Eastern Economy Edition

Second Edition

Design of Bridge Structures

T.R. Jagadeesh

M.A. Jayaram

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T.R. Jagadeesh
Principal
HMS Institute of Technology
Tumkur

M.A. Jayaram
Professor
Siddaganga Institute of Technology
Tumkur

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T.R. Jagadeesh and M.A. Jayaram

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Preface

First and foremost, we express our gratitude to the tremendous response that we got from the readers of this book. This Second edition of this book has three main objectives—first, a general updates of the bridge designs as per revised IRC codes; secondly, an improvement in the presentation of the material, and thirdly and more importantly to make the book complete by incorporating topics like *design of prestressed concrete bridge decks* and *applications of artificial intelligence in bridge engineering*.

In writing this textbook, we have striven to fulfil several needs, within the compass of a single volume, not perhaps met by existing works. First, we have kept the civil engineering students in mind who require a book up to degree or diploma standard covering both hydraulic and structural design aspects of the bridge. Secondly, we have attempted to provide full-fledged design drawings depicting various views of bridges. Thirdly, we have included computer programs to help students take up computer-aided projects in bridge design. Fourthly, the book can be used as a ready reckoner on important design aspects of bridges by practising civil engineers.

The entire bridge design process is covered in two sections. Section I is devoted to hydraulic design requirements of bridges. It begins with a chapter on the importance of hydraulic factors in bridge design and computation of peak flood flows. Chapter 2 which follows, deals with the methods of analysis of runoff response from catchments. Chapter 3 is devoted to hydraulic geometry of river channels, linear waterways, economic span and scour.

Section II deals with structural design and drawing requirements of bridges. Chapter 4 explains the standard loading conditions developed by the Indian Road Congress (IRC), which form a consistent basis for design procedures used throughout the book. Chapters 5–11 provide elaborate coverage of design of masonry arch bridges, pipe culverts, slab bridges, box culverts, beam and slab bridges, and composite bridges. Chapters 12–14 cover other aspects such as design of substructures, foundations, bearings and expansion joints. Chapter 15 deals with design of prestressed bridge deck, design of prestressed slab and design of post-tensioned T-beam have been elaborated. Chapter 16 explains the applications of artificial intelligence in bridge engineering.

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Numerous solved examples have been included to illustrate both analysis and design calculations. Besides, neatly done drawings will help students grasp the crucial aspects of bridge design.

It has been our endeavour to offer the most complete and practical treatment of every aspect of bridge design for use as a textbook by students of civil engineering. Professionals engaged in bridge design should also find this book useful.

We sincerely thank all those teachers and students who have been of enormous help to us in greatly improving the text. We are also grateful to Sri A.G. Umaprasad, Lab Instructor, for preparing neat drawings with great care and responsibility.

Constructive criticism and suggestions for the improvement of the text will be gratefully acknowledged.

T.R. Jagadeesh
M.A. Jayaram

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Section I

Hydraulic Design

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Chapter 1

Introduction

The design of a bridge across a river demands that detailed attention be paid not only to the route location, potential traffic flow and structural and foundation details, but also to the characteristics of the river beneath. To evaluate the characteristics of the river, it is necessary to collect information on aspects such as channel stability, sediment discharge, scour and sediment deposition and hydrodynamic forces. Predictions about what may happen in particular circumstances also need to be made. For such predictions, one of the important data required is the hydraulic data. This data can be gathered by aerial, hydrographic or hydraulic surveys.

1.1 IMPORTANCE OF HYDRAULIC FACTORS IN BRIDGE DESIGN

The process of arriving at final design of a bridge is very elaborate and complex, involving geotechnical, hydraulic and structural attributes. These attributes are adjusted in such a way that they satisfy functional, economical and aesthetical constraints. The hydraulic parameters which influence sequential phases of bridge design can be summarized as follows:

Phase I. During this phase, site reconnaissance, review and analysis of available river data are undertaken with a view to selecting possible bridge locations that are compatible with the proposed communication route.

Phase II. At each of the possible bridge sites, the hydrographic and hydraulic surveys are conducted.

Phase III. From the available data, the following hydraulic parameters are assessed:

1. Maximum flood flow
2. Design flood flow
3. Maximum flood level
4. Navigational requirements
5. Bed and bank characteristics
6. Approach velocity and direction
7. Flood plain width
8. River meandering characteristics.

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Phase IV. During this phase, linear waterway, normal scour depth, afflux. backwater effect, flow velocity, including the works requirements for a suitable river training scheme are determined.

Phase V. Various alternative methods of construction taking into account factors such as structural loading, soil characteristics, economy of construction, availability of manpower (skilled and unskilled) and materials of construction, access to the site, prevailing climate, environmental impact, and maintenance are considered. Detailed investigation of certain other factors, which affect the configuration of the bridge, is also necessary at this stage. Proper freeboard, vertical clearance, height of the bridge and hydrodynamic forces on the pier are estimated. Due consideration is given to proper location and geometty of the piers. The geometry of the piers should be such as to minimize the backwater effect and scour. The piers should be aligned with the principal direction of flow so that a streamline flow is maintained.

Phase VI. For the proposed configuration of the bridge, normal scour, maximum scour and backwater effect are computed. For more refinement in computations, if needed, the number of piers is reduced and also if more reduction in scour is required, the local and general scour effects can be altered by adjusting the waterway opening.

Phase VII. The cost of alternative schemes for each location is appraised in this phase. If the cost of the scheme is outside the budgetary constraints, savings may be possible by altering the designs.

