

(g) Extent of catchment area

3.3.3. Criteria for avoiding/selection of submersible structures

In the absence of any directives/guidelines by the user authority, the following criteria may be followed for selection of suitable type of submersible structures including causeways on different categories of roads.

(1) These should be avoided on National Highways

(2) These may not be considered for adoption in the following situations:

(i) Roads of economic importance, roads linking important towns or industrial areas or areas with population more than 10,000 where alternative all weather route with reasonable length of detour is not available

(ii) On roads which are likely to be upgraded or included, from future traffic considerations, in the National Highway network

(iii) If the length of a high level bridge at such crossings would be less than 30 m except where construction of high level structure is not economically viable

(iv) Maximum mean velocity of stream during floods is more than 6 m/sec

(v) If the cost of submersible bridge with its approaches is estimated to be more than approximately 70% of the cost of high level bridge with its approaches, near about the same site

(vi) If firm banks are available and approaches are in cutting or height of embankment for submersible portion of approaches is more than 2 m

(vii) Where there are faults in the river bed

(viii) If after completion of the submersible structures, the number of interruptions in a year caused to traffic and duration of the interruptions are likely to exceed the suggested values given in **Table 3.1** below.

Table 3.1: Permissible Number and Duration of Interruptions

S. No.	Category of Roads	Maximum No. of permissible interruptions in a year	Duration of interruption in hours at a time
1.	State Highways, M.D.Rs., roads linking important towns, industrial estates.	6	2-6 h duration, less than 2 h not to be considered and more than 6 h not acceptable
2.	O.D.Rs,Village Roads	6	6-12 h duration, less than 6 h not to be considered and more than 12 h not acceptable

3.3.4. Fords

Fords (i.e. unpaved causeway), though the cheapest type of crossing, should be avoided as far as possible and its adoption should be limited to sites where stream is wide, shallow with depth of water not more than 200 mm, velocity of flow is low (less than 2 m/sec), bed is firm, volume of traffic is low and the water is not likely to become muddy due to the traffic, endangering the aquatic life in the watercourse or the environment.

3.3.5. Causeways

Causeways for crossing a wide watercourse with low banks and having not too large but perennial flow should be proposed with caution. These should be proposed on rural and less important link roads, not likely to generate much traffic in near future due to situations like dead end, low habitation and difficult terrain conditions. The causeways may be proposed on streams of flashy nature with high frequency of short duration floods or at sites where construction of submersible bridge is not economically viable.

3.3.6. Submersible bridges

These can be provided in all situations other than those mentioned in paras 3.3.4 and 3.3.5 above where provision of submersible structures is technically feasible and economically viable.

3.4. Geometric Standards

3.4.1. General

(a) A road conforming to sound geometric standards results in economical operation of vehicles and ensures safety. Geometric standards for approach roads to a submersible bridge or causeway depends on the classification of road (i.e. State Highway (SH) or Major District Road (MDR) or Rural Road (RR) which include Other District Road (ODR) and Village Road (VR), location (i.e. in urban or non-urban area), terrain (i.e. plain or rolling or mountainous or steep), length of crossing and requirements of the user authority (i.e. local, State Govt. etc.).

(b) The geometric standards in general should conform to relevant IRC Publications (i.e. IRC:5, IRC:38, IRC:52, IRC:73, IRC:86, IRC:SP:20, IRC:SP:23 and IRC:SP:48).

(c) There is no specific separate guideline in the IRC codes regarding geometric design standards for submersible structures including immediate approaches except in IRC:5, which stipulates that vented causeways/submersible bridges shall provide for at least two lanes of traffic (7.5 m wide carriageway) unless one lane of traffic (4.25 m wide carriageway) is specially permitted in the design. However, the provision for single lane width is likely to be revised and has been increased in these guidelines. Refer **Table 3.2.**

3.4.2. Width of cross drainage structures

Cross-drainage structures are difficult to widen at a later date. As such, road width should be selected carefully at the planning stage itself. In case a road is likely to be upgraded in the foreseeable future, it is desirable to adopt higher roadway width.

Minimum carriageway width of submersible structures, measured at right angles to the longitudinal center line of the structure, between the inner faces of discontinuous kerbs/safety kerbs wherever provided or between the guideposts/stones (without kerbs), should be as given in **Table 3.2**.

Table 3.2: Minimum Width of Carriageway for Submersible Structures

Category of road	Minimum Width of Carriageway*(m)	
	Plain & Rolling Terrain	Mountainous and Steep Terrain
Single lane	6.8	5.5
Two lanes	7.5	7.5

Note: * Minimum width of carriageway should be suitably increased as per IRC:73 in case of structures located on curves.

In case footpaths are provided, the width of footpaths should not be less than 1.5 m each. The width of discontinuous safety kerbs, if provided should not be less than 600 mm.

Overall width between the outer faces of discontinuous kerbs/safety kerbs wherever provided or guideposts/stones/railings (without kerbs) of the structures with length upto 30 m should preferably be a little more to match with the roadway width of immediate approaches **Table 3.3**.

3.4.3. Geometrics of approach roads

(i) Alignment of the road generally governs the site of a submersible structure if the length of crossing is less than 60 m. However, if the length of the crossing is more than 60 m, the suitability of the site for the submersible structure and the geometric design of immediate approaches both should be considered together. In case the length of crossing is more than 300 m, the most suitable site for the bridge should be the governing criteria.

(ii) The approaches on either side of a straight submersible bridge should have a minimum straight length of 30 m and should be suitably increased, where necessary, to provide for the minimum sight distance for a vehicular speed of 35 km/h.

(iii) Horizontal curves in immediate approach roads for a length of about 100 m on either side of a submersible structure or causeways should be avoided. If horizontal curves have to be provided in the approaches, the same should be located beyond the straight portion on either side and the minimum radius of curvature, the super-elevation and transition length should be provided in accordance with relevant stipulations contained in IRC:38. Radii of horizontal curves in case of immediate approach roads however should, not be less than 60 m in case of plain and rolling terrain and 30 m in case of hilly terrain from road user safety consideration.

3.4.4. Design speed

From consideration of safety of road users, lower design speed than that recommended in IRC:73 should be adopted for the immediate approaches to a submersible bridge or causeway. The informatory boards installed on approaches should indicate permissible speed of 35 km/h in case of plain and rolling terrain and 20 km/h in case of mountainous and steep terrain irrespective of any higher speed adopted in the design of the road.

3.4.5. Roadway width

Width of roadway should be as shown in **Table 3.3.**

Table 3.3: Width of Roadway (m)

S.No.	Road Classification	Plain & Rolling Terrain	Mountainous & Steep Terrain **
1.	**State Highways**		
	i) single lane	12.0*	6.25##
	ii) two lanes	12.0	8.8
2.	**Major District Roads**		
	i) single lane	9.0	6.25##
	ii) two lanes	9.0	8.8
3.	**Rural Roads**		
	i) single lane	7.5***	6.0##
	ii) two lanes	9.0	7.5

Notes: 1 * For single lane State Highways, width of roadway might be reduced to 9 m if the possibility of widening, the carriageway to two lanes is considered remote.

2. ** The roadway widths in mountainous and steep terrain, given above are exclusive of parapets (usual width 0.6 m) and side drains (usual width 0.6 m).
3. *** Roadway width for rural roads in plain and rollers terrain also may be reduced to 6.0 m in case where traffic intensity is less than 100 motor vehicles per day and traffic is not likely to increase due to situations like dead end, low habitation and difficult terrain conditions.
4 ## On roads subject to heavy snow fall, where regular snow clearance is done over long periods to keep the road open to traffic, the roadway width may be increased by 1.5 m.
5 The roadway widths for Rural Roads are on the basis of a single lane carriageway of 3.75 m.
6 In hard rock stretches, or unstable locations where excessive cutting might lead to slope failure, width of roadway may be reduced by 0.8 m on two-lane roads and 0.4 m in other cases.
7. On horizontal curves, the roadway width should be increased corresponding to the extra widening of carriageway for curvature.

3.4.6. Camber/crossfall

The camber/crossfall on straight sections of immediate approaches and on submersible structures should be unidirectional towards the downstream and as recommended in **Table 3.4** depending on type of surface of pavement

Table 3.4: Pavement Camber/Crossfall

Surface Type	Unidirectional Cross fall (%)
For all categories of roads	
1. High Type bituminous surfacing or cement concrete	2.0
2. Thin bituminous surfacing for approaches	2.5
3. Brick/stone set pavement	3.0

Note: Shoulders of approach roads likely to be submerged during floods should be paved to same cross fall towards downstream as for pavement.

3.4.7. Superelevation

Superelevation to be provided on horizontal curves is calculated from the following formula subject to the maximum values indicated in **Table 3.5**.

Superelevation in m per m = (Design speed in km/h $)^2$ /(225 x radius of curve in m)

Table 3.5: Maximum Permissible Superelevation

1.	Plain/rolling terrain and snow bound hill roads	7%
2.	Hill roads not affected by snow	10%

3.4.8. Gradients

As a general rule, values of ruling gradients specified in IRC:73 should be adopted. However, in case of immediate approaches to submersible structures, carrying substantial slow traffic, flatter gradients than ruling values should be preferred. Nevertheless, gradients in immediate approaches unless, otherwise permitted by user authority, should not exceed 5.0% (1 in 20) irrespective of nature of terrain.

4. HYDROLOGY AND HYDRAULICS

4.1. Hydrology

4.1.1. General

For the design of an efficient and economical hydraulic structure, knowledge of hydrology and the characteristics of the Stream/River are of paramount importance. A brief about hydrology is given in **Appendix 4.1**. In most cases hydrological record of the stream particularly data regarding floods may not be available. A rational estimation of design flood discharge for the specified return period leads to economical design of bridge foundations for submersible bridges. The failures of hydraulic structures are very expensive as in most cases, the indirect costs are many times larger than the direct cost of bridge replacement. Some hydraulic structures especially bridges have failed in the past mainly due to inadequate assessment of HFL/ Design flood discharge and rarely due to structural failures. Due attention to the determination of hydrology of the structure needs to be paid as an irrational approach can lead to loss and destruction of the structure due to floods higher than the design floods.

4.1.2. Determination of design discharge

The design discharge for which the waterway of most of the bridge including submersible bridges is to be designed should be based on the flood discharge corresponding to highest observed flood level, irrespective of the return period of that flood or the flood of 50 years' return period whichever is higher, except in the case of important bridges when return period may be taken as 100 years. The design discharge can be determined by the following methods:

(1) Empirical Methods
(2) Slope Area Method
(3) Rational Method
(4) Unit Hydrograph Method

4.1.2.1. Empirical methods

Based on studies conducted, some empirical formulae for specific regions have been evolved. The empirical formulae for flood discharge suggested are in the form:

$$Q = CA^n \qquad \dots \qquad (4.1)$$

Where,

Q = Max. flood discharge in m^3/s
A = Catchment Area in sq. km
C = An Empirical Constant, depending upon nature and location of catchment
n = A Constant

The most commonly adopted empirical formulae and recommended for use are:

(i) Dicken's formula based on data of rivers in Central India, (ii) Ryve's formula based on Rivers in South India and (iii) Inglis formula based on West Indian rivers in the old Bombay Province. Details of these emperical formulae are given in **Appendix 4.2**.

The empirical formule should, however, be used with due caution as given below:

(i) These were developed for particular region and for small catchments and, therefore, have obvious limitations. The value of 'C' at the best is valid only for the region for which it has been determined, as each basin has its own characteristics affecting run-off.

(ii) These involve only one known variable factor viz. area of the catchment and therefore a large number of remaining factors that affect the run-off such as shape, slope, permeability of catchments etc. are to be accounted for in selecting an appropriate value of the coefficient 'C'.

(iii) A correct value of 'C' can only be derived for a given region from an extensive analytical study of the measured flood discharge vis-a-vis characteristics of the basin. The value of 'C' will therefore be valid only for the region for which it has been determined, as each basin has its own characteristics affecting run-off. A new designer should use these formulae only under the guidance of an experienced designer or expert.

4.1.2.2. Slope – area method

In this method the maximum water level reached in a historic flood is estimated on the evidence of local witnesses, which may include identification of flood marks on structures or trees close to the bridge site. The discharge is then calculated by:

$$Q = A\,V \qquad \ldots \qquad (4.2)$$

Where, Q = discharge in m^3/s and A = wetted area in m^2

V = velocity of flow in m/sec which can be calculated by the Manning's formula:

$$V = 1/n\ R^{2/3}.\ S^{1/2} \qquad \ldots \qquad (4.3)$$

Where, R = hydraulic mean depth, S = the energy slope which may be taken as equal to bed slope and n = rugosity coefficient.

The details of the method are given in **Appendix 4.3**.

This method has also considerable room for error due to:

(i) The variability of bed profile slope etc. during floods from those measured during survey.

(ii) The computation of stream velocity is dependent upon a subjective selection of an Empirical Coefficient of rugosity for different conditions of bed out of the various values recommended by Manning.

4.1.2.3. Rational method

The rational method for flood discharge takes into account the intensity, distribution and duration of rainfall as well as the characteristics of the catchment area, such as shape, slope, permeability and initial wetness of the catchment.

The rational formula is as follows:

$$Q = A\, I_0\, \lambda \qquad \ldots \qquad (4.4)$$

Where,

Q = Maximum flood discharge in m^3/s

A = Catchment area in hectare

I_0 = Max. intensity of rainfall in cm/h

λ = Function depending upon characteristics of the catchment in producing peak run-off and given by –

$$\lambda = \frac{0.056\, f\, P}{t_c + 1} \qquad \ldots \qquad (4.5)$$

Where, 'f' is the area correction factor, 't_c' is the time of concentration in hours and 'P' is permeability coefficient of the catchment depending on the soil cover conditions and slope of catchment etc. The details about Rational Method are given in **Appendix 4.4**. The formulae may generally be adopted for catchment areas upto 500 sq. km and upto 2000 sq. km in exceptional cases.

4.1.2.4. Unit hydrograph method

(i) Unit Hydrograph: The unit hydrograph or unit graph is defined as the hydrograph of storm run-off at a given point in the river, resulting from an isolated rainfall of unit duration (normally taken as 6 h to 12 h) occurring uniformly over the catchment and producing unit run-off. The unit run-off adopted is 1 cm depth over the catchment area.

(ii) A Committee of Engineers appointed by Govt. of India recommended a rational methodology based on use of design storms and unit hydrographs for estimating design floods for different zones/sub-zones of India. A list of these zones and sub-zones is given in **'Annexure A'** of **Appendix 4.5**. The report as prepared jointly by CWC, RDSO (Railways), MoSRT&H and IMD have been published by CWC, Govt. of India. These reports give methodology through a set of charts and graphs for quick estimation of design flood of 25, 50 or 100 years of return periods for ungauged catchments.

(iii) Unit hydrographs are prepared either by computation from direct run-off hydrograph for gauged streams or are synthetically prepared from catchment characteristics for ungauged catchments and then used for finding design flood of desired return period. The detailed procedure for constructing Synthetic unit hydrograph and how to obtain design flood from storm of corresponding return period is illustrated in an example given in **Appendix 4.5.**

(iv) The unit hydrograph method can give fairly precise results for drainage areas upto 5000 sq. km. Variation in assumptions made for larger areas (>5000 sq. km) in the method are usually too great to be ignored.

4.1.2.5. Fixing design discharge

Flood discharge can be estimated by three or more different methods and the values obtained should be compared. The highest of these values should be adopted as the design discharge provided it does not exceed the next highest discharge by more than 50%. If it does, restrict it to that limit.

4.1.3. Discharge through a submersible bridge

The total discharge in the stream after the construction of a submersible bridge can be found by the method suggested by Johnson Victor as given below:

$$\text{Total discharge } Q = Q_a + Q_b + Q_c \quad \ldots \quad (4.6)$$

$$\text{and } \quad Q_a = A_a \times \frac{2}{3} C_a \sqrt{2g} \; \frac{(H + h_a)^{3/2} - h_a^{\;3/2}}{H} \quad \ldots \quad (4.6.1)$$

$$Q_b = A_b \times C_b \sqrt{2g}.\ \sqrt{H + h_a} \quad \ldots \quad (4.6.2)$$

$$Q_c = A_c \times C_c \sqrt{2g}.\ \sqrt{H + h_a} \quad \ldots \quad (4.6.3)$$

Where,

Q_a = Discharge between afflux upstream water level and down stream water level and A_a is its area of flow

Q_b = Discharge between downstream water level and deck level

A_b = Area of flow between downstream water level and deck level

Q_c = Discharge through vents and A_c is the area of vents

C_a, C_b & C_c are coefficients of discharge

H = Afflux

h_a = Head due to velocity of approach.

C_a = 0.625 for equation (4.6.1)

C_b = C_c=0.9 for equations (4.6.2) and (4.6.3)

(Refer Fig. 4.1 for various parameters of flow.)

4.1.4. In cases where the cross-section of the stream has wide spill zones of shallow depth, the discharge through causeway or low level submersible bridge can also be found by adding the calculated discharge of the three parts viz. (a) Discharge through vents of area A1, (b) Flow over the causeway/submersible bridge proper through area A2 and (c) Flow over shallow triangular compartments of area A3 on either side of the main stream at the crossing. (See **Fig 4.2**).

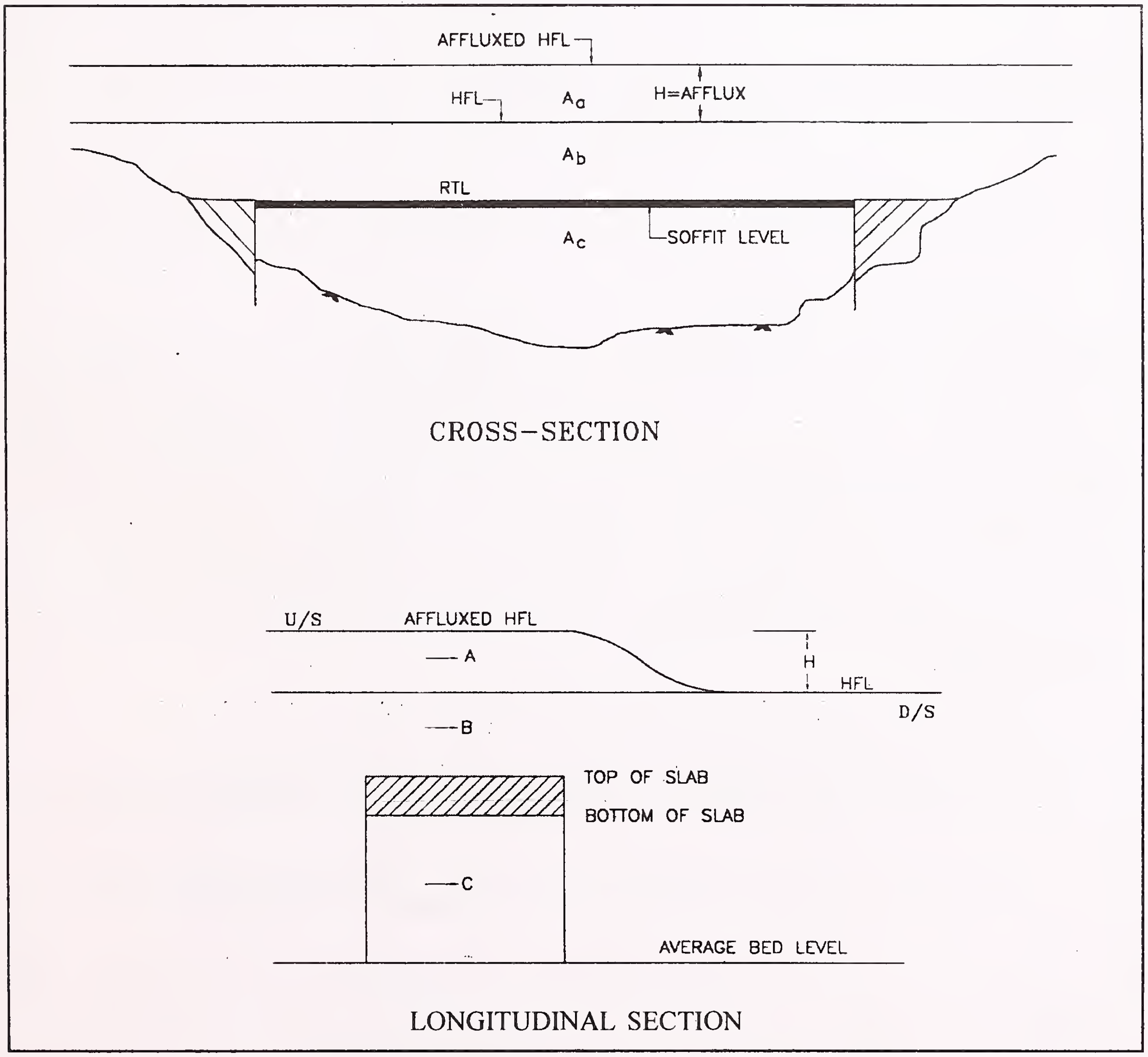


Fig. 4.1. Total Discharge at a Submersible Bridge

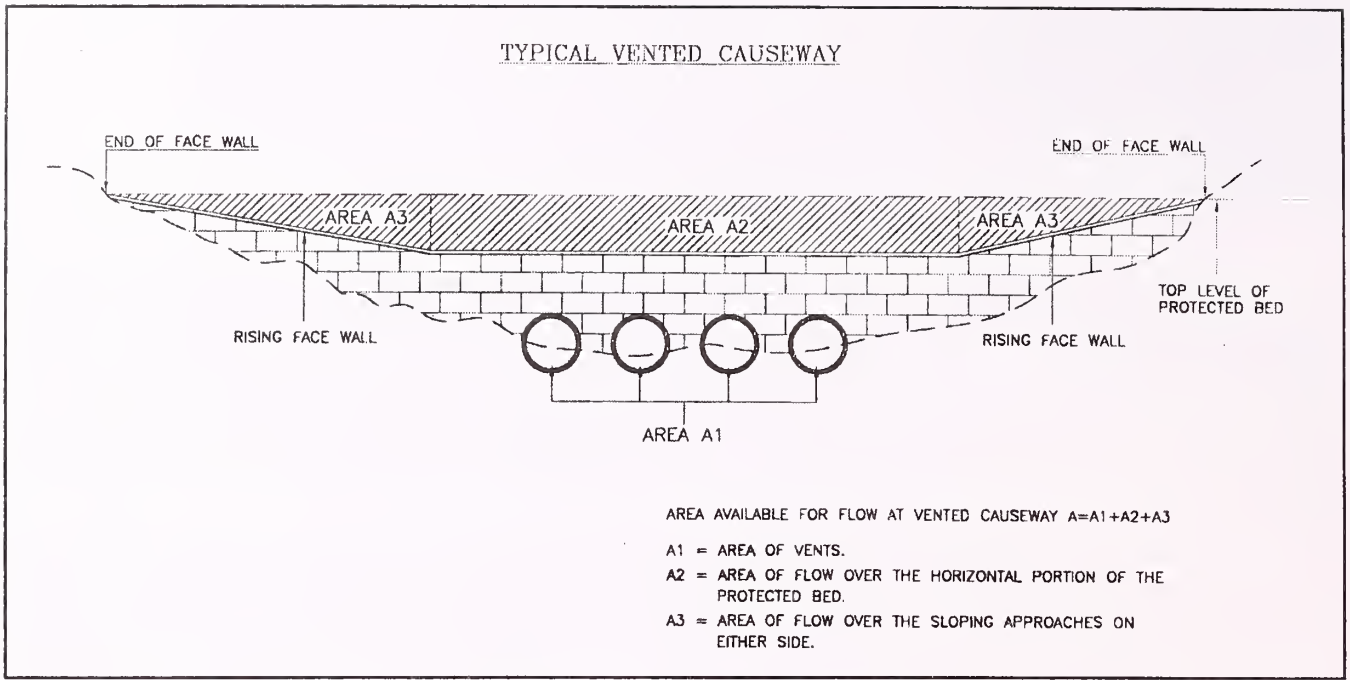


Fig. 4.2 Typical Vented Causeway

4.2. Forces due to Water

4.2.1. Hydro Static Force

Force of stationary water on a solid surface is called the hydrostatic force. It includes force due to the afflux head and the force of buoyancy. A body submerged in water experiences an upward force due to water pressure and this force is called 'Buoyancy'. It must be considered for stability of structure if there is possibility where while considering combination of forces, stability of the structure is to be affected. It is recommended that while checking for minimum pressure on foundation, the maximum uplift pressure at high water level should be considered. Further, while checking for maximum pressure the minimum uplift pressure at the low water level should be taken into account. In case of submersible bridges, full buoyancy effect on the superstructure also needs to be considered.

4.2.2. Hydrodynamic force of water current

4.2.2.1. Water current forces on foundation above scour level and on substructure

Water current causes hydrodynamic force on the submerged part of a body. These forces on a member can be calculated by the following formula as given in Clause 213 of IRC:6.

$$P = 52KV^2 \qquad \ldots \qquad (4.7)$$

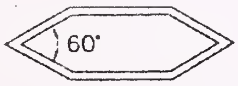
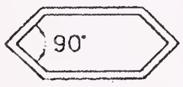
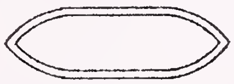
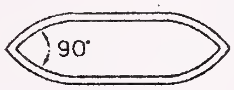
Where,

P = Intensity of pressure due to water current in kg/m^2

V = The velocity of the current at the point where the pressure intensity is being calculated in meter per second and

K = A constant having the following values for different shapes of members as given in **Table 4.1.**

Table 4.1: Shapes of Bridges Piers & Value of K

VALUES OF K	SHAPES OF MEMBERS IN PLAN	
1.50		MEMBER WITH SQUARE ENDS (AND FOR SUPERSTRUCTURE)
0.66		CIRCULAR OR SEMICIRCULAR ENDS
0.50		TRIANGULAR (THE ANGLE INCLUDED BETWEEN THE FACES BEING 30 DEGREES OR LESS)
0.50 TO 0.70		TRIANGULAR (THE ANGLE INCLUDED BETWEEN THE FACES BEING MORE THAN 30 DIGREES BUT LESS THAN 60 DEGREES)
0.70 TO 0.90		**TRIANGULAR (THE ANGLE INCLUDED BETWEEN THE FACES BEING 60 TO 90 DEGREES OR LESS**
0.45		EQUILATERAL ARCS OF CIRCLES
0.50		INTERSECTING AT 90 DEGREES

The maximum velocity at the top surface of flow shall be assumed to be $\sqrt{2}$ times the maximum mean velocity of the current. Square of velocity at a height X from the point of deepest scour = $U^2 = \dfrac{2\,\overline{V}^2 X}{H}$ ... (4.8)

Where $\overline{V}$ is the maximum mean velocity.

The value of $\overline{V}^2$ in the equation (4.8) is assumed to vary linearly from zero at the point of deepest scour to the square of the maximum velocity at the free surface of water **(Fig. 4.3).**

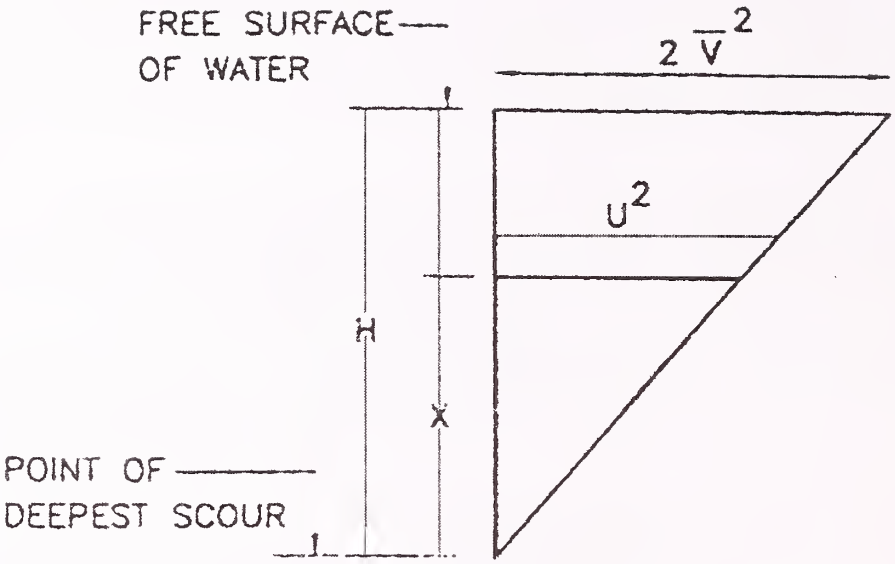


Fig. 4.3

4.2.2.2. Water current forces on superstructure

(i) The importance of water current forces on the superstructure is significant due to the extent of obstruction offered by the bridge superstructure and its location. Since the submerged area of superstructure exposed to water current forces is sufficiently large and the velocity of current at its level is also high, the stresses on foundations due to water current forces acting on the submerged superstructure are quite pronounced.

(ii) Flowing water produces two types of forces on a submerged or partially submerged superstructure viz. the drag force and the lift force. These are characterized by two factors i.e. the drag force co-efficient (C_d) and coefficient of lift (C_L). Both drag force and lift force depend largely on the shape of the body and several other factors and these can be best determined by conducting hydraulic model studies, as explained in **Appendix 4.6**.

(iii) The results of model studies conducted so far do not conclusively recommend any generalized values of co-efficient of drag (C_d) and co-efficient of lift (C_L). However, presently the following method is adopted for calculation of drag force and uplift pressure on superstructures, in cases where it is not feasible or economically viable to conduct hydraulic model studies:

(a) The expression $P = 52\ KV^2$ as given in para 4.2.2.1 be adopted with value of K as 1.5 for drag force.

(b) The expression $p = wh$ may be adopted for calculating uplift pressure,

Where 'w' is the unit weight of water and 'h' is the uplift head under the deck and can be estimated as h = thickness of slab + wearing coat and afflux after deducting the head loss due to increase in velocity through vents.

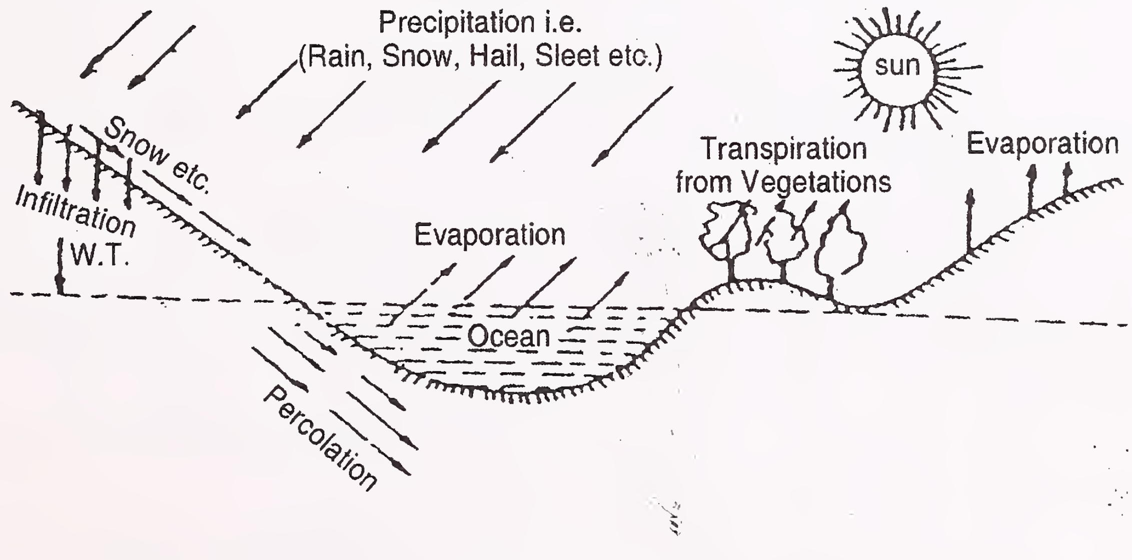
The head loss is given by the expression $(V_v^2 - V^2)/2g$,

Where V_v is the velocity through vents and V is velocity of approach.

A BRIEF ON HYDROLOGY

Hydrology deals with depletion and replacement of our water resources. The basic knowledge of this science is must for Civil Engineer, particularly the one who is engaged in design planning and construction of hydraulic structures such as Bridges.

I. The Hydrologic Cycle: Most of the earth's water sources such as rivers, lakes, oceans and underground sources, etc. get their supply from the rains, while the rain water in itself is the evaporation from these sources. Water is lost to the atmosphere as vapour from the earth, which is then precipitated back in the form of rain, snow, hail, dew, sleet or frost, etc. This evaporation and precipitation continues forever and thereby, a balance is maintained between the two. This process is known as Hydrologic Cycle. It can be represented graphically, as shown in **Fig. 4.4**.



Representing Hydrologic Cycle

Fig. 4.4

II. Run-off

Run-off and surface run-off are two different items and should not be confused. Run-off includes all water flowing in the stream at any given section, and therefore it can also be named as 'Discharge of the Stream' while surface run-off includes only the water that reaches the stream channel without first percolating down to the water table. Run-off consists of the following **(Fig. 4.5)**.

(i) Direct precipitation over the surface of the stream and its distributaries, this is very small and ignored.

(ii) Surface run-off consisting of true surface run-off and sub-surface run-off.

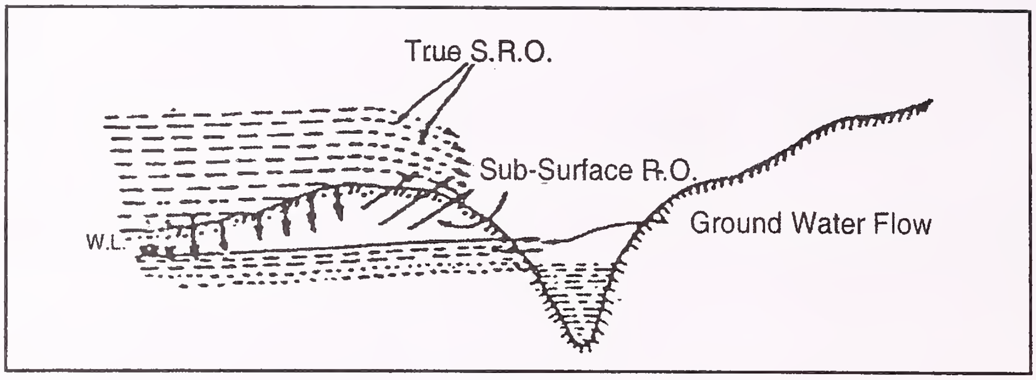


Fig. 4.5

(iii) Ground water flow.

(iv) True surface run-off :- Water that flows directly over the ground surface to the stream.

(v) Sub-surface run-off:- Water that infiltrate the soil, moves laterally and before joining water table it joins the river channel and this quantity of water is known as sub-surface run-off. It behaves nearly like a surface run-off and not like a ground water flow, because it reaches stream so quickly that it is difficult to differentiate from true surface run-off. For this reason Sub-Surface Run-off is always treated as surface run-off.

Hence, Run-off = Surface Run-off + Ground Water flow.

The ground water is often times, long delayed before it reaches the stream. It is to be further noted that Ground water flow is important for 'Minimum flow' in the stream while surface run-off is important for the 'Maximum flow' of the stream.

Run-off depends upon (a) Characteristics of drainage basin and (b) Characteristics of rainfall precipitation which further depends on following factors:

(i) *Characteristics of Drainage basin* depend upon (i) Size, (ii) Shape (fan or fern), (iii) Elevation of water shed. Besides these three important characteristics of the drainage basin, the arrangement of the stream channels formed by nature within the basin, the type of soil, the type of vegetation cover are various other factors influencing the run-off.

(ii) *Characteristics of Rainfall Precipitation* depend upon (i) Slope of Channel, (ii) Shape in plan (layout) (iii) Nature of bed (iv) Sub-soil storage characteristics of the bed and banks (v) Status of flow at commencement of precipitation.

III. The Rainfall precipitations are of following types: -

(i) Cyclonic Precipitation – Cyclonic precipitation is of two types: Tropical and Extra Tropical. The tropical cyclones originate in the open ocean and are primary source of monsoon rainfall in the country. Extra tropical cyclonic precipitation is responsible for most of the winter rains in North- Western India.

(ii) Convection Precipitation – Convection precipitation generally occurs in tropics in the form of showers of high intensity and short duration.

(iii) Orographic Precipitation :- It is most important precipitation and is responsible for most of the rains in India. Orographic precipitation is caused by air masses which strike some topographic barriers like mountains and can not move forward, hence rise up causing condensation and precipitation, and greatest amount of precipitation falls on wind ward side. A striking example of such natural barriers is in southern slopes of the hills of Meghalaya.

The rainfall is dependent on various factors and combination which are numerous such as:

(a) Duration (b) Quantum (c) Intensity (d) Direction of storm (e) Special distribution of rain over the catchments (f) Temperature and Humidity (g) Velocity and duration of wind

IV. Point/Station Rainfall

Point rainfall, also known as station rainfall refers to the rainfall data of a station. Depending upon the need, data can be listed as daily, weekly, monthly, seasonal or annual values for various periods. In practice, however, hydrological analysis requires a knowledge of the rainfall over an area, such as over a catchment. To convert the point rainfall values at various stations into an average value over a catchment, the following three methods are in use:

(a) Arithmetic-mean method,
(b) Thiessen-polygon method, and
(c) Isohyetal method.

(a) Arithmetic-Mean Method

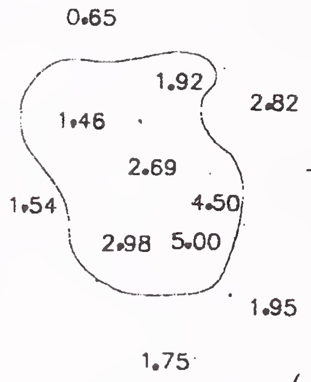
When the rainfall measured at various stations in a catchment show little variation, the average precipitation over the catchment area is taken as the arithmetic mean of the station values. Thus if $P_1, P_2, \ldots\ldots, P_n$ are the rainfall values in a given period in n stations within a catchment, then the value of the mean precipitation $\overline{P}$ over the catchment by the arithmetic-mean method is

$$\overline{P} = \frac{P_1 + P_2 + \ldots\ldots + \ \ldots\ldots + P_n}{n} = \frac{1}{n}; \sum_{i=1}^{n} P_i$$

This method is explained in **Fig. 4.6(a).**

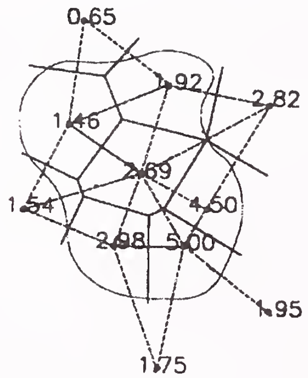
(b) Thiessen-Mean Method

In this method the rainfall recorded at each station is given a weightage on the basis of an area closest to the station. The procedure of determining the weighting area is as follows: Consider a catchment area shown in **Fig. 4.6 (b)** containing eleven raingauge stations, of which five lie outside the catchment but in its neighbourhood. The catchment area is shown and positions of the



$$\frac{1.46+1.92+2.69+4.50+2.98+5.00}{6} = 3.09 \text{ cm}$$

(a) Arithmetic mean method

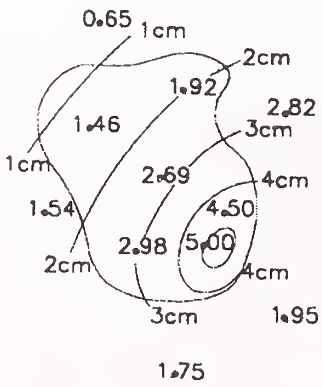


Observed precip (cm)	Area* (sq km)	Percent total area	Weighted precipiation (col.1 x col.3)
0.65	7	1	0.01
1.46	120	19	0.28
1.92	109	18	0.35
2.69	120	19	0.51
1.54	20	3	0.05
2.98	92	15	0.45
5.00	82	13	0.65
1.50	76	12	0.54
	626	100	2.84

Average = 2.84 cm

* Area of corresponding polygon within basin boundary

(b) Thiessen method



Isohyetal (cm)	Area* enclosed (sq km)	Net Area (sq km)	Avg precip. (cm)	Precipiation volume (col.3 x col.4)
5	13	13	5.0	65
4	90	77	4.5	347
3	206	116	3.5	406
2	402	196	2.5	490
1	595	193	1.5	290
<1	626	31	0.8	25
		626		1623

Average = 1623 ÷ 626 = 2.59 cm

*polygon within basin boundary

(c) Isohyetal method

Rainfall in cm at different stations shown (*)

Fig. 4.6. Areal Averaging of Precipitation

eleven stations marked on it. Stations 1 to 11 (not indicated) are joined to form a network of triangles. Perpendicular bisectors for each of the sides of the triangle are drawn. These bisectors form a polygon around each station. The boundary of the catchment, if it cuts the bisectors is taken as the outer limit of the polygon. These bounding polygons are called Thiessen Polygons. The areas of these eight Thiessen polygons are determined either with a planimeter or by using an overlay grid, If $P_1, P_2, \ldots, P_n$ are the rainfall magnitudes recorded by the stations 1, 2, ..., n respectively, and $A_1, A_2, \ldots A_n$ are the respective areas of the Thiessen polygons, then the average rainfall $\overline{P}$ over the catchment is given by

$$\overline{P} = \frac{P_1 A_1 + P_2 A_2 + \ldots + P_n A_n}{A_1 + A_2 + \ldots + A_n}$$

(c) Isohyetal Method

An isohyet is a line joining points of equal rainfall magnitude. In the isohytal method, the catchment area is drawn to scale and the raingauge stations are marked. The recorded values for which areal average P is to be determined are then marked on the plot at appropriate stations. Neighbouring stations outside the catchment are also considered. The isohyets of various values are then drawn by considering point rainfalls as guides and interpolating between them by the eye (**Fig. 4.6 (c)**). The procedure is similar to the drawing of elevation contours based on spot levels.

The area between two adjacent isohyets are then determined with a planimeter. If the isohyets go out of catchment, the catchment boundary is used as the bounding line. The average value of the rainfall indicated by two isohyets is assumed to be acting over the inter-isohyet area. Thus $P_1, P_2, \ldots, P_n$ are the values of isohyets and if $a_1, a_2, \ldots, a_{n-1}$ are the inter-isohyet areas respectively, then the mean precipitation $\overline{P}$ over the catchment area is given by

$$\overline{P} = \frac{a_1 \dfrac{(P_1 + P_2)}{2} + a_2 \dfrac{(P_2 + P_3)}{2} + \ldots + a_{n-1} \dfrac{(P_{n-1} + P_n)}{2}}{a_1 + a_2 + \ldots \ldots + a_{n-1}}$$

Fig. 4.6. Shows areal averaging of precipitation by the three methods.

V. Snowfall/Snow Melt

In India snowmelt is of importance in Himalayan region. Snowmelt run-off adds to the flood by augmenting the rainfall run-off. Two situations are usually considered:

(i) A short period accumulation of an optimum snow cover in a fairly wet drainage basin, followed by a rainfall of a maximum probable magnitude for the season.

(ii) A maximum snow accumulation and melting under a critical temperature sequence, along with a rainstorm at the time of maximum snowmelt run-off. Methods of estimation of maximum probable snowmelt flood are being developed. Some methods, which are in general use, are described in a manual published by U.S. Bureau of Reclamation.

Appendix 4.2

EMPIRICAL FORMULAE FOR CALCULATION OF DISCHARGE

(i) **Dickens' Formula:**

$$Q = CA^{3/4} \quad \dots \quad (4.2.1)$$

Where,

Q = run-off in m^3/s, A is catchment area in sq. km and C is a constant,

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

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

= 22 in Western Ghats.

(ii) **Ryve's Formula:**

$$Q = CA^{2/3} \quad \dots \quad (4.2.2)$$

Where,

Q = run-off in m^3/s, A is catchment area in sq. km and C is a constant, having values as :

C = 6.8 for areas within 25 km off the coast

= 8.5 for areas between 25 km & 160 km off the coast

= 10.0 for limited areas near the hills

(iii) **Inglis Formula**:

Col. Inglis, who was working in old Bombay Presidency, after study of the run-off and floods in the region, evolved a formula:

$$Q = \frac{125A}{\sqrt{A+10}} \quad \dots \quad (4.2.3)$$

Where,

Q = Run-off in m^3/s and

A = Catchment area in sq. km.