Phase VIII. After studying the alternative bridge designs for each of the possible bridge locations, the best scheme is selected for detailed design. Based on the recommendations, a decision to verify the hydraulic parameters by model investigations is normally considered during this phase.

1.2 COMPUTATION OF PEAK FLOOD FLOW

Predicting peak discharge rates or synthesizing complete discharge hydrographs for use in the design of minor and major bridge structures are two of the more challenging aspects of bridge hydrology. Generally, a hydrologist is required to provide peak rates of discharge and stages at a design frequency, or synthesize a complete discharge hydrograph for a design storm.

Hydrologic design aspects of a complete bridge structure are considerably more complex than those of a small bridge or culvert. The economic selection of waterway from various possibilities dictates the final design and is a function of the degree of protection to be provided, project economy, agency policy and construction standards.

Most of the infomation and techniques presented in this chapter are directed towards prediction of peak discharges with particular reference to small, medium and large size catchments, based on availability of rainfall-runoff data including the catchment characteristics.

1.2.1 Empirical Methods

The empirical formulae for prediction of peak discharges are employed only when sufficient data are not available for detailed and precise analysis of catchment response. One of the

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drawbacks of these empirical methods is that each formula is applicable only to the catchment for which it is developed.

The general form of the empirical equation is

$$Q = CA^n \tag{1.1}$$

where

Q = peak discharge

A = area of the catchment

C and n are constants, which absorb storm and catchment characteristics.

Dicken's formula

Dicken for the first time in 1885 made an attempt and derived a formula of the following form to predict the maximum flood on the basis of the studies conducted on Indian rivers, for determining waterway for bridges.

$$Q = CA^{3/4} \tag{1.2}$$

The above formula is applicable to catchments in Central and North India. The constant C varies from 2.80 to 5.6 for plain catchments and from 14 to 28 for mountainous regions. From the recent studies [1] conducted to make Dicken's formula applicable to different parts of India, the constants as listed in Table 1.1 are suggested:

Table 1.1 Dicken's constants for different watersheds

Type of watershed	*Dicken's constant C*
Bare catchments covered with precipitous hills	19.6–28.0
Catchments with hills on the skirts with undulating country	14.0–16.8
Undulating country with hard indurated clay	11.2–14.0
Flat, sandy, absorbent or cultivated plains	2.8–7.0

Ali Nawaz Jung Bahadur's formula

This formula [2] involves a logarithmic function of areas as the exponent. Jung Bahadur was of the opinion that the formula to be applied should be simple but rational without involving much of the judgement on the part of the hydrologist selecting the coefficient.

$$Q = C(0.368A)\left(0.925 - \frac{\log 0.386A}{14}\right) \tag{1.3}$$

The value of C is 49 and 60 for regions in South and North India respectively.

Ryve's formula

Ryve modified the Dicken's formula for application to catchments in southern regions of India. The modified formula is

$$Q = CA^{2/3} \tag{1.4}$$

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where

A = area of the catchment in sq. km

C = 6.74 for areas up to 24 km from the coast

= 8.45 for areas 24–161 km from the coast

= 10.1 for hilly areas.

The modified Ryve's formula can also be written as

$$Q = CA^{2/3} - C_1 A_1^{2/3} \tag{1.5}$$

where

C_1 = modified Ryve's constant which varies from 1/3 to 1/5 of C

A = combined area of the catchment in sq. km

A_1 = intercepted area of the catchment in sq. km.

Inglis formula

Inglis [3] studied the available hydrologic records of catchments of erstwhile state of Bombay and of a few catchments outside the state to arrive at a simple relationship as follows:

$$Q = \frac{124A}{\sqrt{A - 10.24}} \tag{1.6}$$

This formula is applicable to regions in Maharashtra and Deccan plateau.

1.2.2 Envelope Curves

For developing envelope curves, the maximum flood discharges and the respective areas of the drainage basin possessing similar hydrometerologic characteristics are compiled. The maximum flood discharges versus the drainage areas are plotted on the log-log scale and a smooth curve known as the envelope curve is fitted to pass through the plotted points: By using this curve, the maximum flood discharge can be estimated for a given drainage basin. Envelope curves for Indian rivers developed by Kanwar Sain and Karpov [1] are shown in Fig. 1.1.

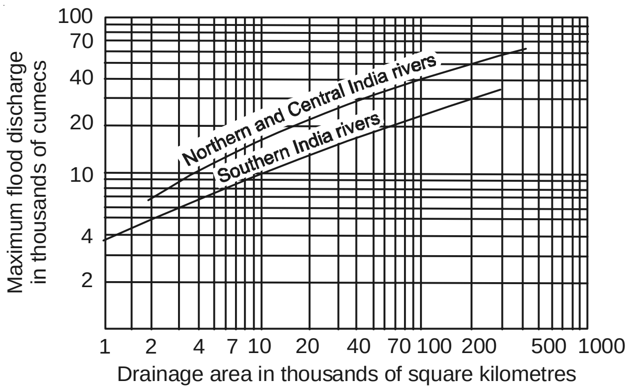

Fig. 1.1 Envelope curves for Indian rivers.

1.3 FLOOD FLOWS AND CATCHMENT SCALE

When the intensity of rainfall exceeds the abstractive capability of the catchment, surface runoff occurs. Eventually, large amounts of surface runoff concentrate to provide large flow rates referred to as floods. The study of floods, their occurrences, causes, transport and effects is the essence of flood hydrology.