Appendix 4.3

DISCHARGE BY SLOPE-AREA METHOD

This method is applicable where reliable data regarding the highest level of discharge at or close to the site is available but not regarding velocity. It is generally easy to obtain highest level of discharge data by local enquiries from the oldest inhabitants in the area or by observing old flood marks on the trees and buildings near the project site. The determination of flood discharge can then be done by applying formulae for determining discharge in open channels. Site data to be collected for this purpose are:-

1. Cross-Section of the river at the site of the probable scoured bed line
2. Observation of the nature of river
3. Slope of the surface of the water in the stream noted by observations during floods or from flood marks

For this purpose three cross-section of the river should be taken one at the proposed site of the crossing, one upstream and one downstream, distances being as given in **Table 4.2** below.

Table 4.2: Spacing of Cross–Section on Streams

Catchment Area of stream/River	Distance apart for Cross-Section
3.0 km^2 or less	100 m (Scale not less than 1 cm to 10 m or (1/1000)
3.0 to 15.0 km^2	300 m (Scale not less than 1 cm to 10 m or 1/1000)
Over 15.0 km^2	One half km or width between the banks whichever is more (Scale 1 cm to 50 m or 1/5000

The average of the three cross-sectional areas should be used for computation. Where a number of cross-section have been taken, the mean is arrived as follows.

$$A = \frac{A_1 + 2A_2 \text{———} + 2A_{n-1} + A_n}{2(n-1)}$$

Where,

'n' number of cross sections, 'A' mean area of flow in the stream, $'A'_1$, $'A'_2$ etc. areas of flow at different cross-sections.

When the cross-section is not plotted to the natural scale (the same scale horizontally and vertically), the wetted perimeter (P) cannot be scaled 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. as shown in **Fig. 4.7.**

Consider one such part say PQ, let PR or QR be its horizontal and vertical projections. The PQ= $(PR^2 + QR^2)$. Now PR can be measured on the horizontal scale of the given

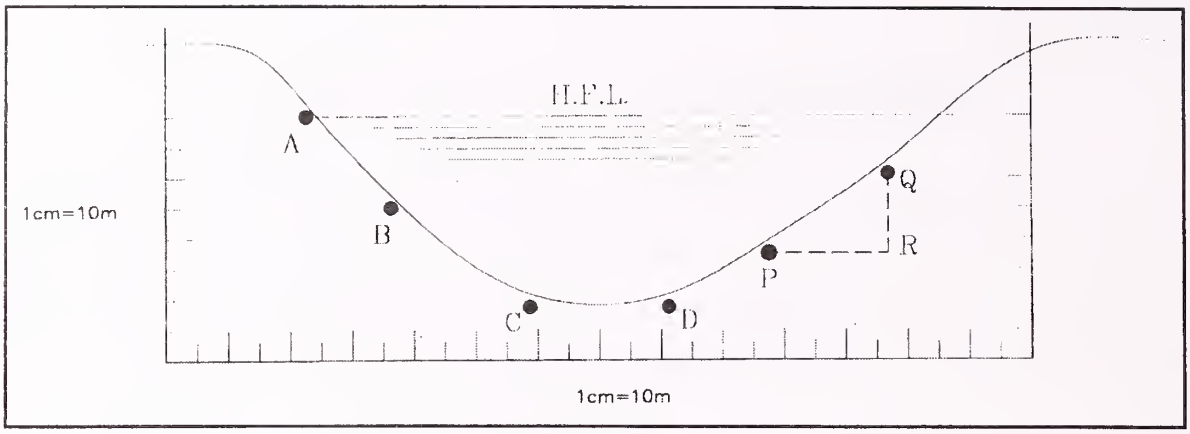


Fig. 4.7

cross-section and QR on the vertical scale. PQ can then be calculated. Similarly the length of each part is calculated. Their sum gives the wetted perimeter.

The velocity of discharge is calculated by **Chezy Formula** or **Manning's formula.** Generally, **Manning's formula** (in metric units) which is simpler, is used

$$V = \frac{1}{\eta} R^{2/3} S^{1/2} \qquad \ldots(4.1)$$

Where,

V = Velocity of flow in m/sec considered uniform throughout the section

R = Hydraulic mean depth that is A/P (in m)

η = Rugosity coefficient

S = Flood slope of the river usually taken as bed slope in absence of precise data **(Fig 4.8).**

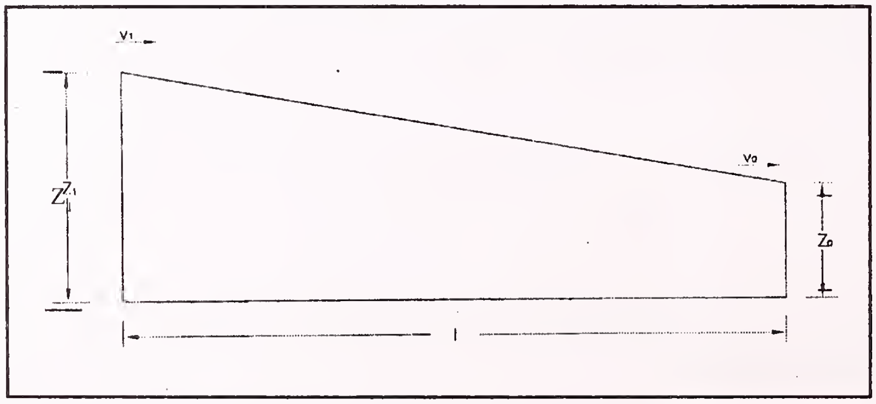


Fig. 4.8

Slope S may be corrected for the kinetic energy difference at the two ends and is given by:

$$S = \frac{Z_1 - Z_0 + \left[\frac{V_1^2 - V_0^2}{2g}\right]}{l} \qquad \ldots(4.2)$$

The second term is kinetic energy difference, which is negligible and can be neglected where the reach is sufficiently long or the slope is not too flat.

The value of rugosity coefficient 'η' is given in the following **Table 4.3.**

Table 4.3: Value of η (Rugosity Coefficient)

Sl.No.	Surface	Perfect	Good	Fair	Bad
1.	Natural Streams 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 deep pools	0.05	0.06	0.07	0.08
8.	Very weedy reaches	0.075	0.1	0.125	0.15

Calculation of Discharge (Q)

$$Q=AV \qquad \ldots (4.3)$$

$$V = \frac{1}{\eta} R^{2/3} S^{1/2} = \lambda S^{1/2}$$

Where,
$$\lambda = \frac{1}{\eta} R^{2/3}$$

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

If the shape of the cross-section is irregular as happens when a stream rises its banks and shallow overflows are created, it is necessary to sub-divide the channel into two or three subsections. Then 'R' and 'η' are found for each sub-section, and their velocities and discharges are computed separately and then added together to get the total discharge.

Appendix 4.4

DISCHARGE BY RATIONAL METHOD

A rational method for estimation of flood discharge should take into account the intensity, distribution and duration of rainfall as well as characteristics of the catchment area. It should also take into account the discharge characteristics of the catchment area which depend on its shape, slope, permeability and initial wetness of the catchment.

1. Govardhanlal, in his method, applied the following Rational formula where by knowing the highest observed rain fall at a representative gauging station in an hour and knowing characteristics of the catchment area and rainfall, one find the discharge safely for areas upto 500 sq. km.

   The formula is as follows

$$Q = A\ I_0\ \lambda \qquad \dots (4.4)$$

Where,

- Q = Maximum flood discharge in m^3/s
- A = Catchment area in hectare
- I_0 = Max. intensity of rainfall in centimeter per hour
- λ = Function depending upon characteristics of the Catchment in producing peak run-off and given by

$$\lambda = \frac{0.056\, f\, P}{t_c + 1} \qquad \dots (4.5)$$

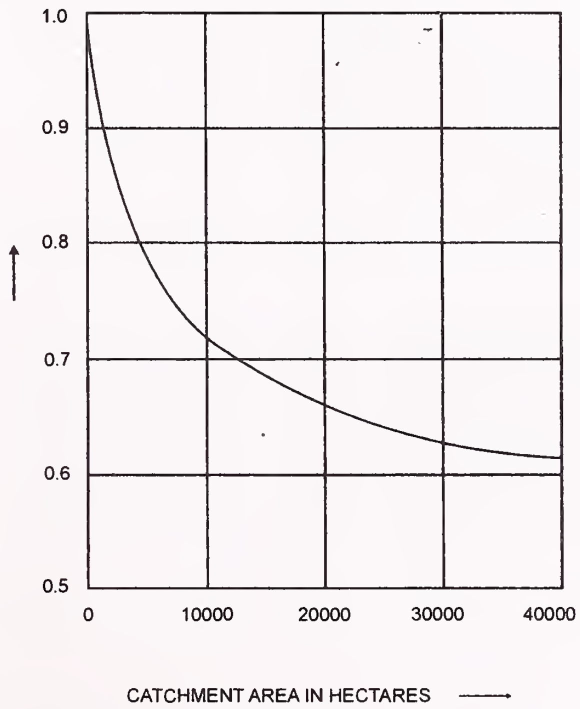
- P = percentage coefficient of surface run-off for the catchment characteristics as given in (**Table 4.4**). Considerable judgment and experience are called for in assessment value of P. Any error in the later will diminish the reliability of the results of laborious calculations involved in this method.
- f = Factor to correct for the variation of intensity of rainfall over the area of the catchment. (**Graph 4.1**).
- t_c = time of concentration in hours

2. **Estimation of time of Concentration (t_c)**

   It is the time taken by the run-off from the farthest point on the periphery of the Catchment (called the critical point) to reach site of Bridge. The concentration time depends on (i) the distance from the critical point upto the Bridge site and (2) the average velocity of flow which depends upon the slope, the roughness of drainage channel and depth of flood. Complicated formulae exists for determining the time of concentration (t_c) from characteristics of the catchment.

Table 4.4: Maximum Value of P in the Formula

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- lightly cultivated	0.30
Sandy soil, light growth	0.20
-do- covered, heavy bush	0.10



f curve

Graph 4.1

3. The time of concentration (t_c) can be obtained by using the State of California formula. This formula has also been recommended for application in India, in IRC:SP:13, para 4.7.5.2 and which is as follows:

$$t_c = 0.87 \times \left(\frac{L^3}{H} \right)^{0.385} \qquad \ldots (4.6)$$

Where,

L = distance from the farthest point in a catchment to the site in km

H = fall in level from the farthest point to the bridge site in m and t_c is the time of concentration

The value of A,L, and H can be obtained from Survey of India Topographical maps. I_0 has to be obtained from Meteorology Department. I_0 of region have not to be found for each design problem, it is characteristic of the whole region and applies to pretty vast areas having the same weather conditions. I_0 of a region should be found once for all and should be known to local engineers. The Metrological Department, Govt. of India have data for heaviest rainfall in centimeter/hour collected for various places in India and are to be obtained from them.

Rational Method may be applied safely for areas upto 500 sq. km and upto 2000 sq. km in extreme cases. The use of Rational Method for small catchments have been advised in IRC:SP:13 vide clause 4.7.14 stating that since the average designer cannot rely so much on his judgement and intuition for selecting value of 'C' in Empirical formulae he should adopt 'Rational Method' which has been outlined in detail in IRC:SP:13, Para 4.7.

Appendix 4.5

PROCEDURE FOR ESTIMATION OF DESIGN FLOOD BY UNIT HYDROGRAPH METHOD

A typical example with reference to Bridge catchment (treated as ungauged) is worked out for illustrating the procedure to compute 50-year design flood and is given below:

The particulars of the catchment under study are as under:

(i)	Name & number of sub-zone	:	Mahi & Subarmati Sub-zone-3(a)*
(ii)	Name of site (i.e. point of study)	:	Bridge No. 129
(iii)	Name of section	:	Dehod-Ratlam
(iv)	Name of tributary	:	Kali Nadi
(v)	Shape of the catchment	:	Oblong
(vi)	Site location	:	Latitude-22°52` 00" Longitude-74° 22` 00"
(vii)	Topography	:	Moderately steep slope

Note: *(See Annexure A for Sub-zones of river systems)

The procedure comprises of assessing A (area), L (Length of longest channel), S (Equivalent Slope of channel) and then finding out Synthetic unit hydrograph ordinates by using these values and relationship derived by research study for the sub-zone concerned. Estimation of effective rainfall unit is done using 50 year 24 hour point rain fall values given in the report for design storm duration, calculated earlier considering design loss rate. With unit hydrograph & 1 hour effective rainfall, 50 year flood is estimated considering design base flow.

The procedure is explained stepwise as follows:

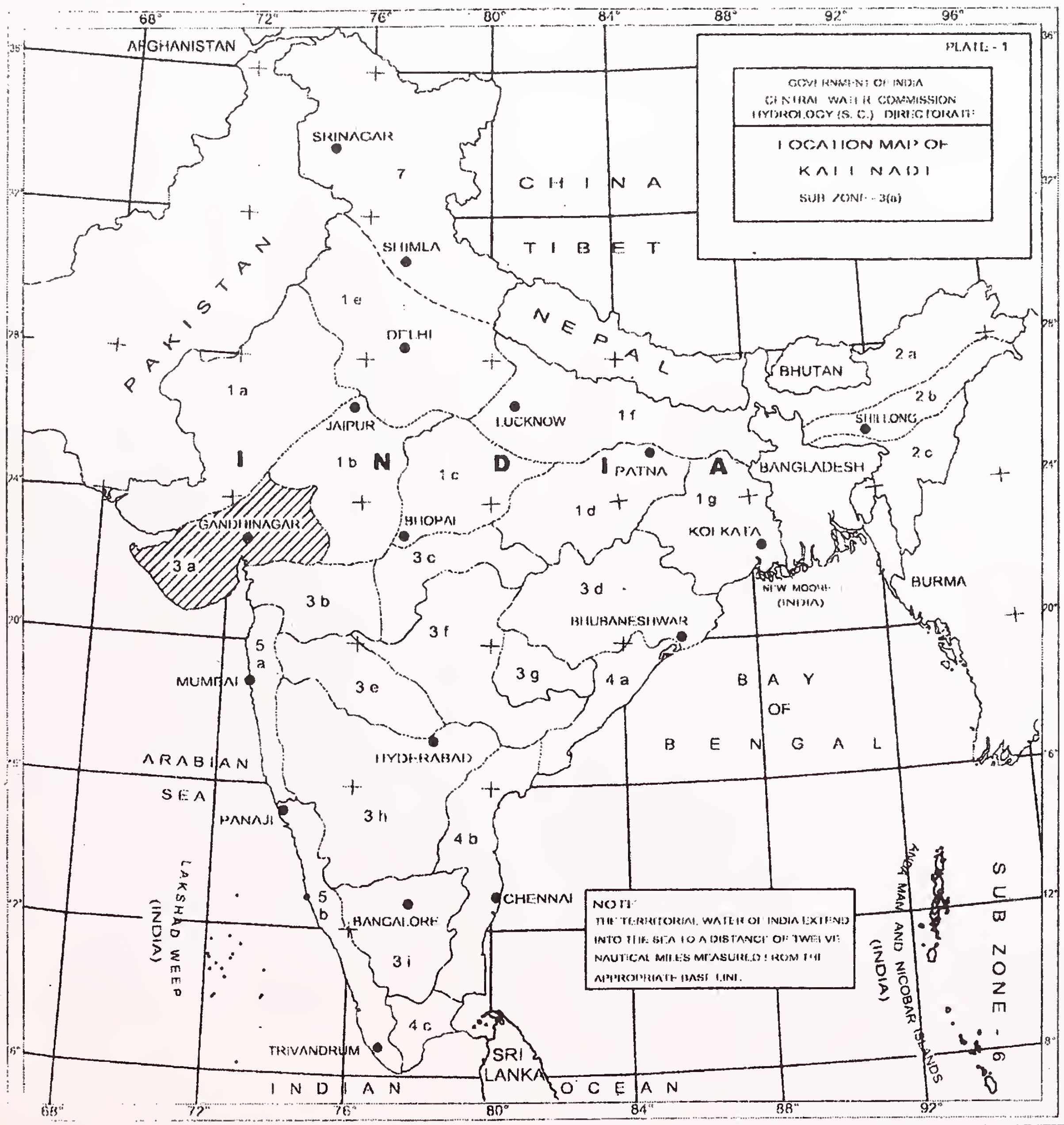
Step I: Preparation of Catchment Area Plan

The point of interest (i.e. bridge site in this case) was located on the Survey of India toposheet and catchment boundary was marked using the contours along the ridge line and also from the spot levels in the plains. A catchment area plan **(Plate 1)** showing the rivers, contours and spot levels was prepared.

Step: 2: Determination of Physiographic Parameters

The following physiographic parameters were determined from the catchment area plan i.e. **Plate 1.**

(i)	Area (A)	:	136.36 sq. km
(ii)	length of the longest stream (L)	:	33.50 km



LIST OF FLOOD ESTIMATION REPORTS PUBLISHED

A. UNDER SHORT TERM PLAN

1.	Estimation of design Flood Peak	(1973)

B UNDER LONG TERM PLAN

1.	Lower Ganga Plains subzone - 1 (g)	(1978)
2	Lower Godavari subzone - 3 (f)	(1981)
3.	Lower Narmada & Tapi subzone - 3 (b)	(1982)
4	Mahanadi subzone - 3 (d)	(1982)
5	Upper Naramada & Tapi subzone - 3 (c)	(1983)
6.	Krishna & Pennar subzone - 3 (h)	(1983)
7.	South Brahmaputra subzone - 2 (b)	(1984)
8.	Upper Indo-Ganga Plains subzone - 1 (e)	(1984)
9.	Middle Ganga Plains subzone - 1 (f)	(1985)
10.	Kaveri Basin subzone - 3 (i)	(1986)
11.	Upper Godavari subzone - 3 (e)	(1986)
12.	Mahi & Sabarmati subzone - 3 (a)	(1987)
13	East Coast subzone - 4 (a), (b) & (c)	(1987)
14.	Sone subzone - 1 (d)	(1988)
15.	Chambal subzone - 1 (b)	(1989)
16.	Betwa subzone - 1 (c)	(1989)
17	North Brahmaputra subzone - 2 (a)	(1991)
18	West Coast Region subzone - 5 (a) & (b)	(1992)
19	Luni subzone - 1 (a)	(1993)
20.	Indravati subzone - 3 (g)	(1993)
21.	Western Himalayas zone - 7	(1994)

C. REVISED UNDER LONG TERM PLAN

1	Lower Ganga Plains subzone - 1 (g)	(1994)
2.	Lower Godavari subzone - 3 (f)	(1995)

REPORTS BROUGHT OUT
BY THE CENTRAL WATER COMMISSION

Annexere–A of Appendix 4.5

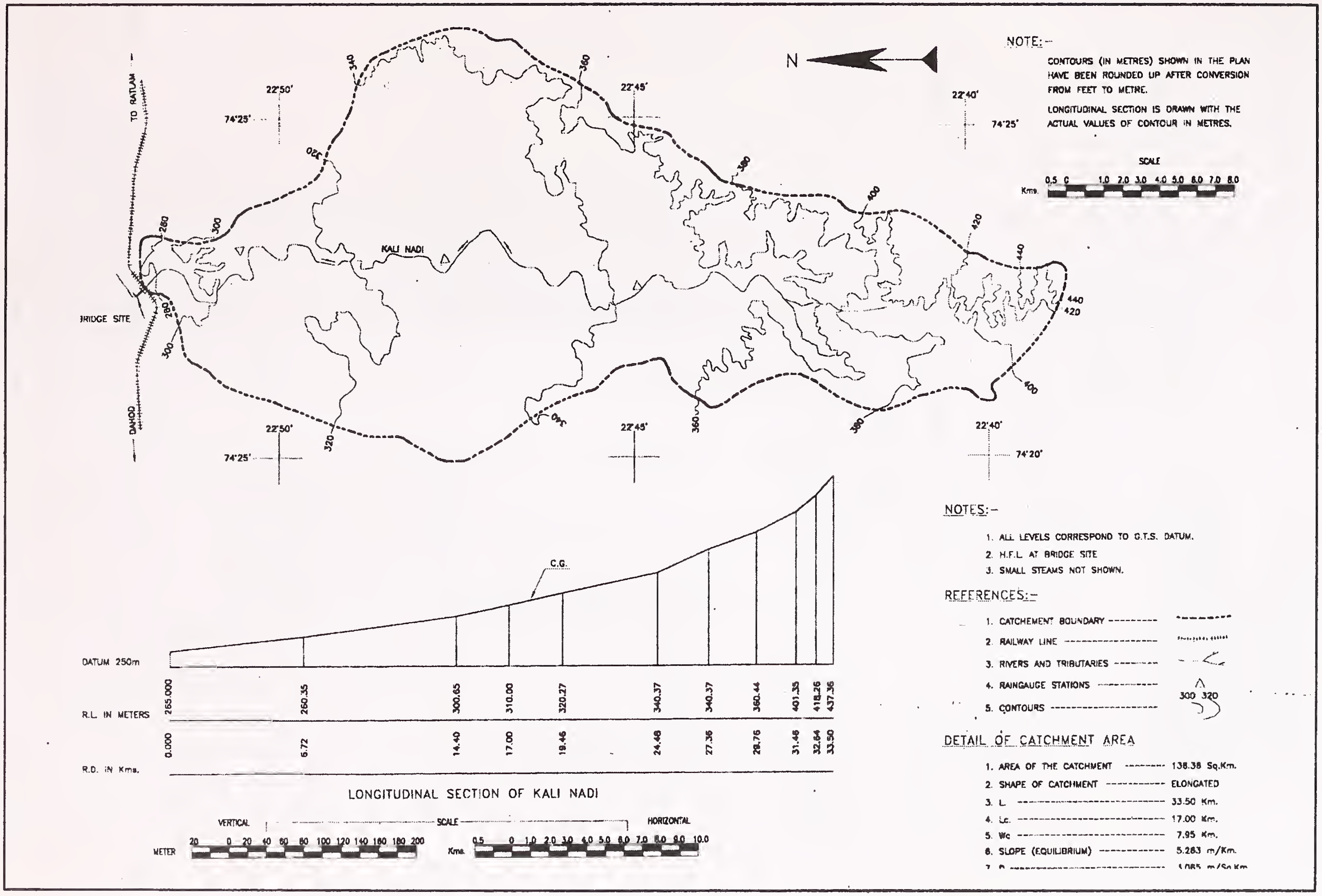

NOTE:-
CONTOURS (IN METRES) SHOWN IN THE PLAN HAVE BEEN ROUNDED UP AFTER CONVERSION FROM FEET TO METRE.
LONGITUDINAL SECTION IS DRAWN WITH THE ACTUAL VALUES OF CONTOUR IN METRES.
SCALE
Kms. 0.5 0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0
N
TO RATLAM
DAHOD
BRIDGE SITE
KALI NADI
22°50'
74°25'
22°45'
22°40'
74°20'
280
300
320
340
360
380
400
420
440
NOTES:-
1. ALL LEVELS CORRESPOND TO G.T.S. DATUM.
2. H.F.L. AT BRIDGE SITE
3. SMALL STEAMS NOT SHOWN.
REFERENCES:-
1. CATCHEMENT BOUNDARY
2. RAILWAY LINE
3. RIVERS AND TRIBUTARIES
4. RAINGAUGE STATIONS
5. CONTOURS
300 320
DETAIL OF CATCHMENT AREA
1. AREA OF THE CATCHMENT ---- 138.38 Sq.Km.
2. SHAPE OF CATCHMENT ---- ELONGATED
3. L ---- 33.50 Km.
4. Lc ---- 17.00 Km.
5. Wc ---- 7.95 Km.
6. SLOPE (EQUILIBRIUM) ---- 5.283 m/Km.
C.G.
DATUM 250m
R.L. IN METERS
265.000
260.35
300.65
310.00
320.27
340.37
340.37
360.44
401.35
418.26
437.36
R.D. IN Kms.
0.000
6.72
14.40
17.00
19.46
24.48
27.36
28.76
31.48
32.64
33.50
LONGITUDINAL SECTION OF KALI NADI
VERTICAL
SCALE
HORIZONTAL
METER 20 0 20 40 60 80 100 120 140 160 180 200
Kms. 0.5 0 1.0 2.0 3.0 4.0 5.0 6.0 7.0 8.0 9.0 10.0

Plate 1

(iii) Length of the longest stream from a point opposite to centroid (C.G.) of the catchment area to the gauging site along the main stream (Lc) : 17.00 km

(iv) Equivalent stream slope (S) : 3.26 m /km – For determining (S), observe reduced level of river bed at different points starting from bridge site.

Following methods are adopted for computation of equivalent stream slope (S):

(a) By Mathematical Calculation:

The computation of (S) with reference to **Plate 1** is explained in the **Table 4.5** below:

Table 4.5

Sl. No.	Distance starting from bridge site (km)	Reduced level of river bed (m)	Length of each segment, L_i (km)	Height above the datum*, D_i (m)	$D_{i-1} + D_i$ (m)	$L_i(D_{i-1} + D_i)$ (km x m)
1.	0	265.00	0	0	0	0
2.	6.72	280.33	6.72	15.33	15.33	103.02
3.	14.40	300.65	7.68	35.65	50.98	391.53
4.	19.68	320.97	5.28	55.97	91.62	483.75
5.	24.48	340.57	4.80	75.57	131.54	631.39
6.	27.36	362.21	2.88	97.21	172.78	497.61
7.	29.76	380.44	2.40	115.44	212.65	510.36
8.	31.68	401.95	1.92	136.95	252.39	484.59
9.	32.64	418.26	0.96	153.26	290.21	278.60
10.	33.50	437.96	0.86	179.96	326.22	280.55
					Sum =	3661.40

Note:
*Datum is reduced level of river bed at point of study = 265.00 m

$$\text{Equivalent Stream Slope} = \frac{\Sigma L_i(D_i\text{-}1+D_i)}{L^2} = \frac{3661.40}{(35.5)^2} = 3.26 \text{ m/km}$$

(b) By Graphical Method

Draw a longitudinal section of the longest main stream from contours crossing the stream and the spot levels along the banks from the source to the point of study from the catchment plan shown in **Plate 1.** Draw a sloping line by trial on the L-Section line from the point of study such that the area above and below the L-Section line are equal. Then compute the slope of this line which gives the equivalent stream slope (S).

Step 3: Determination of 1-hour Synthetic Unit Hydrograph Parameters:

The following relationships have been derived from the studies carried out by CWC & IMD for estimating the 1-hour unit hydrograph parameters for an ungauged catchment in the subzone 3(a). (Similar relationship for other sub-zones are also prepared and may be seen in reports of relevant sub-zones)

(i) $t_p = 0.433\ (L/\sqrt{S})^{0.704}$

(ii) $q_p = 1.161\ (t_p)^{-0.635}$

(iii) $W_{50} = 2.284(q_p)^{-1.00}$

(iv) $W_{75} = 1.331\ (q_p)^{-0.991}$

(v) $W_{R50} = 0.827(q_p)^{-1.023}$

(vi) $W_{R75} = 0.561(q_p)^{-1.037}$

(vii) $T_B = 8.375(t_p)^{0.512}$

(viii) $T_m = t_p + t_r / 2$

(ix) $Q_p = q_p \times A$

Where,

t_p = Time from the center of unit rainfall duration to the peak of unit hydrograph in hour

q_p = Peak discharge of unit hydrograph per unit area in cumecs/sq.km

W_{50} = Width of unit hydrograph measure at 50 per cent max discharge ordinate (Q_p) in hour

W_{75} = Width of the unit hydrograph measured in 75 per cent max discharge ordinate (Q_p) in hour

W_{R50} = Width of the rising side of the unit hydrograph measured at 50 per cent of maximum discharge ordinate (Q_p) in hour

W_{R75} = Width of the rising side of the unit hydrograph measured at 75 per cent of maximum discharge ordinate (Q_p) in hour.

T_b = Base width of unit hydrographs in hour.

T_m = Time from the start of rise of the peak of unit hydrograph in hour.

Q_p = Peak discharge of unit hydrograph in cumecs.

L = Length for longest main stream along the river course in km.

S = Equivalent stream slope in m/km.

t_r = Unit rainfall duration adopted in a specific study in hour

A = Catchment area in sq.km.

The above equations were used to compute the unit hydrograph parameters with the known values of A, L and S as under:

(i)	t_p	$= 0.433\ (33.50/\sqrt{3.26})^{0.704}$	=	3.38 hrs, rounded off to 3.5 hour
(ii)	q_p	$= 1.161\ (3.5)^{-0.635}$	=	0.254 cumecs/sq.km
(iii)	W_{50}	$= 1.331\ (0.254)^{-1.000}$	=	4.36 hour
(iv)	W_{75}	$= 0.827(0.254)^{-0.991}$	=	2.52 hour
(v)	W_{R50}	$= 0.561(0.254)^{-1.023}$	=	1.60 hour
(vi)	T_{R75}	$= 8.375(0.254)^{-1.037}$	=	1.10 hour
(vii)	T_B	$= 8.375\ (3.5)^{0.512}$	=	15.90 hour say 16 hour
(viii)	T_m	$= 3.5 + 1/2$	=	4.0 hour
(ix)	Q_p	$= 0.254 \times 136.36$	=	71.50 cumecs

Step 4: Drawing of a Synthetic Unit Hydrograph

Estimated parameters of unit hydrograph in Step 3 were plotted to scale on a graph paper as shown in **Plate 2.** The potted points were joined to draw a synthetic hydrograph. The discharge indicated by unit hydrograph is calculated graphically by $\Sigma Q_t t_t$. This discharge is compared with theoretical discharge given by formula $Q_{th} = Ad/0.36 \times t$. The above two values of discharge i.e. obtained by graphical and theoretical method are compared. In case graphical value is different from theoretical, the limb of synthetic unit hydrograph is adjusted to make it equal to theoretical value. The discharge ordinates (Q_i) of the unitgraph at t_i=t_r=1 hr interval were summed up and multiplied by t_r=1 i.e. $\Sigma Q_t t_t = 378.8$ cumecs as shown in **Plate 2.** The theoretical volume of 1.00 cm direct run-off depth over the catchment was computed from the formula.

$$Q_{th} = A \times d/0.36 \times t = 136.36 \times 1/\ 0.36 \times 1 = 378.8 \text{ cumecs}$$

Where,

A = catchment area in sq km

d = 1.0 cm depth

t_i = t_r (the unit duration of the UG) = 1.00 h

Therefore, the sum of 1 hr U.G. = 378.8 cumecs. The sum of hourly ordinates ($\Sigma Q_t \times t_t$) of 1 h U.G. was compared with the sum of 1 h U.G. obtained from the above formula.

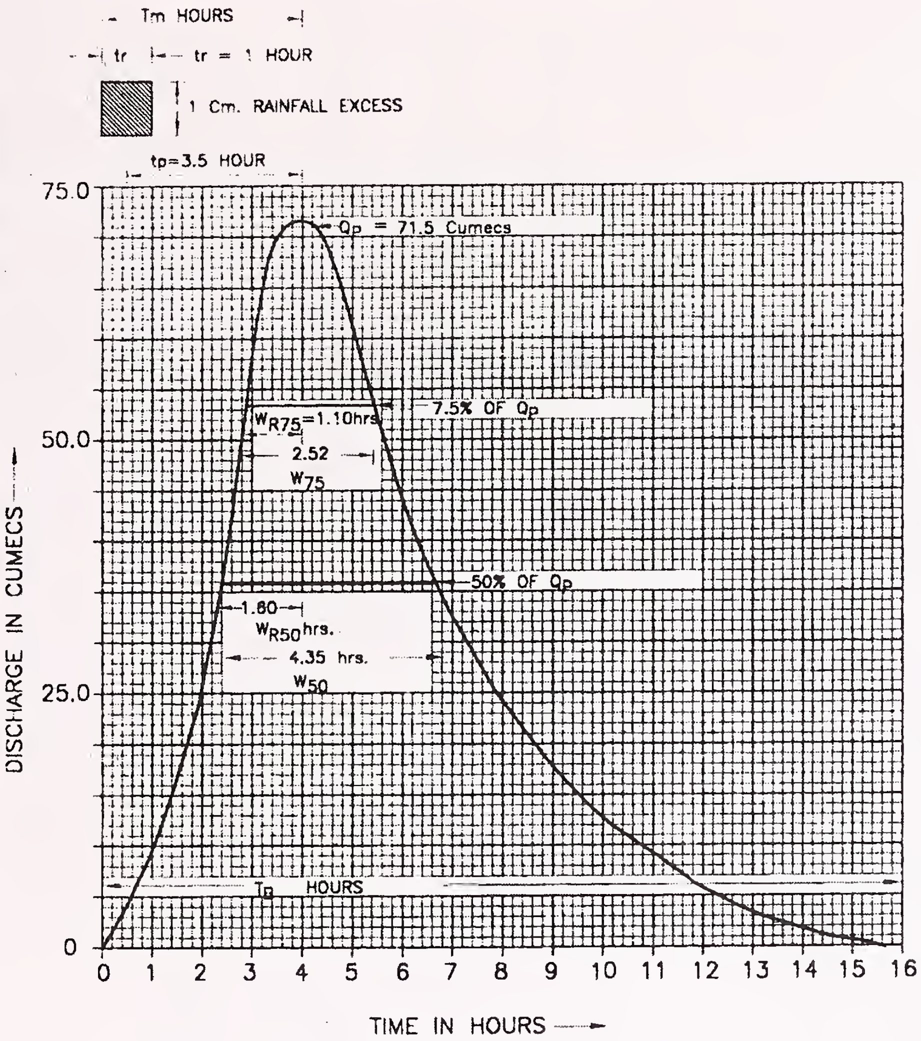
In case the $\Sigma Q_t t_t$ for the unit hydrograph drawn is higher or lower than the volume worked out by the above formula, then the falling limb and/or rising limb may be suitably modified to get the correct volume under the hydrograph, taking care to get the smooth shape of the unit hydrograph.

Step 5: Estimation of Design Storm Duration

The design storm duration $(T_p) = T_b = 16.00$ h

Step 6: Estimation of Point Rainfall and Areal rainfall for storm duration

The site under study was located on **Plate 3** of this Annexure showing 50-year 24-h point rainfall. 50 year 24-h point rainfall=32.0 cm. Conversion factor of 0.905 was read from



SYNTHETIC U.G. PARAMETERS

C.A. = 136.36 SQ. KM.

t_p = 3.5 hrs.

T_B = 16.0 hrs.

Q_p = 71.5 Cumecs.

W_{50} = 4.36 hrs.

W_{75} = 2.52 hrs.

W_{R50} = 1.60 hrs.

W_{R75} = 1.10 hrs.

q_p = 0.524 Cumecs/Sq. Km.

$\frac{L}{\sqrt{S}}$ = 19.0

Tm = 4.0 hrs.

d = 1.00 Cm.

$$\Sigma Q_j = \frac{A \times d}{tp \times 0.36}$$

$$= \frac{1.36 \times 1.00}{1.00 \times 0.36}$$

= 378.80 Cumecs.

TIME IN (HOURS)	1 HOUR SYNTHETIC U.G. ORDINATES (CUMECS)
0	0
1	9.00
2	25.50
3	58.00
4	71.50
5	61.00
6	44.50
7	32.70
8	24.50
9	18.00
10	12.70
11	9.50
12	5.70
13	3.50
14	1.90
15	0.80
16	0
TOTAL	378.80

Plate 2. Synthetic Unit Hydrograph

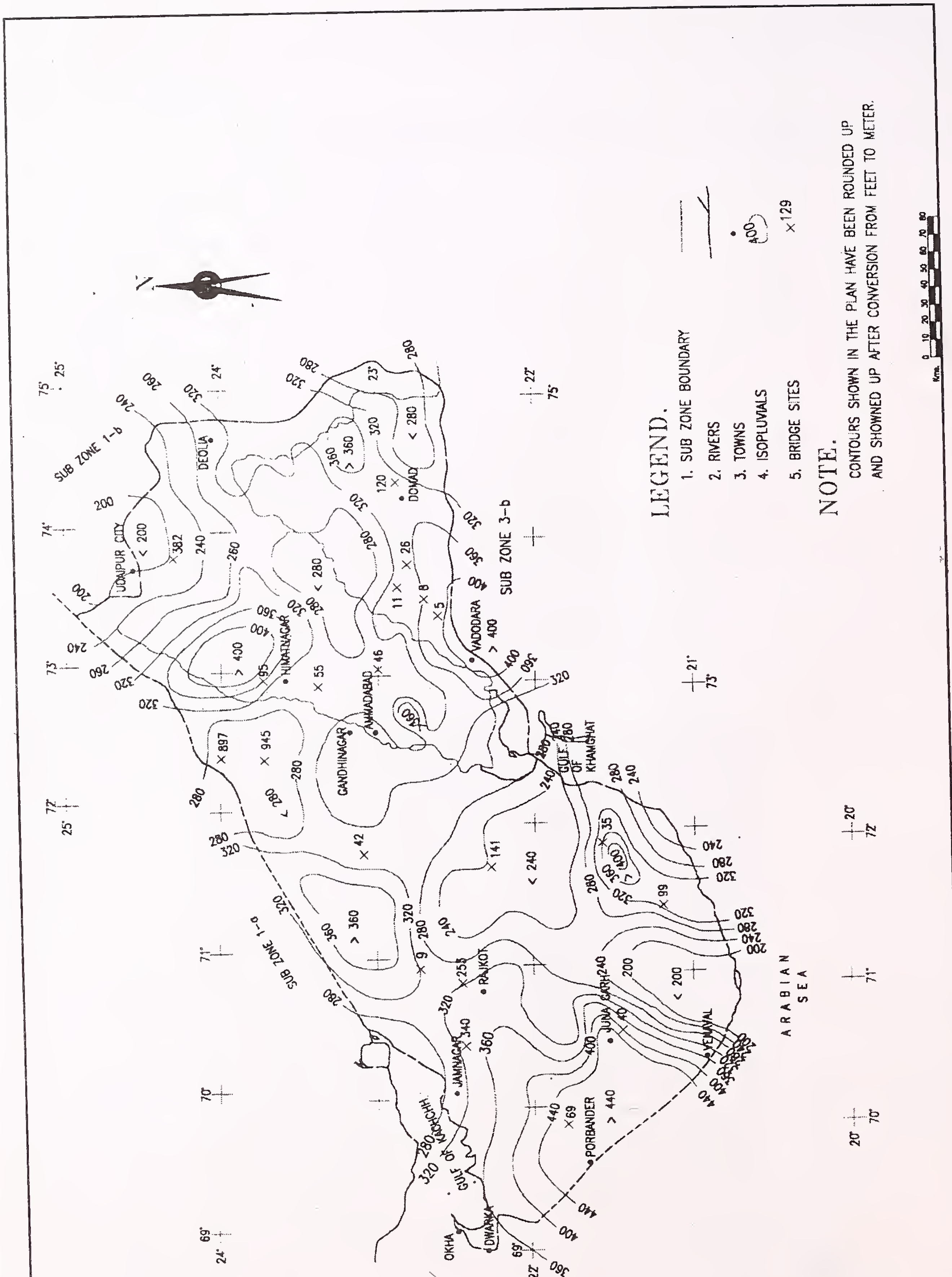

LEGEND.
1. SUB ZONE BOUNDARY
2. RIVERS
3. TOWNS
4. ISOPLUVIALS
5. BRIDGE SITES
NOTE.
CONTOURS SHOWN IN THE PLAN HAVE BEEN ROUNDED UP AND SHOWNED UP AFTER CONVERSION FROM FEET TO METER.
SUB ZONE 1-b
SUB ZONE 3-b
SUB ZONE 1-a
UDAIPUR CITY
DEOLIA
DOHAD
HIMATNAGAR
VADODARA
AHMADABAD
GANDHINAGAR
GULF OF KHAMBHAT
RAJKOT
JAMNAGAR
JUNAGARH
PORBANDER
VERAVAL
OKHA
DWARKA
GULF OF KACHCHH
ARABIAN SEA

Plate 3

Plate 4 for conversion of 50 year 24-h point rainfall to 50 year 16-h point rainfall since T_D = 16 h 50 year 16 h point rainfall thus worked out to be 32.00 x 0.905 = 28.96 cm. Areal reduction factor of 0.915 corresponding to a catchment area of 136.36 sq.km for T_D = 16 h was interpolated from **Fig. 4.9** for conversion of point to areal rainfall. 50 year 16 h areal rainfall = 28.96 x 0.915 = 26.50 cm.

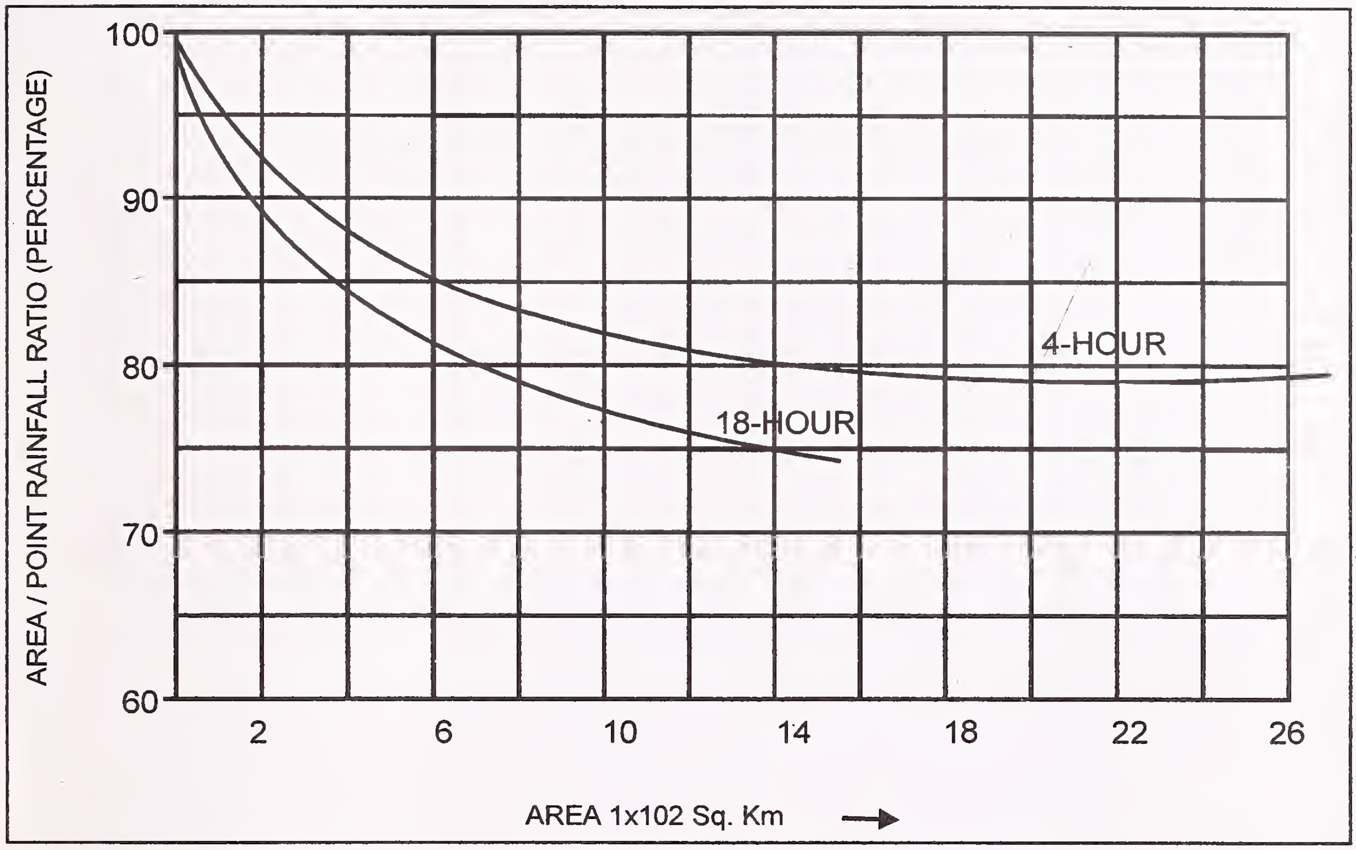


Fig. 4.9

Step 7: Time Distribution of Areal Raifnall

50-year 16-h areal rainfall = 26.50 cm was distributed with the distribution coefficients (Col.16 of **Table 4.6)** to get 1 h rainfall increments as follows:

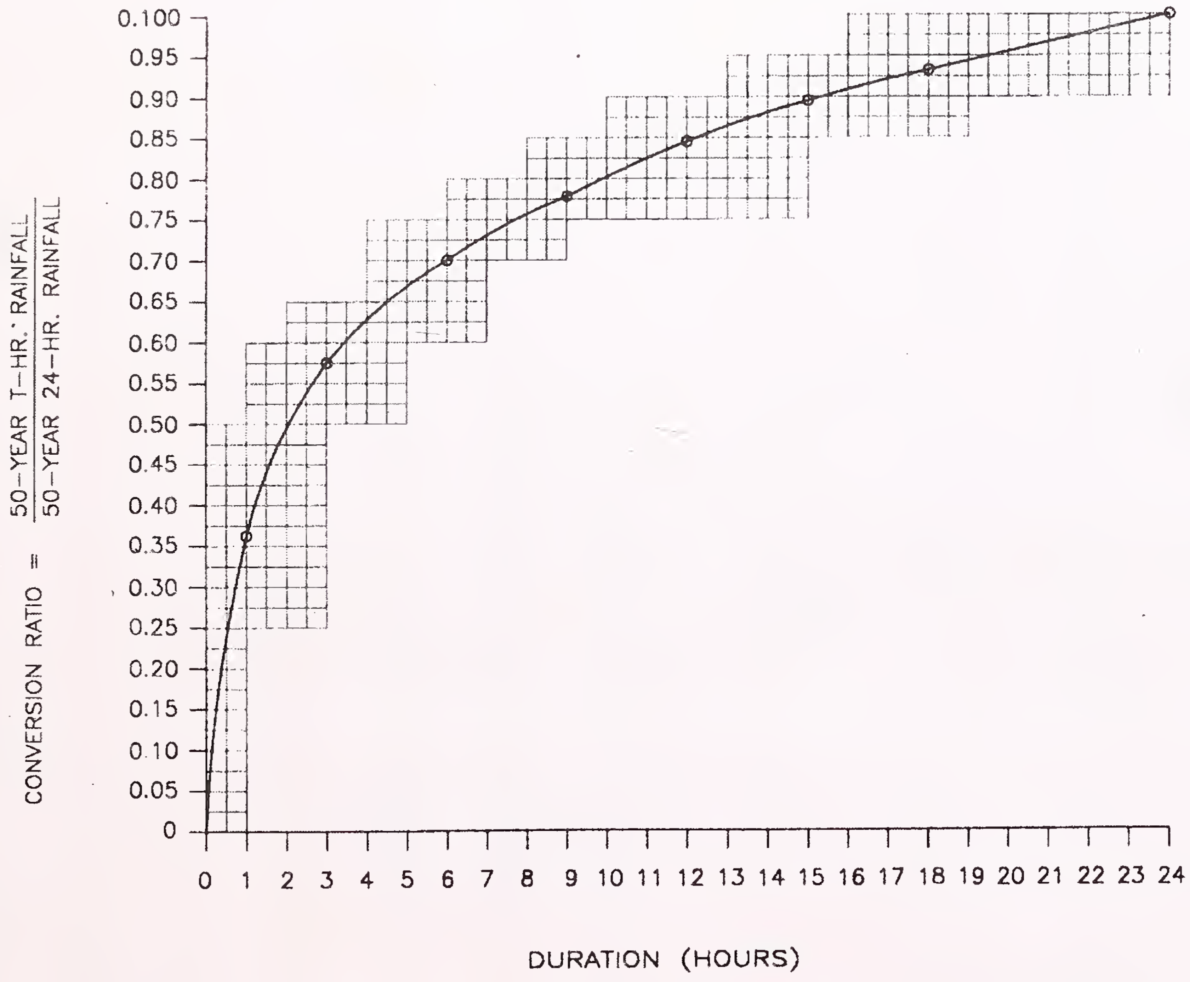
The hourly rainfall increments in col. (4) of the above table were obtained by subtracting the successive rainfall values from 1 h onwards.

Step 8: Estimation of Effective Rainfall Units

Design loss rate of 0.45 cm/h has been adopted for this sub-zone

Table 4.7 the computation of 1 h effective rainfall units in Col. (4) by subtracting the design loss rate in Col. (3) from 1-h rainfall increments in Col. (2).

The Column (2) in **Table 4.8** is taken from col. (4) of **Table 4.7**



RATIOS OF 24-HR. POINT RAINFALL TO SHORT DURATION RAINFALL.

DURATION (HRS.)	RATIO
1	0.360
2	0.505
3	0.570
4	0.620
5	0.665
6	0.700
7	0.730
8	0.760
9	0.780
10	0.805
11	0.825
12	0.850
13	0.860
14	0.875
15	0.890
16	0.905
17	0.920
18	0.930
19	0.945
20	0.960
21	0.970
22	0.980
23	0.990
24	1.000

Plate 4

Table 4.6: Time Distribution Coefficients of Areal, Rainfall, Mahi and Sabarmati Basin, Subzones (a)

Time in Hour			Distribution			Coefficient for Design Storm Duration of 2-24 h																		
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)
24																							1.00	24
23																						1.00	0.99	23
22																					1.00	0.99	0.98	22
21																				1.00	0.99	0.98	0.97	21
20																			1.00	0.98	0.98	0.97	0.95	20
19																		1.00	0.99	0.97	0.96	0.95	0.94	19
18																	1.00	0.98	0.98	0.96	0.95	0.94	0.93	18
17																1.00	0.99	0.97	0.97	0.95	0.93	0.92	0.90	17
16															1.00	0.99	0.97	0.96	0.95	0.93	0.90	0.89	0.87	16
15														1.00	0.99	0.98	0.96	0.94	0.93	0.90	0.88	0.86	0.84	15
14													1.00	0.98	0.98	0.97	0.94	0.91	0.89	0.88	0.85	0.84	0.82	14
13												1.00	0.98	0.97	0.96	0.94	0.93	0.88	0.86	0.84	0.83	0.80	0.78	13
12											1.00	0.98	0.97	0.95	0.94	0.93	0.90	0.85	0.84	0.82	0.80	0.78	0.77	12
11										1.00	0.99	0.96	0.94	0.92	0.91	0.89	0.87	0.82	0.80	0.78	0.77	0.76	0.74	11
10									1.00	0.99	0.98	0.94	0.92	0.91	0.88	0.86	0.85	0.78	0.76	0.75	0.74	0.73	0.71	10
9								1.00	0.99	0.97	0.96	0.94	0.89	0.87	0.86	0.83	0.82	0.76	0.74	0.72	0.71	0.69	0.67	9
8							1.00	0.98	0.99	0.96	0.93	0.87	0.86	0.83	0.82	0.80	0.76	0.72	0.70	0.68	0.67	0.65	0.63	8
7						1.00	0.98	0.96	0.95	0.92	0.88	0.83	0.82	0.79	0.77	0.76	0.73	0.66	0.65	0.63	0.62	0.61	0.59	7
6					1.00	0.98	0.96	0.93	0.90	0.86	0.84	0.78	0.76	0.75	0.73	0.70	0.68	0.63	0.61	0.59	0.58	0.57	0.55	6
5				1.00	0.97	0.96	0.92	0.86	0.84	0.82	0.79	0.72	0.70	0.68	0.66	0.64	0.62	0.56	0.55	0.53	0.52	0.51	0.50	5
4			1.00	0.96	0.93	0.87	0.84	0.82	0.77	0.75	0.73	0.66	0.64	0.62	0.60	0.58	0.57	0.50	0.48	0.47	0.46	0.43	0.42	4
3		1.00	0.95	0.93	0.87	0.80	0.74	0.63	0.71	0.67	0.65	0.58	0.56	0.55	0.53	0.52	0.51	0.41	0.40	0.39	0.38	0.36	0.35	3
2	1.00	0.94	0.88	0.82	0.76	0.68	0.66	0.60	0.58	0.57	0.52	0.47	0.45	0.43	0.41	0.40	0.39	0.33	0.32	0.31	0.30	0.28	0.27	2
1	0.87	0.75	0.68	0.61	0.54	0.50	0.43	0.42	0.39	0.37	0.36	0.32	0.30	0.29	0.28	0.24	0.22	0.22	0.21	0.18	0.16	0.14	0.13	1

Note: Hourly rainfall distribution coefficients are given in the vertical columns for various design storm durations from 2 to 24h

Table 4.7

Durations (h)	Distribution Co-efficients (Col. 16 of Table 2)	Storm rainfall= Rainfall x Distribution coefficient (cm)	1-h rainfall increment (cm)
(1)	(2)	(3) = (2) x 26.50	(4)
1.	0.28	7.42	7.42
2.	0.41	10.86	3.44
3.	0.53	14.04	3.18
4.	0.60	15.90	1.86
5.	0.66	17.49	1.59
6.	0.73	19.34	1.85
7.	0.77	20.40	1.06
8.	0.82	21.73	1.33
9.	0.86	22.79	1.06
10.	0.88	23.32	0.53
11.	0.91	24.11	0.79
12.	0.94	24.91	0.80
13.	0.96	25.44	0.53
14.	0.98	25.97	0.53
15.	0.99	26.23	0.26
16.	1.00	26.50	0.27

Table 4.8

Durations (hrs)	1-hour rainfall (cm)	Design loss rate (cm/h)	1-hour effective rainfall (cm)
(1)	(2)	(3)	(4)
1.	7.42	0.45	6.97
2.	3.44	"	2.99
3.	3.18	"	2.99
4.	1.86	"	1.41
5.	1.59	"	1.14
6.	1.85	"	1.40
7.	1.06	"	0.61
8.	1.33	"	0.88
9.	1.06	"	0.61
10.	0.53	"	0.08
11.	0.79	"	0.34
12.	0.80	"	0.35
13.	0.53	"	0.08
14.	0.53	"	0.08
15.	0.26	"	-
16.	0.27	"	-

Step-9: Estimation of Base Flow

The design base flow for this sub-zone is recommended to be computed by the following formulae:

$q_b = 0.109/A^{0.126}$

$q_b = 0.109/(136.36)^{0.126} = 0.059$ cumecs/sq.km

Total Base Flow = 136.36 x 0.059 = 8.04 cumecs

Step 10: Estimation of 50-Year Flood (Peak Only)

For the estimation of the peak discharge the effective rainfall units were re-arranged against the unitgraph ordinates such that the maximum effective rainfall was placed against the maximum U.G. ordinate, (obtained from the UG diagram plotted after Step 4), the next lower value of rainfall effective against the next lower value of U.G. ordinate and so on as shown in Cols. (2) and (3) and summation of the product of U.G. ordinate and rainfall gives the total direct run-off as in **Table 4.9** below:

Table 4.9

Time	U.G. Ordinate (cumecs)	1-h Effective Rainfall (cm)	Direct Runoff (cumecs)
(1)	(2)	(3)	(4)
1.	9.00	0.34	3.06
2.	25.00	1.14	29.07
3.	58.00	2.73	158.34
4.	71.50	6.97	498.35
5.	61.00	2.99	182.39
6.	44.50	1.41	62.74
7.	32.70	1.40	45.78
8.	24.50	0.88	21.56
9.	18.00	0.61	10.98
10.	12.70	0.61	7.75
11.	9.50	0.35	3.32
12.	5.70	0.08	0.46
13.	3.50	0.08	0.28
14.	2.00	0.08	0.16
			1024.24 Base Flow 8.04
			50-Year Flood Peak 1032.28 cumecs

Unit Hydrograph Method can not be applied safely to large catchments more than 5000 sq.km. and therefore for large Bridge projects one should go in for detailed analysis supported by Project specific Hydro-meteorological investigations. The total drainage area has to be divided into a number of sub basins. Separate hydrographs may be derived for each sub basin from analysis of different storms by using routine method. Calibration of flood hydrographs and flood routine parameters is essential. For large catchments, flood frequency analysis is preferred method.

Appendix 4.6

A NOTE ON MODEL STUDIES FOR DETERMINING DRAG AND LIFT FORCES

1. Both drag force and lift force depend largely on shape of a body and several other factors and as such analytical calculations for these forces cannot be done accurately, therefore recourse to hydraulic models studies is generally taken to determine magnitude of drag and lift coefficient in the case of important structures.

2. Model studies began to be used for the study of water flow phenomenon in the later part of nineteenth century and today model-studies are accepted as useful for Engineering practice. Model studies cost an insignificant fraction of the expenditure of a project, but these suggest vital improvement in design as these studies enable the designers to visualize the whole problem, eliminating doubts and indecisions due to close conformity between the model and the prototype. Therefore recourse to model studies is generally taken to determine the magnitudes of coefficient of lift (C_L) and coefficient of drag (C_d) for evaluation of these forces on superstructures of important submersible bridges.

3. **Model Studies carried out for Bridges in the former Central Province**

3.1. Several submersible bridges situated in former Central Province (CP) were damaged during floods of 1938-39 and the deck slabs were carried away bodily, Govt. of CP got model studies done at Central Water and Power Research Station, Khadakvasla,(CWPRS), Pune. Annual reports of 1938-39, 1939-40 and 1941-42 of CWPRS, inter-alia deal with the coefficient of drag on submerged bodies particularly with reference to bridge superstructure. The results arrived at by the Research Station are summarized below.

(a) Drag Force: -

The drag on a body kept in steady flow is expressed as

$$F_d = C_d A \tfrac{1}{2} \rho V^2 \qquad \ldots \qquad (4.8)$$

Where,

C_d = co-efficient of drag
A = characteristic projected area of the body
ρ = Mass density of fluid
V = Undisturbed free stream velocity

The value of C_d is dependent on shape of body, roughness of the surface and Reynolds number (R_e) which is expressed as VD/υ or ρ VD/μ where V=Velocity, D=Representive dimension of body, μ = Dynamic viscosity of fluid, υ=Kinematic viscosity of fluid, ρ =Mass density of fluid.

The drag force mentioned above takes into account 'Skin-Friction-Drag' and 'Form-Drag'.

(i) Skin-Friction-Drag: It depends upon viscosity of the fluid and it forms only a small part of the total drag force (10 to15%)

(ii) Form-Drag: It is independent of viscosity and depends largely on shape of the body immersed in fluid and therefore form drag can be found accurately in geometrically similar models with similar Froud's number

The ratio of Inertia force (F_1) and Gravity force (F_g) is Froud's number i.e.

$$\frac{F_1}{F_g} = \frac{\rho D^2 V^2}{\rho g D^3} = \frac{V^2}{g D} \qquad \ldots \quad (4.9)$$

The square root of this ratio, is $\left[\frac{V}{\sqrt{gD}}\right]$

Where D is representative characteristic dimension of the body

The non-dimensional ratio $\left[\frac{V}{\sqrt{gD}}\right]$ is called Froud's Number (F_r or F) and it is the ratio of dynamic force to weight. It has greater significance while carrying out model studies for free surface or open channel flow. The nature of free surface flow (i.e. rapid or tranquil) depends upon whether Froud's number is greater or less than unity.

(b) Lift-Force – Lift force is the fluid force component on immersed body acting vertically at right angle to the approach velocity. The lift force largely depends upon the shape of the body and comprises hydrostatic force and hydrodynamic force. The hydrostatic force is generally referred as Buoyancy and acts vertically upwards. This force is independent of the shape of the body. When a body is immerged in a flowing fluid the body experiences in addition to hydrostatic force, a force due to 'Kinematic energy' of flow and is termed as Hydrodynamic Uplift force. This force depends on velocity of flow, Reynold's Number and shape of body. This lift force on a body can be expressed in form of following equation:

$$F_L = C_l A \rho \frac{V^2}{2} \qquad \ldots \quad (4.10)$$

Where,

C_l = coefficient of Uplift, A= plan or chord area

ρ = Mass density of fluid, V=Stream Velocity

The ratio of Intertia Force (F_i) and Viscous Force (F_υ) is Reynold's Number i.e.

$$\frac{F_1}{F_\upsilon} = \frac{\rho D^2 V^2}{\mu V D} = \frac{\rho V D}{\mu} = \frac{VD}{\upsilon} \qquad \ldots \quad (4.11)$$

Where, $\frac{\mu}{\rho} = \upsilon$, the kinematic viscosity of the fluid.

This non-dimensional ratio $\frac{VD}{\upsilon}$ is called Reynold's Number (R_e or N_r) and it is the ratio of dynamic force to viscous force. A critical value of Reynold's Number makes distinction between Laminar and Turbulent flow. Studies have shown that both drag and lift force are highly sensitive to Reynold's Number, particularly in lower range of Reynold's Number. Its value beyond 1.0×10^6 gives steady value of C_l.

Notes:

(1) If 'Frictional or Viscous force' governs the motion, then Reynold's Number will be applicable.

(2) If 'Gravity' is the only force producing the motion then 'Froud's Number' will be applicable.

3.2 The above results give value of coefficient of drag on various shapes of solid slabs having aspect ratio (width/depth) in range of 12 to 15 and, therefore, are not applicable for box section having aspect ratio usually in range of 5 to 5.33. Further, the CWPRS station, Pune in 1940 did not estimate the lift force on such shapes and values for the same even for rectangular shapes are not available.

4. In the absence of information on coefficient of drag and lift forces for box-type superstructures for submersible bridges, model studies were got done to study the effect of water current forces on the following for submersible bridges.

(i) Submersible Bridge on 'Chambal River' on NH-3 near Dholpur (Rajasthan)

(ii) Submersible Bridge on 'Bhima River at Sangam Village on Tembhurni-Akluj Road-Distt. Solapur (Maharashtra)

During these model studies it was decided to observe value of coefficient of drag (C_d) coefficient of lift (C_l) on box-girder superstructure for various depths of submergence and various velocities of water current.

4.1. **Submersible Bridge at Chambal River – on (NH-3) Near Dholpur (Rajasthan)**

(a) In absence of value of 'k' for box-type deck submersible bridge in the IRC codes, Rajasthan P.W.D. and Ministry of Shipping Road Transport & Highways (MoSRT&H) took decision that model studies be got done at Indian Institute of Technology, Mumbai, for the box section adopted for superstructure for the reconstruction of damaged submersible bridge at Chambal (NH-3), so that values of coefficient of drag (C_d) and coefficient of lift (C_l) are evaluated and accordingly precautions are taken in design and construction for drag and lift forces. At present, IRC Codes does not provide any value for C_d and C_l for Aerofoil box Section.

The shape and dimensions of the Aerofoil box superstructure adopted at Chambal bridge are given below in **Fig. 4.10**.

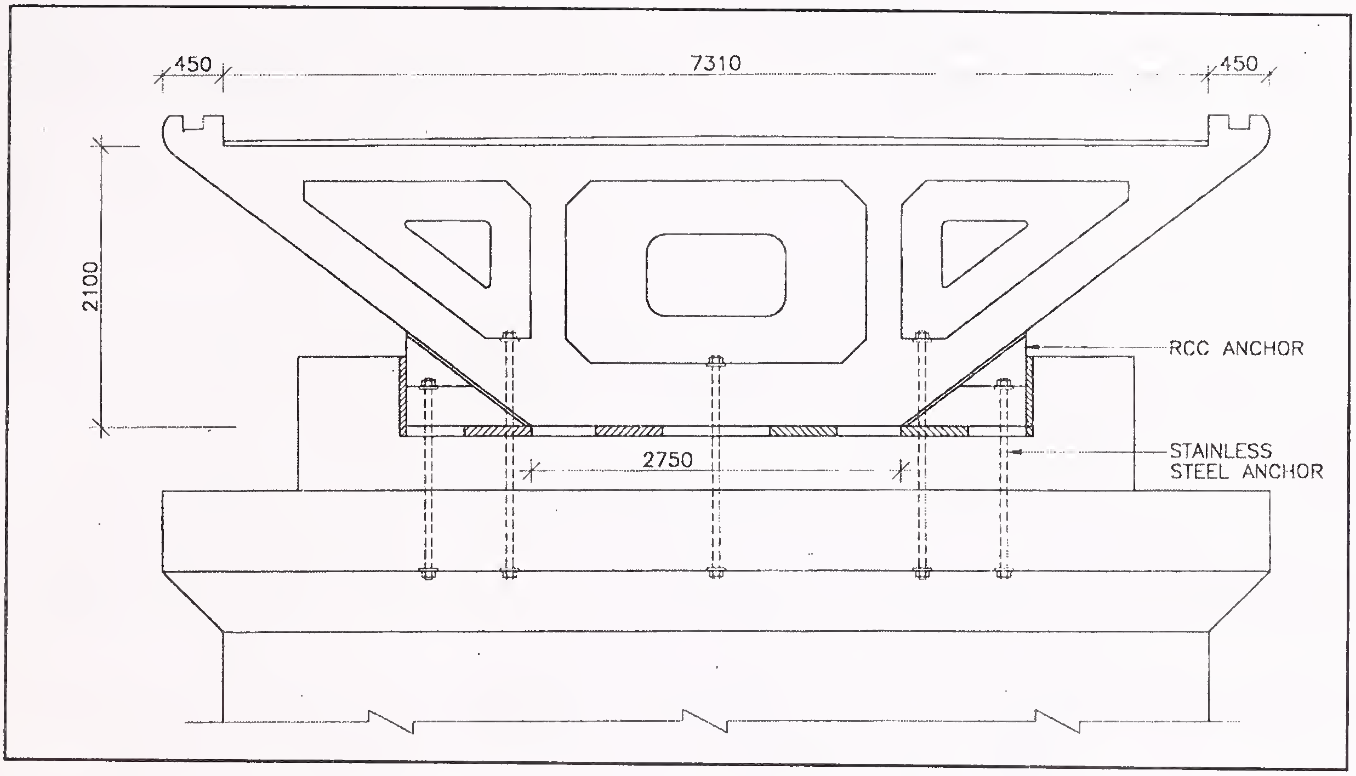


Fig. 4.10. Cross-Section at Support of Chambal Bridge (42.70 m Span)

(b) The brief technical details for the Chambal submersible bridge are as follows:

(i) Maximum design discharge = 5097.6 cumec

(ii) Maximum design velocity of flow = 4.57 m/sec

(iii) Maximum depth of flow = 27.13 m

(iv) Deck level is at 8.23 m below the maximum design flood level

(v) Length of individual span = 43.28 m

Experiments were carried out for two conditions of flow:

(a) Corresponding to maximum designed flood level passing over the bridge deck.

(b) Upstream water just grazing at bridge deck level.

The main objective of this study was to determine the coefficient of drag, (C_d) and the coefficient of lift (C_l)

(c) For similitude between prototype and model, the scales selected were 1:25 and 1:75 geometric. The former for detailed measurement of coefficient of lift (C_l) with the help of piezometric taps, and coefficient of drag (C_d) was measured with help of strain gauge on 1:75 scale model.

(d) Results of model studies on Submersible Bridge at Chambal on NH-3 are shown in **Table 4.10**.

Table 4.10 Model – Test-Results

S.No.	Froud's No.	Flow condition	Results in 1:75 model			Results in 1:25 model		
			C_d Total	C_d Pressure	C_l	C_d Total	C_d Pressure	C_l
			Flow Normal					
1.	0.265	Design conditions at (15 ft/sec) 4.573 m/s velocity	1.79	1.32	0.63	1.60	1.22	2.10
2.	0.27	Upstream water just grazing the top of deck at (12 ft/sec) 3.66 m/s velocity	1.55	1.36	0.54	1.70	1.53	0.60
			Flow at 28° oblique					
3.	0.265	Design conditions at (15 ft/sec) 4.575 m/s velocity	1.75	-	-	1.60	1.26	2.04
4.	0.27	Upstream water just grazing the top of deck at (12 ft/sec) 3.66 m/s	1.46	-	-	1.26	1.11	0.04

(e) As can be seen form result in above table that variation in value of coefficient of drag (i.e. C_d) obtained during model studies at IIT Mumbai was very small, maximum value of C_d obtained is 1.79 and minimum 1.55 for normal flow. Also there is no substantial difference in value of C_d under oblique flow. However, it is seen that **value of C_d (Coefficient of drag) are more than 1.5 i.e. the values recommended by (IRC:6), and therefore, thrust-blocks were provided to prevent sliding of the superstructure.** (ref. Drawing **Annexure-A-1 & A-2).**

(f) It was informed by IIT, Mumbai that the coefficient of lift (C_l) observed during model experiments includes buoyancy i.e. hydrostatic effects also. The variation in coefficient of lift (C_l) values obtained for different conditions of submergence were large varying from 0.04 to 2.10 as given in **Table 4.10 above. The value of C_l obtained in model studies gave indication that streamlined shape of superstructure are also likely to be unstable against lift forces and need extra anchorages to prevent lifting of the superstructure.**

4.2. Submersible Bridge at Bhima River–Maharashtra

In absence of more information for value of 'k' in IRC code for Aerofoil box type superstructure it was decided by Maharashtra Govt. to get model studies done at CWPRS, Pune. The Aerofoil deck adopted for submersible bridge at Bhima river is given in **Fig. 4.11.**

(a) The brief technical details of Bhima Bridge are given below:

Length of the bridge	350 m
Design discharge	2436.2 cumecs

Design flood velocity	5.26 m/sec
Design maximum high flood level	463.95 m
Design deck level	458.83 m

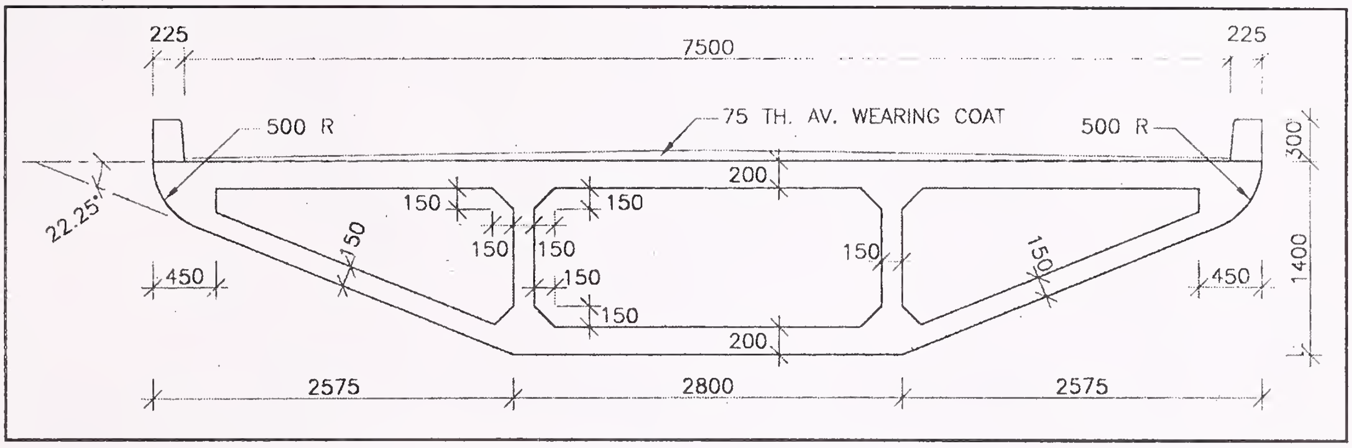


Fig. 4.11. Cross-Section of Submersible Bridge on Bhima River

The model studies were carried out to evaluate coefficient of drag and lift forces for different depths of submergence and velocities on Aerofoil shaped box girder.

(b) The piezometric observations were done on models with approach velocities ranging from 6.0 m/sec for submergence of the deck slab by 5.13 m, and other with water level grazing deck slab's top and having approach velocity 4.0 m/sec. The **coefficient of drag** (C_d) varied from 0.37 to 2.10 for the above range of velocities. These variations in C_d values are plotted as a function of Reynold's Number for different submergences and are shown **in Fig. 4.12.**

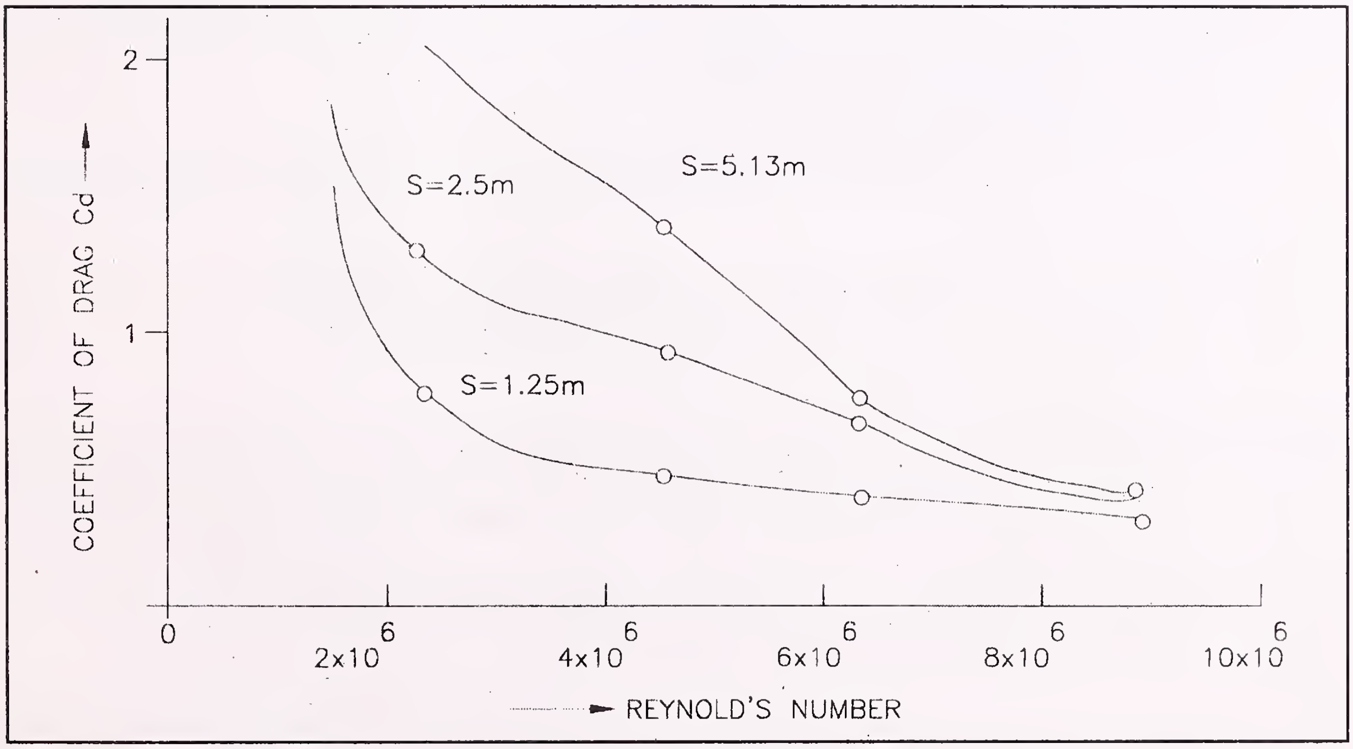


Fig. 4.12

(c) The coefficient of lift (C_l) for Phase I studies for the velocities as given in para (b) above varied from 0.04 to 0.41. These C_l values were worked out by excluding the hydrostatic force which is to be accounted for separately. The variation of C_l v/s R_e is shown in **Fig. 4.13.**

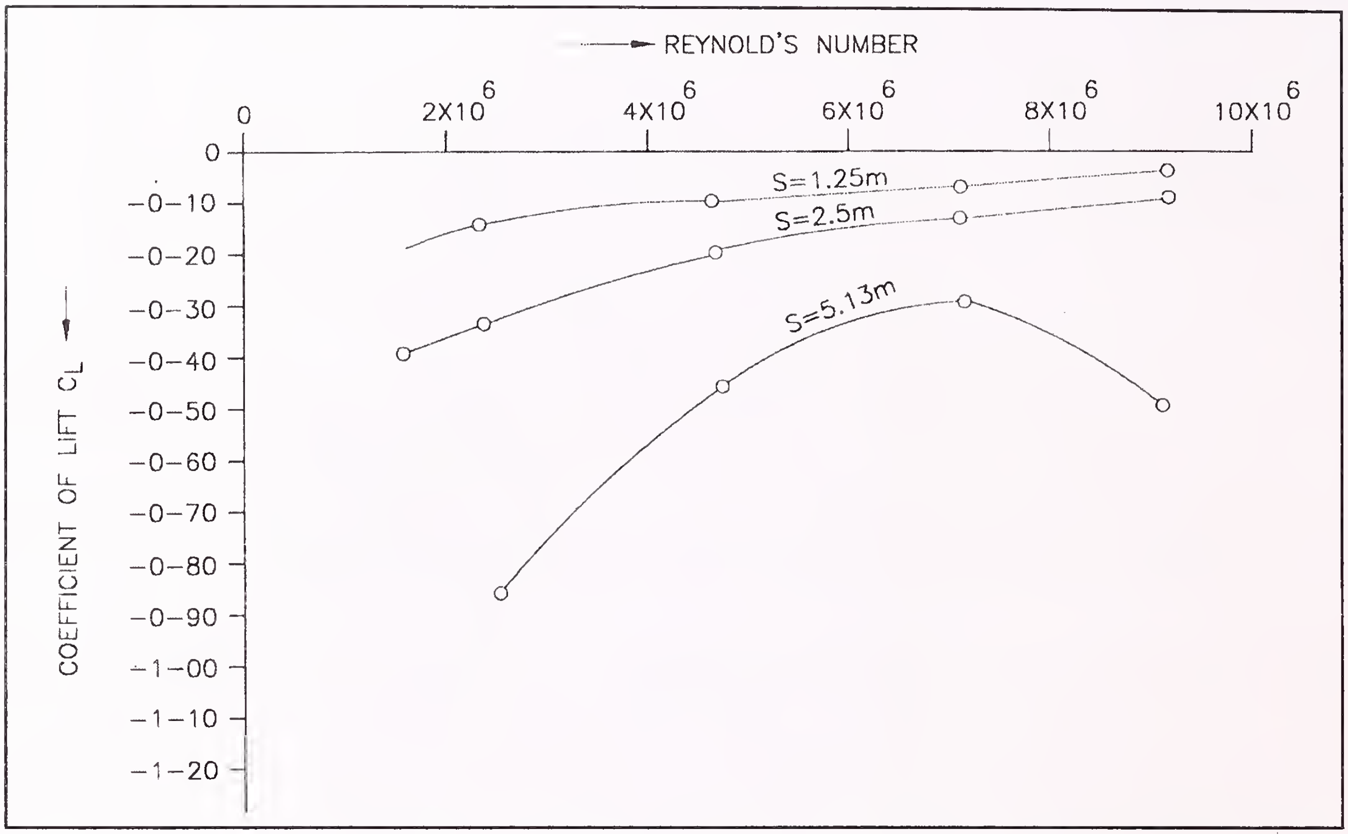


Fig. 4.13

(d) From the above values of C_d and C_l as worked out after model studies on Bhima Bridge, the following inferences can be drawn:

(a) **Coefficient of Drag (C_d)**

(i) Values of C_d shows a tendency to decrease as value of R_e (Reynold's Number) increases.

(ii) The coefficient of drag (C_d) varies with depth of submergence

(iii) For very low R_e the value of C_d is abnormally high.

(b) **Coefficient of Lift (C_l)**

Values of C_l vary with R_e (Reynold's Number). For low R_e and maximum submergence, the coefficient of lift C_l is the highest attaining value of (-) 0.86.

4.3 State of Maharashatra also got carried out model studies from CWPRS Pune for conventional rectangular shape of box superstructure **(Fig 4.14).**

(a) On above rectangular box section CWPRS, Pune carried out model studies for different conditions of submergence and velocity. The results indicate that coefficient of drag varied

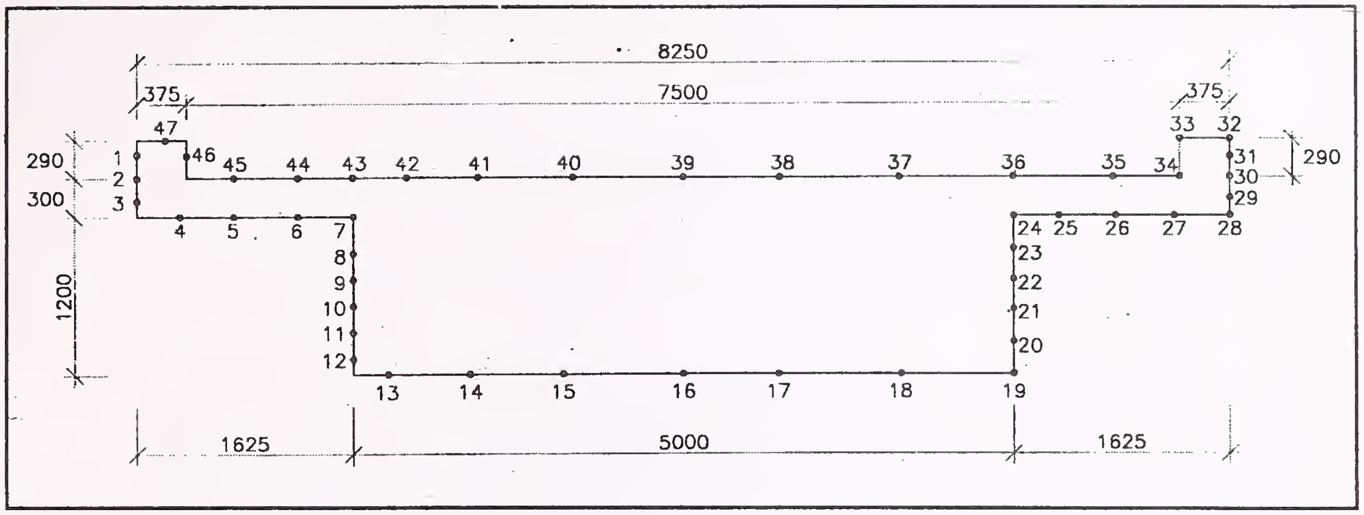
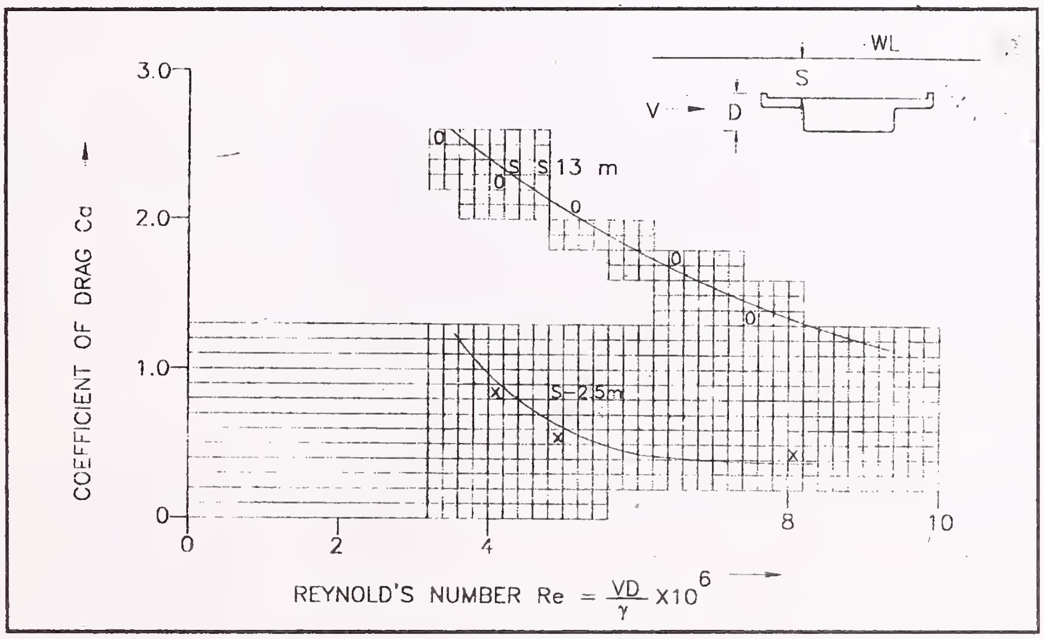


Fig. 4.14. Models of Box Type Submersible Bridge

from about 0.5 to 2.8 under various depths of submergence and approach velocity. It was also seen that value of C_d varied substantially for the submergence of 5.13 m and 2.5 m, the variation in values of submergence for 2.5 m to 1.25 m was negligible. The graphs showing variation of **C_d Vs R_e** are given below in **Graph 4.2.**



Graph 4.2

(b) The coefficient of lift (C_L) was varying from (-) 0.2 to (-) 0.9. and are shown in **Fig 4.15.**

4.4. Brief discussion on results of studies carried out in the following cases:

(i) CWPRS, Pune model studies (1938 to 1942), for bridges in Central Province

(ii) IIT Mumbai model studies on submersible Chambal bridge on NH-3 – Dholpur (Rajasthan)

(iii) CWPRS, Pune model studies on submersible bridge at Bhima River (Maharashtra)

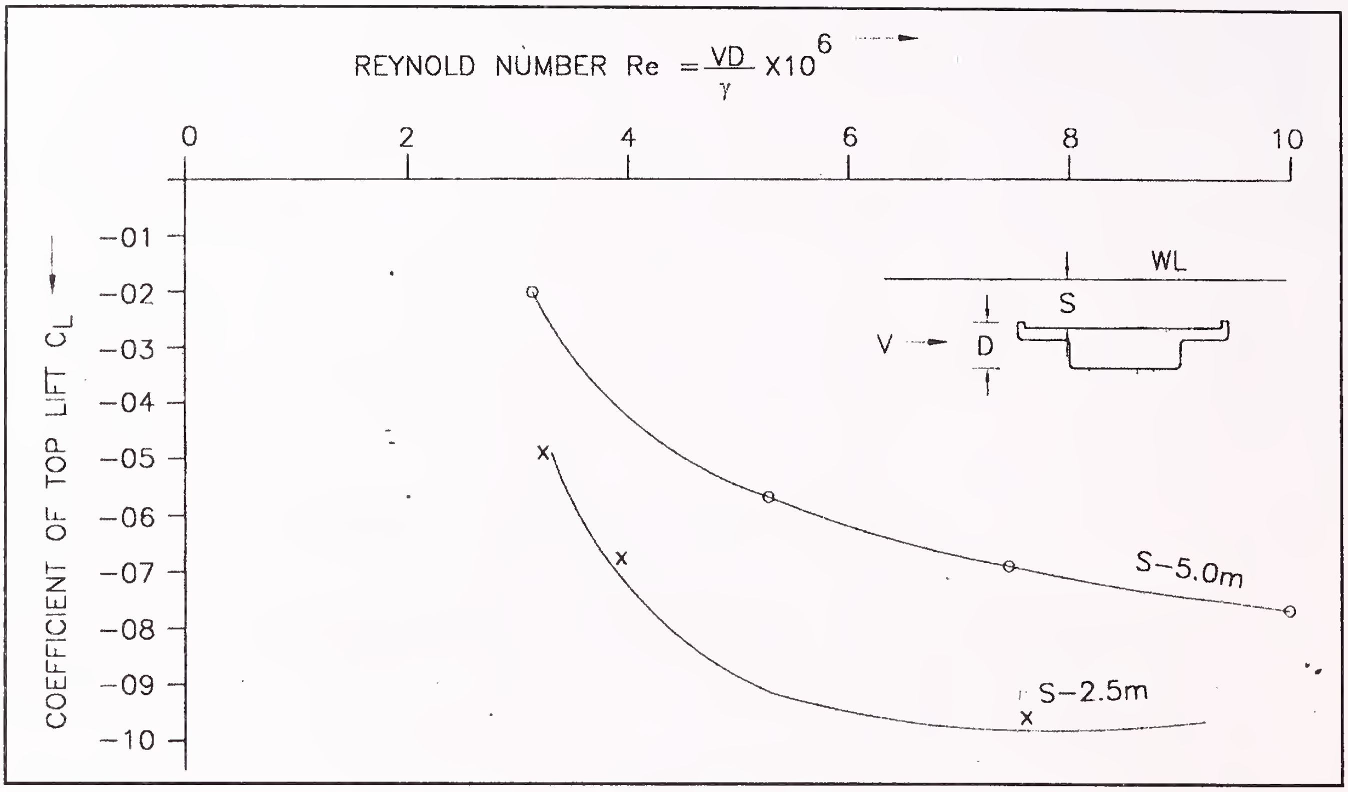


Fig. 4.15

4.4.1. The Tests carried out by Central Water and Power Research Station (period 1938 to 1942) pertain to the solid slab deck where the width to depth ratio is very high (in range of 12 to 15) and velocities simulated were in low range of 1.83 m/sec to 2.44 m/sec in prototype i.e. 0.56 m/sec to 0.74 m/sec in model. Comparison of these results with recent model studies done for Box Aerofoil Superstructures for Chambal and Bhima river bridge will not be correct. Moreover, observations carried out by the Central Water and Power Research Station in 1940 were only qualitative in nature for estimating the lift force.