In general, rainfall varies in space and time. However, under certain given conditions, it is possible to assume the rainfall to be either (1) constant in both space and time or (2) constant in space but varying in time or (3) varying in both space and time. The catchments scale helps to determine which one of these assumptions is justified on practical grounds. Generally, small catchments are those in which runoff can be modelled by assuming constant rainfall in both space and time. Midsize catchments are those in which runoff can be modelled by assuming rainfall to be constant in space but varying in time. Large catchments are those in which runoff can be modelled by assuming rainfall to vary in both space and time.

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Chapter 2

Catchments

2.1 SMALL CATCHMENTS

2.1.1 Characteristics

The characteristics of small catchments can be stated as follows:

1. Rainfall is assumed to be uniformly distributed both in time and space.
2. Storm duration generally exceeds the concentration time.
3. Runoff is primarily by overland flow.
4. Channel storage processes are negligible.

It is difficult to define the upper limit of size of small catchments. Sizes ranging from 0.65 to 12.5 sq. km have been mentioned in the literature [4, 5]. The current practice is to use 1.3 to 2.5 sq. km, as the upper limit. There is no theoretical lower limit, and, catchments as small as 1 ha also fall into this category. Some authorities regard small catchments as those whose concentration time is 1 h or less. It may also to be noted that small catchments are either concentrated or superconcentrated.

2.1.2 Analysis of Runoff Response

The following methods are used for analysis of runoff response from small catchments:

1. Parametric approaches, such as the rational method, which lump all relevant hydrologic processes into a few descriptors like rainfall intensity and catchment area.
2. Deterministic methods such as overland flow models. These methods are beyond the scope of this book.

Rational method

The rational formula was introduced in the United States by Kuichling in 1886[6]. Since then, it has become the most widely used method for the analysis of runoff response from small

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catchments. This method provides only the value of peak discharge. The peak discharge is primarily due to overland flow rather than stream flow.

The rational method takes into account the following parameters:

1. Rainfall intensity
2. Rainfall duration
3. Rainfall frequency
4. Catchment area
5. Hydrologic abstractions
6. Runoff concentration
7. Runoff diffusion.

The equation for the rational formula can be written as

$$Q = 0.278CIA \tag{2.1}$$

where

Q = peak discharge in cumecs, corresponding to a given rainfall intensity, duration and frequency

C = runoff coefficient, a dimensionless parameter that includes the abstraction and diffusive properties of the catchment

I = rainfall intensity in mm/h, averaged in time and space

A = catchment area in sq. km.

2.1.3 Runoff Concentration

Runoff concentration implies that the flow rate at the outlet will gradually increase until rainfall from the entire catchment has had time to travel to the outlet and is contributing to the flow at that point. At that time, the maximum or equilibrium, flow rate is reached, implying that the surface runoff has concentrated at the outlet.

2.1.4 Concentration Time

The time that is required for a parcel of water to travel from the farthest point of the divide to the catchment outlet is referred to as the time of concentration of the catchment.

Concentration time formulae

Most of the concentration time formulae are empirical in nature and therefore have a somewhat limited application.

A well-known formula relating the concentration time to slope and length parameters is the Kirpich formula [7], applicable to small agricultural watersheds with drainage areas of less than 80 ha. This formula is

$$t_c = \frac{0.06628L^{0.77}}{S^{0.385}} \tag{2.2}$$

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where

t_c = concentration time in hours

L = length of the principal watercourse, from the outlet to divide, in km

S = slope between the maximum and minimum elevations in m per m.

Another formula used for estimation of concentration time is that of Hathaway [8]

$$t_c = \frac{0.606(Ln)^{0.467}}{S^{0.234}} \tag{2.3}$$

in which n is the roughness factor of the catchment surface and all other terms are the same as in Kirpich formula. The applicable values of n for Hathaway equation are given in Table 2.1.

Table 2.1 Values of n for Hathaway equation

Type of surface	*Value of n*
Smooth impervious	0.02
Smooth bare picked soil	0.10
Grass, row crops or moderately rough bare soil	0.20
Pasture	0.40
Deciduous timber land, or coniferous timber land or deciduous land with deep litter or grass	0.80

EXAMPLE 2.1

Using Kirpich and Hathaway formulae, estimate the value of concentration time for a catchment having the following characteristics: L = 0.8 km, S = 0.005, and type of surface being smooth impervious soil.

Solution

From Kirpich formula, we have

$$t_c = \frac{0.06628 \times 0.8^{0.77}}{0.005^{0.385}} = 0.429 \text{ h}$$

Using Hathaway formula, we get

$$t_c = \frac{0.606(0.8 \times 0.2)^{0.467}}{0.005^{0.234}} = 0.303 \text{ h}$$

2.1.5 Application of the Rational Method

The drainage boundaries may be estimated from topographic maps or aerial photographs. The catchment area is calculated either by a planimeter or by some other suitable means. For

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application of the rational formula, the primary requirement is that the catchment should be small. The drainage survey should also include:

1. Land use and land use changes
2. Percentage of imperviousness
3. Characteristics of soil and vegetative cover that may affect the runoff coefficient
4. General magnitude of ground slopes and catchment gradient necessary to determine the value of concentration time.

The concentration time of the catchment can he computed by one of the following methods:

1. Using an empirical formula.
2. Using a value of flow velocity based on hydraulic properties and then calculating the travel time through the catchment's hydraulic length.
3. Calculating the steady equilibrium flow velocity (using Manning equation) and then computing the associated travel time through the hydraulic length.