4.4.2. Recent studies carried out by I.I.T. Mumbai for Chambal river bridge and by Central Water Research Station for Bhima river bridge have been examined in depth. It will be noticed that though the shape of deck of Chambal bridge and Bhima river bridge are apparently similar but there are some difference's also. The overall width of decks of both the bridges are more or less the same as also the width of straight portion of soffit, but the depth of the box being different, therefore, the angle subtended by inclined soffit with deck top for Chambal bridge is 37.875° and the same for Bhima river bridge is only 22.25°. Due to this, there is a difference in the aspect ratio (width to depth). In the former case this ratio is 3.85, while in the latter it is 5.678 and this inequality could give rise to different streamlines. As a result, the values of C_d and C_l are bound to be different in these two case studies also.

4.4.3. **Inferences from Model Studies at IIT, Mumbai for Chambal Bridge on NH-3.**

(i) The Indian Institute of Technology (IIT) Mumbai carried out model studies by adopting geometric scale of 1:25, with model Reynold's Number of about $1x10^5$ which is considered close to hydraulically unsteady zone. Therefore, it was necessary to adopt more appropriate scale.

(ii) In IIT Mumbai model studies, the variation in values of total drag was very small. Maximum C_d total is 1.79 and minimum C_d total is 1.55 for normal flow. Also, there is no substantial difference in value of C_d total' under oblique flow. However, the values of C_d obtained are more than 1.5, the value as recommended by the Indian Roads Congress for superstructure.

(iii) Variations in coefficient of lift (C_l) obtained under different conditions are very large giving its values from 0.04 to 2.10. As a matter of fact, these values give rise to an apprehension that such streamlined shaped bodies are likely to be unstable against flowing water and, therefore, should be used with caution and need extra anchorages to prevent lifting up of the sub-structure, are to be provided (as done in case of Chambal bridge at Dholpur).

4.4.4. Inferences from Model Studies at CWPRS Pune for Bhima River Bridge-Maharashtra

The graph of C_d vs. R_e obtained in model tests carried out at CWPRS, Pune, gives higher values of C_d for rectangular shape box than stream line box under maximum submergence. As for condition for water level at deck level, coefficient of drag for rectangular box section is less than the stream line section. It is difficult to explain why C_d is lower for rectangular shaped box compared to Aerofoil shaped box.

5. WATERWAY AND AFFLUX

5.1. Waterway

5.1.1. General

The area through which the water flows under a bridge superstructure is known as the waterway of the bridge. The linear measurement of the waterway along the bridge is known as linear waterway. The linear waterway is equal to the sum of the length of all the clear spans. The natural waterway is the unobstructed area of flow of the river/stream at the bridge site.

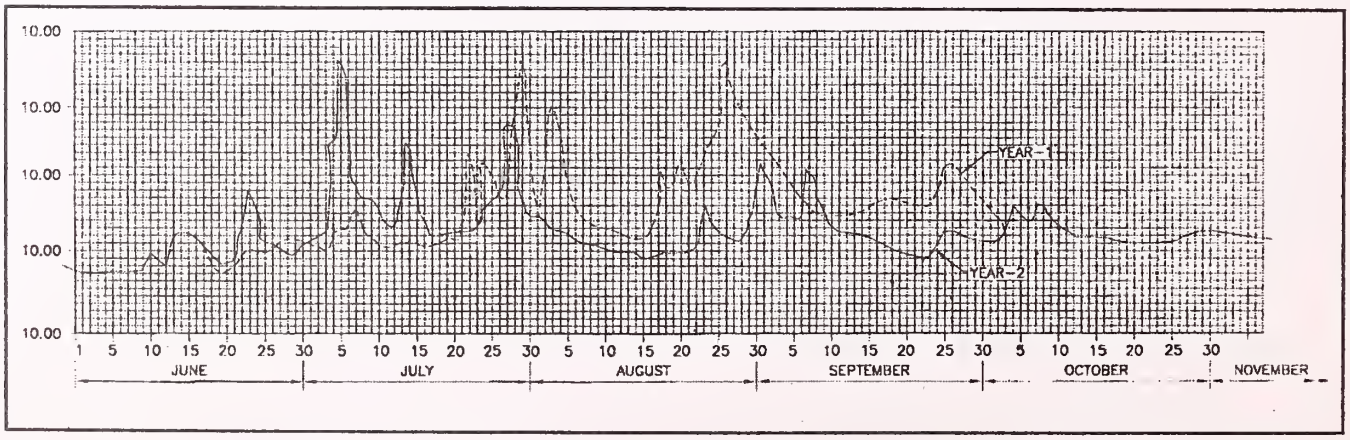
The waterway adopted should be adequate to pass the design flood of specific Return Period. The opening has to be capable of passing the design-flood without overtopping the deck in case of high level bridges, and the design floods estimated upto a level at which the deck is fixed in case of submersible bridges, without endangering these structures.

5.1.2. Fixing deck level of submersible bridges

(i) Specific number of overtopping the deck is permitted during annual floods in case of submersible structures, which are primarily low cost and economical solutions compared to high level structures. It is, therefore, necessary to first decide the permissible duration and frequency of such overtoppings.

(ii) Generally, it has been observed that high flood occurs three to four times during monsoon, but water level rises so fast and falls again so rapidly that the peak level of these floods lasts only for a short time. If level of floods is plotted on vertical axis and dates of floods on horizontal axis, then one can easily decide about the deck level as seen from **Graph 5.1** which is a typical example of yearly floods in a river during monsoon.

(iii) Based on the flood data during monsoon season, the deck level of submersible bridge is so fixed that the facility will satisfy the criteria of frequency and time period of interruptions to traffic as specified by user Authority and as indicated in



Graph 5.1

Table 3.1 of these guidelines. The concerned Department has to collect flood data for a representative monsoon season and decide the OFL above which deck level of submersible bridge is so fixed that it satisfies the criteria of frequency and time period of interruptions of traffic.

5.1.3. Constriction of waterway

(i) Any constriction of waterway either laterally or vertically reduces the natural waterway of stream which results in change in normal flow pattern from that existing before the constriction and in afflux on upstream. Higher the constriction of natural waterway, higher will be the afflux and the velocity of flow through the vents. It is therefore, desirable to keep the constriction of waterway to the minimum in order to reduce expenditure on providing raised face walls and protection of bed. However constriction to varying degrees becomes unavoidable, depending on the type of structure that may be selected for adoption based on various other technical and economic considerations. The constriction of waterway that can be permitted in any particular case depends on several site specific conditions the more important ones being the nature of soil in the river bed and the adopted Road Top Level (RTL) in relation to the design HFL.

(a) If the bed material is easily erodible, it would be desirable to avoid high constriction to keep the velocity of flow through the vents within manageable limits.

(b) Similarly, higher constriction can be provided for low level submersible structures like causeways but, if the depth of flow below RTL in relation to the depth below the design HFL is high as would generally be the case when higher submersible bridges are provided, the constriction must be kept low so as to keep the hydrostatic forces on the structure within manageable limits.

(ii) Several States in the country, which have been constructing submersible structures for a long time, have their own practices with regard to the permissible constriction, based on their experience and site conditions prevailing in the respective States. These practices may vary from State to State and it is recommended that the States may continue to follow their successful practices in this regard. Alternatively, the following recommendations may be followed:

(a) For low level submersible structures like causeways, provide a vent area of about 40 per cent but not less than 30 per cent of the unobstructed area of the stream measured between the proposed road top level and the stream bed. In scanty rainfall areas where annual rainfall is less than 600 mm, the vent area can be reduced upto 20 per cent to 30 per cent of unobstructed area. However, the available area of flow under design HFL condition should always be at least 70 per cent of the unobstructed area of flow between the design HFL and the stream bed i.e. the obstruction under design HFL condition should not be more than 30 per cent.

(b) For submersible bridges, which would generally be provided with relatively higher road top level, the available area of flow under the structure should not be less than 70 per cent of the unobstructed area of the stream measured between the stream bed profile and the proposed road top level.

(iii) RTL should not be abnormally high over vent opening as this causes heading up of water on u/s which in turn may result in high velocity (it can even be in the range of hypercritical) leading to failure and out flanking. Hence RTL should be kept as low as possible.

(iv) The increase in velocity under the bridge should be kept below the allowable safe velocity for the bed material. Typical values of safe velocities for different bed material are as below:

Type of Material	Safe Velocity (m/sec)
Loose clay and fine sand	upto 0.5
Coarse sand	upto 1.0
Fine gravel, sandy or shift clay	upto 1.5
Coarse Gravel/Weathered rock/ Boulders upto 200 mm size	upto 2.5
Larger boulders (200 – 800 mm size) or rocky strata	2.5 to 6.0

(v) In case the velocity exceeds the above specified values for scourable beds, then bed protection consisting of flooring with proper cut-off wall should be provided on both upstream and downstream side on the bridge as discussed in detail in para 6.4 of Chapter 6. A typical arrangement of floor protection works is given in **Fig. 5.1**.

(vi) In the post protection works, the velocity of flow under structure should not exceed 2 m/sec. The depth of drop wall should be such that it does not get undermined. If a flooring is not provided, then maximum depth of scour should be calculated carefully and depth of foundations be provided accordingly.

5.2. Afflux

5.2.1. General

(i) Afflux can be defined as a rise/heading up of water surface above normal water level on the upstream side of a bridge. It is caused when the effective 'linear waterway' through the bridge is less than the natural width of the stream immediately upstream of obstruction. Afflux can also be caused in case of a bridge where there is reduction in overall width of the waterway over the natural width of stream, due to the obstruction of piers and projecting abutments as indicated in **Fig. 5.2**.

(ii) Afflux governs the dynamic action of water current. The greater the afflux greater will be the fall of water level from upstream to downstream and therefore greater

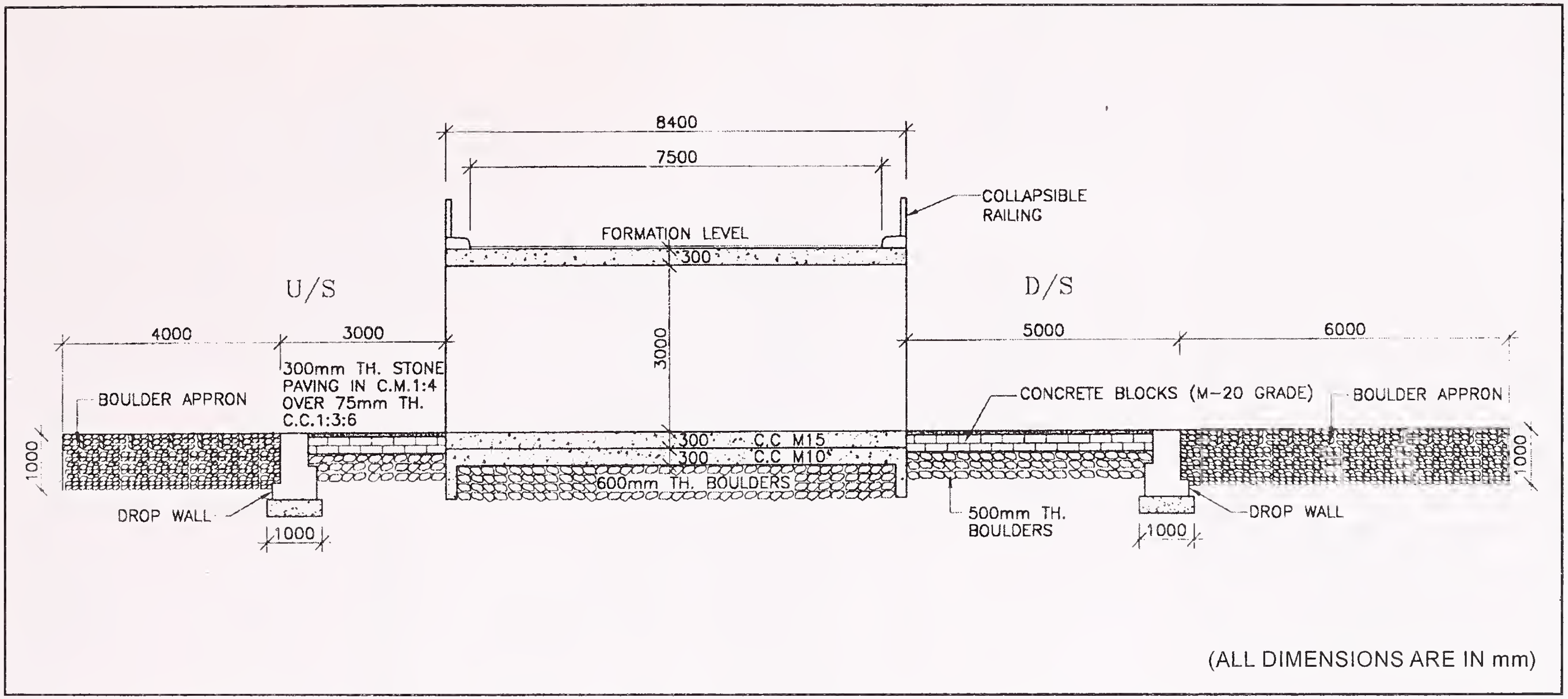


Fig. 5.1. Typical Section through Bridge Floor

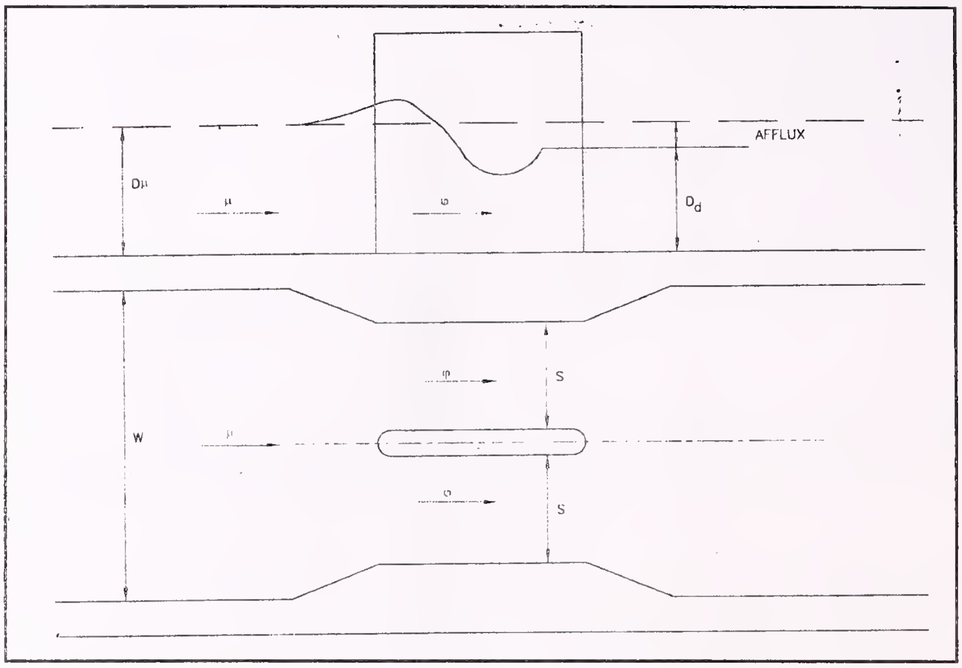


Fig. 5.2. Afflux caused by obstruction of Piers & Abutments

will be the velocity of flow on the downstream side leading to greater scour, thereby requiring deeper foundations.

(iii) An estimation of afflux is necessary (i) for fixing the bottom face-line of the bridge deck after allowing for adequate free board, (ii) for fixing levels of the approach road (iii) for determining the increased velocity as required for designing the foundation and bed protection works.

5.2.2. Estimation of afflux for non-scourable beds may be computed approximately by use of empirical formulae given below :

(a) Molesworth formula
(b) Rebbock's formula
(c) Varnell's formula

However, **Molesworth Formula** which is given below is usually adopted to estimate the afflux at bridge constrictions:

$$h = \left[\left(\frac{V^2}{17.9} \right) + 0.015 \right] \left[\left(\frac{A}{a} \right)^2 - 1 \right] \qquad \ldots \qquad (5.1)$$

Where,

h = afflux in m
V = velocity of approach in m/sec
A = natural waterway area of the stream in sq.m.
a = constricted area in sq.m.

5.2.3. Estimation of afflux by broad crested weir and orifice formulae

The afflux (h), the discharge (Q) the unobstructed stream width (W) and the linear waterway (L) are all interrelated. Greater the reduction in linear waterway, the greater is the afflux. Since downstream depth (D_d) is not affected by the bridge obstruction as the same is governed by the hydraulic characteristics of the channel downstream, it can be safely assumed that the upstream (u/s) depth which prevailed before the bridge construction is same as the downstream depth (D_d) that prevails even after the bridge construction. Hence (D_d) is the depth that prevailed at bridge site before the construction of the bridge. To estimate afflux we must know the discharge (Q) in the channel, the value of D_d, value of W and L, then afflux can be calculated by applying weir and orifice formula as given below.

(a) Broad Crested Weir Formula*

The weir formula as given below applies only when standing waves are formed i.e. so long as the afflux (D_u-D_d) is not less than ¼ D_d:

$$Q = 1.706\ C_w L \left(D_u + \frac{u^2}{2g} \right)^{3/2} \quad \ldots \quad (5.2)$$

Where,

C_w = coefficient to account for losses in friction

D_u = upstream water depth and D_d = Downstream water depth

$\frac{u^2}{2g}$ = head due to velocity of approach

The parameters are indicated in **Fig. 5.3.**

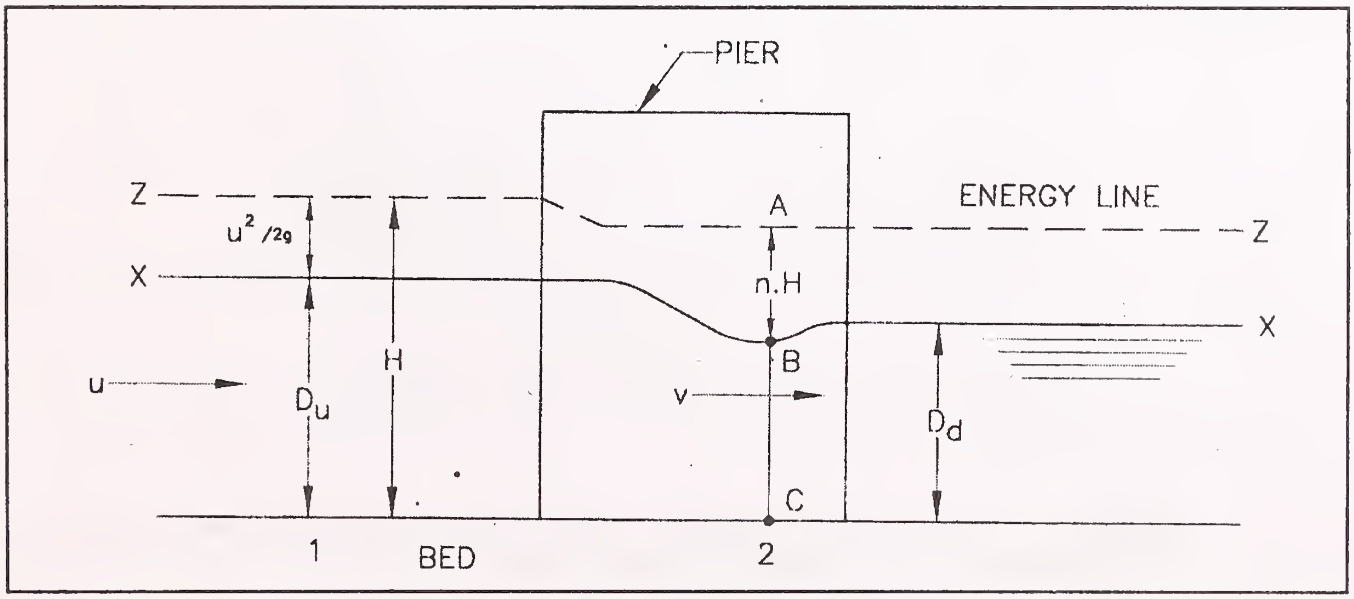


Fig. 5.3. Determination of Afflux by Broad Crested Weir Formula

Values of C_w for different types of opening are given in **Table 5.1** below :

Table 5.1. Value of C_w

Type Bridge Opening	Value of C_w
1. Narrow Bridge* openings with or without floor	0.94
2. Wide Bridge openings with floor	0.96
3. Wide Bridge openings with out bed floor	0.98

* when span is less than downstream depth

(b) The Orifice Formula:

The Orifice formula given below will be applicable with suggested values of coefficient C_o & e. When downstream depth is more than 80% of upstream depth, $D_u - D_d$ is less than ¼ D_d.

$$Q = C_o \sqrt{2g}\ L\ D_d \left(h + (1+e)\frac{u^2}{2g} \right)^{1/2} \qquad \ldots \qquad (5.3)$$

The parameters are indicated in **Fig. 5.4.**

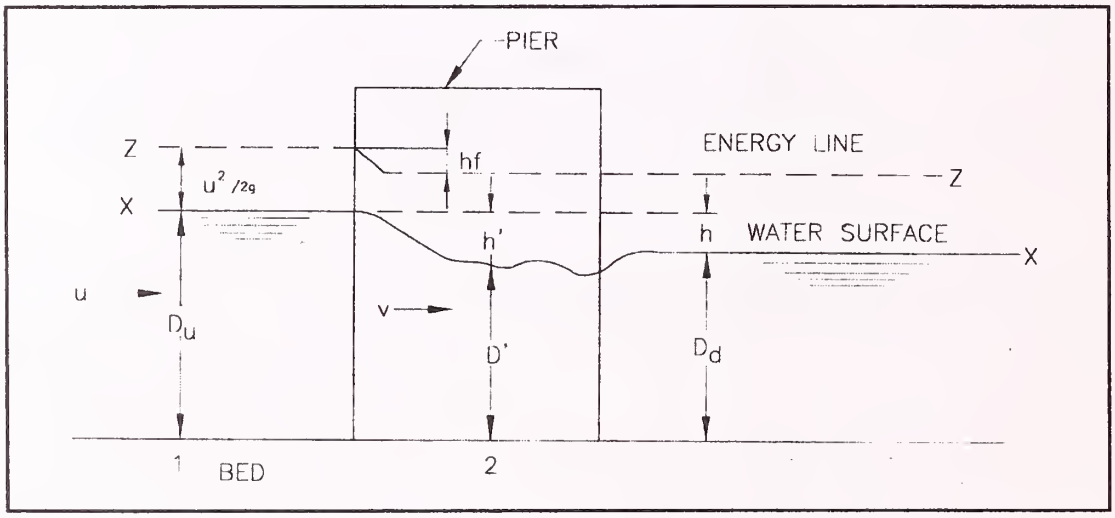


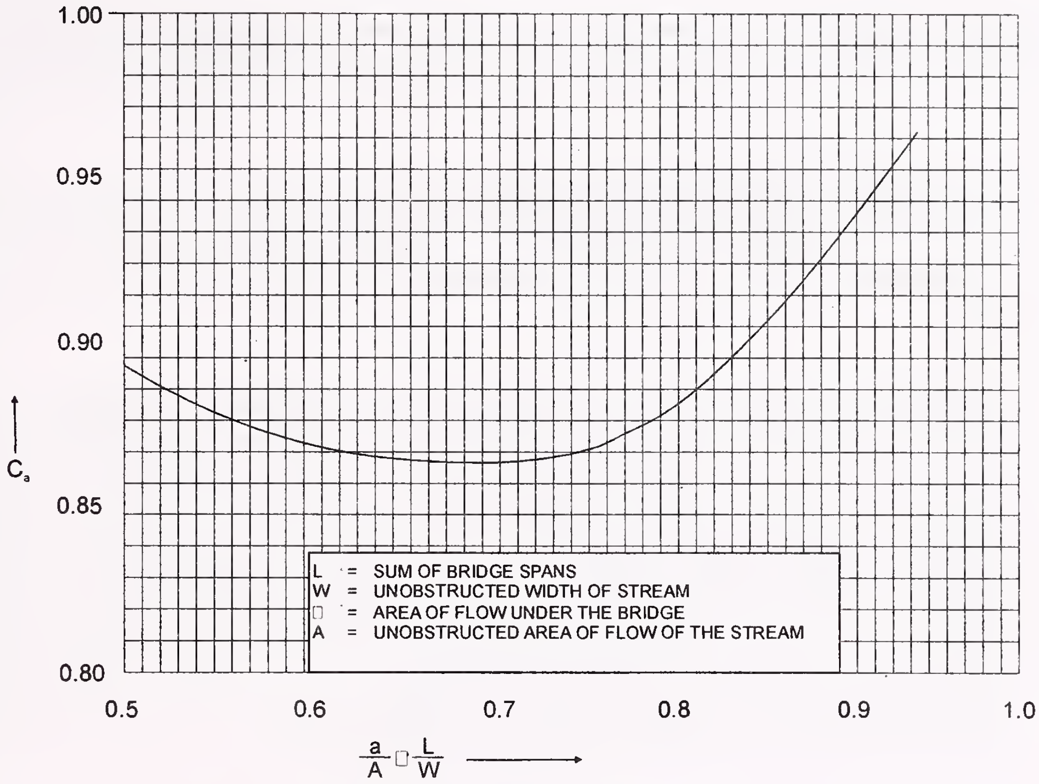
Fig. 5.4. Determination of Afflux by Orifice Formula

The values of coefficient C_o and 'e' may be taken from curves given in **Graphs (5.2 and 5.3)**.

An example of calculation of afflux using the broad crested weir and orifice formulae is given in **Appendix 5.1.**

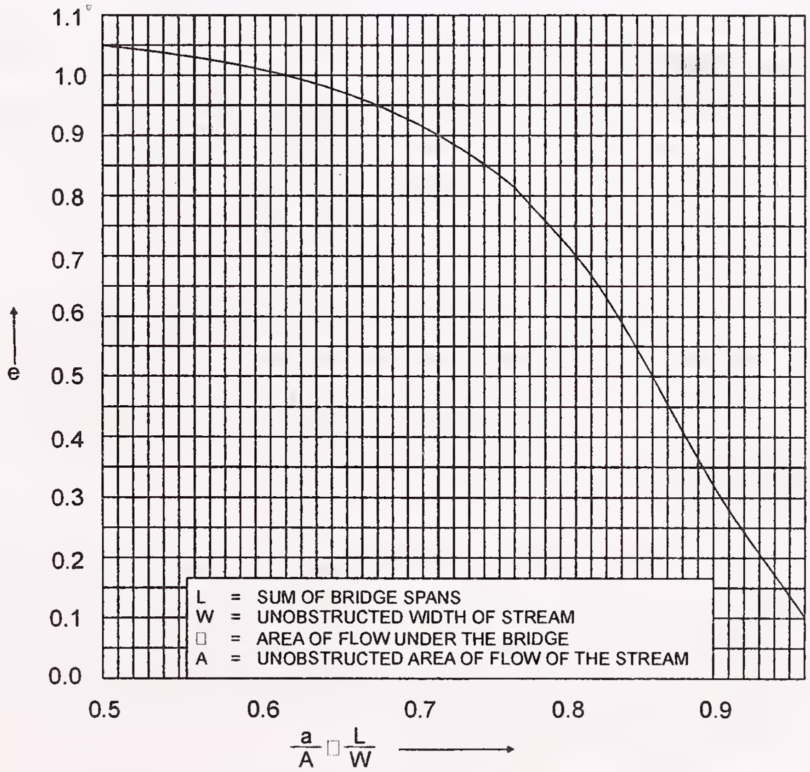
5.2.4. Precautions to be taken in the design for reducing afflux and its effects on the submersible bridge and approaches.

In a submersible bridge the entire cross-sectional area of the superstructure also obstructs natural waterway, so afflux is more in case of submersible bridge than that of high level bridge.



Orifice Formula Coefficient "C_o"

Graph 5.2



Orifice Formula Coefficient "e"

Graph 5.3

ELEMENTS OF THE HYDRAULICS OF FLOW THROUGH BRIDGE

Further high afflux increases velocity which causes high moments on pier foundations. In order that excessive afflux and thereby hypercritical velocity are not created, following precautions are to be taken:

(i) The deck level of submersible bridge should be kept low. If the deck level is kept high i.e. near the HFL where velocity of flow is the highest, then greater afflux is created and moments due to hydrodynamic forces are more severe at the foundation level.

(ii) The bed erosion and afflux are interlinked and dependent on one hand on the amount of constriction and obstruction caused by the bridge and approaches and also on the other hand on the type of hydrograph for the river obtained. In a river with gradually increasing and sustained floods, the full bed scour would develop giving negligible afflux while in flashy rivers the time for bed scour may not be adequate thereby causing very high afflux. It has been observed that the afflux upstream of Mokamah Bridge on Ganga River which has sustained floods, was much less than the afflux created on upstream of the bridge on Luni River in Rajasthan with predominantly sandy beds but which has flash floods.

(iii) The percentage of obstructions to the flood discharge is maximum when the flood rises just upto the top level of the submersible bridge. It is well known that percentage of obstruction to flood water goes on diminishing as the flood water rises above the submersible bridge, since the area obstructed remains constant and at any further higher floods the percentage of obstruction becomes much smaller. Obviously the conditions causing erosion of banks and subsequent outflanking are when the afflux is maximum i.e. water level including afflux is touching top of deck of the submersible bridge. Therefore, in short, this is one of the critical conditions for the design of submersible bridge.

(iv) In order to ensure that the actual afflux as created remains low and does not exceed the calculated value, so that the road surface becomes dry as quickly as possible for the passage of the traffic, it is necessary to adopt methods as suggested below so as to maximize the discharging capacity of the causeways/submersible bridge for a given afflux:

(a) Increasing the coefficient of discharge of the vents. The co-efficient of discharge through normal kind of vents used in causeways/submersible bridge is of the order of about 78 to 80 per cent but experiments have indicated that a provision of bell-mouth entries for the vents on the upstream side increases the co-efficient to about 88 per cent.

(b) Similarly the upstream edges of the approach roads under submergence should also be rounded off to ensure streamline flow and enhance the discharging capacity.

(c) In alluvial bed, even where rigid flooring is not provided, suitable relief due to scouring of bed may not be considered in the calculation of area of flow and afflux.

(d) The size of vents should be fixed such that the obstruction to the flow of water at the stage when it touches the top of deck slab, is less than 60 per cent or at the most 70 per cent.

Appendix 5.1

AN EXAMPLE OF CALCULATION OF AFFLUX USING BROAD CURBED WEIR AND ORIFICE FORMULAE

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 down stream depth is 1 m.

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

Discharge through the bridge by the Orifice Formula

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

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

Afflux corresponding to this, $C_0 = 0.867$, $e = 0.85$, $g = 9.8$ m/sec²

$$70 = 0.867 \times 4.43 \times 25 \times 1 \left(h + \frac{1.85\,u^2}{2g} \right)^{1/2}$$

$$\therefore h + 0.0943\,u^2 = 0.53 \qquad \text{..... (i)}$$

Also, just upstream of the bridge

$Q = W\,(D_4 + h)\,u$

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

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

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

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

Substituting for u in (ii)

$h = 0.263$ m

Alternatively, assume that h is more than ¼ D_d and apply the Weir Formula

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

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

$\therefore H = 1.45$ m

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

Or; $D_u = 1.45$ m (approx)

Now,

$Q = W\,D_u\,u$

$70 = 33 \times 1.45\,u$

$u = 1.46$

$$\frac{u^2}{2g} = 0.1086 \text{ m}$$

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

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

Since "h" is actually more than 1/4 D_d, the value of afflux arrived by the Weir Formula is to be adopted.

Therefore adopt h = 0.3414 m.

6. SCOUR AND FOUNDATIONS

6.1. Scour

6.1.1. General

Scour is the erosive action of flowing water and carrying away of the resultant loose material from the watercourse bed. The deepening of bed (extent of scour) mainly depends on characteristics of watercourse and its bed (e.g. presence of rifts, deep pools, weeds, stones/boulders, bends, longitudinal slope, straight/meandering reach, particle size of bed material, velocity of water during floods, resistance of bed to erosion etc.) and duration of floods. The scouring action in natural watercourses is also not generally uniform throughout the bed width and it inter alia depends on concentration of flow and constriction caused by the structure. The scour is deeper in the vicinity of the pier foundation because the velocity and turbulence of flow around piers and its foundation is higher than the average velocity of flow. The scour is an important parameter for fixing the size, type, depth of foundations and also span arrangements of a submersible structure and should therefore, be carefully determined keeping in view the site conditions, observed values of normal scour in the unobstructed reach of watercourse and maximum observed scour around the foundations of structures in the vicinity on the same watercourse etc.

In case of submersible bridges in erodible bed, the scour will be more than that for a high level bridge for any particular value of discharge. The main reason for this is the increased obstruction in the waterway due to submergence of super structure which increases both the velocity of flow and its turbulence and the difference is likely to the maximum when the flood level is just about to overtop the deck. After this level, the percentage of obstruction with respect to the total cross-sectional area of the channel reduces appreciably as may be seen in a worked out example at **Appendix 6.1.** This will, however happen only if the duration of the flood corresponding to the deck level, is long enough to scour the bed and transport the bed materials. The scour depths for submersible structures should therefore, be calculated under different flood levels i.e. Ordinary Flood Level (OFL), flood level just touching the deck level and HFL to determine the design scour depth.

The scour under the submersible structures especially around pier foundations can, if required, be controlled either by siting the bridge where rock is available at shallow depth or by the provision of suitably designed floor protection around the structure.

6.1.2. Design discharge for determination of scour depth

6.1.2.1. In order to provide for an adequate margin of safety, IRC:78 recommends that the scour for foundations (except raft foundations) should be determined based on a discharge larger than the design discharge determined as per Chapter 4 of these guidelines. The values of increment to be adopted for different ranges of catchment areas are given in **Table 6.1** for ready reference. Unless otherwise specified by Design Engineer, these values may be adopted in case of submersible structures.

Table 6.1

Catchment area in km²	Increase over design discharge in per cent
0 – 3000	30
3000 – 10000	30 – 20*
10000 - 40000	20 – 10*
Above 40000	10

* Percentage increase over design discharge for intermediate values of catchment areas may be worked out by interpolation.

6.1.3. Normal scour

6.1.3.1. Normal scour level, is the level of the scoured bed that is likely to occur during design floods. As regime conditions seldom exist in natural streams and because of limited duration of any flood, the relationships given by Lacey for alluvial streams are not strictly applicable. However, in the absence of any other recommendations based on further research work/experience gained, these relationships are widely adopted to get an idea of expected scour and the same may also be adopted in case of submersible structures.

6.1.3.2. According to Lacey, the scour parameters in case of alluvial streams depend only on discharge (Q) and silt factor (K_{sf}). The value of silt factor depends on size and looseness of the grains of the alluvium and may be determined by the following expression:

$$K_{sf} = 1.76 \sqrt{d_m} \qquad \ldots \qquad (6.1)$$

Where d_m is the weighted mean diameter of the bed particles in millimetre.

The basic parameters are calculated from the following expressions:

(i) Wetted Perimeter (P) = $C \sqrt{Q}$ ... (6.2)

Where Q is discharge during floods and C is a constant dependent on local conditions and may vary from 4.5 to 6.3. However, value of 4.8 is generally adopted in design.

(ii) Hydraulic mean radius (R) $= 0.473\ (Q/K_{sf})^{1/3}$ ... (6.3)

In case of wide alluvial streams, width (W) is assumed equal to the wetted perimeter (P) and normal depth of bed (normal scour depth) below water level equal to 'R'.

6.1.4. Mean depth of scour

6.1.4.1. The mean scour depth below designed flood level for natural channels flowing over scourable bed is generally calculated from the following equation:

$$d_{sm} = 1.34\ [\ D_b^{\ 2} / K_{sf}]^{\ 1/3} \qquad \ldots \qquad (6.4)$$

Where, D_b = Design discharge for foundation per metre width of effective waterway,*

K_{sf} = Silt factor for a representative sample of bed material obtained upto the level of anticipated deepest scour, given by the Expression 6.1 [$1.76 \sqrt{d_m}$] and d_m is the mean weighted mean diameter in millimetre.

* The discharge per meter width may be calculated from the following two conditions:-

(i) When the flood level touches the top of the wearing coat

(ii) During floods, based on the width of compartments of the flow area under consideration and the calculated discharge for the same (As described in para 4.1.4)

The weighted mean diameter of particles for a stratum may be worked out as per procedure illustrated in **Appendix 6.2.**

6.1.4.2. In the absence of sieve analysis of the bed material of different strata upto anticipated deepest scour, the values of K_{sf} for various grades of bed materials as given in **Table 6.2** may be adopted for estimation of scour depth for the preparation of preliminary project reports. The detailed design of foundations should, however be based on the sieve analysis of different strata upto the anticipated deepest scour likely to be met with at the site.

Table 6.2

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 0.7
Medium sand	0.233 to 0.505	0.85 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

6.1.5. The anticipated maximum depth of scour should be determined after considering all the relevant local conditions over a reasonable period of time. In this regard the following aspects may be kept in view in deciding the maximum scour depth:

(a) Wherever possible, soundings for the purpose of determining the depth of scour should be taken in the vicinity of the site proposed for the bridge. Findings of such soundings are best when taken during or immediately after a flood before the scour holes have had time to silt up appreciably

(b) The design discharge being greater than the flood discharge at the time of taking soundings

(c) The increase in velocity due to obstruction to flow caused by the construction of the bridge; bed protection, approaches, training works etc. and

(d) The increase in scour depth in the proximity of piers and guide bunds.

(e) If there is any appreciable concentration of flow in any part of waterway due to bend of the watercourse in immediate upstream or downstream or for any other reason, like wide variation in the type of bed material across the width of channel, the waterway may be divided into compartments as per the concentration of flow and the mean scour depth may be calculated separately for each compartment.

(f) If the watercourse is of a flashy nature and bed does not lend itself to the scouring effect of floods, the theoretical expressions for d_{sm} derived from equation 6.4 and maximum depth of scour indicated under para 6.2 below should not be applied and maximum depth should be assessed from actual observations.

6.1.6. No rational formula or data for determining scour depth for bed material consisting of gravels and boulders (normally having weighted diameter more than 2.00 mm) is available. In such cases, scour depth may be determined by actual observations and mean scour depth fixed based on such observations and theoretical calculations.

6.1.7. Similarly in the absence of any established relationship for determining the value of K_{sf} or scour depth in clayey bed, the following theoretical calculations for determining the value of silt factor (K_{sf}) for clayey bed material may be adopted:

(i) In case of soil having Ø (angle of internal friction) <15° and
c (cohesion of soil) >0.2 kg/cm²,

$$K_{sf} = F(1 + \sqrt{c}) \qquad \dots \qquad (6.5)$$

Where,

c is in kg/cm²

$$\begin{aligned} F &= 1.50 \text{ for } \text{Ø} > 10^0 \text{ and } <15^0 \\ &= 1.75 \text{ for } \text{Ø} > 5^0 \text{ and } < 10^0 \\ &= 2.00 \text{ for } \text{Ø} <5^0 \end{aligned}$$

(ii) Soils having Ø>15° may be treated as sandy soil even if value of *c* is more than 0.20 kg/cm² and silt factor worked out as per Expression 6.1.

6.2. Maximum Depth of Scour for Design of Foundations

6.2.1. For piers and abutments

6.2.1.1. Following flood conditions should be considered for the assessment of maximum depth of scour for design of piers and abutments;

(i) Flood level equal to deck level and
(ii) H.F.L.
(iii) L.W.L.

6.2.1.2. The maximum depth of scour below the designed flood level for the design of piers and abutments of submersible bridges having individual foundations without any floor protection may be considered as follows.

(I) Flood without seismic combination:

(i) For piers - $2.0d_{sm}$ ... (6.6)

(ii) For abutments (a) $1.27d_{sm}$ with approach retained or lowest bed level whichever is deeper ... (6.7)

(b) $2.0d_{sm}$ with scour all around. ... (6.8)

(II) Flood with seismic combination:

The values of maximum depth of scour around piers and abutments under flood conditions given in (I) above, may be reduced by multiplying factor of 0.9 as given below:

(i) For piers - $1.80d_{sm}$ ... (6.9)

(ii) For abutments (a) $1.143d_{sm}$ with approach retained in front or lowest bed level whichever is deeper ... (6.10)

(b) $1.80d_{sm}$ with scour all around .... (6.11)

(III) Low water level or without flood condition with seismic combination:

The values of maximum depth of scour around piers and abutments under low water level or without flood condition with seismic combination, the values given under (I) above, may be reduced by multiplying factor of 0.8 as given below:

(i) For piers - $1.60d_{sm}$ ... (6.12)

(ii) For abutments (a) $1.016d_{sm}$ with approach retained in front or lowest bed level whichever is deeper ... (6.13)

(b) $1.60d_{sm}$ with scour all around ... (6.14)

6.2.2. For floor protection

In the absence of actual observed data on similar structures in the vicinity of the proposed site, maximum scour below designed flood level for the design of floor protection works for raft or open foundations may be based on following relationships. However for raft foundation d_{sm} may be based on design discharge without any increase.

(i) In a straight reach and in a bend with angle of deviation of less than 15° — $1.27d_{sm}$ ... (6.15)

(ii) In a bend with angle of deviation

(a) between 15° and 45° — $1.50d_{sm}$ ... (6.16)

(b) between 45° and 60° — $1.75d_{sm}$ ... (6.17)

(c) between 60° and 90° — $2.00d_{sm}$ ... (6.18)

(iii) For Cut off/Curtain walls

Cut off/Curtain walls are generally designed as fully protected against scour by flexible aprons. The depths of cut-off walls are determined based on safe exit gradient (creep theory) explained in para 6.4 of these guidelines. However, it is advisable to fix the foundation level of the cut-off walls below the anticipated scour level.

6.2.3. For guide bunds/guide walls

(a)	Upstream curved mole head	$2.00d_{sm}$-$2.50d_{sm.}$	...	(6.19)
(b)	Straight reach including tail on down stream side	$1.50d_{sm}$	...	(6.20)

6.2.4. For submersible approaches

(a)	**Downstream side embankment slope and apron**			
(i)	Around abutments and for 30 per cent of the length of submersible approach (L) subject to minimum of 15 m from the abutment then gradually reducing in next 10 per cent of L to $1.50d_{sm}$	$2.00d_{sm}$	...	(6.21)
(ii)	For next 30 per cent of the length of submersible approach then gradually reducing, in next 5 per cent of length of submersible approach (L), to $1.27d_{sm}$	$1.50d_{sm}$	...	(6.22)
(iii)	For next 20 per cent of the length of submersible approach	$1.27d_{sm}$	...	(6.23)
(iv)	Balance portion upto bank gradually reducing to 0 near bank	$1.27d_{sm}$		
(b)	**Upstream side embankment slope and apron**			
(i)	For 30 per cent of the length of submersible approach subject to minimum of 15 m beyond the portion covered under (a) (i) above (i.e. flank protection around abutment) then gradually reducing in next 20 per cent of L to $1.27d_{sm}$	$1.50d_{sm}$	...	(6.24)
(ii)	For next 30 per cent of the length of submersible approach	$1.27d_{sm}$	...	(6.25)
(iii)	Balance portion up to bank gradually reducing to 0 near bank	$1.27d_{sm}$		

(c) **In case of structures with flooring** (bed protection), the above-mentioned lengths should be reckoned beyond the edge of the flooring.

6.2.5. Special studies for the increased discharge calculated as per para 6.1.2, should be undertaken for determining the maximum scour depth for the design of foundations of major submersible bridges in all situations where abnormal conditions, such as the following are encountered:

(i) the site being located in a bend of the river involving a curvilinear flow, or excessive shoal formation, or

(ii) where the deep channel in the watercourse hugs to one side or

(iii) very thick piers/arch structures, are existing or being proposed inducing heavy local scours, or

(iv) where the obliquity in the watercourse is considerable, or

(v) site is in the vicinity of a dam or weir, barrage or other irrigation structures where concentration of flow, aggradation/ degradation of bed etc. are likely to occur.

6.2.6. Steps for determination of anticipated maximum scour level

(i) Calculate the value of d_{sm} based on increased value of discharge (as per **Table 6.1**) and K_{sf} from sieve analysis (**Appendix 6.2**) or **Table 6.2**, using Expression 6.4. (In case of raft foundations d_{sm} may be based on the design discharge without any increase) D1

(ii) Find out the depth of scour holes or deepest bed level from the cross sections taken at different locations (i.e. u/s, d/s and at site) D2

(iii) Maximum depth of scour holes away from piers, vented causeways in existing structures in the vicinity of proposed site D3

(iv) Select the highest value among the D1, D2 and D3.

(v) Determine maximum depth of scour as per para 6.2. above.

For Submersible bridges, the scour level is determined for the following two conditions

(i) When the flood level is at the top of the bridge with live load on it.

(ii) When the flood level is HFL with no live load.

A worked out example of determination of anticipated maximum scour depths for a typical case is given in **Appendix 6.1.** It is seen that the scour level at former condition is different than the later (HFL) case.

6.3. Foundations

6.3.1. General

As mentioned in Chapter 3 of these guidelines, submersible bridges should normally be sited where rocky or firm strata is available at relatively shallow depths and construction of high embankments in submersible portion of immediate approaches is not involved. As such these structures would generally have shallow foundations, rendering open or raft foundation suitable. In case of sandy strata or soft soils requiring deeper foundations, well or pile foundations can be adopted. In case of bridges/causeways on erodible bed, the bed is protected against scour by provision of suitably designed flooring with cut off walls and launching aprons on either side.

However, in case the bed protection is not feasible or economically viable, deep well or pile foundations are also adopted.

6.3.2. Types of foundations

Choice of type of foundations for the selected type of structure and span arrangement has to be based on the sub-soil investigations and data of existing structures in the vicinity. Foundations can be broadly divided into two categories viz. shallow foundations and deep foundations.

6.3.2.1. Shallow foundations

Shallow foundations are adopted where suitable hard strata which is not erodible and having adequate safe bearing capacity (SBC) of about 150 kN/m^2 or more is available at shallow depth below bed level of the watercourse. Shallow foundations are preferred for submersible bridges because they are economical and easy to execute. In case of sandy stratum extending to considerable depth, shallow foundations are adopted by restricting the scour to the top of suitably designed bed protection. Shallow foundations are categorized into following types:

(i) **Isolated open foundations:**
Isolated open foundations are generally adopted where the safe bearing capacity (SBC) of about 150 kN/m^2 or more is available at shallow depths. However, if sub soil is porous and water table is high this type of foundation would be feasible only upto 3 to 4 m below the bed.

(ii) **Raft foundations:**

Foundation block covering the entire length and width of the proposed bridge structure is commonly known as raft foundation. It is adopted when good founding strata is not available within reasonable depth or SBC. of top stratum is less than 100 kN/m^2. The concrete raft serves to distribute pressure evenly to the sandy surface/ clayey strata and also the arch action of the foundation block helps in distributing the loads evenly. Another economical alternative in such situation is the adoption of multiple continuous box type structure using full box section as a beam on elastic foundation.

6.3.2.2. Deep foundations

If suitable founding stratum is available only at a depth greater than 6 m with substantial depth of standing water and large scour depth then execution of open foundations and the bed protection works becomes difficult. In such a situation it is advisable to adopt well foundations or pile foundations

(a) **Well Foundations:**

This is one of the most popular types of deep foundations in India, due to various reasons like simplicity, requirements of very little equipment for execution, and adaptability to different subsoil conditions/difficult site conditions like deep standing water and availability of good founding strata at large depths etc.

(b) Pile Foundations: - Pile foundations are also viable alternative. Large diameter piles some times prove to be a good alternative to conventional well type foundations. IRC:78 may be referred for detailed information with regard to various types of pile foundations and their suitability.

6.3.3. Minimum depth of foundation

Foundations of any structure including cut-off walls should be deep enough and safe to transfer loads and force transmitted through sub-structure under worst combination of loads and forces specified in IRC:6, IRC:78 and Chapter 7 of these guidelines. However the minimum depth of open foundations shall be as follows:

(i) In erodible strata - not be less than 2.0 m below the scour level or the protected bed level.

(ii) In Rock

(a) For hard rock, with an ultimate crushing strength of 10 MPa or more, arrived at after considering the overall characteristics of the rock such as fissures, bedding planes etc. : 0.6 m

(b) All other cases : 1.5 m

6.4. Bed Protection or Floor Protection for Shallow Foundations

6.4.1. General

In case of causeways and submersible structures, where adoption of shallow foundations becomes economical by restricting the scour, bed (floor) protection to the structures has to be provided. The bed protection is provided to guard against scour and undermining (washing away or disturbance by piping action etc.) which can be quite severe in the case of vented causeways and submersible structures. The bed/floor protection consists of the following components:

(i) upstream flexible apron;
(ii) upstream cut-off wall;
(iii) rigid flooring;
(iv) downstream cut-off wall and;
(v) downstream flexible apron

The sizes of various components of bed/floor protection are fixed on the basis of experience of previous such works and then checked for the adequacy for the proposed structure. The length of flexible apron is fixed keeping in view the shape of apron in launched position to protect the foundations of cut-off walls as well as main foundations of the structure from being undermined. However, it is essential that the performance of rigid flooring and flexible apron is kept under watch, especially on the downstream side, for first few floods after construction and remedial measures should be taken immediately in the form of replenishment of boulders disturbed during the floods. In extreme cases, it may become necessary subsequently to increase the length of downstream apron in case the performance proves it to be inadequate.

The work of bed protection should be completed simultaneously along with the work on the causeway or foundations of the structure to prevent any damage due to unexpected floods even during the construction stage.

6.4.2. The floor protection should be properly designed as per the detailed guidelines contained in IRC: 89. However the minimum specifications for floor protection are given below for guidance. These may be adopted in the absence of any other more rigorous requirements/rational design.

(i) The post construction velocity under the structure should not exceed 2 m/s and the intensity of discharge is limited to 3m^3/m except in the case of properly designed raft foundation with adequate protective works.

(ii) The rigid flooring under the structure should extend for a distance of at least 3 m on upstream side and 5 m on downstream side of the structure. However if splayed wing walls are provided, the flooring should extend upto the line connecting the end of the wing walls on either side of the structure. In case of well designed raft foundation with upstream and downstream cut-off walls, the depth of cut-off walls and length of flooring (apron) could be suitably changed depending on the successful practice followed in the State.

(iii) The top of the flooring should be kept 300 mm below the lowest bed level to prevent the flooring from acting as a weir when retrogression of levels take place.

(iv) The flooring may consist of 150 mm thick flat stone/bricks on edge in cement mortar1:3 laid over 300 mm thick cement concrete M15 grade over a layer of 150 mm thick cement concrete M10. In case of streams carrying abrasive particles with velocities higher than 4 m/sec., an alternative specification of flooring comprising of 450 mm thick concrete layer in M20 over 150 mm thick concrete layer in M15 grade can be adopted. Spacing of the joints should be limited to about 20 m.

(v) The rigid flooring should be enclosed by cut-off/curtain walls (tied to the wing walls) with a minimum depth below floor level of 2 m on upstream side and 2.5 m on down stream side. The cut-off/curtain walls should be in cement concrete M15 grade/brick/ stone masonry in cement mortar 1:3. The rigid flooring shall be continued over the top width of the cut-off/curtain walls. Horizontal/vertical joints in curtain/cut-off walls should be avoided.

(vi) Flexible apron 1 m thick comprising of loose stone boulders (weighing not less than 40 kg) should be provided beyond the curtain/cut-off walls. The length of apron on the downstream side should be adequate to launch upto the designed maximum scour level in a slope of 2 horizontal to 1 vertical and the length of apron on the upstream should not be less than about 0.7 times of the same on downstream side subject to a minimum length of 4 m on upstream side and 6 m on downstream side. In case required size of stones are not economically available, stones in wire crates or cement concrete blocks may be used in place of specified stones.

(vii) Crated boulder aprons should also be preferred at sites near inhabited areas so as to discourage removal of stones by anti-social and unscrupulous elements.

(viii) It is essential that the work of bed protection is simultaneously completed along with the work on the foundations of the structure to prevent any damage to the foundations.

(ix) Typical arrangements of flooring with cut-off walls etc. are indicated in **Fig. 5.1** in Chapter 5 and **Fig. 6.1.**

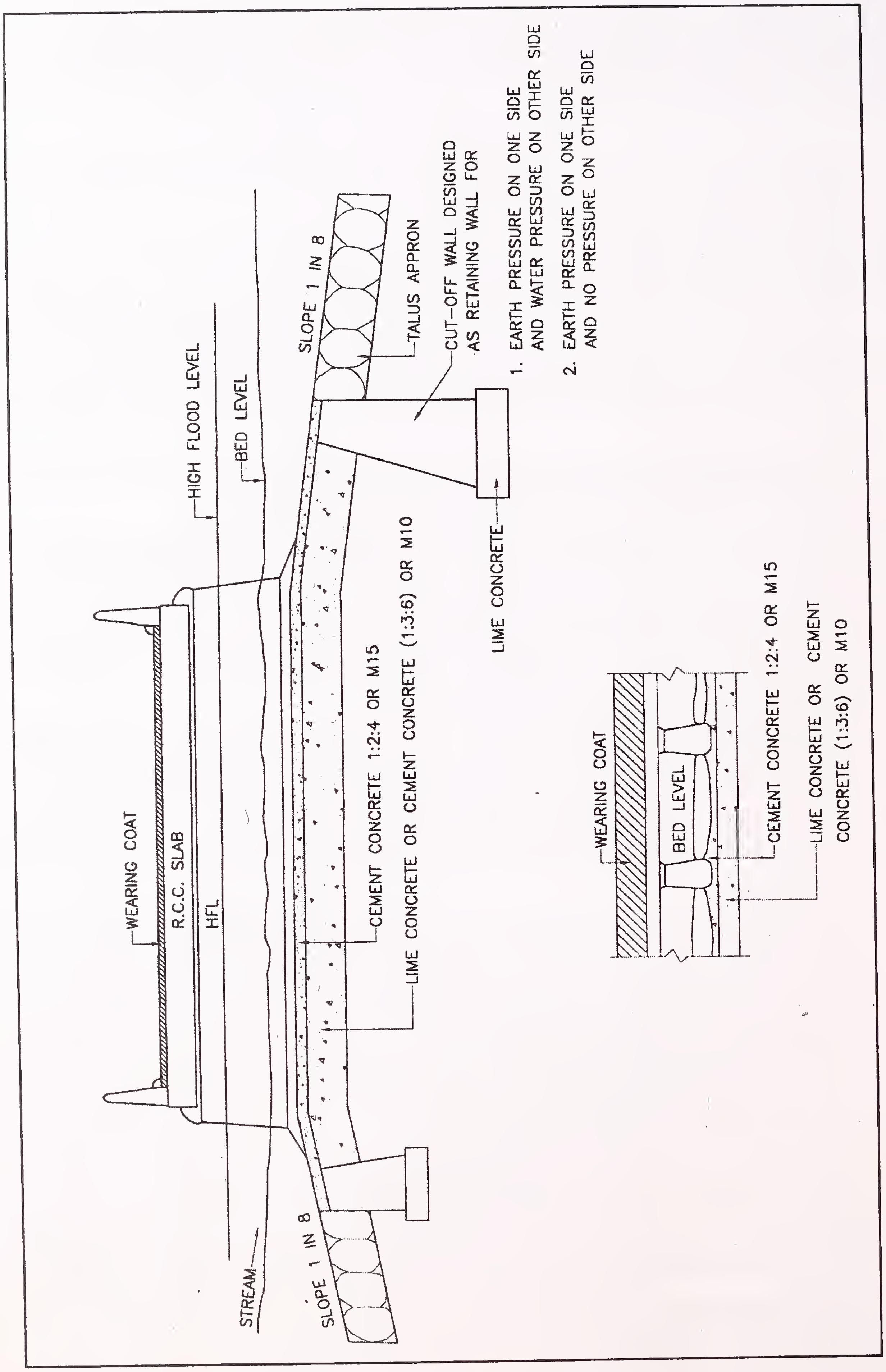


Fig. 6.1. Typical detail of florring with apron

6.4.3. Design

6.4.3.1. When a rigid (Pukka) floor is provided, the water in addition to flowing over the floor also creates a path for itself along the surfaces of the upstream cut-off wall and the under side of the floor protection to finally emerge on the downstream side. If the exit gradient of water when it emerges is high then it would be capable of carrying soil particles with it. This leads to piping action, which can cause progressive loss of material from under the floor resulting finally in damage or collapse of the floor. To prevent this from happening, the dimensions of the rigid flooring have to be such as to keep the exit gradient quite low. The theories used for design of weirs on alluvial soils are generally used for checking the exit gradient of water under the rigid flooring of the bridge. However, in case of causeways/ submersible bridges, difference in depth of water on the upstream side and downstream side (pressure head) is only due to the afflux. Since a large value of afflux cannot be permitted in the design of submersible structures, the pressure head is very small when compared with the depth of water on downstream side. However, for purposes of design, difference of depth of say 1 m may be assumed between the upstream and downstream sides of submersible bridges and vented causeways with floors and 500 mm in case of flush causeways which generally span the whole waterway.

6.4.3.2. There are three methods of checking the adequacy of the proposed sizes of different components of bed protection, viz:

(i) Bligh's creep theory

(ii) Lane's weighted creep theory, and

(iii) Khosla's method of independent variables applied to potential theory.

The salient points useful for the design of cut-off walls and aprons for finding out the hydraulic gradient at exit for the flooring are given below for ready reference and guidance.

(i) **Bligh's Creep Theory**: As per this theory the total equivalent length of water path (creep length) is given by

$$C \times H = L + 2\,(d_1 + d_2) \qquad \ldots \quad (6.26)$$

Where,

H = Pressure head, taken as equal to depth of water on upstream side including afflux (-) depth of water on downstream side.

(Minimum values of pressure head for submersible bridges/vented causeways and flush causeway may be assumed as 1.00 m and 500 mm respectively.)

L = Total length of all horizontal contact surfaces of water path.

d_1 = depth of upstream cut-off wall

d_2 = depth of downstream cut-off wall

(According to Bligh's theory, vertical cut-offs are twice as effective as horizontal flooring in contributing to creep length and the rate of head loss along the creep length is assumed to be linear.)

C = a constant called creep coefficient. The values of C for different soils are as given in **Table 6.3**.

Table 6.3: Creep Coefficients

Material	Bligh's Creep coefficient (C)	Lane's weighted Creep coefficient (C_1)
Very fine sand and or silt	18	8.5
Fine sand	15	7.0
Coarse sand	12	5.0
Gravel and sand	9	3.0 to 3.5
Boulders, gravel and sand	9	2.5 to 3.0
Clayey soils	4 to 6	1.6 to 3.0

(ii) **Lane's Theory:** This is based on statistical study of performance of actual structures on permeable foundations, a greater weightage is given to the vertical cut-offs than horizontal water contacts and is assumed in the ratio of 3:1. The creep length (equivalent length of water path) works out to be

$$C_1 H = 1/3\ L + (d_1 + d_2) \quad \ldots \quad (6.27)$$

Where C_1 = weighted creep coefficient as given in **Table 6.3**.

Lane's approach, though an improvement over Bligh's creep theory, still has the limitation of an empirical method. This is however, commonly used to cross check the adequacy of the proposed designs based on Bligh's Creep Theory.

(iii) **Khosla's Theory:** When the subsoil is homogeneous, Khosla's method of independent variables applied to the theory of sub-soil flow is the most reliable. In case of floor protection with cut-off walls on either end, the **exit gradient (G_E)** is worked out by the Expression:

$$(G_E) = \frac{H}{d_2} \times \frac{1}{\pi \sqrt{\lambda}} \quad \ldots \quad (6.28)$$

Where,

$$\lambda = \frac{1 + \sqrt{(1+\alpha^2)}}{2} \quad \ldots \quad (6.29)$$

α = L/d_2

H = head of the water (1 m or 0.50 m as the case may be)

d_2 = depth of downstream cut off wall.
As per Khosla's theory, the depth of up-stream cut off wall does not have any effect on exit gradient and, as such, only the depth of down-stream cut off wall is considered)

L = length of the floor

To safeguard against undermining, the exit gradient should not exceed the numerical value of unity. However, in order to provide adequate factor of safety, the dimensions of the cut-offs and floor should be selected in such a way that the exit gradient does not exceed the permissible values indicated in **Table 6.4.**

Table 6.4: Permissible Exit gradient for different materials

S. No.	Material	Permissible Exit Gradient
1.	Shingle	1/4 to 1/5
2.	Coarse sand	1/5 to 1/6
3.	Fine sand	1/6 to 1/7
4.	Clay	1/3 to 1/4

6.4.3.3. To find the required thickness of floor, the uplift pressures at salient points are calculated and the thickness necessary to counter this uplift pressure is determined. To investigate the adequacy of the proposed thickness of flooring, the values recommended in 'Design of Weirs on Permeable Foundation' – Central Board of Irrigation Publication No.12 are generally adopted (Relevant **Table** and Graphs are reproduced at **Appendix 6.3**).

6.4.3.4. Beyond the rigid flooring, flexible aprons of designed length are provided on either side to keep the scour holes at safe distance. The size and weight of stones/boulders required for launching apron to resist mean design velocity (average velocity) is given by the expression:

$$v = 4.893\ (d)^{1/2}$$

Where,

v = mean design velocity in m/s

d = equivalent diameter of stone/boulder in m.

The weight of the stone/boulder can be determined by assuming the spherical stone/boulder having the average specific gravity of 2.65 and are given in **Table 6.5** for reference and adoption.

Table 6.5

S. No.	Mean design velocity (m/s)	Minimum size and weight of stone/boulder	
		Diameter (cm)	Weight (kg)
1.	Upto 2.5	30	40
2.	3.0	38	76
3.	3.5	51	184
4.	4.0	67	417
5.	4.5	85	852
6.	5.0	104	1561

Note: No stone weighing less than 40 kg should be used in apron. Where required size stones are not economically available, cement concrete blocks (M15 grade) or stones in wire crates or cement concrete blocks and stones in combination may be used in place of isolated stones of equivalent weight. Cement concrete blocks should however, be preferred wherever possible.

In all the above cases, the pressure at the base of the foundation should be well within the safe bearing capacity of the foundation material.

Worked out example for investigating the adequacy of bed protection, depth of cut off walls is given in **Appendix 6.4**.

Appendix 6.1
(Reference Para 6.2.6)

WORKED OUT EXAMPLE OF DETERMINATION OF ANTICIPATED MAXIMUM SCOUR DEPTH

I. Hydraulic Data

(i)	Catchment area	9500 sq. km.	
(ii)	H.F.L. (with return period of 50 years)	RL	95.22 m
(iii)	Flood Level (Return period of 10 years)	RL	92.2 m
(iv)	O.F.L.	RL	89.0 m
(v)	L.W.L.	RL	84.6 m
(vi)	Lowest bed level (LBL)	RL	83.2 m
(vii)	Velocity		
	(a) at H.F.L.	4.61 m/sec	
	(b) at Flood Level (10 years)	2.80 m/sec	
	(c) at O.F.L.	2.0 m/sec	
(viii)	Discharge worked out as per procedure outlined in Chapter 4 of these guidelines		
	(a) At H.F.L.	12625 cumecs	
	Percentage increase over design discharge for calculations of foundation design as per para 6.1.2.1 – **Table 6.1**	21%	
	∴ Design discharge = 1.21 X 12625 = 15276 cumecs		
	(b) Design discharge at M.F.L.	5659 cumecs	
	(c) Design discharge at O.F.L.	2870 cumecs	
(ix)	Water Spread at		
	H.F.L.	550 m	
	F.L (10 years)	390 m	
	O.F.L.	250 m	

II. Bed Material

Medium/coarse ~~sand and scourable~~.
Banks – quasi alluvial

III. Proposed Structure

22 spans of 18 m c/c with following salient details:

(i)	Deck level	RL	= 92.0
(ii)	Depth of superstructure		= 1.0 m

(iii) Height of discontinuous kerb = 0.3 m
(iv) Width of pier cap =1.2 m
(v) Width of solid pier =0.9 m

Step I

Assessment of discharge at bridge deck level (RL 92.0) during H.F.L. .... Q_1

Q_1 = Design discharge – discharge likely to flow over the bridge deck (i.e. between RL 95.22 and R.L. 92.0)

= 15276- [(Length of water spread x velocity of flow at R.L. $\frac{95.22 + 92.0}{2}$ = 93.61) x depth of flow]

Water spread at R.L. 93.61 = 472.50 m

Calculation of velocity of flow at R.L. 93.61
(Refer clause 213.2 of IRC:6)
$2v^2$ at R.L. 95.22 = 2 x 4.61^2 = 42.50
$2v^2$ at R.L. 93.61 = 42.50 X 12.61/14.22 = 37.69
$v = \sqrt{18.845} = 4.34$ m/sec
Q_1 = 15276-472.5 x 4.34 x 3.22
= 15276-6603
= 8673 cumecs, say 8700 cumecs

Step II

Assessment of scour depth at H.F.L. with bridge in submerged condition

Assuming wide alluvial stream, scour depth (hydraulic mean radius-R) and value of K_{sf} from **Table 6.2** for medium/coarse sand from Expression 6.3

$$\text{Scour depth} = 0.473 \left(\frac{Q}{K_{sf}}\right)^{1/3}$$

$$= 0.473 \left[\frac{15276}{1.25}\right]^{1/3}$$

$$= 10.9 \text{ m}$$

Unobstructed natural area of flow at H.F.L $= \left(\dfrac{Q}{v}\right) = \left(\dfrac{15276}{4.61}\right) = 3314$ sq m

Obstructed natural area of flow at H.F.L. = Area of superstructure between the faces of abutments + No. of piers (area of pier cap + pier shaft)

= 394 x 1.30 + 21 (1.2 x 0.6 + 6.8 x 0.9)
= 655.8 sq m
= say 656 sq m

Modified scour depth $= 10.90 \left(\dfrac{\text{Unobstructed area}}{\text{Net area}}\right)^{0.61}$

$= 10.90 \left(\dfrac{3314}{3314-656}\right)^{0.61}$

$= 12.47$ m

Scour level = RL 95.22 – 12.47 = RL 82.75

Step III

Scour depth when flood level is at R.L. 92.0

Scour depth $= 0.473 \left(\dfrac{6750}{1.25}\right)^{1/3}$

= 8.3 m

Unobstructed natural area of flow at R.L. 92.0 $\dfrac{Q}{v} = \dfrac{6750}{2.80} = 2411$ sq m

Increased scour depth done to obstruction $= 8.3 \left(\dfrac{2411}{2411-655.8}\right)^{0.61}$

= 10.1 m

Scour level for flood level at R.L. 92.0 = RL 92.0-10.1 = RL 81.9

<u>Second Method</u>

Scour depth during flood level of RL 95.22 using Expression 6.4

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

Effective linear waterway is assumed as water spread at H.F.L. is 550 m

$$D_b = \frac{Q}{\text{Water spread}} = \frac{15276}{550} = 27.77 \text{ cumecs/m}$$

say 28 cumecs/m

$$d_{sm} = 1.34 \left(\frac{28^2}{1.25}\right)^{1/3}$$

Scour level = R.L. 83.75 say Deepest Bed Level (DBL) RL 83.2

With submerged structure

$$\text{Scour depth} = d_{sm}\left(\frac{\text{unobstructed area}}{\text{Net area}}\right)^{0.61}$$

$$= 11.47\ (3314/2658)^{0.61}$$
$$= 13.14 \text{ m}$$

Scour level at H.F.L. R.L. 82.08 say RL 82.0

(b) Scour depth during flood level at RL 92.0
Effective linear waterway = 390 m

$$D_b = \frac{6750}{390} = 17.31 \text{ cumecs/m}$$

$$d_{sm} = 1.34 \left(\frac{17.31}{1.25}\right)^{1/3} = 8.32 \text{ m}$$

With reduced vent area

$$\text{Scour depth} = 8.32 \left(\frac{2651}{1748}\right)^{0.61} = 10.73 \text{ m}$$

Scour level = R.L. = 81.27

Above calculations indicate that scour is more when the flood level is at deck level rather than when it is at H.F.L.

Scour level may be assumed as under
During flood level at R.L. 95.22 = RL 82.00
During flood level at R.L. 92.00 = RL 81.20

At O.F.L. i.e. R.L. 89.0

Effective waterway as per Clause 104 of IRC: 5
=394 – 21 x 0.9 = 375.2 say 375 m

$$D_b = \frac{2870}{375} = 7.65 \text{ cumecs/m}$$

$$d_{sm} = 1.34\ (7.65^2/1.25)^{1/3} = 4.83 \text{ m}$$

Scour level = 89.0 – 4.83 = RL 84.17
Deepest Bed Level (DBL) = RL 83.2

Assume scour level as D.B.L. i.e. RL 83.2 being on conservation side.

Maximum Scour

	At H.F.L.	At F.L. (10 Years)	At O.F.L.
Around Piers Expression 6.6 2 dsm	2[95.22-82.0=13.22] = 26.44 m	2[92.2-81.2]= 22.0 m	2 x 4.83 = 9.66 m
R.L.	68.78	70.20	79.34
Around Abutments Expression 6.8	68.78	70.20	79.34
Expression 6.7 1.27 d_{sm}	78.41	78.23	L.B.L. 83.20

Appendix 6.2
(Reference Para 6.1.4.1)

PROCEDURE TO WORK OUT THE WEIGHTED MEAN DIAMETER OF PARTICLES FOR A STRATUM

The weighted mean diameter of particles for a stratum may be worked out as per method illustrated in the **Table 6.6.**

Table 6.6

Sieve Designation	Sieve Openings (mm)	Weight of soil retained (gm) (w_i)	Percentage of weight retained (p_i) = col (3) x 100/ ΣW_i (Total weight of sample)	Average size of opening (mm)	Y_i= [Col (5)] percentage of weight of material retained on the next small size sieve] x [Col (4)]
1	2	3	4	5	6
5.60 mm	5.60	Say 0	0	-	0
				4.80*	4.80 x p_1
4.00 mm	4.00	w_1	p_1		
				3.40	3.40 x p_2
2.80 mm	2.80	w_2	p_2		
				1.90	1.90 x p_3
1.00 mm	1.00	w_3	p_3		
				0.712	0.712 x p_4
425 micron	0.425	w_4	p_4		
				0.302	0.302 x p_5
180 micron	0.180	w_5	p_5		
				0.127	0.127 x p_6
75 micron	0.075	w_6	p_6		
				0.0375	0.0375 x p_7
< 75 micron in Pan		w_7	p_7		
	W_i (Total weight of sample)	100			Y_i

* (5.60+4.00)/2 = 4.80 mm and so on

$$d_m = \frac{\Sigma Yi}{100} \quad \text{(rounded off to two decimal places)}$$

Appendix 6.3
(Reference: para 6.4.3.3)

Table 6.7: Thickness of Slope Patching for various Grades of Sand and Slopes of Rivers

Bed Slope (10^{-5})	4.73	14.2	18.9	28.4	37.9
Sand classification	Thickness of stone pitching in (cm)				
Very coarse	41	48	50	64	71
Coarse	58	64	71	79	86
Medium	71	79	88	94	102
Fine	86	94	102	109	117
Very Fine	102	109	117	124	132

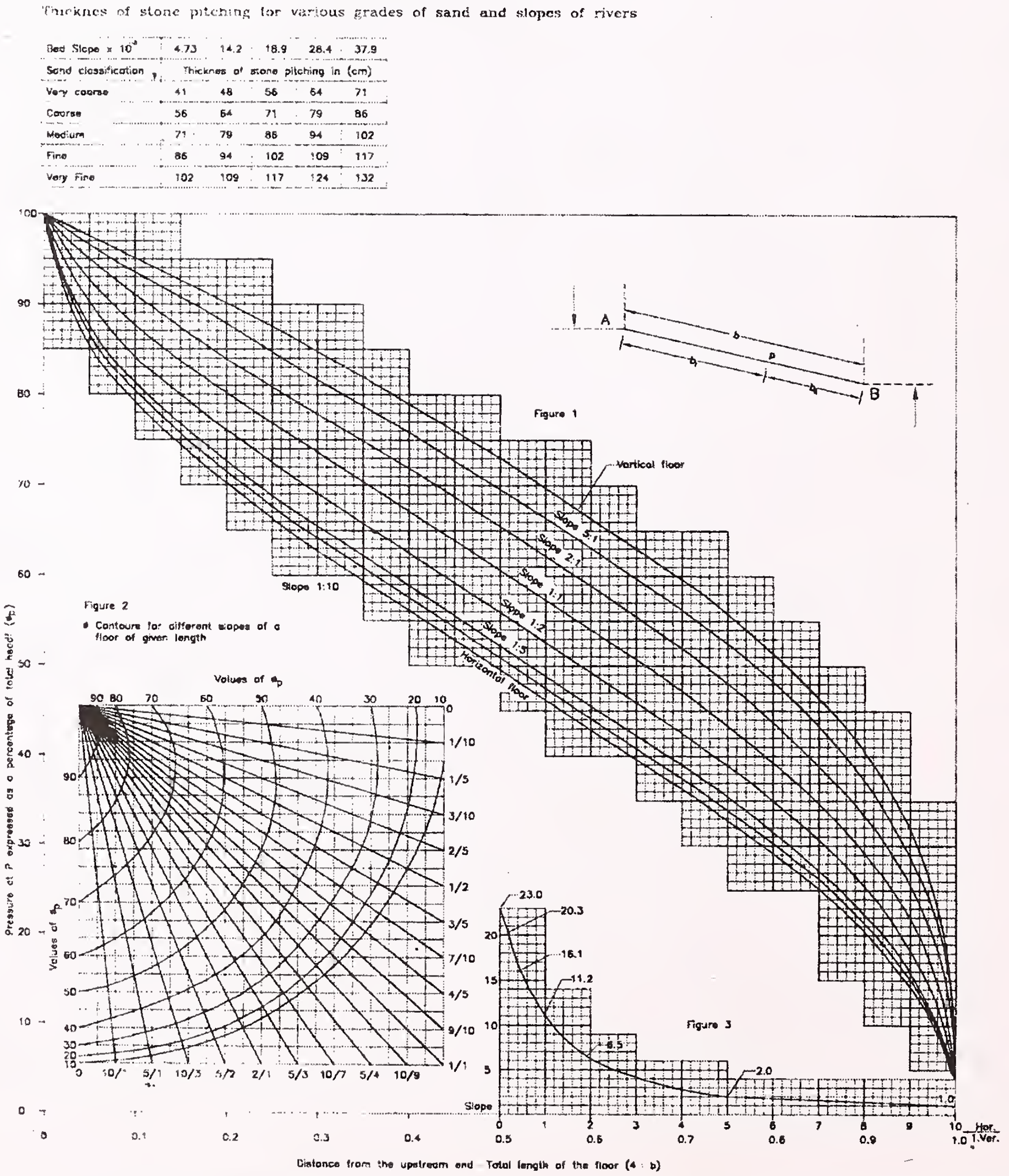


Fig. 6.2. Showing Pressure Distribution under Floors with different Slopes

Appendix 6.4
(Reference para 6.4.3.4)

DESIGN OF BED PROTECTION
(Worked out example)

Given Data:

Design discharge			=	1000 cumecs
Total effective waterway			=	250 m
Depth of flow above floor level at designed flood level			=	2.70 m
Afflux			=	500 mm
Width of the foundation			=	10.0 m
Mean designed velocity			=	2.0 m/s
(Maximum permissible velocity as per IRC:89 is 2.0 m/s)				
Area available for discharge	=	250 x 2.70	=	675 m^2
Discharging capacity	=	675 x 2	=	1350 cumecs
				> 1000 cumecs
				Hence OK
Width of the foundation			=	10.0 m
Length of flooring as per IRC:89	=	10.0 + 3.0 + 5.0	=	18.0 m
{Para 6.4.2 (ii) refers}				
Depth of upstream side cut-off wall			=	2.50 m
(Minimum required as per IRC:89 is 2.0 m)				
Depth of downstream side cut-off wall			=	3.50 m
(Minimum required as per IRC:89 is 2.50 m)				

(i) As per Bligh's Creep Theory

Total length of water path =2 x 2.50 + 18 + 2 x 3.50 = 30.0 m

Assume difference of head between upstream and downstream sides as 1.0 m as per Para 6.4.3.1 against value of afflux of 500 mm.

Hydraulic gradient = 1.0/30.0 i.e. 1 in 30

As the gradient of 1 in 30 is far less than even the flattest permissible gradient of 1 in 18 (value of creep coefficient for very fine sand or silt), as per **Table 6.4**, the length of flooring provided is adequate.

(ii) As per Lane's Weighted Creep Theory

The equivalent length L_w = 1/3 x 18.0 + 2 x 2.50 + 2 x 3.50 =18.0 m

Creep ratio C_l = 18/1.0 = 18.0

Thus the creep ratio is much higher than the requirement (**Table 6.4** refers). Hence flooring length provided is adequate.

(iii) As per Khosla's Theory

Using Expressions 6.31 and 6.32 for exit gradient i.e.

$$G_E = \frac{H}{d_2} \times \frac{1}{\pi \sqrt{\lambda}}$$

Where

$$\lambda = \frac{1 + \sqrt{(1+\alpha^2)}}{2} \quad \text{and} \quad \alpha = L/d_2$$

H is the head of water	=	1.0 m
d_2 is the depth of downstream cut-off wall	=	3.50 m
L is the length of floor	=	18 m
α = 18.0/3.50	=	5.14

$$\lambda = \frac{1 + \sqrt{(1+\alpha^2)}}{2} = 3.12$$

$$\text{Exit gradient } (G_E) = \frac{1}{3.5} \times \frac{1}{\pi \sqrt{3.12}} = 1/19.42$$

The exit gradient is within the permissible values (**Table 6.5**)

Hence the proposed length of the floor as 18 m and depth of cut-off wall as 3.5 m are adequate.

Check for adequacy of the thickness of the proposed flooring by Khosla Theory

Assume the following composition of the flooring as per Para 6.4.2 (iv):

150 mm thick cement concrete levelling course (M10)

+ 300 mm thick cement concrete (M15)

+ 150 mm thick brick masonry in cement mortar 1:3.

$1/\alpha = d_2/L = 3.50/18.0 = 0.194$

From Khosla's graph,

Pressure at the bottom of the cut-off wall on the u/s side

$= 100 - \varphi_D = 100 - 26.2 = 73.8\ \%$

Pressure at point of beginning of the floor on the u/s side

$= 100 - \varphi_E = 100 - 38.3 = 61.7\ \%$

Loss of pressure from bottom of u/s cut-off walls to beginning of floor = 12.1%

$$\text{Correction for thickness of floor} = \frac{12.1 \times 0.6}{3.5} = 2.07\ \%$$

Correction for interference of d/s cut-off wall

$$C = 19\sqrt{d_2/L'} \times \frac{d_1 + d_2}{L}$$

Where,

d_1 = depth of u/s cut-off wall and d_2 = depth of d/s cut-off wall

L' = distance between the two cut-off walls

L = length of floor

As cut-off walls are proposed at the end of flooring therefore L = L'

$$C = 19\sqrt{3.5/18} \times \frac{3.5+2.5}{18} = 2.79\ \%$$

Corrected pressure at underside of floor where it begins

= 61.7 + 2.07 + 2.79 = 66.56 %

Average density of floor = [0.450 (concrete) x 2.2 + 0.15 (brick masonry) x 1.9] ÷ 0.6

= 2.125 t/m^3

Density of floor allowing for 100 % buoyancy = 1.125 t/m^3

$$\text{Thickness of floor required} = \frac{0.6656 \times 1}{1.125} = 0.592 \text{ m}$$

against proposed thickness of 0.6 m.

Hence OK

7. DESIGN

7.1. Site Inspections

7.1.1. General

Site inspections play an important role right from the conception stage to the end of pre-construction stage for successful implementation of any project. The purpose of site inspection ranges from identification of data needs and collection of raw data at elemental level to an overall appraisal of the project to aid in analysis, decision making and financing. As such, each inspection should be carefully planned keeping in view the requirements of the user authority and to aid judgments for most economical and feasible solution. The number of site inspections depend upon the type of proposed submersible structure (i.e. flush/vented causeways, bridge) length, site conditions, importance of the crossing etc.

The site inspections should be carried out by experienced bridge engineers as these form the basis for arriving at the basic design parameters for the project, especially in case of submersible structures and immediate approach roads which are subjected to frequent submergence.

7.1.2. Types of site inspections

Based on the stage of the project, site inspections can be classified as under:

I Pre-construction stage inspection

- (i) Pre-feasibility stage inspection
- (ii) Feasibility stage inspection
- (iii) Detailed engineering stage inspection

II Construction/Implementation stage inspection

III Post construction/Maintenance stage inspection

7.1.3. Broad requirements of site inspection

7.1.3.1. Pre-feasibility stage inspection

A reconnaissance visit to the area of the intended submersible structure site is generally sufficient to examine the general area and to identify the project, its requirements, utility, present arrangements of crossing, nature of the crossing, traffic intensity, land marks and broad features for the crossing etc. The inspection report at this stage should give broad idea of area, intended project and in general inter-alia cover the following:

- (i) Requirements of the user authority in charge of the road and its limitations, if any, regarding availability of funds for the project;
- (ii) Category of road (i.e. SH/MDR/RR/E&I road/industrial road/link road etc.);

(iii) Population of the area likely to be benefitted with the construction of the proposed facility

(iv) Existing alignment of road indicating deficiencies/constraints, if any

(v) Salient details of the existing road i.e. condition of the road, type of surfacing, carriageway width, number of lanes, formation width, shoulders etc.

(vi) Type of terrain, type of present traffic (i.e. slow moving/ fast moving etc.)

(vii) Present traffic intensity including broad breakup (on rough percentage basis) of traffic i.e. heavy/light commercial vehicles, passenger cars, slow moving vehicles e.g. bullock carts, tongas etc.

(viii) Nature of the water channel (i.e. Perennial, seasonal, steady, flashy, approximate rate of rise of flood level etc.)

(ix) Approximate velocity of stream and material likely to float down in the channel during floods i.e. debris/ branches of trees and their approximate size

(x) Condition of banks (i.e. their slopes, whether low or high, erodible or non-erodible)

(xi) Present arrangement for crossing during dry season and floods (i.e. boats, motorized ferry etc), availability of alternative route and length of detour etc.

(xii) Approximate depths of water and water spreads during dry season and floods (at HFL, OFL and LWL)

(xiii) Characteristics of river bed (i.e. alluvial, quasi-alluvial, presence of big boulders affecting the velocity of stream, scourable or non-scourable etc.)

(xiv) Presence of rocky strata

(xv) Possible location of submersible structure with respect to the most suitable site for the high level bridge

(xvi) Period of cut-off of the area with duration at a time and number of such interruptions in a year and population affected

(xvii) Expected land acquisition problems, if any, for immediate approaches

(xviii) Broad details of existing CD works (bridges or causeways) on the same channel in the vicinity. The details should include;

- (a) description like type, distance from the proposed site etc.
- (b) approximate length and depth of submergence and frequency (including duration) of interruptions per year to traffic
- (c) number and length of spans/size of vents, clear waterway, adequacy or otherwise of waterway with special reference to silted up spans or signs of under scour or attacks on abutments and approaches in case of bridges etc.

(xix) Broad justification for the project and

(xx) Any other aspect considered important by the inspecting officer.

7.1.3.2. Feasibility stage inspection

During the feasibility stage, data in general, should be collected in line with the provisions of IRC:5, IRC:78 & IRC:SP:54 (Project Preparation Manual) to enable the controlling authority to take a decision about the type, length, foundation etc. of submersible structure.

At least one visit should be carried out in the beginning of this stage, to identify the data required to be collected pertaining to the items mentioned under para 7.1.3.1 above in various stages and to establish coordination with different departments responsible for supply of data etc. In case of a flush causeway, one or two site visits may be sufficient. However, in case of vented causeway and submersible bridge, higher number of inspections for site data/conditions at various stages should be planned to avoid major changes in the proposal at a later date. Officers from design office should also inspect sites and review the adequacy of the data collected.

This stage is considered very important as the review of the data collected, recommendations of the inspecting officers from field as well as from design wing form the basis of final selection of the site, alignment, most importantly type of the structure i.e. flush or vented causeway or submersible bridge and detailed engineering.

Since the selection of submersible structure, its design i.e. deck level, length of the bridge, design of approaches mainly depends on the correctness of the flood levels, their spread, subsurface investigations, due attention is to be paid to the analysis of the data. The assumptions made in analysis, results and recommendations should have the approval of the competent authority.

A joint inspection by a team of officers of user authority and the executing agency (P.W.D. or any other agency) of the finally selected site and approach alignment should invariably be conducted in case of important submersible structures having length more than 60 m to avoid revision/modifications in the proposal at a later date i.e. during detailed engineering or execution stage.

7.1.3.3. Detailed engineering stage inspection

Frequent inspections during this stage are required to be carried out to ensure adequacy and accuracy of investigations, data collection for proper identification of sources for materials, preparation of working drawings for the structure at the finally approved site along with geometric parameters with respect to the needs of the project and user authority, detailed specifications for different items of work, bill of quantities and rates for realistic cost estimation.

7.1.3.4. Construction/Implementation stage inspection

Site inspections are carried out to ensure that assumptions made during the detailed design are realised at site and to give further instructions to analyse the additional data collected during actual execution/effect modifications in the working design drawings, quality assurance measures, to remove bottlenecks/constraints for successful timely completion of the bridge project.

7.1.3.5. Post construction/maintenance stage inspection

Regular inspections of submersible structures should be carried out as per the guidelines given in IRC:SP:18 (Manual for Highway Bridge Maintenance Inspection), IRC:SP:35 (Guidelines for Inspection and Maintenance of Bridges) and MOSRT&H Manual for Maintenance of Roads 1983.

Box girder sections of submersible bridges shall be cleared of silt deposit after each flood season.

7.2. Site Selection

7.2.1. The location of causeways should generally be governed by the approach roads alignment except in difficult site conditions.

7.2.2. The site for submersible structures should generally be selected down stream side of the most suitable site for the high level bridge so that the same serves as diversion during the construction of high level bridge at a later date. The distance between the finally selected site for the submersible bridge from the most suitable site for the high level bridge should be at least equal to the tentative length for the high level bridge at that site, but not less than 50 m in any case. Submersible bridge should generally be proposed on the upstream side of the existing causeway so as to ensure that the afflux caused by the construction of the submersible bridge does not reduce the functioning of the existing causeway.

7.2.3. Factors affecting site selection

Some of the important factors requiring due consideration in the selection of a site are listed below. Though it may not be possible to satisfy all attributes simultaneously, the selected site should represent the most desirable mix of the attributes consistent with overall economy.

(a) The reach of the watercourse on both u/s and d/s of the proposed site should be straight to the extent possible. The straightness of reach of the watercourse ensures uniform distribution of discharge and velocity. Curvature in the watercourse especially on u/s side leads to concentration of flow and higher scour on concave side. If the bank on the concave side is erodible then it may lead to heavy recurring expenditure in protecting the abutment and approach on that side.

(b) The watercourse should not have the history of meandering i.e. changes in the course at the site. (This can be ascertained from the past maps prepared over a long time).

(c) The bridge site should be sufficiently away from confluence of large tributaries, where turbulence and obliquity of flow can be expected which results in higher unpredictable scour and water current forces on the submersible structure, washing away of approaches, bank erosion, out skirting the structure etc.

(d) The banks should be well defined and fairly high at least for the OFL.

(e) The watercourse should be narrow to ensure large average depth of flow compared to maximum depth of flow in some reaches.

(f) The site should offer possibilities of constriction in the waterway in the range of $^2/_3$ to ¾ of the waterway required for a high level bridge.

(g) The bed conditions should offer good foundation conditions, preferably non-scouring type (i.e. rock etc).

(h) The site should offer straight approaches and square crossing.

(i) The site should not involve construction of high embankment in submersible portion of approaches and/ or over exposed rocky strata.

7.3. Essential Design Data

7.3.1. General

Due attention should be paid to the collection and analysis of data and fixing of salient design details of submersible bridges to avoid major changes in the proposal during execution or excessive maintenance problems during service. Correct and proper presentation of data enables the controlling authority to accord expeditious approvals to the details like length of the submersible structure, approaches, type of the structure, foundations, deck level, bed and slope protection to the approaches etc.

Hydrologic data and river characteristics play an important role in deciding the salient details of submersible structures as these are subjected to frequent submergence with possibility of washing off /dislodgment of superstructure from the substructure/excessive damage to sub structure and foundation because of higher water forces, deeper scour as compared to high level bridges. Similarly, portions of approaches are submerged during floods with possibility of breaches or excessive damages/washing off and require heavy and properly designed protection works with proper drainage system. As such a dedicated team of officers and workers should be assigned the job of collection of essential data in case of submersible structures to avoid major changes in the proposal at a later date or heavy recurring maintenance cost.