Since the rational method is applicable to concentrated and superconcentrated catchments, knowing the value of concentration time, the storm duration is made equal to the concentration time. Then, a rainfall frequency based on importance of the project is chosen. Later, for the selected storm duration and rainfall frequency the value of rainfall intensity is obtained from the appropriate IDF (intensity-duration-frequency) curve.

Once the values of rainfall intensity and catchment area have been arrived at, an appropriate runoff coefficient can be selected from Table 2.2 and finally the value of peak flow can be computed.

Table 2.2 Average runoff coefficient for use in the rational method

Topography and vegetation	*Soil texture*		
	Open sandy loam	*Clay and silty loam*	*Tight clay*
Woodland			
Flat	0.10	0.30	0.40
Rolling	0.25	0.35	0.50
Hilly	0.30	0.50	0.60
Pasture			
Flat	0.10	0.30	0.40
Rolling	0.16	0.36	0.55
Hilly	0.22	0.42	0.60
Cultivated land			
Flat	0.30	0.50	0.60
Rolling	0.40	0.60	0.70
Hilly	0.52	0.72	0.81

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EXAMPLE 2.2

A slab culvert is proposed to pass a peak flow from a catchment of 2 sq. km, with a return period of 10 years. Using the IDF function of the following form, compute the magnitude of the peak flow.

$$I = \frac{100\,T^{0.2}}{(t_r + 20)^{0.7}}$$

where

I = rainfall intensity in mm/h

T = return period in years

t_r = rainfall duration in minutes.

Take C = 0.4, length of the principal course = 0.75 km, and slope = 0.005

Solution

Concentration time is given by

$$t_c = \frac{0.06628(0.75)^{0.77}}{0.005^{0.385}} = 0.40\text{ h} = 24.0\text{ min}$$

For concentrated catchments, $t_r = t_c$ = 24.0 min

Rainfall intensity is given by

$$I = \frac{100 \times 10^{0.2}}{(24.0 + 20)^{0.7}} = 11.2\text{ mm/h}$$

The peak discharge is given by

$$\begin{aligned} Q &= 0.278\ CIA \\ &= 0.278 \times 0.4 \times 11.2 \times 2 \\ &= 2.5\text{ cumecs} \end{aligned}$$

2.1.6 Composite Catchments

A composite catchment is one that produces its peak flow on account of the contribution from two or more adjacent subareas of widely differing characteristics. For instance, consider that a catchment has two subareas C and D with values of concentration time t_c and t_d respectively (say $t_c < t_d$) (see Fig. 2.1).

To compute the peak flow using the rational method, several rainfall durations are chosen, ranging from t_c to t_d, in suitable increments. The calculations proceed by trial and error, and each trial is associated with each of the rainfall durations. To calculate the partial concentration from subarea D, an assumption is made regarding the rate at which the flow is concentrated at the catchment outlet. The rainfall duration that gives the highest peak flow is taken as the design rainfall duration.

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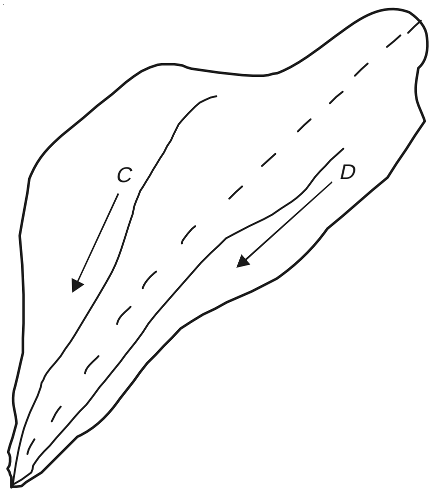

Fig. 2.1 Composite catchment.

EXAMPLE 2.3

A culvert is proposed at a site to pass the peak flow from a 1.5 sq. km composite catchment. Using the following data, compute the value of peak flood flow.

	Subarea C	*Subarea D*
Area (sq. km)	0.5	1.0
Runoff coefficient	0.4	0.2
Concentration time (min)	30	60

Assume a return period of 5 years and the following IDF function:

$$I = \frac{100\,T^{0.2}}{(t_r + 20)^{0.7}}$$

where

I = rainfall intensity in mm/h

T = return period in years

t_r = rainfall duration in minutes

Solution

To compute the contribution from subarea *D*, take the rainfall duration between 30 and 60 minutes at intervals of 10 minutes. For each rainfall duration, calculate the rainfall intensity by using the given IDF function. By assuming linear concentration, obtain the contributing areas for subarea *D* for each rainfall duration. The calculations are shown in Table 2.3.

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Table 2.3 Calculations for Example 2.3

Rainfall duration t_r (min)	*Rainfall intensity I* (mm/h)	*Contributing area of D* (km^2)
30	88.23	0.50
40	78.54	0.66
50	70.50	0.83
60	64.21	1.00

For t_r = 30 min, the peak flow contributed by subareas *C* and *D* is given by

$$Q = 0.278\ CIA$$
$$= 0.278 \times 88.23\ \{0.4 \times 0.5 + 0.2 \times 0.5\}$$
$$= 7.36 \text{ cumecs}$$

Successive trials with rainfall durations of 40, 50 and 60 min result in peak flows of values 7.24, 7.17 and 7.14 cumecs, respectively, which are all less than the above value computed for rainfall duration of 30 min. Hence, the peak flow is 7.36 cumecs and the design rainfall duration is 30 min.