7.3.2. Collection and presentation of design data

Collection and presentation of design data should be done as per the provisions contained in IRC:5, IRC:78 and IRC:SP:54 on the subject.

Some of the essential maps and plans to be prepared and design data to be collected is listed in **Appendix 7.1** for guidance.

7.4. Design Flood Level, Road Top Level/Deck Level

Design flood level is fixed based on the number and duration of interruptions to traffic in a year or in specific period i.e. annually/ 5 years etc. depending on the present volume of traffic and importance of road.

Deck level of submersible bridge is then arrived at keeping in view two aspects i.e.

(i) Flood level during which the submersible bridge is to serve as high level bridge.

(ii) Flood level (with specific return period) during which the structure is over- topped but depth of water over the deck is safe for vehicular traffic say 200 mm with number of and duration of interruptions to traffic not exceeding the permissible values.

In absence of any other guidelines following criteria may be adopted;

(i) Number and duration of interruptions to traffic may be reckoned from the flood level 200 mm above RTL/deck level.

(ii) In case of submersible bridge, level of deck may be fixed so that the structure serves as a high level bridge during OFL but to be over topped during higher floods.

(iii) The deck level of the submersible bridge should not be higher than RTL of approaches likely to be submerged during floods, otherwise the approaches may be breached resulting in major portion of flow to pass through breaches in place of the main structure, change of course of active channel, extensive recurring maintenance/ repair costs etc.

(iv) RTL/deck level should be as low as possible in order to have economical design. RTL in case of flush causeways/fords may be kept at bed level and may follow the cross-section of the channel to the extent possible

(v) RTL in case of vented causeways may be kept keeping in view the minimum vertical size of vent as 1000 mm preferably 1200 mm for proper maintenance, minimum heading up of water of 500 mm to generate velocity and minimum concrete cushion of 300 mm over hume pipes. The height of vented causeways may be restricted to 3 m above the deepest bed level

(vi) Area of vents in case of vented causeways and waterway of submersible bridge may be worked out as per procedure explained in Chapter 5 of these guidelines.

7.5. Design Procedure for Causeways

A design procedure and a worked out example of vented causeway is given in **Appendix 7.2. Fig. 7.1** shows cross-section of a typical vented causeway.

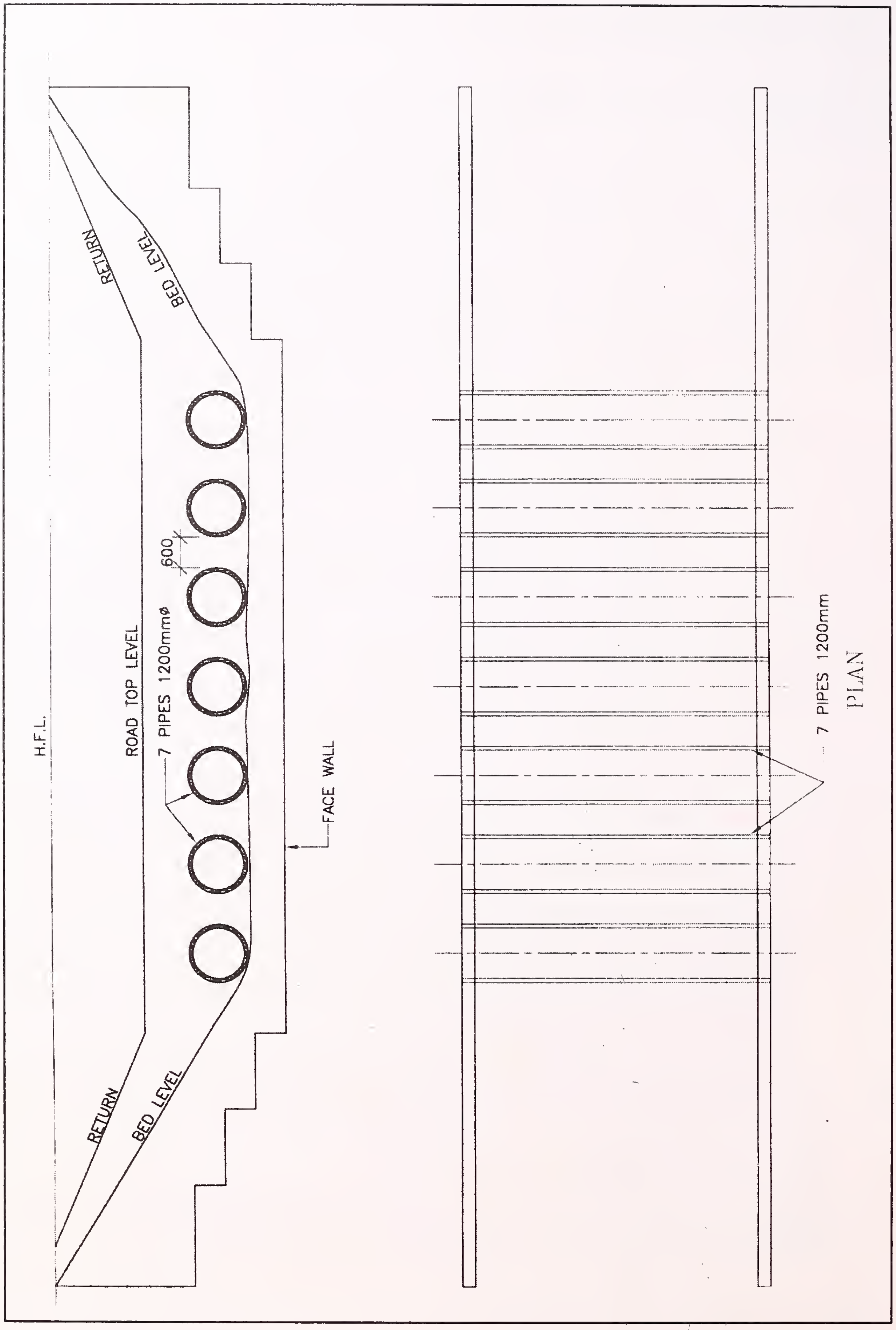


Fig. 7.1 Typical Cross Section of Vented Causeway

7.6. Length of Submersible Structure

Length of a submersible structure is dependent on the water spread at design flood level based on return period and percentages of total discharge, which are expected to pass through the structure and over the approaches without damaging the structure or appurtenances. Therefore, economical length of a submersible structure is the length, which will be able to pass the maximum flows that are expected without endangering the structure and appurtenances by scour, without creating major maintenance problems, without causing unacceptable backwater effects upstream or affecting the legitimate interests of the local people to unacceptable limits. The length should also be able to pass through likely quantities of debris or logs or rolling boulders without endangering the structure or other property/locals as a result of accumulation.

Smaller length of the structure with less initial cost generally requires extensive protection/ training works in addition to increased recurring cost of maintenance and repairs and as such may not be economical for all situations even with protection works. It is, therefore, always advisable that bridge engineer strikes a balance between the cost of the structure and protection/training works on the one hand and the expected recurring cost of maintenance on the other hand.

Following criteria may be adopted while selecting the length of the submersible structure;

(i) The length of submersible structure across a watercourse with regular path should normally be neither less than water spread at designed flood level (which is based on return period) plus 2 m.

(ii) The deck level of the structure should be as low as possible in order to have economical design of substructure and foundations.

(iii) Abutments should not be located in active channel of the watercourse.

(iv) Afflux at HFL (with return period of 50 years) is not abnormally high and unacceptable from environmental considerations or endangering the safety of people or properties.

7.7. Span Arrangement

7.7.1. As any obstruction to free flow of water in a watercourse affects the flow pattern, the number of piers (intermediate supports) should be as small as possible because these cause obstruction to the flow. In other words, the arrangement selected should have minimum number of spans. This would have the added advantage of lesser number of expansion joints and bearings.

7.7.2. Single span structure, if feasible should be preferred especially in mountainous regions, where torrential velocities prevail. Single span arrangement avoids construction of piers and its foundations in the middle of the watercourse (stream) where the water is the deepest. This also alleviates the chances of washing away of partly constructed foundations and substructure in case of watercourses of flashy nature. However, while this approach eliminates the cost of piers and its foundations, it requires a stronger superstructure of bigger span and the total cost of the project may remain unaltered.

7.7.3. From economic and aesthetic point of view, the proposed structure should preferably have odd number of spans (1,3,5 etc.) of equal length. Such an arrangement avoids construction of pier foundations in the middle of the active channel of the watercourse having maximum depth of water, higher velocity of flow and thus reduces the cost of substructure and foundations.

7.7.4. Span/vent arrangement should be evenly distributed in the active channel portion to ensure that it does not cause the water to flow parallel to the carriageway i.e. along the approaches under normal conditions of flow.

7.7.5. When vents are concentrated at the central portion of causeway, heavy scouring occurs on the sides due to high velocity jets reaching on still water region. Therefore, the vents should be uniformly distributed all along the length of Causeway. In the case of circular vents the clear spacing between the vents should be atleast half the diameter of the vents.

7.7.6. In the case of submersible structures (except for major bridges), the usual practice is to adopt multiple span arrangements with small span lengths of equal length involving simple design, repetitive operations, maximum utilization of local resources, easy in construction and rectifications/ modifications during execution stage. Such arrangements generally do not cause major time/cost overrun. However, depending on site conditions, span arrangements with varying span lengths, may prove more economical.

7.8. Span Length

7.8.1. Selection of span length depends on the materials to be used in substructure and capacity of the founding strata to carry the loads and forces likely to be transferred from the substructure and superstructure.

7.8.2. From the point of view of proper maintenance, length of span (even in case of vented causeways) should preferably not be less than 1.5 m and internal diameter of hume pipe not less than 1000 mm, preferably 1200 mm.

7.8.3. In the case of submersible structures with raft foundations or isolated open foundations with founding level not deeper than say 4 m below the anticipated maximum scour level (i.e. top of bed protection for a majority of submersible structures) and solid wall type piers, economical designs would generally be given by the following expressions:

(i) Masonry arch structures

Span = 2 x total height of substructure (upto introdos of the key stone) from bottom of its foundation in metres.

(ii) Reinforced Cement Concrete slab structures

Span = 1.5 x total height of substructure from bottom of its foundation in metres.

(iii) Typical arch span arrangement of submersible bridge is shown in **Fig. 7.4.**

7.8.4. Some typical span arrangements and cross-sections successfully adopted in the past for submersible structures, are shown in **Figs. 7.2 to 7.7.**

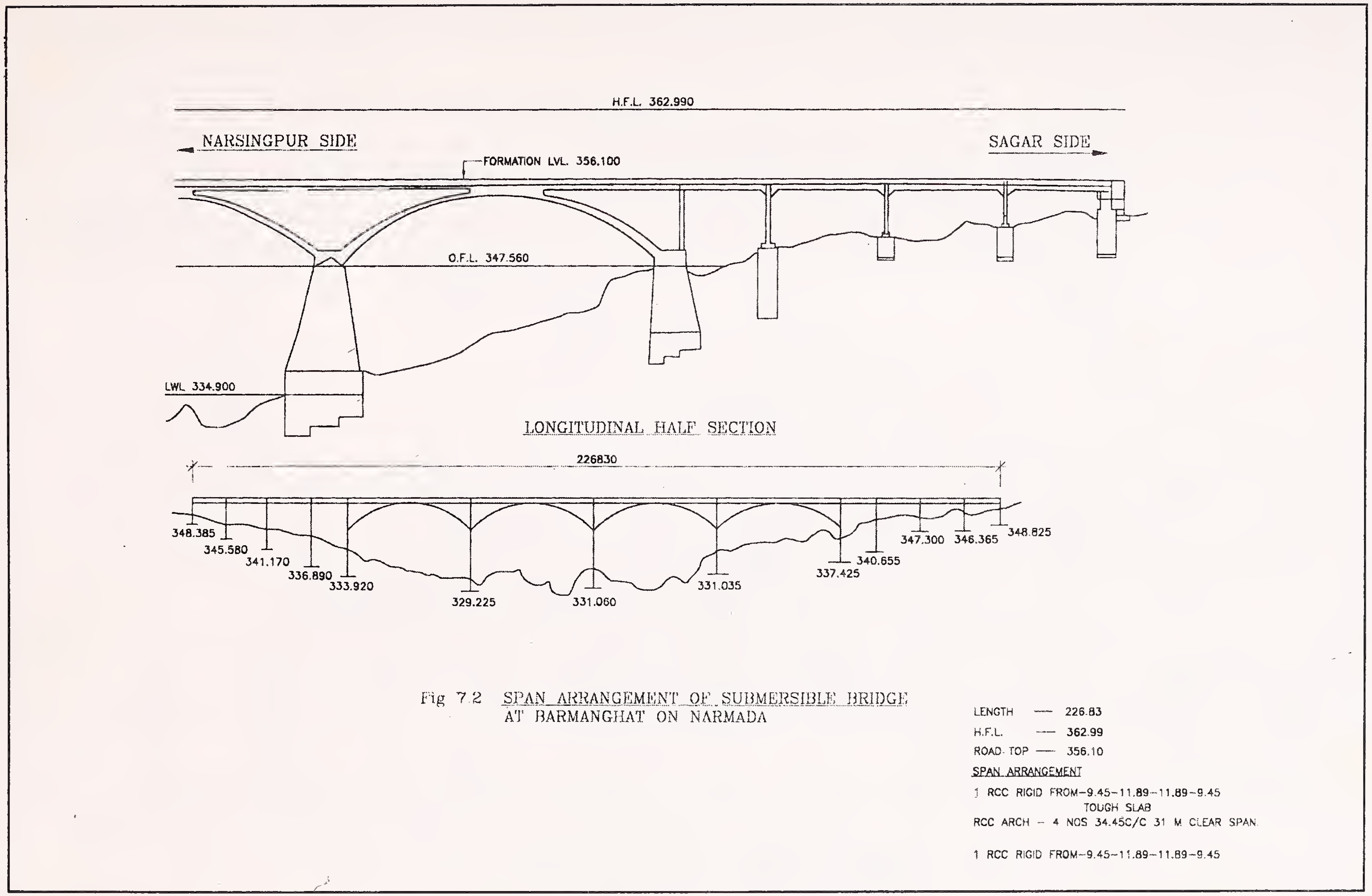


Fig. 7.2. Span Arrangement of Submersible Bridge at Barmanghat on Narmada

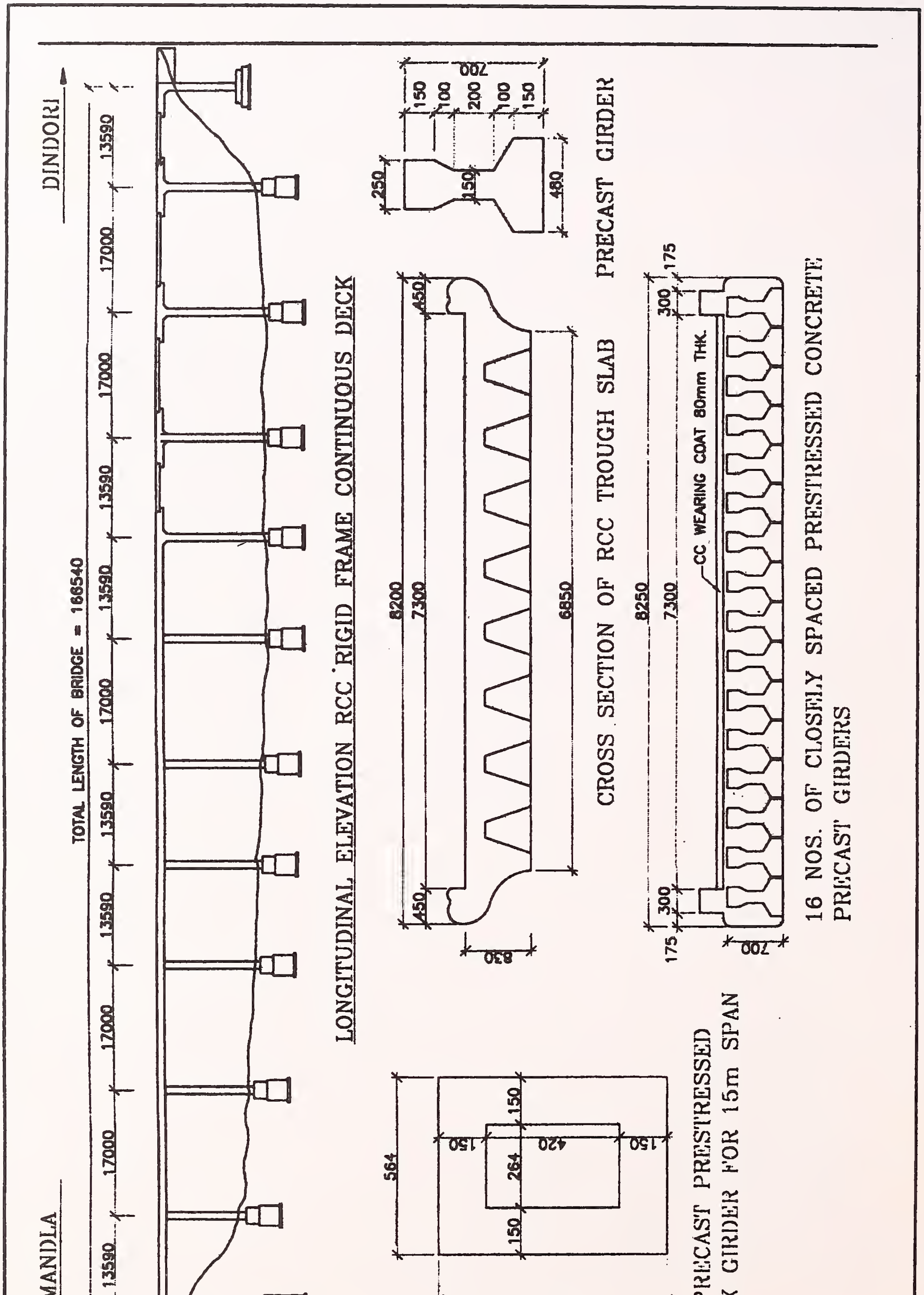


Fig. 7.3. Typical Continuous Deck with Precast Elements

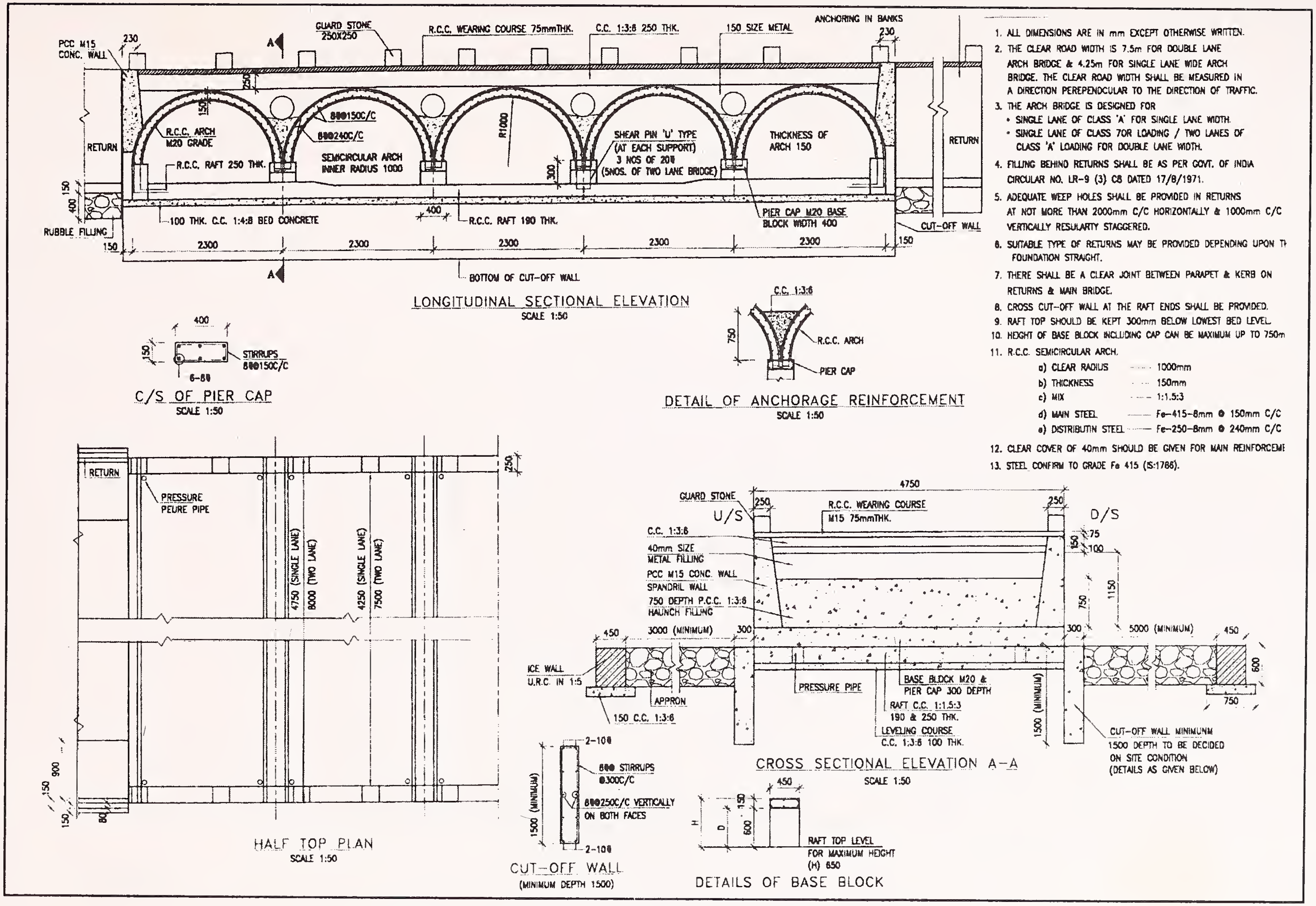


Fig. 7.4. Arch Type Vented Causeway (Single Lane)

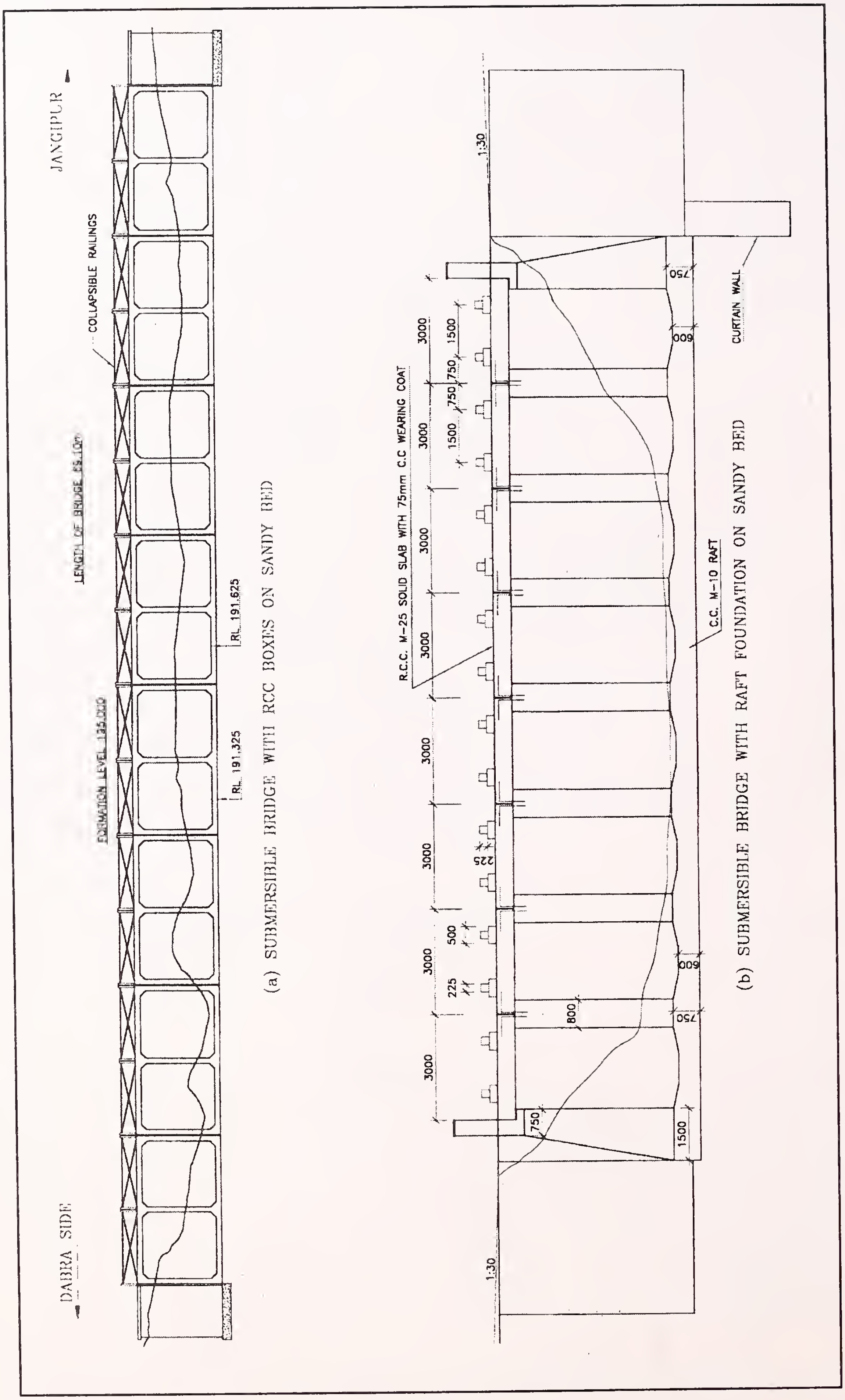


Fig. 7.5. Submersible Bridges on Sandy Beds

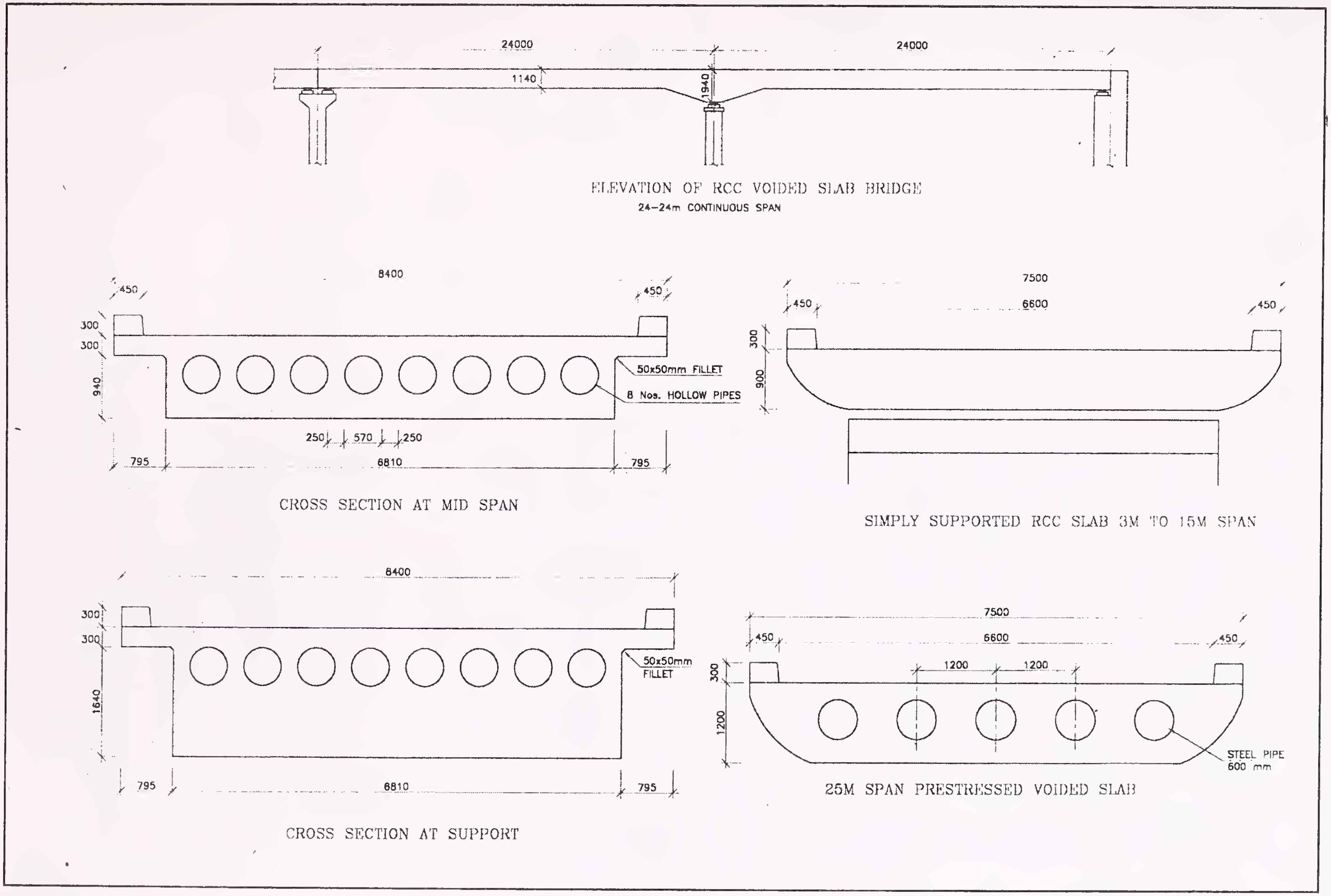
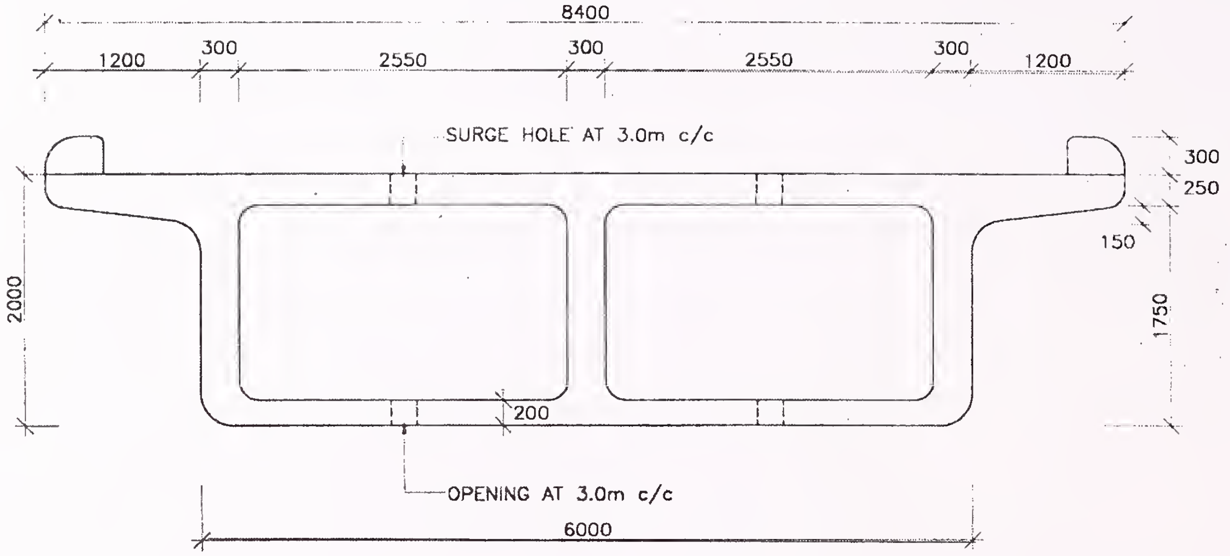
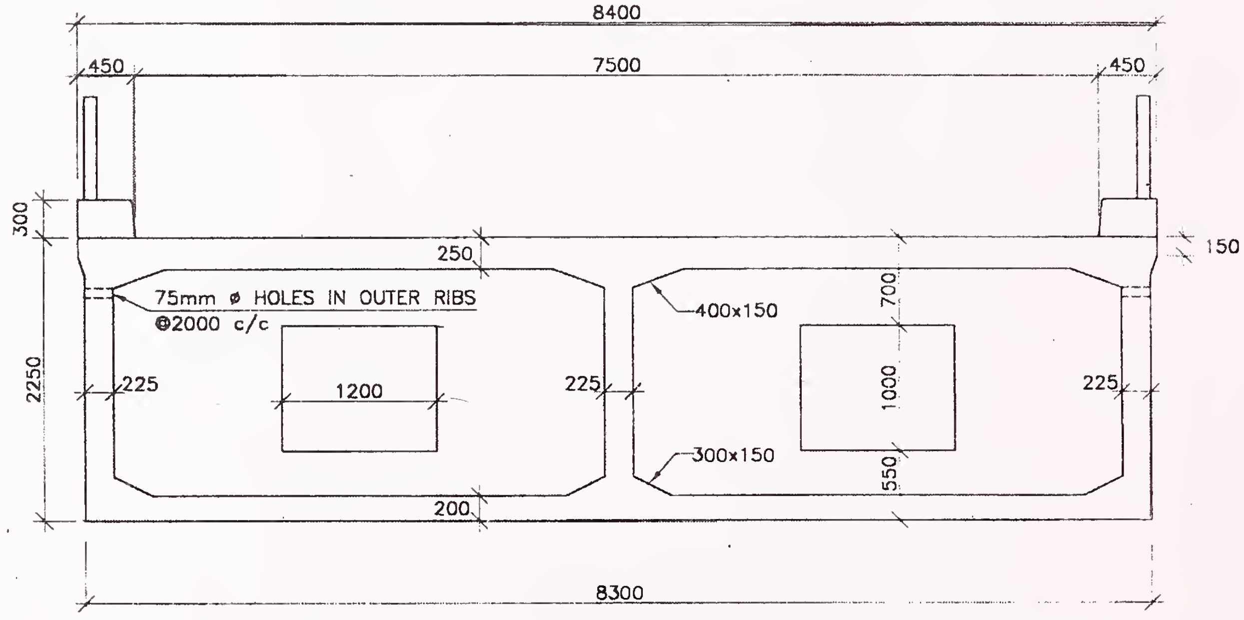


Fig. 7.6. Cross Section of Typical Voided and Solid Slab Type Submersible Bridges



(a) CROSS SECTION OF PRESTRESSED CONCRETE BRIDGE ON SHABARI (30m SPAN)



(b) CROSS SECTION OF NARMADA BRIDGE AT PUNASA (50m SPAN)

Fig. 7.7. Cross-Section of Typical Box Type Submersible Bridges (Note the Surge Holes)

7.9. Types of Superstructure

7.9.1. The type of superstructure to be adopted should be selected keeping in view the economics, aesthetics and client's requirements. Each type has limitations with respect to economy, requirement of special type of machinery/technology/supervision. Since the main consideration in adoption of submersible structures in place of high level structures is initial low cost, it is necessary to adopt the most suitable and economical type of superstructure for submersible structures.

7.9.2. Executing agencies (Contractors) generally prefer the type with minimum construction time, not requiring specialized skilled labour and supervision.

7.9.3. Slender sections with complicated formworks requiring special precautions/ supervision for proper compaction of concrete or involving stage concreting or which are likely to be displaced during floods or cause higher hydrodynamic forces on substructure and foundations should be avoided.

7.9.4. In the case of structures in mountainous reaches, cast in-situ construction may pose some problems due to high velocity of stream and as such, arrangement with beams cast in casting yard at site and launched in position may be more appropriate.

7.9.5. Vent size of the arrangement should not be less than 1.50 m (horizontal) and 1.20 m (vertical) for proper maintenance.

7.9.6. For major bridges over rivers with high velocities during floods or meandering nature, it is advisable that the shape and type of superstructure is selected based on the model studies for the proposed site.

7.9.7. Balanced cantilever, suspended span or light/steel truss type superstructure are unsuitable for submersible bridges and should not be provided.

7.9.8. Integral bridges viz framed structures, arches, multi-cell box structures should be preferred for submersible structures.

7.9.9. In order to reduce water current forces on the superstructure and turbulence, aerofoil type cross section of the superstructure should be adopted.

7.9.10. Different types of superstructures with suggested span range suitable for submersible structures are shown in **Table 7.1** for ready reference and guidance.

Table 7.1

S.No.	Type of superstructure	Suggested Span range (m)	Remarks
1.	Masonry Arch	1.5 to 6.0	Segmented or semicircular arch structures with short height of substructure and raft foundations upto 4.0 m have been extensively constructed in the past. Use of stone/ brick masonry arches may be restricted to seismic Zones II and III with height of substructure not exceeding 6m above foundation level. The floor can be either horizontal slightly below the deepest bed level or inverted shaped arch (such type of construction is most suitable for the sites having low safe bearing capacity say 150 kN/m^2. Series of arches may be used with 3rd or 5th pier designed as abutment pier. In order to reduce horizontal forces due to water current forces spandrel arch or circular corrugated/hume pipe openings may be provided in the spandrels.

2.	R.C.C. arch	3.0 to 15.0	Larger span (not larger than 15.0 m in seismic Zone V) say upto 35.0 m requiring deep foundations may be proposed only if required by the client. In order to reduce horizontal forces due to water current forces, spandrel arch or circular corrugated/hume pipe openings may be provided in the spandrels.
3.	R.C.C. Box Cell type structure	1.5 to 5.0	Suitable for sites with founding strata having low safe bearing capacity say below 150 kN/m^2.
4.	Simply supported R.C.C. solid slab	1.5 to 10.0	Very suitable for submersible structures on account of ease of constructions as these are generally located in isolated places.
5.	Simply supported R.C.C. voided slab	12.0 to 25.0	Should be preferred over T-beam and slab arrangement or R.C.C. box girder type.
6.	3 to 4 span continuous R.C.C. closely spaced T-beam and slab	10.0 to 25.0	The spacing of the longitudinal beams is to be closer in case of superstructures for submersible bridges (say not exceeding the depth of the girder) so as to reduce the depth. May be considered in view of reduction in expansion joints and bearings provided a rocky stratum is available at shallow depths.
7.	Simply supported cast-in-situ R.C.C.Box Girder	20.0 to 30.0	Prestressed concrete construction should be preferred to R.C.C. type being submersible structure.
8.	Prestressed Concrete simply supported box girder (cast-in-situ and post tensioned)	30.0 to 45.0	Minimum inside height of the cell should not be less than 1.2 m for proper maintenance.
9.	Prestressed Concrete simply supported voided slab (cast-in-situ and post- tensioned)	15.0 to 30.0	May be preferred to box type superstructures.
10.	Prestressed Concrete simply supported voided slab (precast and pre/ post- tensioned)	15.0 to 25.0	To be considered in case of watercourses with high velocities.
11.	R.C.C. Rigid –Frame or Portal type 3-4 continuous spans	10.0 to 20.0	Most suitable for sites where rocky stratum is available at shallow depths.

7.10. Piers and Abutments

7.10.1. General

Location of piers in the middle of active channel and abutments in active channel should be avoided.

Size of substructure (piers and abutments) and foundations depends on the material used, capacity of the founding strata and height of deck from foundation level. Therefore, the piers/ abutments should be as short as possible with maximum possible span for a site.

7.10.2. Piers

Solid wall type piers are very common being simpler in design and construction. This type is economical for smaller heights up to 4.0 m and most suitable for watercourses with or without floating debris. Single column or multicolumn arrangements connected by diaphragm walls are economical for watercourses requiring higher substructure. However if the stream carries floating debris the diaphragm walls connecting the columns would have to be of full height so as to avoid the possibility of the debris getting entangled between the columns.

Hollow type piers though economical for larger heights, require complicated form work, closely spaced reinforcement cage and precautions during concreting for proper compaction, are not recommended for submersible structures over watercourses of high velocity, carrying large amounts of debris, floating big size stones etc.

Piers should be provided at both ends (upstream and downstream sides) with suitably shaped cut and ease waters extending upto full height.

7.10.3. Abutments

Open or spill through abutments though economical, are not recommended for adoption for submersible structures.

Solid wall type abutments are most common because of easy construction and simple formwork and are economical for small heights say up to 4.0 m.

For larger heights either box or counter-fort type are economical. The only disadvantage in both types is complicated formwork and more construction time when compared with solid wall type abutments. Box type also provides opportunity to design engineers to reduce the length of cantilever return wall.

Abutments of submersible bridges should be designed as abutment piers so that if required, the same could be converted to the similar shape as pier at a later date.

Abutments should preferably be located in banks to avoid embankment for approaches. In case it is not possible at a particular site, it is necessary to provide straight return walls in full length of embankments (anchoring in banks). Alternatively, embankment of approaches likely to be submerged during floods should be provided with suitably designed protection work i.e. side pitching and toe wall/flexible apron as per the provisions of IRC: 89 applicable for spurs to avoid excessive damages/washing away/ maintenance cost.

7.11. Structural Aspects

7.11.1. Design live loads

The design live loads for submersible bridge should be in accordance with relevant provisions of IRC: 6, depending on the width of carriageway.

7.11.2. Forces

7.11.2.1. Adequacy/stability of the structures needs to be investigated for critical condition. Generally the critical condition for a submersible structure is when the affluxed flood level on upstream side of superstructure is just at the deck level or RTL, and there is a trough of standing wave on the downstream side due to high velocity through vents.

7.11.2.2. Following pressures may be considered at the critical condition:

(a) **Pressure due to static head due to afflux on upstream side and trough of standing wave on downstream side:**

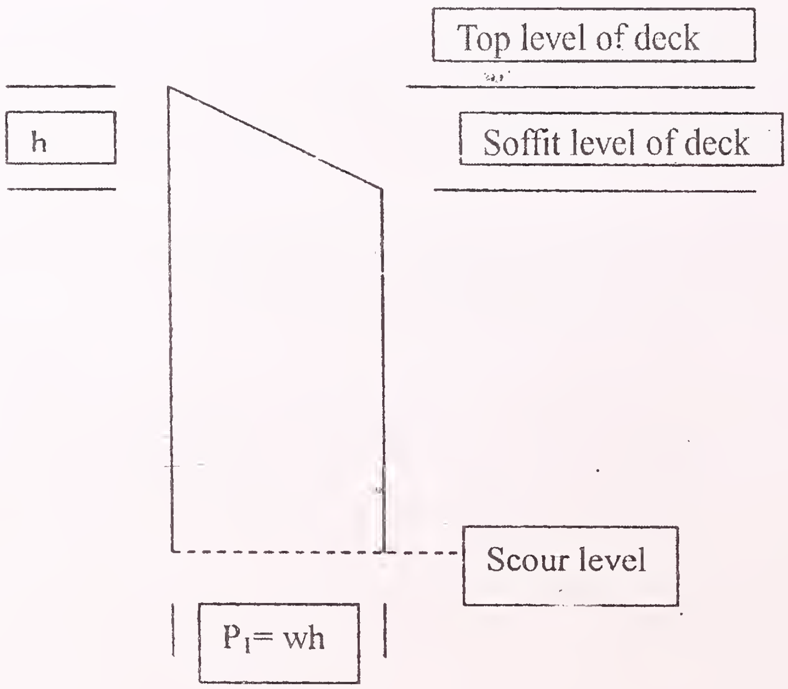
The pressure due to static head can be calculated as follows:

The pressure due to static head will be zero at the surface of water and will increase linearly to $P_1 = w\,h$ at depth 'h' from the surface, below which it will be constant as indicated in the sketch below.

Where,

w = unit weight of water

h = afflux or depth of superstructure (including wearing coat) whichever is more.



(b) Pressure due to velocity head

The pressure 'P_2' due to velocity head shall be determined as detailed in para 4.2.2.

(c) Pressure due to eddies

$$P_3 \text{ (Pressure due to eddies)} = \frac{w\,(V_v - V)^2}{2g}$$

Where,

V_v = velocity of flow through the vents

V = velocity of approach

(w = unit weight of water; g = Acceleration due to gravity)

(d) Pressure due to friction of water against piers and bottom of slab

$$P_4 \text{ (Pressure due to friction)} = f\,\rho\,(C \times V_v)^2$$

Where,

f	=	friction coefficient = 1
ρ	=	mass density of water (w/g)

To obtain different force effects, the individual values of pressure (P_1, P_2, P_3, P_4) are to be multiplied by the surface area of pertinent bridge components (of Super structures and Sub-structures) normal to the direction of flow.