2.1.7 Types of Catchment Response

Concentrated catchment

In this type of catchment response, the runoff concentrates at the outlet and reaches its maximum rate after an elapsed time equal to concentration time t_{cr}. Rainfall stops at this time, and the subsequent flows at the outlet are no longer concentrated, because not all the catchment is contributing. Therefore, the flow gradually recedes to zero (see Fig. 2.2). In practice, the recession limb is asymptotic. The effective rainfall P_e is also shown in the figure.

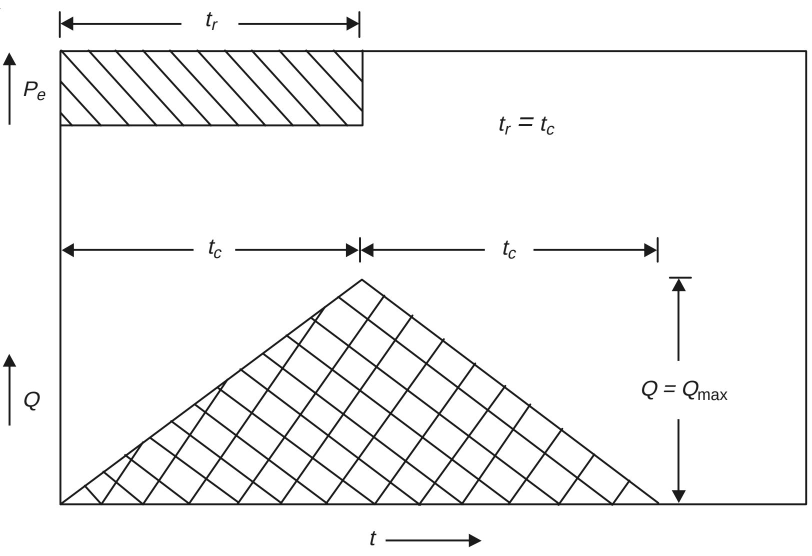

Fig. 2.2 Concentrated catchment response.

Superconcentrated catchment

This type of response occurs when the rainfall duration t_r exceeds the concentration time t_c. In this case, the runoff concentrates at the outlet, reaching its maximum (equilibrium) rate after an elapsed time equal to the concentration time. Since rainfall continues to occur, the runoff continues too. After the rainfall has stopped, the flow gradually recedes to zero (Fig. 2.3).

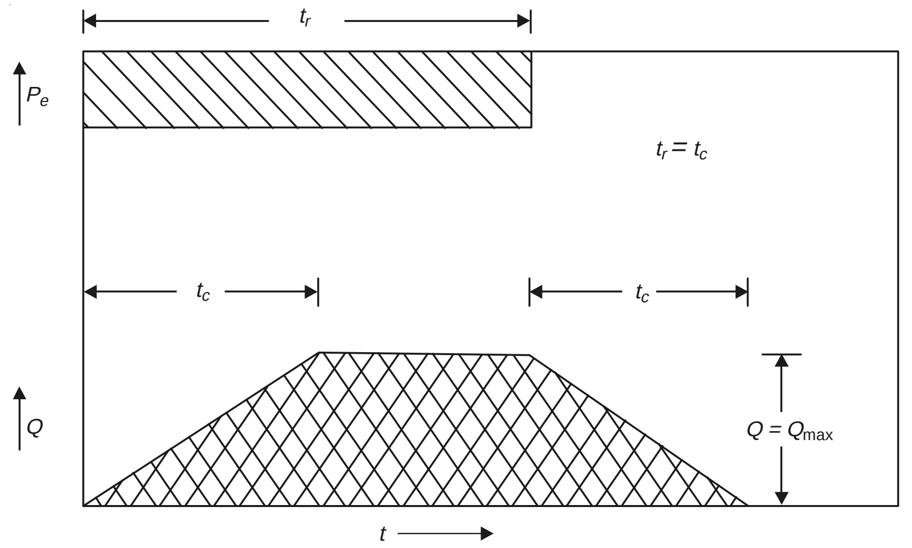

Fig. 2.3 Superconcentrated catchment response.

Subconcentrated catchment

In this type of response, the rainfall duration t_r is shorter than the time of concentration t_c, and the flow at the outlet does not reach the equilibrium state. After the rainfall has stopped, the flow gradually recedes to zero (Fig. 2.4).

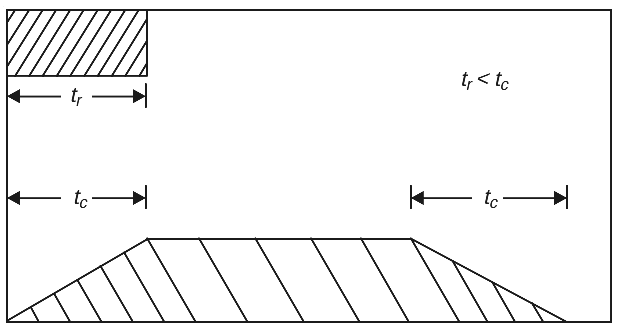

Fig. 2.4 Subconcentrated catchment response.

2.2 MIDSIZE CATCHMENTS

2.2.1 Characteristics

The characteristics of midsize catchments are enumerated below:

1. The intensity of rainfall is not the same throughout the duration of the storm.
2. Rainfall is assumed to be uniformly distributed in space.
3. Runoff response is due to both overland flow and stream-channel flow.
4. Channel storage processes are negligible.

Just as the demarcation between the small and midsize catchments is not clear, the demarcation between the midsize and large catchments is also not well defined. For midsized catchments, the runoff response is primarily the function of the catchment characteristics of the storm hyetograph, with concentration time playing a secondary role. Because of the large drainage area and the associated reduction in the overall catchment gradient, runoff diffusion is increased and so also the concentration time. Hence, the catchment response is subconcentrated. Therefore, the concentration time cannot be used as a divider for the catchment scale. Values ranging from 100 to 5000 sq. km have been used to define the demarcation between the midsize and large catchments [9]. The current practice is towards the lower value.

2.2.2 Analysis of Runoff Response

Runoff curve numbers

The SCS (US Soil Conservation Service) procedure consists of selecting a storm and computing the direct runoff by the use of curves on field studies of the amount of measured runoff from numerous soil combinations. A runoff curve number (CN) is selected from Table 2.4. This selection of the runoff curve number is dependent upon antecedent conditions and the types of covers. Soils are classified as A, B, C, or D according to the following criteria:

A (Low runoff potential). Soils having high infiltration rates, even if thoroughly wetted, and consisting chiefly of well- to excessively-drained sands or gravels are called the type A soils. These soils have a high rate of water transmission.

B Soils having moderate infiltration rates, if thoroughly wetted, and consisting chiefly of moderately-deep to deep, moderately-well to well-drained soils with moderately-fine to moderately-coarse texture are called the type B soils. These soils have a moderate rate of water transmission.

C Soils having slow infiltration rates, if thoroughly wetted, and consisting chiefly of soils with a layer that impedes the downward movement of water, or soils with moderately-fine to fine texture. These soils are called the type C soils have a slow rate of water transmission.

D (High runoff potential). Soils having slow infiltration rates, if thoroughly wetted, and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high

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Table 2.4 Runoff curve numbers

Land use of cover	*Cover* *Treatment or practice*	*Hydrologic condition*	*Hydrologic soil group* *A*	*B*	*C*	*D*
Fallow	Straight row	—	77	86	91	92
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and terraced	Poor	66	74	80	82
	Contoured and terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured and terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
	Contoured and terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77

watertable, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material are all called the type D soils. These soils have a very slow rate of water transmission.

The composite curve number (CN) for a watershed having more than one land use, treatment, or soil type can be found by weighting each curve number according to its area. The curve numbers in Table 2.4 are applicable to average antecedent moisture conditions. The other antecedent moisture conditions (AMC) are:

AMC I. A condition of watershed soils where the soils are dry but not to the wilting point, and when satisfactory ploughing or cultivation can take place. (This condition is not considered applicable to the design of flood computation methods.)

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AMC II. The condition representing the average case of annual floods, that is, the average of the conditions that have preceded the occurrence of the maximum annual flood on numerous watersheds.

AMC III. A condition of soils where heavy rainfall or light rainfall at low temperatures has occurred during the five days preceding the given storm and when the soil is nearly saturated.

The corresponding curve numbers for conditions I and III can be obtained from Table 2.5 if the curve number for AMC II is known.

Table 2.5 Corresponding runoff curve numbers for the three AMC conditions

AMC II	*AMC I*	*AMC III*	*AMC II*	*AMC I*	*AMC III*
100	100	100	60	40	78
99	97	100	59	39	77
98	94	99	58	38	76
97	91	99	57	37	75
96	89	99	56	36	75
95	87	98	55	35	74
94	85	98	54	34	73
93	83	98	53	33	72
92	81	97	52	32	71
91	80	97	51	31	70
90	78	96	50	31	70
89	76	96	49	30	69
88	75	95	48	29	68
87	73	95	47	28	67
86	72	94	46	27	66
85	70	94	45	26	65
84	68	93	44	25	64
83	67	93	43	25	63
82	66	92	42	24	62
81	64	92	41	23	61
80	63	91	40	22	60
79	62	91	39	21	59
78	60	90	38	21	58
77	59	89	37	20	57
76	58	89	36	19	56
75	57	88	35	18	55
74	55	88	34	18	54
73	54	87	33	17	53
72	53	86	32	16	52
71	52	86	31	16	51
70	51	85	30	15	50
69	50	84	—	—	—
68	48	84	25	12	43
67	47	83	20	9	37
66	46	82	15	6	30
65	45	82	10	4	22
64	44	81	5	2	13
63	43	80	0	0	0
62	42	79	—	—	—
61	41	80	—	—	—

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EXAMPLE 2.4

A watershed has the following hydrologic conditions: (1) a pasture, in fair hydrologic condition, soil group B, covering 22% and (2) a meadow, soil group B, covering 55%, and (3) woods, in poor hydrologic condition, soil group C, covering 23%. Determine the runoff *Q*, in centimetres, for a 15.2 cm rainfall. Assume an AMC II antecedent moisture condition.

Solution

From Table 2.4, we have

For the Ist type of soil, the curve number is 69.

For the IInd type of soil, the curve number is 58.

For the IIIrd type of soil, the curve number is 77.

Therefore, the weighted value of CN is

$$\text{CN} = 0.22 \times 69 + 0.55 \times 58 + 0.23 \times 77 = 65$$

From Table 2.5, we get

For AMC II corresponding to CN 65 the curve number under AMC III is 82.

For CN = 82 and rainfall P = 15.2 cm (6 inch), a value of Q = 4.2 inch (10.67 cm) is obtained as the direct runoff from Fig. 2.5.