V_v	=	velocity of flow through the vents (m/sec)
C	=	value of constant generally taken as 10%

(e) Force due to uplift head under superstructure

This force acts vertically upwards and is given by

Uplift force = wh x A_{sp}

Where,

A_{sp}	=	area of the superstructure in plan
h	=	the uplift head under the deck slab which may be taken as higher of the following two values:

(i) Afflux

(ii) Thickness of superstructure including wearing coat - head loss due to increase in velocity through vents (V_v)

Head loss due to increase in velocity through the vents is calculated by following expression

$$h_1 = \frac{(V_v^2 - V^2)}{2g}$$

Where,

V_v	=	velocity of flow through the vents
V	=	velocity of approach

7.11.2.3. The sum total of all the horizontal forces may be taken as acting at the top of the pier to test the stability of the structure against overturning.

7.11.3. Combination of loads and forces and additional design considerations

7.11.3.1. All load combinations as specified in IRC:6 should be investigated under the following flood conditions:

(a) Flood level just clear of soffit of the superstructure when the structure is to serve as high level bridge

(b) Flood level at deck level

(c) HFL

(d) LWL (for design of abutments only)

7.11.3.2. Structural adequacy of substructure and foundations should be investigated for all combinations as specified in Clause 706 of IRC:78.

7.11.3.3. In case of pre-stressed and arch type super structure, effect of buoyancy should also be considered while working out stresses likely to develop in the superstructure during flood conditions.

7.11.3.4. Additional load of 150 mm thick silt with density equal to 15 kN/m^3 spread over the entire soffit (in case of box girders) and deck slabs of all types of superstructure should also be considered.

7.11.3.5. In absence of model studies, horizontal forces due to water currents on submerged superstructure as per Clause 213 of IRC:6 should be based on the value of 'K' as 1.5. for superstructure.

7.11.3.6. Effects of buoyancy should be considered in the design of abutments assuming that the fill behind abutment has been removed by scour (with scour all around)

7.11.3.7. Relief due to vent holes provided in deck slab/T beam girders/soffit slab of box cell type superstructure and in spandrels of arch structure should be ignored.

7.11.3.8. Restraining devices/stoppers should be designed to cater for the entire horizontal forces due to water currents, wind and seismic effects as per the load combination given in IRC:78 ignoring friction.

7.12. Other Design Precautions

7.12.1. Structural members should be designed for severe conditions of exposure.

7.12.2. Anchor rods of stainless steel bolts should be provided between the deck slab and the Piers/abutments to prevent uplift of the deck.

7.12.3. Properly designed stoppers as per para 7.11.3.8 above shall be provided on pier/abutment caps to avoid sliding of superstructure in transverse direction.

7.12.4. Superstructure and substructure should be given stream line shape on upstream and downstream side.

7.12.5. Sufficient number of vent holes of 100 mm diameter, minimum three numbers per span or at a spacing of 3 m in the longitudinal direction, should be provided in the deck and soffit slabs and webs of T-beam/box type superstructure to improve stability during floods. Vent holes of 100 mm diameter spacing not exceeding 3 m centre to centre in horizontal direction and 1 m centre to centre in vertical direction should be provided in the spandrels of arch bridge.

7.13. Bearings

7.13.1. General: Bearings are vital components of a bridge since these allow longitudinal and/or transverse rotations and/or movement of superstructure with respect to the substructure (thus relieving stresses due to expansion, contraction and rotation) and effectively transfer loads and forces from superstructure to substructure. Adequate care should therefore, be exercised in selection of the right type of bearings for submersible bridges based on the following guidelines:

(i) Solid Slab Superstructure with span length upto 10 m may generally be proposed to rest directly on unyielding supports (pier/abutment caps) without any bearings.

(ii) Metallic bearings (mild/cast steel) are not suitable for submersible structures which are subjected to severe conditions of exposure during floods and should be avoided.

(iii) R.C.C. bearings, though adopted in past involve complicated detailing of reinforcement, formwork, more construction time, special precaution/supervision for proper compaction of concrete and are difficult to repair/replace if required at a later date and as such should be avoided.

(iv) Copper alloy bearings with copper, tin and lead in 70:5:25 ratio with proper anchorage system may be considered for spans between 10 m to 20 m.

(v) Elastomeric bearings with central anchorage may be considered for span range of 10 m to 30 m.

(vi) PTFE bearings with stainless steel components and with galvanised anchorage may be considered for longer spans than 25 m.

(vii) Minimum 3 number of bearings should be provided at each end of the superstructure from stability considerations.

7.13.2. The design of elastomeric and PTFE bearings should be in conformity with IRC: 83 Part II and III respectively.

7.13.3. In order to cater for any possible relative longitudinal undue movement of bearings over the abutment resulting in superstructure ends jamming against the dirt wall, a larger gap may be provided between the superstructure end and the dirt wall.

7.13.4. All bearing assemblies should be installed in accordance with the instructions contained in IRC codes and specifications and shown on the approved drawings. In particular the following important points should not be lost sight of:

(i) All bearings should be set truly level so as to have full and even seating. Thin mortar pads (not exceeding 12 mm) may be used to meet this requirement.

(ii) The bottoms of girders resting on the bearing should be plane and truly horizontal.

(iii) For elastomeric bearing pad, the concrete surface should be level such that the variation is not more than 1.5 mm from a straight edge placed in any direction across the area.

(iv) For spans in grade, the bearings should be placed horizontal by using tapered sole plates or suitably designed R.C.C. pedestals.

(v) Placing of bearings of different sizes next to each other to support a span should be avoided.

(vi) Installation of multiple bearings one behind the other for a single line of support should not be permitted.

(vii) The bearings should be so protected while concreting the deck in situ, so that there is no flow of mortar or any other extraneous matter into the bearing assembly and particularly on to the bearing surfaces. The protection should be such that it can be dismantled after the construction is over without disturbing the bearing assembly.

(viii) Special attention should be given to the temporary fixtures to be provided for the bearings during the concreting of superstructure in order to ensure that the bearings do not get displaced during the initial installation itself. The temporary fixtures should be removed as soon as the superstructure has attained its required strength.

(ix) Bearings provided at any end of superstructure should be along a single line of support and of identical dimensions.

7.14. Expansion Joints

7.14.1. General: The primary requirement of an expansion joint is that it should be capable of accommodating all movements of the deck viz..translation and rotation and in the process; it must not cause unacceptable stresses either in the joint itself or in the structure by way of restraint and impact. The replacement of an expansion joint always involves traffic interruption. Therefore, expansion joints should be robust, suitable for all loads and local actions under all weather conditions and durable, specially for submerged conditions. The replacement of all wearing parts should be possible in a simple way. In general, the expansion joints should perform the following basic functions:

(i) Should permit the expansion/contraction of the span/spans to which it is fixed without causing any distress or vibration to the structure.

(ii) Cause no inconvenience/hazard to the road user and offer good riding comfort.

(iii) Should be capable of withstanding the traffic loads including dynamic effects.

(iv) Be watertight and be capable of expelling debris without clogging/without imparting higher force on the structure than what it is designed for. For this, it is desirable to have expansion joint extending for full width including the kerb as well as in footpath portion. However, specifications of joints provided in footpath and kerb may be different than that provided in the main carriageway portion.

(v) Surface exposed to traffic should be skid free and resistant to polishing.

(vi) Ensure accessibility for inspections and easy maintenance with all parts vulnerable to wear being easily replaceable.

7.14.2. Expansion joints should be designed as per provisions of IRC:SP:69 and steel components glavanized prior to installation.

7.14.3. No joint is needed for a movement upto 6 mm as such open joint with appropriate edge/nose protection and joint filer may be considered.

7.14.4. Filler joints are suitable for movements upto 10 mm or a span of 10 m. The components of this type of joint are corrugated copper plate (minimum 2 mm thick), 20 mm thick compressible fiber board to protect the edges, 20 mm thick pre-moulded joint filler and joint sealing compound.

7.14.5. Compression seal joint consisting of galvanized steel armoured nosing at two edges of the joints gap, suitably anchored to the deck concrete and a pre-formed chloroprene elastomer/ closed cell foam joint sealer, compressed and fixed into the joint gap with special adhesive binder may be considered for movements upto 40 mm.

7.15. Wearing Coat

Unless otherwise specified, 75 mm thick R.C.C. wearing coat (not monolithic with deck slab) in concrete grade of minimum M 30 and water cement ratio not exceeding 0.4 should be considered for submersible structures. The reinforcements placed at the middle depth of the wearing coat may consist of 8 mm diameter bars @ 200 mm centres reducing to 100 mm centers in both directions over a strip of 300 mm near the expansion joint.

7.16. Material Specifications

7.16.1. Materials used in construction of submersible structures shall conform to relevant IRC Codes.

7.16.2. In view of the likelihood of the structure getting submerged during floods, the following additional criteria may also be considered for adoption;

(i) Minimum strength of concrete, cement contents (kg/cum) and maximum water cement ratio should be as suggested in **Table 7.2.**

Table 7.2

Structural Member	Minimum strength of concrete	Minimum cement content (kg/cu.m)	Maximum water cement ratio
Plain Cement Concrete members (PCC members)	M 20	310	0.45
Reinforced Cement Concrete members (RCC members)	M 25	360	0.40
Prestressed Concrete member (PSC)	M35	400	0.40

Notes:

(i) The above minimum cement content is based on 20 mm aggregate (nominal maximum size). For larger size aggregates, it may be reduced suitably but the reduction should not be more than 10 per cent or 30 kg per cu.m whichever is lower. For 12.5 or 10 mm size aggregates, it shall be increased suitably but the increment should not be less than 10 per cent or 40 kg per cu.m whichever is higher.

(ii) Hand mixed concrete shall be avoided but if unavoidable for small isolated causeways, the cement content shall be increased by 10 per cent.

(iii) Leveling course for masonry abutment, pier, return/wing/toe wall should be M15.

(iv) Concrete for piers should not be leaner than M30.

7.16.3. Brick/stone masonry work should be in not leaner than cement mortar of 1:3.

7.16.4. Annular space around foundations in rock should be filled with cement concrete of minimum grade of M15.

7.16.5. Use of thermo-mechanically treated (T.M.T.) bars conforming to IS: 1786 should be preferred.

7.16.6. In case of streams carrying abrasive particles and velocity higher than 4 m/sec, the substructure should be provided with an additional sacrificial cover and richer concrete.

Appendix 7.1
(Reference: Para 7.3.2)

COLLECTION AND PRESENTATION OF DESIGN DATA

Some of the essential maps and plans to be prepared and design data to be collected and presented in the report are listed below for ready reference and guidance.

I. Maps, plans and topographical features

(i) An index map (toposheets in scale one cm to 500 m or 1/50,000) should show the north line, location of the project area, name of district and state, possible sites for submersible structure along with alignments of approaches, overall road network, nearby important towns/ villages, structures on the watercourse in the vicinity, some landmarks for easy identification during reconnaissance etc.

(ii) A contour survey plan of the watercourse showing topographical features and extending u/s and d/s of the sites for submersible structure considered and also most suitable site for high level bridge (to be constructed later on). The distances to which the contour plan should extend depends on the extent of catchment area and should be slightly more than the coverage in site plan (vide sub para 3 below) as given in **Table 7.3.**

Table 7.3

S.No	Catchment area (km^2)	Distance to which survey plan to be extended (m)	Scale
1.	3	150	1 cm to 10 m or 1/1000
2.	3 to 15	400	
3.	>15	1.5 km or width between the banks whichever is more	Not less than 1 cm to 50 m or 1/5000
4.	Meandering watercourses (streams)	To be decided by an experienced design engineer	

(iii) A site plan drawn to a suitable scale should extend at least 100 m u/s and d/s from the centerline of the crossing or at least two loop length on u/s and d/s of the proposed site in case of meandering watercourses and should show the following details:

(a) sites considered and site selected for the submersible structure along with the chainages, north line and latitude and longitude of the site as measured from the survey of India maps;

(b) most suitable site for high level bridge (to be constructed at a later date);

(c) approaches to sufficient lengths indicating the portions likely to be submerged during floods. In case of structures with length more than 60 m, the length of road alignment shown should not be less than 500 m on either side of the submerged portion of approaches;

(d) private land boundaries, permanent buildings, services, location of deep channels, ponds, marshes, wells, rock out crops, places of worship, graveyards etc. if any near to the proposed site which may affect the approach alignments;

(e) course/(s) of the watercourse;

(f) name of watercourse, road, name of ferry, location, direction of flow during HFL and OFL presence of islands if any, bank lines, angle of skew/square crossing, alignment of approaches;

(g) names of nearby town/locality and road leading to the site;

(h) reference and R.L. of permanent station/ bench marks/GTS benchmarks if available;

(i) location and reduced level of the temporary bench mark used as datum location of the longitudinal section (LS) and cross sections (CS);

(j) location of cross-sections (CS) of the stream taken within the area of the plan;

(k) location of trial pits/borings with their identification number;

(l) existing crossing structures on the same watercourse;

(m) any other feature considered necessary by the survey party.

(iv) Catchment area map prepared from the toposheets/ contour survey plan, is required in order to assess the basic parameters of discharge etc. In case the catchment area of the watercourse is restricted area, the concerned department should make efforts to obtain the toposheets for the project. The scale and size to be used in the catchment area map depends on size, nature and needs of each submersible structure. The catchment area of a submersible structure should be identified and marked clearly on the topographical map. The identified catchment should also include the contour, slope both in longitudinal and cross directions, existing land use pattern like forests, cultivated land, barren land, desert, natural and artificial storage areas etc., to the extent possible.

(v) Cross-section of the channel at/near the proposed site of the submersible structure and a few cross-sections at suitable distances as given in **Table 7.4**, both upstream and downstream drawn to a horizontal scale not less than 1 cm to 10 m or 1/1000 and vertical scale not less than 1cm to 1 m or 1/100 should indicate:

(a) name of project, watercourse, villages/localities on either side;

(b) bed levels at close intervals depending on the cross slope of bed in the channel and banks, with reference to the temporary benchmark and ground levels for sufficient distance beyond the edge of the channel;

(c) LWL, OFL and HFL;

(d) distance from the proposed site for the crossing;

(e) location of the trial pits/bores;

(f) nature of the subsoil in the bed, banks, approach portions, depth of trial pits/bores with proper identification number and type of sub-soil strata;

(g) outcrops of rocks, pools/dips if any in the approach portion and

Table 7.4

S.No.	Size of watercourse/channel (Ws - water spread at H F L)	Distance in m (upstream and downstream of crossing) at which cross-sections should generally be taken unless otherwise specified by the Design Engineer/fixed after site inspection.
1.	Very small (Ws<30 m)	50
2.	Small (Ws>30 m but <60 m).	100
3.	Medium (Ws>60 m but <300 m)	300
4.	Large (Ws>300 m)	500

Three numbers of cross-sections (one at the proposed site for the submersible structure and one each at u/s and d/s of the proposed site) are generally sufficient to yield necessary data for the design of waterway, length of the structure , location of piers/ abutments, deck level for the proposed structure and height of immediate approach embankment. In case an existing road or cart track crosses the watercourse at the site selected for the submersible structure, the cross section should not be taken along the centerline of the road/track as the same will not represent the natural shape and size of the watercourse. In such cases, the cross section should be taken at a short distance on the d/s of the selected site.

(vi) A longitudinal section of the channel along the approximate center line of the active (deep water) channel between the boundaries of the survey plan drawn to suitable horizontal scale and vertical scale, not less than 1 cm to 10 m or 1/1000, should show:

(a) Locations of the proposed site for crossing and cross-sections taken;

(b) H.F.L., O.F.L., L.W.L.;

(c) Bed levels at suitable intervals and

(d) Name of the project and channel if any.

II. Hydraulic data and watercourse characteristics

The basic purpose of collecting hydrological data is to study the rain fall pattern like intensity, duration, frequency and run-off characteristics of the basin under consideration and thereby assess the likely discharge through the watercourse.

Hydraulic data collected for the purpose of the preliminary project report (PPR) has to be good enough for the detailed engineering also. No separate hydraulic data collection is envisaged for detailed engineering except that for model studies, if any conducted for structures across large rivers. The hydraulic data collected should include:

(i) Catchment area map, cross-sections and longitudinal section prepared as per sub-paras (iv) to (vi) of I above respectively;

(ii) HFL ascertained from watermarks if any on the permanent objects on the banks supplemented by local enquiry from nearby inhabitants as to the highest flood levels reached during their living memory;

(iii) OFL and LWL ascertained with reference to watermarks if any on the permanent objects on the banks supplemented by local enquiry from nearby inhabitants;

(iv) Velocity of flow and presence of floating debris etc., during floods from local enquiry. Velocity of flow should preferably be ascertained during floods by the use of floats by determining the time to traverse two fixed points at measured distance apart;

(v) In case a causeway or the existing bridge is of insufficient waterway resulting in afflux, the extent of such afflux be ascertained for arriving at the rough assessment of discharge;

(vi) Names and approximate discharges of all tributaries joining the river within a reasonable distance u/s of the site under consideration;.

(vii) Skew angle of crossing, if any, should be ascertained correctly. Skew angle should be measured in relation to the direction of flow at/near designed flood level (i.e. OFL) and not in relation to the bank line.

(viii) Rainfall data indicating

(a) Maximum precipitation in one hour and 24 hours

(b) Rainfall distribution in catchment

(c) Duration and frequency of floods

(d) Rain gauge data of storms for which corresponding watercourse gauge data is available (data for unit hydrograph)

(e) Average annual rainfall characteristics supported with relevant meteorological records.

III. Watercourse/channel/river characteristics:

(i) All details of configuration of the watercourse as may be relevant to hydrological analysis (given below for ready reference) may be obtained from ground survey. All controls, natural (drops, rapids, bends, debris) and artificial (dams/ weirs /spurs and bridges etc.) should be identified and effect on the discharge at the site should be assessed. Degradation of the watercourse channel may invite higher flood discharge whereas aggradation may result in higher flood levels and bank spills. These factors have direct bearing on design of waterway clearances and approaches and structure.

(a) Seasonal or perennial

(b) Braided, meandering or straight

(c) Other classifications like bouldery, flashy, well defined, presence of pools, weeds etc.

(d) Conditions of banks i.e. erodible or non-erodible, high or low or flat, khadir width

(e) Sediment load aggradation or degradation behaviour.

(ii) Flood flow data

A reliable and correct collection of flood data forms the basis of decision about the type of structure (i.e. high level or low level structure), deck level of the structure and height of embankments etc. Historical and flood data records maintained by the irrigation or other authority helps in arriving at a realistic assessment of likely discharge, frequency, HFL, OFL influence of afflux on project areas etc.

(iii) Data regarding existing bridge structures:

Data regarding flood records, scour observations, waterway, functioning of the existing bridges on the same stream in the vicinity of the proposed site helps in fixation of design data, identification of additional data required to be collected and planning of the project in a systematic manner. The data should include the following:

(a) Description with sketches showing relevant dimensions or general arrangement drawing.(GAD) indicating the salient hydraulic data;

(b) Observed HFL, OFL ascertained from records or marks on substructure;

(c) Length and depth of submergence, number and sizes of vents, and frequency (including duration) of interruptions to traffic in case of causeways/ submersible structures;

(d) Number and length of spans, clear waterway, adequacy of otherwise of waterway with special reference to silted up spans or signs of unde scour or attacks on abutments and approaches in case of bridges and

(e) Observed afflux, if any.

IV. Sub-soil data

(a) The main aim of sub-surface exploration (investigations) is to collect sub-soil data in order to determine the suitability or otherwise of the available soil or rock and the design parameters for foundations of submersible structure. The sub-surface investigations are carried out in two stages, i.e. preliminary and detailed. However, in case of submersible structures with multiple (more than two) spans of more than 15 m or well/pile foundations or availability of suitable foundation strata at varying depth below bed level with abrupt changes in thickness, additional/confirmatory explorations is carried out. The main objective of additional exploration during execution is to confirm the characteristics of sub-soil materials established during detailed exploration, based on which design was made and to affect suitable modifications to suit the conditions met at specific foundation locations. All in-situ tests should invariably be supplemented by laboratory investigations. Guidance for subsoil investigation may be taken from the Standards listed below:

IS:1498, IS:1888, IS:1892, IS:2131, IS:2132, IS:2720
IS:4434, IS:4968 (Parts 1 to 3)
IRC:75 and IRC:78

Appendix 7.2
(Reference: Para 7.5)

DESIGN PROCEDURE AND WORKED OUT EXAMPLE FOR A TYPICAL VENTED CAUSEWAY

A simple approach for designing typical vented causeways is given below for the guidance of new Engineers.

I. Important components of a vented causeway are vents, bed protection, raised face walls and paved road surface, which together ensure stability and prevent outflanking. The flow conditions are analyzed with reference to top of the protected bed and if the percentage obstruction to flow at the Road Top Level (RTL)/deck level is kept below 60% and at the most 70%, then normally no outflanking would take place. At flood levels higher than the road top level, the percentage obstruction goes on reducing and the structure will be safe.

The critical conditions for design are:

(i) When the flow is at RTL
(ii) When the flow is at HFL

II. Step by step procedure:

(i) Collect normal hydraulic data, such as, catchment area, annual rainfall, HFL, site plan, L-section, tide level, etc.

(ii) Collect hourly/ daily record of flood levels for a representative monsoon period and plot it on a graph as explained in para 5.1.2 of Chapter 5. However, if decided by the competent authority, this step may be skipped in the case of less important crossings with length of causeway less say 30 m.

(iii) Plot defined cross-section in the vicinity of proposed site to a natural scale.

(iv) Plot cross-section of crossing at proposed location to the natural scale showing soil conditions.

(v) With the help of the graph (ii) above, determine the lowest required Road Top Level so as to satisfy the requirements of frequency and duration of submergence indicated in **Table 3.1** of Chapter 3.

(vi) Fix Road Top Level (RTL) keeping in view following guidelines: -

(a) It should be as low as possible but higher than the lowest RTL determined vide step (v) above.

(b) In case of box or simply supported slab/arch type structures, the vent size should not be less than 1.5 m horizontal and 1.2 m vertical. Internal diameter of circular corrugated/RCC hume pipe should not be less than 1.0 m or 1.2 m preferably.

(c) Cushion over the structures should not be less than thickness of proposed road pavement subject to minimum 300 mm.

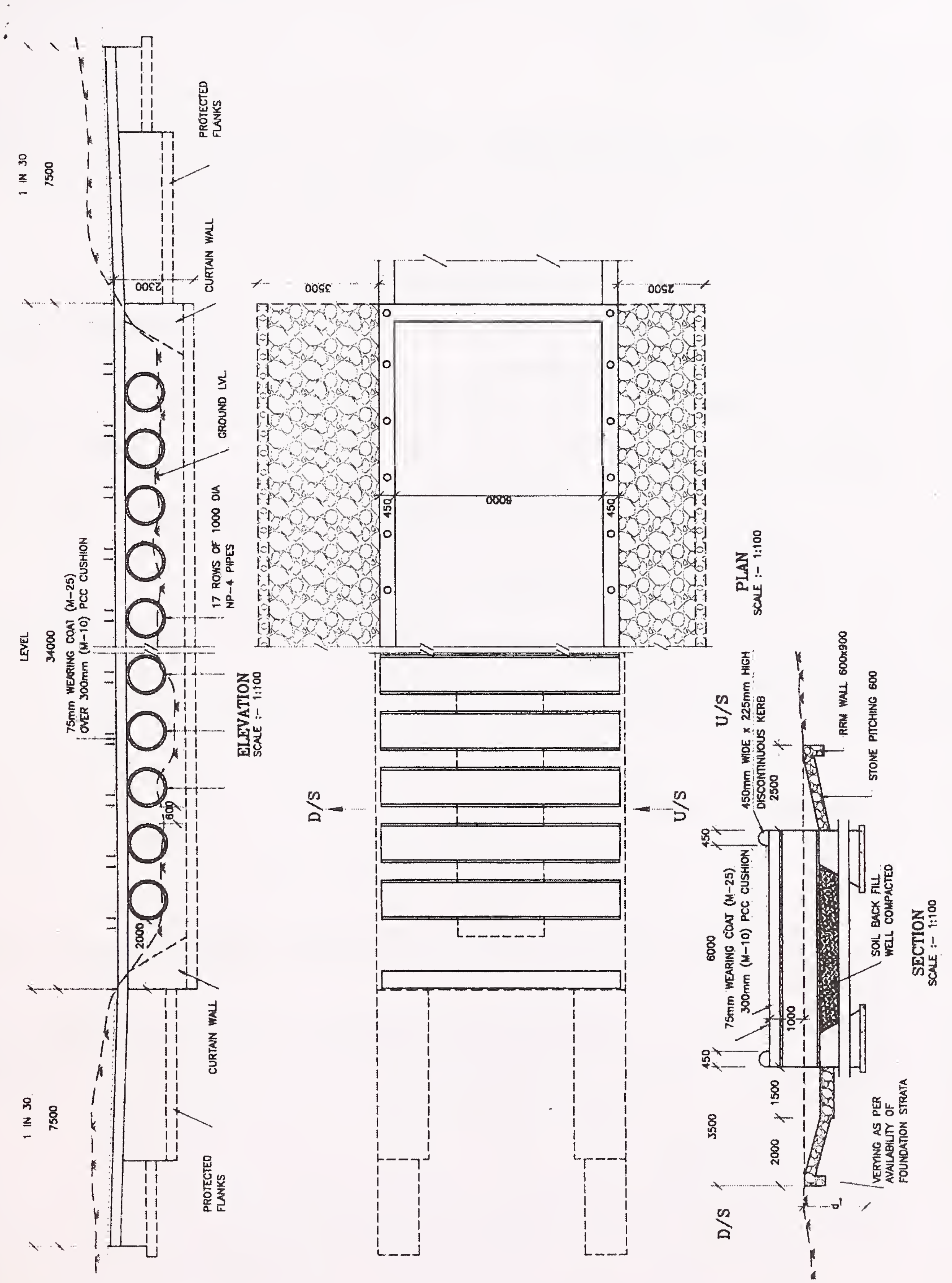


Fig. 7.8. Details of Vented Causeway (Appendix 7.2)

(d) Sill level of vents should be 300 mm below the lowest bed level (LBL) with longitudinal slope (in the direction of flow) nearly same as that of stream bed subject to minimum of 1:100.

(e) The Protected Bed Level (PBL) may be kept equal to the sill level of vents.

(vii) In the case of less important crossings, if it is decided by the competent authority to skip fixing of the lowest required Road Top Level as per rigorous method vide step (v) above, for first trial a level difference of say 1.5 m may be assumed between RTL and PBL.

(viii) Transfer RTL fixed as above to the defined cross-section as first trial.

(ix) Calculate area below RTL at the defined cross-section.

(x) Fix vent area i.e. about 40% but minimum 30 percent, in normal rain fall areas and minimum 20% in case of scanty rainfall areas.

(xi) Determine number of pipes/ number of spans and span length of vents.

(xii) Fix length of horizontal portion of the face wall and length of rising face wall keeping in view following guidelines:

(a) Length of horizontal portion should be equal to bed width of the channel plus minimum 4m.

(b) Gradient of rising face wall should be between 1:15 to 1:30.

(xiii) Calculate the unobstructed natural area of flow at the defined cross section between the bed level and the proposed RTL $= A_1$

(xiv) Calculate the area of flow available at vented causeway upto protected bed

$$= A_2$$

(xv) The percentage obstruction to flood water is calculated by following expression.

$$= \frac{A_1 - A_2}{A_1} \times 100$$

(xvi) If the obstruction is not less than 70 per cent, then steps are repeated by increasing the RTL by 200 mm.

(xvii) The proposal, with percentage obstruction less than 70 per cent is then finalized.

(xviii) This should be checked for flood level at design flood level, for which condition the percentage obstruction should be less than 30%.

III. Worked out example

Determine water way required for the following conditions:

Design discharge	=	682.6 cumecs
Bank width at defined cross-section	=	42 m
HFL	=	102.42 m
LBL	=	98.905 m

Design

(i) Assume sill level of vents and PBL at RL 98.905- 0.3 = RL 98.605 m

(ii) Assume 1 m dia pipes and RTL of 100.5 m which provides for more than adequate cushion over the hume pipes.

(iii) Area of flow at defined cross section below RTL (assuming parabolic profile of bed and channel width of 34 m at RTL).

$= 34 \times (100.5\text{-}98.905) \times 2/3 = 36.15$ sq.m.

(iv) Provide a vent area of 40% to the causeway

$= (36.15 \times 40)/100 = 14.46$ sq.m.

(v) Number of 1000 mm internal diameter pipes required

$= (14.46) / [(p/4) \text{ x } (1.0)^2] = 18.42$ say 19 Numbers

Trial (i) with 19 no. of pipes of 1000 mm inner dia.

The outer dia pipe is 1150 mm (i.e. 1.15 m)

Clear spacing between adjacent pipes shall be 0.6 m

Length of End portion on either side (for safety)= 2.0 m

Total length = 19 x 1.15 + 18 x 0.6 + 2 x 2.0 = 36.65 m

As 19 Nos. of 1 m internal dia pipes can not be accommodated in a bed width of 34 m available at the RTL, the number of pipes need to be reduced.

Trial (ii) with 17 pipes of 1000 mm dia.

Total length = 17x1.15+ 16x0.6+ 2x2.0 = 33.15 m

This can be accommodated with the available 34 m.

Available vent area = 17x $[(\pi/4) \text{ x } (1.0)^2]$= 13.345 sq.m.

Percentage of the area of flow below RTL= 100x13.345/36.15= 36.92%.

This is a little less than 40% but is more than the minimum of 30%. Hence O.K.

Check for obstruction when flood level is at HFL

Assume an approach gradient of 1:30 on either side.
Width of stream at HFL = 34+2x 30 (102.42-100.5) = 149.2 m.
Area available for flow above RTL = [(149.2 +34)/2] x1.92 = 175.872 sq. m.
Therefore total available area for flow = 175.872 + 13.345 = 189.217 sq. m.
Area of obstruction = 34 (100.5 – 98.605) – 13.345 = 51.085 sq. m.
Percentage obstruction = 51,085/ 189.217 x 100 = 27 %
Less than 30%. Hence O.K.

See Fig. 7.8 for details of worked out example.

8. APPROACHES, PROTECTION WORK AND APPURTENANCES

8.1. Approaches

8.1.1. General

(i) The approach roads to Causeways/ Submersible Bridge should preferably be in cutting with the approach gradient not steeper than 1 in 30. The sloping portion of the approach should merge into level portion of the causeway in the arc of a properly deigned vertical curve in order to eliminate bumping as illustrated in **Fig. 8.1**. Similarly wherever there is a change of grades, it is desirable to provide properly designed vertical curve at the junction of the two grades from considerations of user's comfort.

(ii) It is preferable to have the approaches in cutting as the embankments are liable to be washed away during floods. In cutting, however, there may be the problem of silting but the same can be appreciably reduced if the approaches are aligned at a slight angle to the center line of the bridge so that the gradient falls in the direction of the river flow. This, however, may result in sharp curves in the approaches. Straight approaches are always preferable from the point of view of traffic and also ease in construction of wing wall and in actual practice the amount of silting in a well designed submersible bridge with straight approaches may not be excessive. The sides of cutting should be protected by stone revetment upto at least 1 m above the affluxed H.F.L. This would avoid scouring of the sides of cutting and consequent silting of the approaches.

(iii) The approaches in cutting would get submerged for a considerable period therefore these should be provided with safe side slopes considering the submerged condition. Further, deep cutting (say more than 4 m) should invariably be avoided. Wherever steep side slopes are provided for the approaches in cutting it is experienced that the slopes slip over the road pavement and it becomes a recurring problem to clear it, after every flood. The values of safe side slopes for the different types of soil under submerged condition are given in **Table 8.1** for reference and guidance:

Table 8.1

Type of soil	Vertical	Horizontal
(i) Soft soil	1:	2 ½
(ii) B.C. soil	1:	2
(iii) Soft murum	1:	1 ½
(iv) Hard murum	1:	1

(v) Lined drains on either side along the side slopes should be provided. The lining may be either stone or brick or concrete. The side drains should meet the stream proper atleast 10 m away from the edge of the main causeway junction as the flow of water in the drains would erode the banks for certain length in course of time **Fig. 8.2**. In case of straight return walls, minimum 3 m long walls perpendicular to the return walls should be provided to avoid undermining of foundations of return wall or/and abutments.

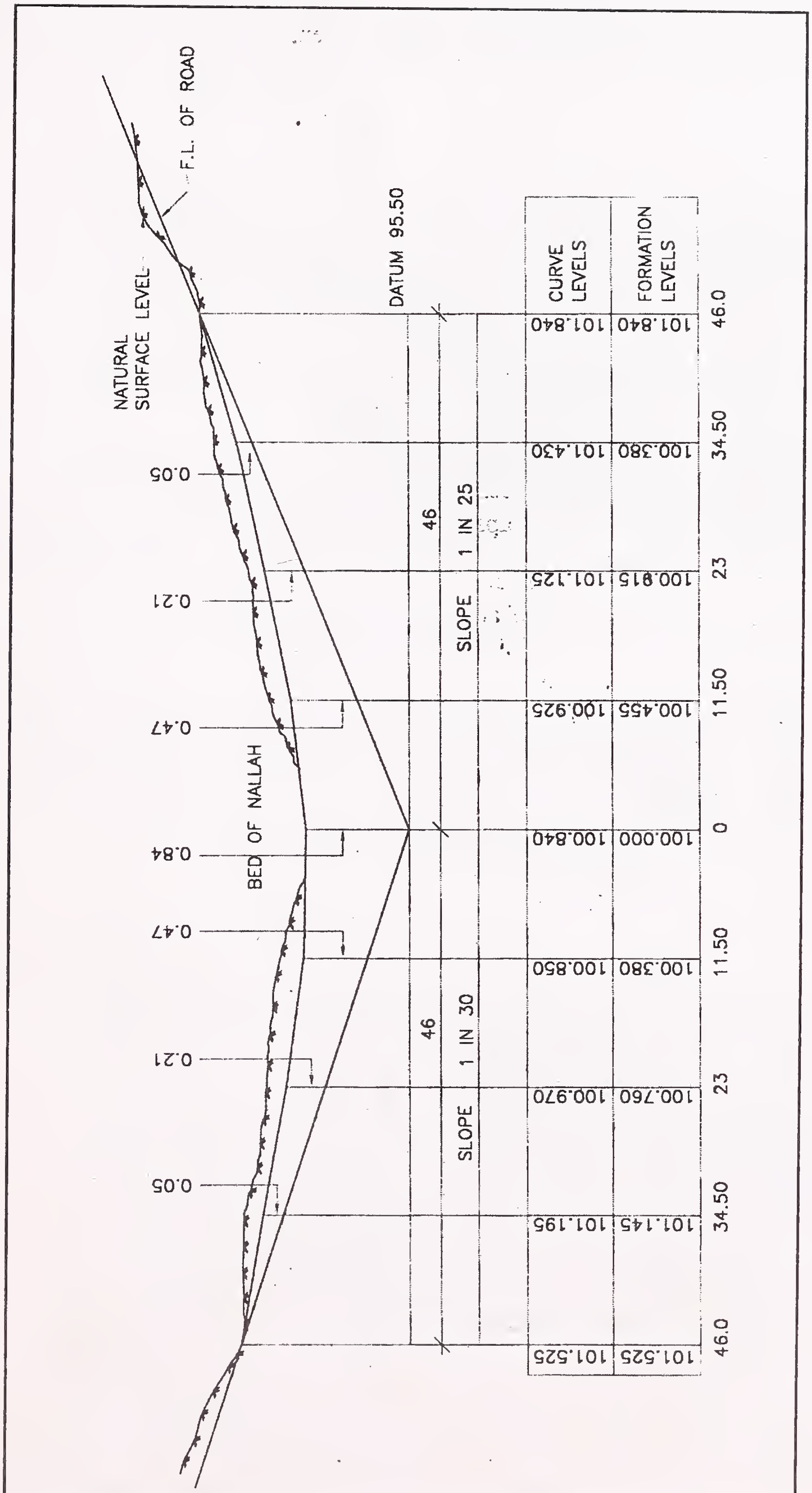


Fig. 8.1. Vertical Curve of a Causeway

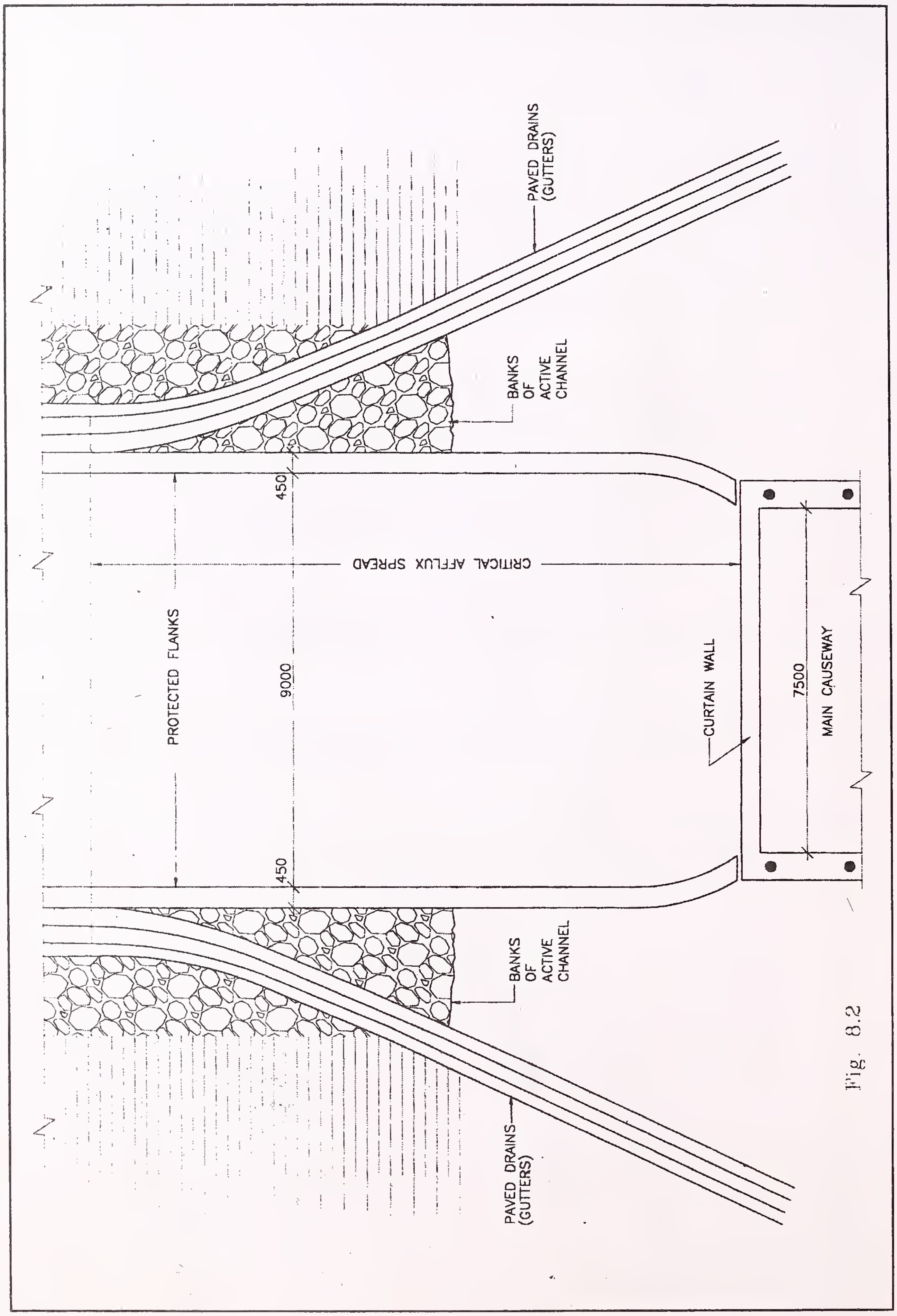

PAVED DRAINS (GUTTERS)
BANKS OF ACTIVE CHANNEL
450
CRITICAL AFFLUX SPREAD
PROTECTED FLANKS
9000
CURTAIN WALL
7500
MAIN CAUSEWAY
450
BANKS OF ACTIVE CHANNEL
PAVED DRAINS (GUTTERS)
Fig. 8.2

Fig. 8.2

(vi) In case of submersible bridges founded on sandy beds, extraction of sand/soft material beyond apron and curtain wall should not be done as this may endanger the safety of the structure.

8.1.2. Pavement for approaches

(i) Length of approaches upto the spread of affluxed highest flood is subjected to repeated inundation and is always prone to out flanking. Therefore the roadway should be paved in a similar manner to that of the main causeway/submersible Bridge. The paved approach roadway in cutting needs to be confined between anchor–walls as shown in **Fig. 8.3**. In case the approach road is not in cutting than unidirectional camber (d/s side) should be provided.

(ii) Length of the remaining approach roads in cutting, beyond spread of the affluxed highest flood, may be constructed with the usual type of pavement with the exception that the metalled surface should be provided for the full width of the roadway. In case of soft soils banks it is preferable to provide this with the anchor walls and side drains **Fig. 8.3.**

(iii) Composition of Pavement

Following minimum pavement composition for approaches to causeways may be adopted unless otherwise required from design consideration.

(a) 200 mm thick compacted mooram/gravel/crushed stones;
(b) 150 mm thick water bound macadam; and
(c) 200 mm thick cement concrete slab of M30 grade.

8.1.3. Face walls/cut-off walls for causeways

(i) A substantial portion of the flood water has to pass over these face walls/ cut-off walls, therefore these walls should be taken down to safe depth and their structure should be strong enough to avoid damages during floods.

(ii) In order to ensure as streamed lined flow as possible and there by restrict the velocities and turbulence on the D/S side to desirable limits and to avoid out-flanking, the elevational profile adopted for these walls should be as close as possible to that of the natural hyperbolic cross section of the watercourse. However, in road geometry it may be difficult to adopt such a section, therefore a modified shape in form of trapezoid may be adopted. This is achieved by keeping the central portion of the walls at one level and raising their levels in the flank portions as shown in **Fig. 8.4.**

(iii) It is desirable to provide length of the level portion of face wall equal to width of the stream at RTL + 2 to 5 m on either side. Similarly the total length of face wall will be equal the width of the stream at OFL + 2 to 5 m. It will be provided for full height of the approaches on either side.