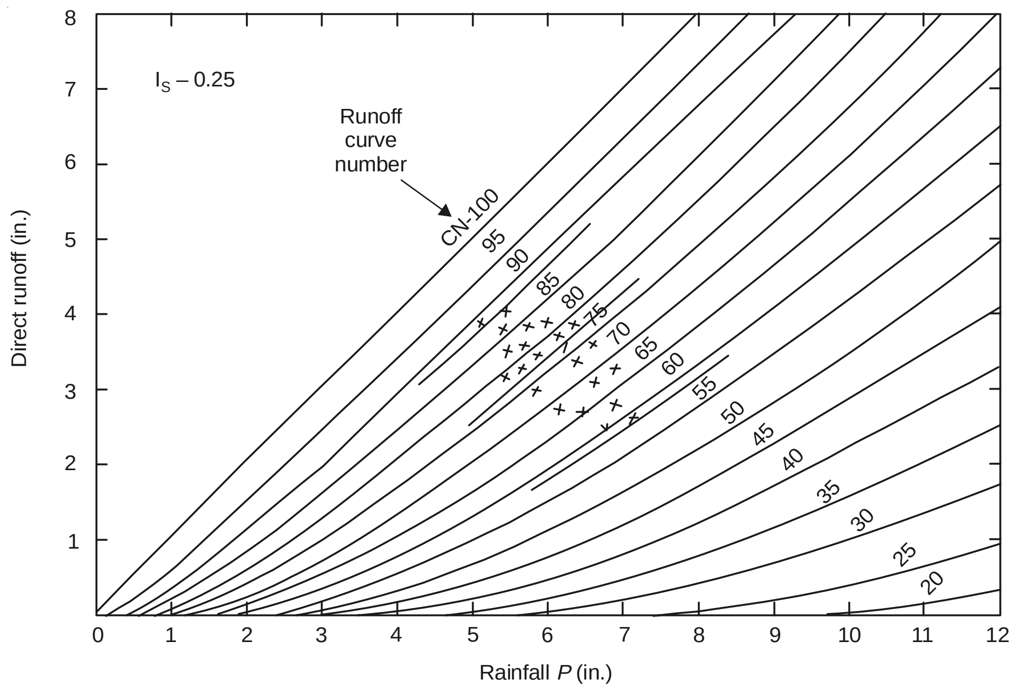

Fig. 2.5 Estimation of runoff curve numbers from measured data.

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Unit hydrograph: direct method

This technique is used for midsize catchments to develop a hydrograph for any given storm. Sherman [10] was the first to coin the concept of unit hydrograph. A unit hydrograph is defined as the hydrograph resulting from one unit (1 cm or 1 inch) of effective rainfall of a specified or unit duration. The unit duration varies from 1–24 h. However, for midsize catchments it varies from 1-6 h.

To develop a unit hydrograph (UH) by the direct method it is necessary to have a gauged catchment, i.e. a catchment equipped with rain gauges and a stream gauge facilily at the outlet, and adequate sets of the corresponding rainfall–runoff data. The rainfall–runoff data should be thoroughly examined to identify storms suitable for UH analysis. Strictly speaking, the storm should have a clearly defined duration, with no rainfall preceding or succeeding it. The selected storm should be of uniform effective rainfall intensity both temporarily and spatially. In practice, as the size of the catchment grows, the requirement of uniform spatial variation of rainfall decreases. The upper limit of applicability of the unit hydrograph is not very well defined. Sherman used it in connection with basins varying in size from 1300 to 8000 sq. km. Linsley et al. [11] have defined the upper limit as 5000 sq. km, to preserve accuracy. But as per Victor Miguel Ponce [9], for midsize catchments, the concept of unit hydrograph is applicable to basins ranging from 2.5 to 250 sq. km.

The following steps are followed to derive a unit hydrograph from the direct runoff hydrograph:

1. Select a hydrograph, which satisfies the requirements of the unit hydrograph.
2. Separate the measured hydrograph into direct runoff hydrograph (DRH) and base flow.
3. Draw a series of ordinates at some uniform time interval covering the entire hydrograph.
4. Compute the direct runoff volume by integrating the direct runoff hydrograph using the Simpson's or Trapezoidal formula as follows:

$$\Sigma\,(O \times t \times 3600)\ \text{cu. m}$$

   where ΣO is the sum of all the direct runoff ordinates in cumecs and t is the time interval in hours between the ordinates.
5. Divide the direct runoff volume by the drainage area (A) to obtain the equivalent depth of direct runoff, which is given by

$$\text{Direct runoff depth} = \frac{\Sigma(O \times t \times 3600) \times 100}{A \times 10^6}$$

$$= \frac{0.36\,\Sigma O \times t}{A}\ \text{cm}$$

   where A is the catchment area in sq. km.
6. Divide the ordinates of DRH by the equivalent depth of DRH to obtain the ordinates of UH.

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EXAMPLE 2.5

A unit hydrograph is to be developed for a 190.8 sq. km catchment with a 3-h rainfall that has produced the following data:

Time (h)	0	3	6	9	12	14	18	21	24
Flow (cumecs)	15	20	55	80	60	48	32	20	15

Based on this data develop a 3-h UH, assuming a base flow of 15 cumecs.

Solution

The calculations are presented in Table 2.6 whose columns 1 and 2 show the respective time and measured flow. Column 4 gives the ordinates of DRH obtained by subtracting base flow from the measured flow. The depth of direct runoff is

$$\frac{0.36 \times 210 \times 3}{190.8} = 1.19 \text{ cm}$$

The ordinates of UH (column 5) are obtained by dividing the ordinates of DRH by 1.19 cm.