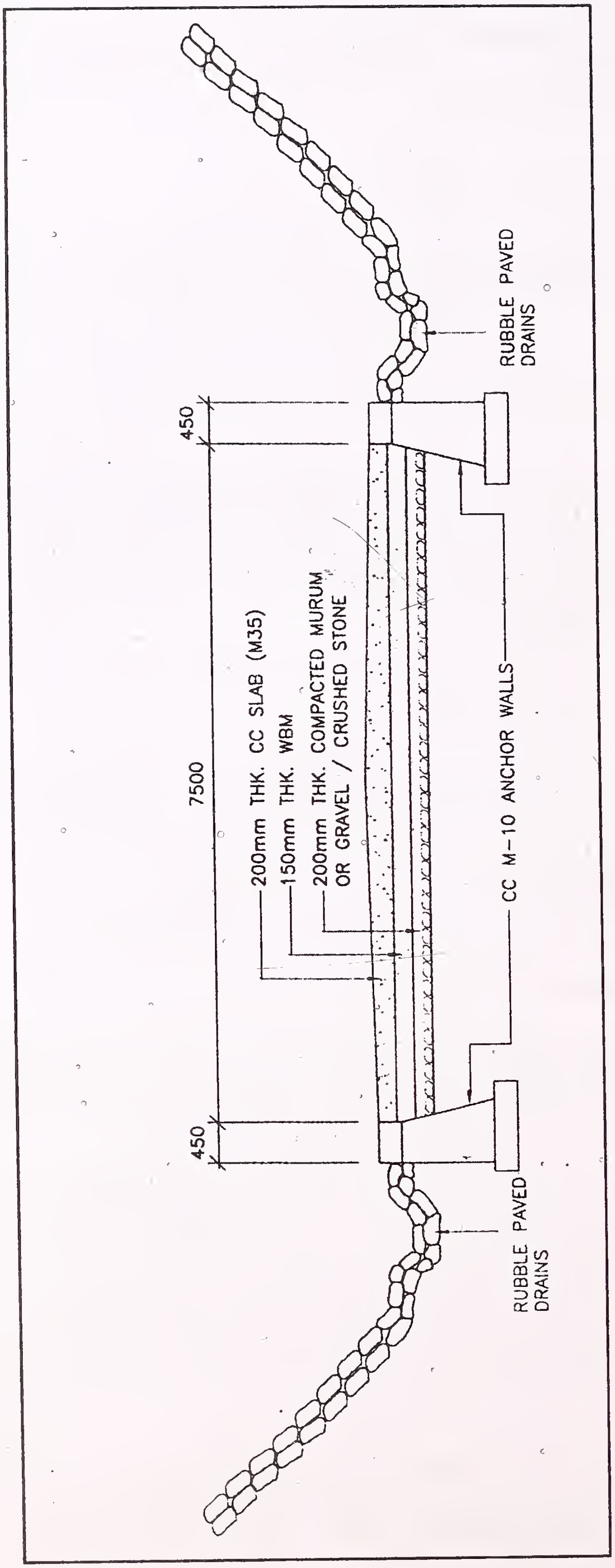


Fig. 8.3. Cross Section of Protected Flanks

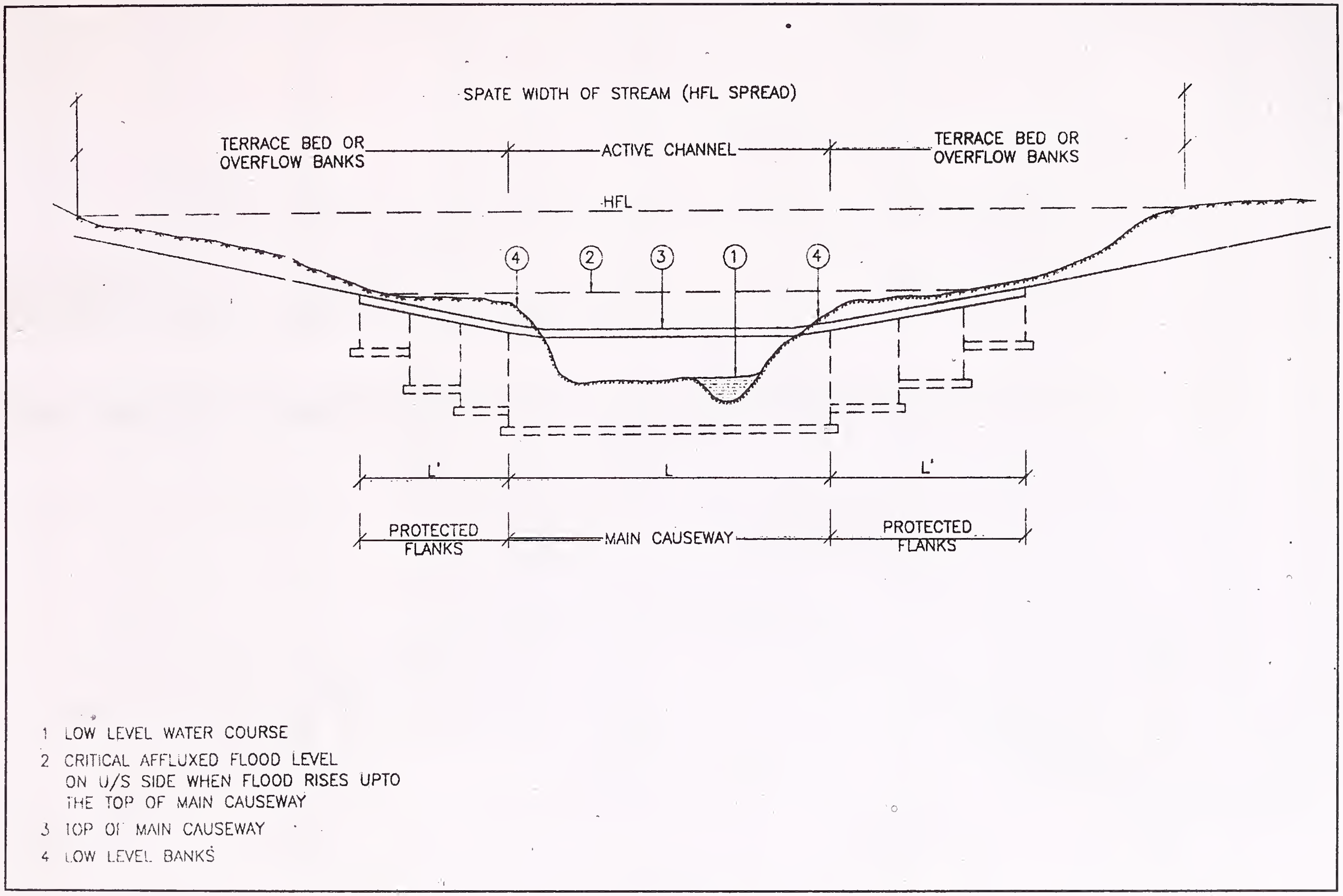


Fig. 8.4. Defined (Virgin) Cross Section of Stream

(iv) The foundation of the level portion and 1 to 2 m raised portion of either side of face wall should be taken sufficiently deep to avoid exposure due to scouring as shown in **Fig. 8.4.**

(v) For the structural stability and better hydraulic performance, it is desirable that batter of face walls should be provided on the outside faces.

8.2. Protection Work

(i) If it becomes necessary to provide the approaches in embankment than proper protection of approaches is to be done with stone pitching etc. Full width of the roadway of approaches likely to be submerged should be paved. Typical cross-sectional details of approaches are shown in **Fig. 8.5 (a), (b) and (c).**

8.2.1. Anchorages and thrust–blocks for submersible–structures

In order to resist the water current forces in the form of drag and uplift forces it is advisable to anchor the deck slabs with pier/abutment caps in case of vented causeways/submersible bridges.

(i) **For solid deck slabs** – Special arrangements, to anchor the deck to the pier/abutment (with stainless steel anchor rods) against uplift or lateral thrust and at the same time to allow longitudinal movements due to contraction and expansion because of temperature effect, are required to be made. Typical details of anchoring and thrust block are shown in **Fig. 8.6(a)** and **8.6 (b).**

(ii) **For hollow box – girder superstructure:** - As a protection measure, the following arrangements are used to withstand safely the effects of water current forces in case of hollow box type superstructure;

(a) For stability against uplift forces acting on hollow boxes superstructure, holding down Stainless steel anchorages are provided as shown in **Fig. 8.7 (a).**

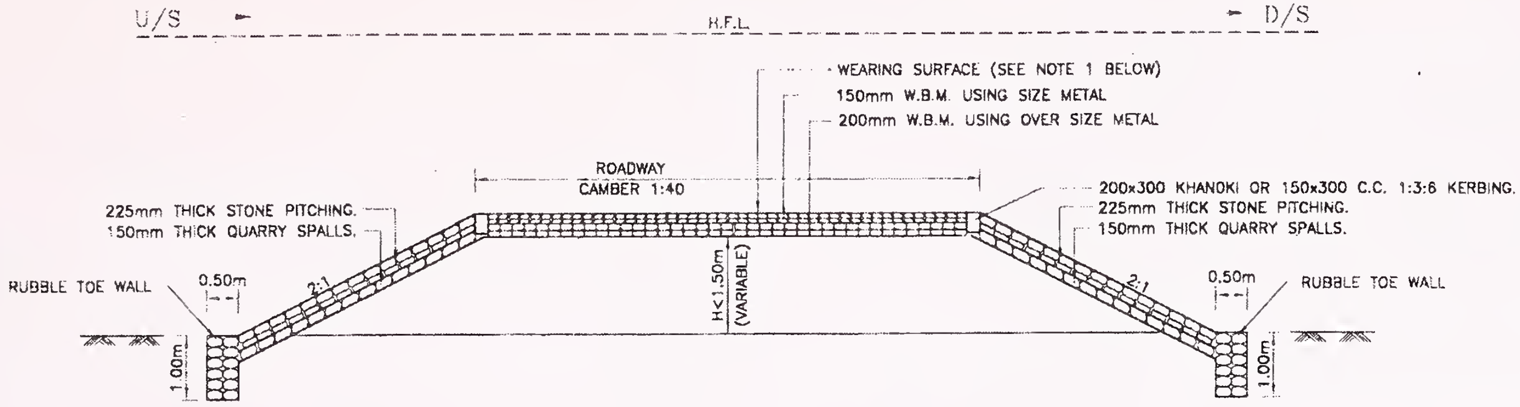
(b) For stability against drag forces, reinforced cement concrete thrust blocks either alone or in combination with side elastomeric pads are provided over pier/abutment cap as shown in **Fig. 8.7 (b).**

8.3. Appurtenances

Appurtenances have their own value for (i) safety, (ii) aesthetics and architectural point of view. General detail, guard stones, and railing are given below:

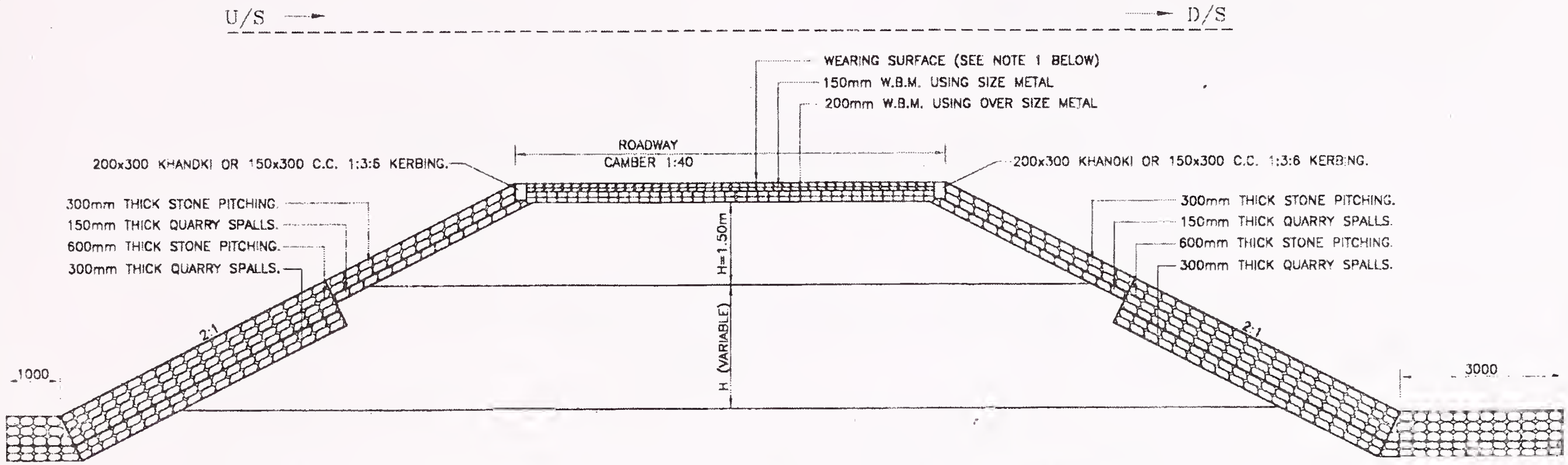
8.3.1. R.C.C. Streamlined guard-stones at 1.5 m c/c are generally provided on vented and non-vented causeways (flush causeways). These are discontinuously provided over the deck & approaches within the zone of the affluxed HFL and should be cast monolithically with slab over headwalls so as to form a firm grip **(Fig. 8.8).** Dressed stones can also be used as guard stones if suitable size of stones (say 225 mm x 225 mm x 400 mm) are available.

In the case of other submersible structures, the use of guard stones should be restricted to the approaches.



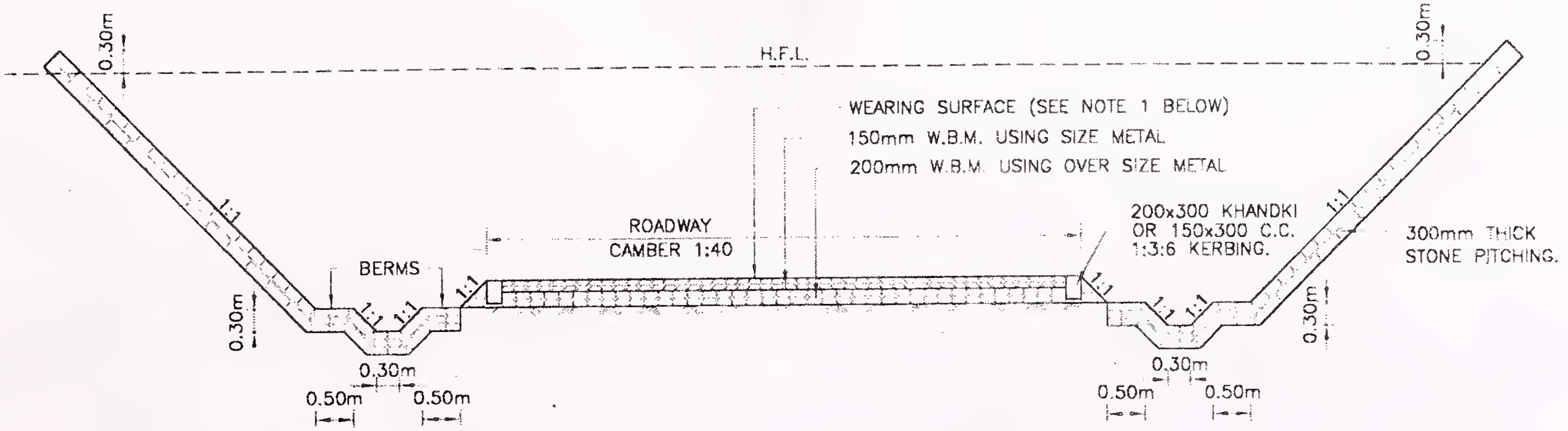
1. TYPE CROSS SECTION OF SUBMERSIBLE APPROACHES IN EMBANKMENT FOR HEIGHT UPTO 1.50m.

Fig. 8.5 (a)



2. TYPE CROSS SECTION OF SUBMERSIBLE APPROACHES IN EMBANKMENT FOR HEIGHT MORE THAN 1.50m UPTO 3.0m.

Fig. 8.5 (b)



3. TYPE CROSS SECTION FOR SUBMERSIBLE APPROACHES IN CUTTINGS.

Fig. 8.5 (c)

1 WEARING SURFACE : FULL WIDTH OF ROADWAY SHALL BE BLACK TOPPED. IF RAINFALL IS >1000mm/YEAR 50mm FULL GROUT WITH SEAL COAT SHALL BE PROVIDED. IF RAINFALL IS <1000mm/YEAR 20mm PREMIXED CARPET WITH SEAL COAT SHALL BE PROVIDED. IT SHALL BE DESIGNED AS PER RELEVENT IRC

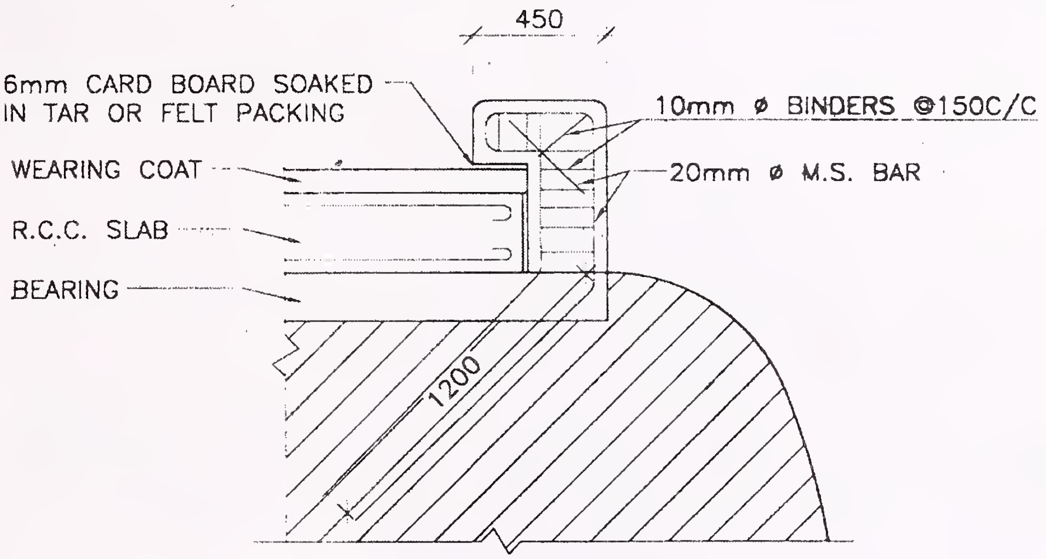
2 THE SLOPE OF CUTTING SHALL GENERALLY BE 1:1 EXCEPT FOR VERY SOFT STRATA WHERE IT SHALL BE 1½:1. FOR HARD STRATA SUITABLE USUAL STEEPER SLOPES SHALL BE PROVIDED. THE SLOPE OF EMBANKMENT SHALL BE 2:1.

3 FOR CROSS SECTION IN CUTTING THE BERMS ON EITHER SIDE OF THE GUTTER SHALL BE 0.50m. WIDE. HOWEVER IN CASE OF VERY SOFT STRATA THE BERM ON CUTTING SIDE SHOULD BE 1.00m. WIDE, KEEPING BERM ON ROAD SIDE AS 0.50m.

4 300mm THICK STONE PITCHING WITH CEMENT POINTING (1:3) SHALL BE PROVIDED FOR GUTTER. THE BERMS AND THE SIDE SLOPES OF THE CUTTING IF STRATA IN SOFT OR VERY SOFT. THIS STONE PITCHING SHALL EXTEND UPTO A LEVEL 0.30m. ABOVE H.F.L. THE PITCHING IS NOT NECESSARY WHERE STRATA IS HARD.

5 TYPE CROSS SECTION SHOWN ARE FOR SUBMERSIBLE APPROACHES IN CUTTING OR EMBANKMENT IN FLOOD ZONE.

6 PROPERLY EMBEDED KERB STONE AT U/S AND D/S EDGES MADE OUT OF KHANDKI STONES OF 200x300 OR 1:3:6 OF 150x300 BE PROVIDED. THIS WILL PREVENT CAVITATION AND DAMAGE TO EDGES.



2 nos. of M.S. rod 1200mm bent to shape as shown built in the piers to hold down the slab against uplift. allowance for expansion & contraction has to be made by inserting card board soaked in tar or felt as shown in section

Fig. 8.6 (a) Thrust Block Anchor

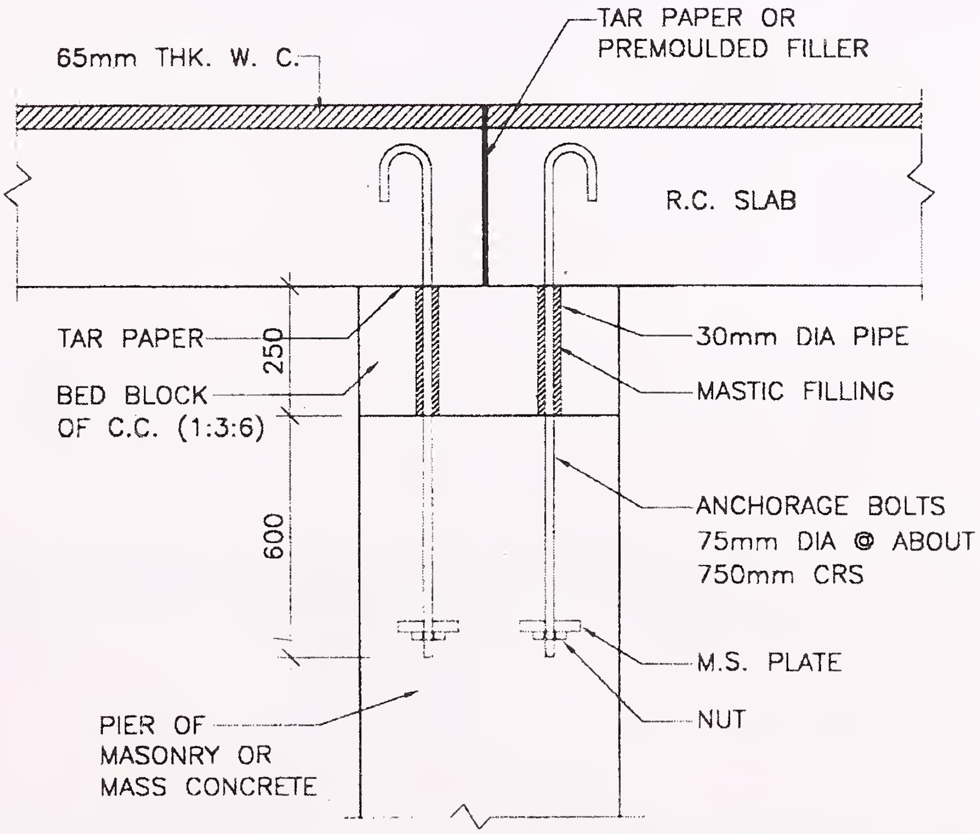


Fig. 8.6 (b) Deck Slab Anchorage

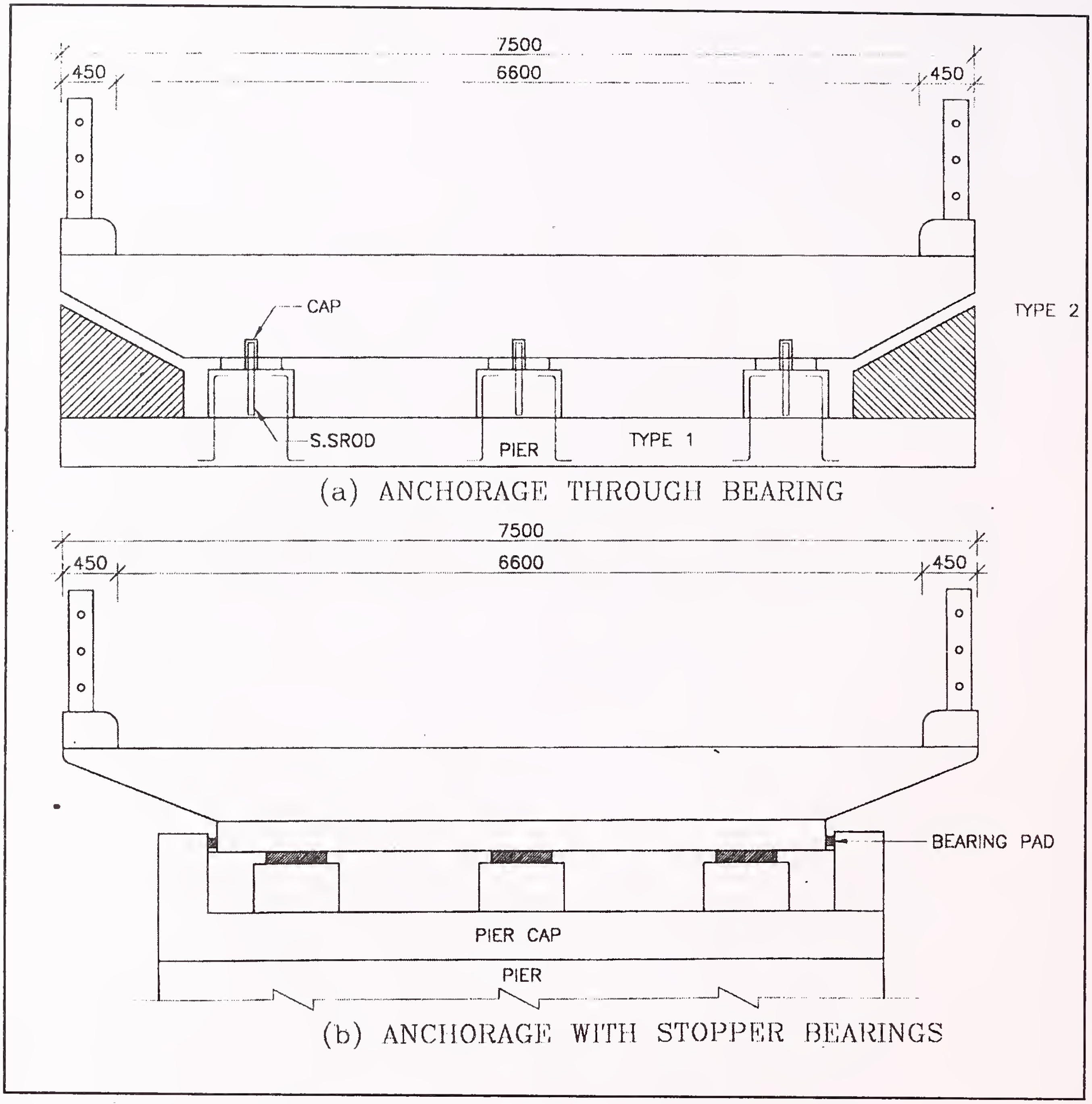


Fig. 8.7. Protection Measures from Water Current Forces

8.3.2. R.C.C. kerb

Discontinuous kerbs with 300 mm wide gap @ 1800 mm centre to centre (1500 mm continuous length of kerbs) should be provided with gaps of opposite kerbs kept in alignment with the flow for streamlining of flow.

8.3.3. Railings

8.3.3.1. Pipe railing: Typical details for pipe railing for submersible bridges are shown in **Fig. 8.9(a)** for reference and guidance.

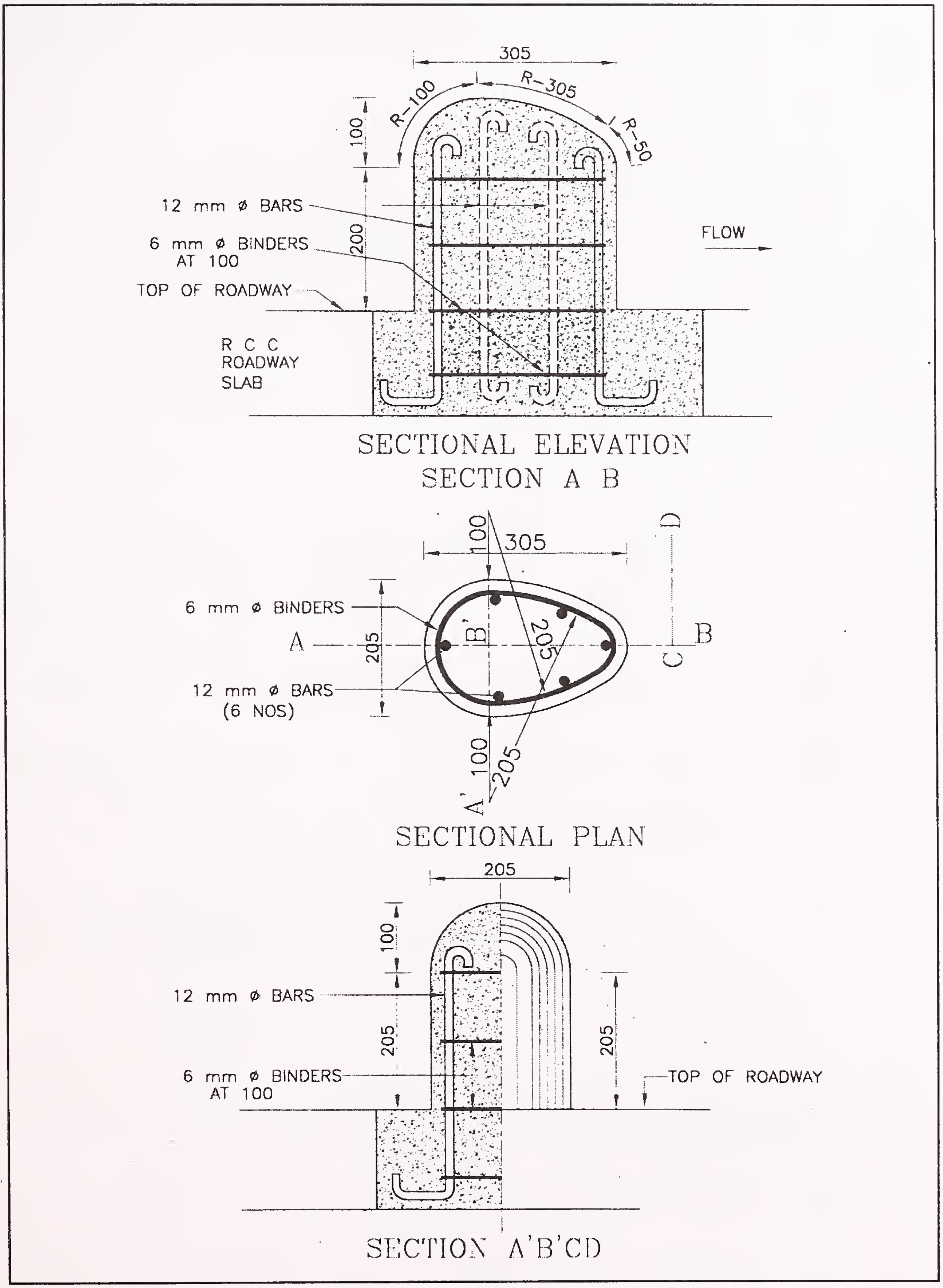


Fig. 8.8. Details of Guard Stones

8.3.3.2. Collapsible railing: The railings are also of steel section. These can be collapsed flat when flood level approaches the kerbs. **Fig. 8.9 (b)** shows the typical details of such type of railings.

8.3.4. Debris arrester

During monsoon particularly during the first couple of rains a lot of debris is carried by floods. This debris gets obstructed by the piers and the superstructure. This causes damages to structure and obstruction to flow of water. The phenomenon is more pronounced in the case of bridges in forest areas where the floods carry a lot of trees and branches. This situation can be quite dangerous for smaller spans. Generally the banks get out flanked if the structure is strong enough to withstand water current forces. It is, therefore, desirable to resort to spans of minimum 8 m or so. This will allow a lot of debris to pass through the bridge. If the spans are smaller in comparison to the size of debris, debris arresters should be provided on up steam of the bridge site so that there is free flow of floodwater through the bridge. **Fig. 8.10 (a)** and **(b)** show typical details of concrete and steel type debris arresters.

8.3.5. Surge holes and inspection holes

(i) Surge holes are provided in the deck slab and in the webs of submersible bridges to relieve uplift under rising flood waters (See **Fig. 7.7).** However, holes in the deck slab should be avoided as these would endanger traffic safety and affect the riding quality of the deck.

(ii) Such openings are also provided in the bottom slab (soffit) of box sections to maintain equal level of flood water both inside and outside.

(iii) Some openings in webs/diaphragms of box girder superstructures across large rivers would also serve the purpose of providing access for inspection. Such openings should therefore be of man-hole size. The minimum required area of all other openings is calculated considering rise in flood level @ 300 mm per hour.

(iv) Silt accumulated inside the box type of submersible bridges is to be removed manually or by water jets at the end of each flood season. The top of the soffit slab of the box girders may be provided with mild cross slope not exceeding 4% to facilitate the removal of silt by jets.

8.3.6. Flood gauges

For safety of road users, flood gauges conforming to IRC:67 at about 15 m c/c to indicate the depth of water over the road surface/deck alongwith danger level should be installed on all submersible structures and approaches likely to be submerged.

8.3.7. Informatory/warning sign boards

Advance warning/cautionary signs giving information about the proximity of the submersible structure, speed limit, depth of water during annual floods, limits of submergence should be installed

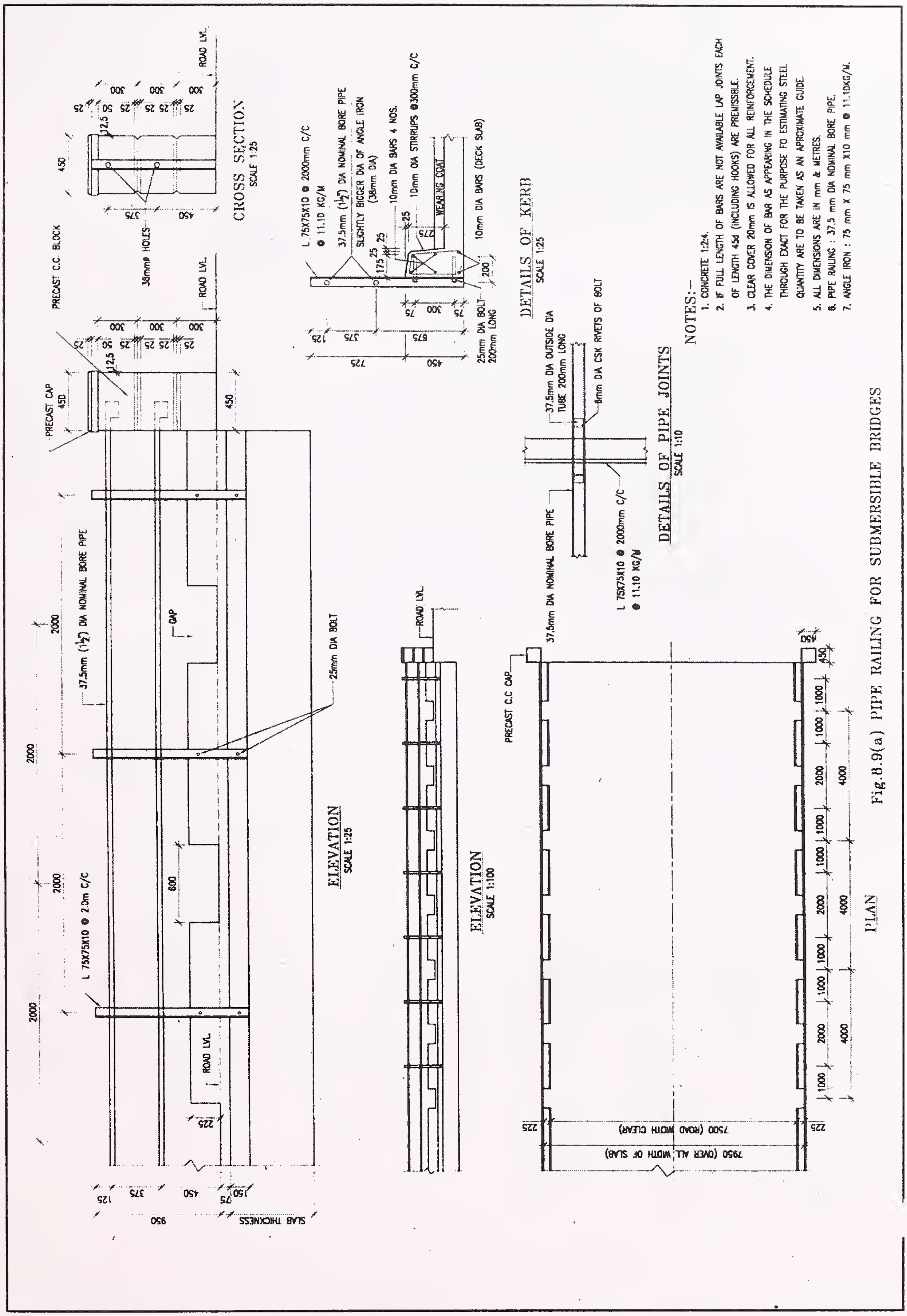


Fig. 8.9 (a) Pipe Railing for Submersible Bridges

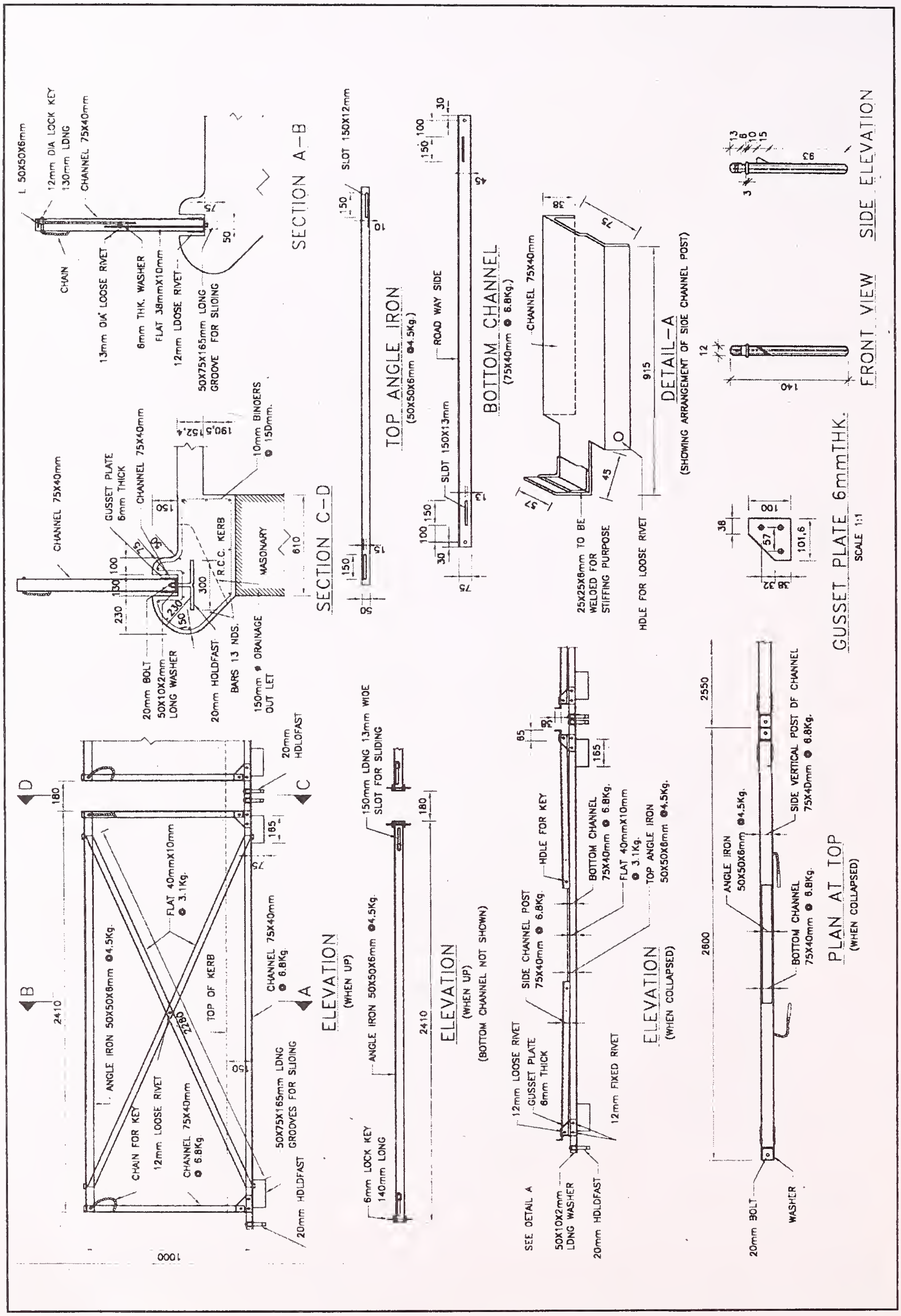


Fig. 8.9 (b) Collapsible Railing for Submersible Bridges

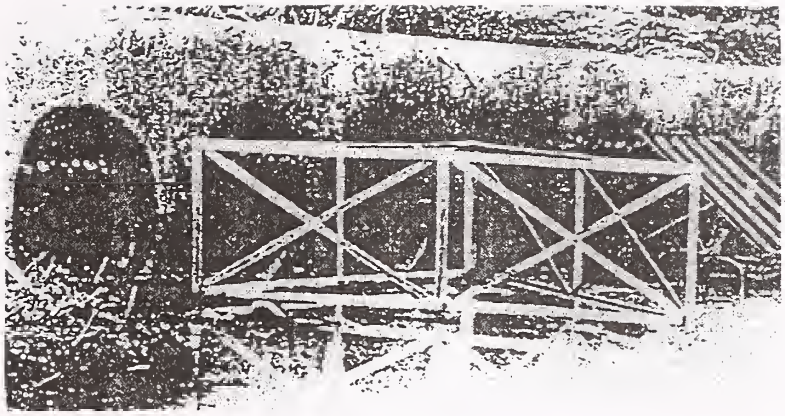


Fig. 8.10 (a)

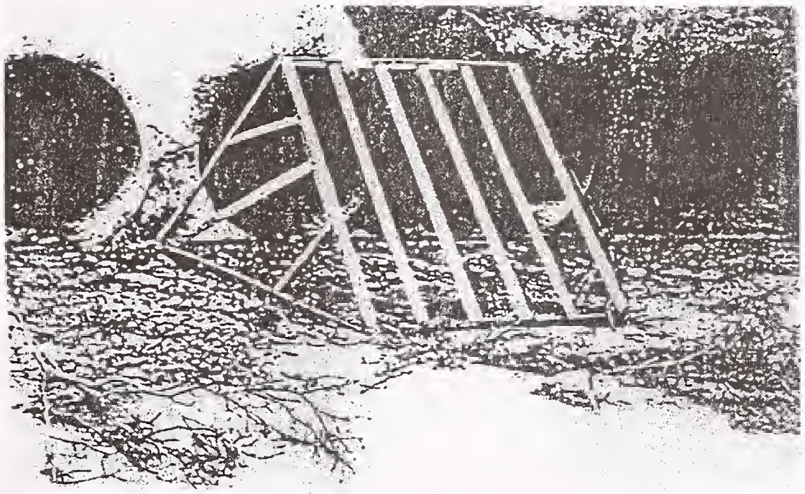


Fig. 8.10 (b)

on either side of submersible structures. Two number of advance warning informatory signs on either approach of the submersible structure one at about 200 m from the start of submerged portion of the approach and the other at 50 m from the start of submersible structure. The signs should contain the following warnings:

(i) **"Slow Down. Submersible Structure 200 m Ahead Speed Limit 15 kmph"**

(ii) **"Dead Slow Submersible Structure 50 m Ahead**

(iii) **"Do not Cross when Flood Water Overtops the Carriageway"**

8.3.8. Rumble Strips alongwith cautionary signs as per IRC:99 shall be provided at 30 m ahead of the submersible bridge on either approach road.

REFERENCES

In this publication reference to the following Standards of IRC, IS, CBIP and others have been made. At the time of publication, the editions indicated were valid. All Standards and Guidelines are subject to revisions and the parties to agreements based on these guidelines are encouraged to investigate the possibility of applying the most recent editions of standards indicated below:

A. CODE OF PRACTICE

1.	IRC:5	Standard Specification and Code of Practice for Road Bridges, Section-I - General Features of Design (Seventh Revision)
2.	IRC:6	Standard Specification and Code of Practice for Road Bridges, Section-II - Loads and Stresses (Fourth Revision)
3.	IRC:38	Guidelines for Design of Horizontal Curves for Highways and Design Tables (First Revision)
4.	IRC:52	Recommendations about the Alignment Survey and Geometric Design of Hill Roads (Second Revision)
5.	IRC:67	Code of Practice for Road Signs
6.	IRC:73	Geometric Design Standards for Rural (Non-Urban) Highways
7.	IRC:75	Guidelines for the Design of High Embankments
8.	IRC:78	Standard Specification and Code of Practice for Road Bridges, Section-VII - Foundations and Substrcuture (Second Revision)
9.	IRC:83 Part II	Standard Specification and Code of practice for Road Bridges, Section-IX–Bearings, Part II - Elastomeric Bearings
10.	IRC:83 Part III	Standard Specification and Code of practice for Road Bridges, Section-IX–Bearings, Part III - POT, POT-cum-PTFE, PIN and Metallic Guide Bearing
11.	IRC:86	Geometric Design Standards for Urban Roads in Plains
12.	IRC:89	Guidelines for Design and Construction of River Training and Control Works for Roads Bridges (First Revision)
13.	IRC:99	Tentative Guidelines on the Provision of Speed Breakers for Control of Vehicular Speeds on Minor Roads
14.	IS:1498	Classification and Identification of Soils for General Engineering Purpose
15.	IS:1786	Specification for High Strength Deformed Bars and Wires for Concrete Reinforcement
16.	IS:1888	Method of Load Tests on Soils
17.	IS:1892	Site Investigation for Foundations for Investigation and Collection of Data

18. IS:2131 Method for Standard Penetration Test for Soils

19. IS:2132 Thin Walled Tube Samples of Soils

20. IS:2720 Methods of Test for Soils

21. IS:4434 In-Situ Vane Shear Test for Soils

22. IS:4968 (Part 1 & 2) Method of Dynamic Cone Penetration Test for Cohesive Soils

23. IS:4968 (Part 3) Method of Cone Penetration for Cohesive Soils

B. PUBLICATIONS

1. Design and Construction Practice of Submersible Bridges and Causeways – Indian Roads Congress, 1990 (Panel Discussion and Important Papers)

2. Pocket Book for Bridge Engineers Ministry of Road Transport and Highways, 2000

3. Central Board of Irrigation and Power Design of Weirs on Permeable Foundation (Publication No. 12)

4. Investigation, Design and Construction of Submersible Bridges D. Johnson Victor Jl.IRC Vol. XXIV, Part 1, 1959

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified therein)