Table 2.6 Development of the unit hydrograph for Example 2.5

Time (h)	*Ordinates of the given hydrograph* (cumecs)	*Base flow* (cumecs)	*Ordinates of the direct runoff hydrograph* (cumecs) (2) – (3)	*Ordinates of the unit hydrograph* (cumecs) (4)/l.19
(1)	(2)	(3)	(4)	(5)
0	15	15	0	0.0
3	20	15	5	4.20
6	55	15	40	33.61
9	80	15	65	54.62
12	60	15	45	37.81
15	48	15	33	27.73
18	32	15	17	14.28
21	20	15	5	4.20
24	15	15	0	0.0
			Σ210	Σ176.45

Check: Area under the unit hydrograph = $\frac{0.36 \times 176.45 \times 3}{190.8} = 1.0$ cm

Changing the duration of unit hydrograph

Sometimes it is necessary to change the duration of the given UH to another duration. Two methods are available to change the duration of a unit hydrograph.

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Superposition method

This method is used to convert a *t*-hour unit hydrograph into an *nt*-hour unit hydrograph, where *n* is an integer. The procedure consists of writing the ordinates of a *t*-hour lagged unit hydrograph, each for an interval equal to the *t*-hour. Summing the ordinates of both the *n* hydrographs at each time interval, and dividing the summed ordinates by *n*, the ordinates of the *nt*-hour unit hydrograph are obtained.

EXAMPLE 2.6

Use the superposition method to convert the ordinates of the 3-h unit hydrograph obtained in Example 2.5 to a 6-h unit hydrograph.

Solution

The calculations are given in Table 2.7. Column 1 shows the time in hours. Column 2 shows the ordinates of the 3-h unit hydrograph. Column 3 gives the ordinates of the 3-h lagged unit hydrograph. Column 4 shows the ordinates of the 6-h unit hydrograph, obtained by summing the respective ordinates of columns 2 and 3 and dividing each resulting sum by 2.

Table 2.7 Superposition method of changing the duration of unit hydrograph for Example 2.5

Time (h)	3-h *UH*	3-h *lagged UH*	6-h *UH* [(2) + (3)]/2
(1)	(2)	(3)	(4)
0	0	—	0.0
3	4.2	—	2.1
6	33.61	0	16.80
9	54.62	4.2	29.41
12	37.81	33.61	35.71
15	27.73	54.62	41.17
18	14.28	37.81	26.04
21	4.2	27.73	15.96
24	0	14.28	7.14
27	—	4.2	2.1
30	—	0	0

S-Hydrograph method (SH)

The S-hydrograph method is used to convert a *t*-h unit hydrograph into a t_1-h unit hydrograph, regardless of the ratio between *t* and t_1. The steps followed for derivation of such a hydrograph are:

1. Calculate the ordinates of the *t*-h S-hydrograph.The *t*-h S-hydrograph is obtained by accumulating the unit hydrograph ordinates at intervals equal to *t*.
2. Lag the *t*-h S-hydrograph by a time equal to t_1-h.
3. Subtract the ordinates of the lagged *t*-h S-hydrograph from those of the *t*-h S-hydrograph.
4. Multiply the resulting hydrograph ordinates by t/t_1 to obtain the ordinates of the t_1-h unit hydrograph.

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EXAMPLE 2.7

The ordinates of a 6-h UH are given below. Derive the ordinates of a 12-h UH by the S-curve technique.

Time (h)	0	6	12	18	24	30	36	40	48	54	60
6-h UH (cumecs)	0	5	13	30	35	32	20	14	8	4	0

Solution

The details of calculations are given in Tables 2.8(a) and 2.8(b).

Table 2.8(a) Derivation of 6-h S-hydrograph for Example 2.7

Time (h)	6-h *UH* (cumecs)	*S-hydrograph additions* (cumecs) (*unit storm after every* 6-h = t_r)	6-h *S-hydrograph ordinates* (cumecs)
0	0	— ...	0
6	5	0 ...	5
12	13	5 0 ...	18
18	30	13 5 0 ...	48
24	35	30 13 5 0 ...	83
30	32	35 30 13 5 0 ...	115
36	20	32 35 30 13 5 0 ...	135
42	14	20 32 35 30 13 5 0 ...	149
48	8	14 20 32 35 30 13 5 0 ...	157
54	4	8 14 20 32 35 30 13 5 0 ...	161
60	0	4 8 14 20 32 35 30 13 5 ...	161
66	0	4 8 14 20 32 35 30 13 5 ...	161

Table 2.8(b) Derivation of 12-h UH by S-hydrograph method for Example 2.7

Time (h)	6-h *S-hydrograph ordinates* (cumecs)	*Lagged* 6-h *S-hydrograph ordinates* (cumecs)	*S-hydrograph difference* (cumecs) (2) – (3)	*Ordinates of the* 12-h *UH* (cumecs) (4) × 6/12
(1)	(2)	(3)	(4)	(5)
0	0	—	—	0
6	5	0	5	2.5
12	18	5	13	6.5
18	48	18	30	15
24	83	48	35	17.5
30	115	83	32	16
36	135	115	20	10
42	149	135	14	7
48	157	149	8	4
54	161	157	4	2
60	161	161	0	0
66	161	161	0	0

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Convolution and composite hydrographs

This technique is based on the principle of linearity and superposition. The volume under the composite hydrograph is equal to the total volume of effective rainfall. The hydrograph convolution is used to develop a composite flood hydrograph based on the unit hydrograph and an effective storm hyetograph.

EXAMPLE 2.8

The following 3-h hydrograph has been derived for a certain catchment.

Time (h)	0	3	6	9	12	15	18	21	24
Flow (m^3/s)	0	10	20	30	25	20	15	10	0

A 9-hour storm covers the entire catchment and is distributed in time as follows:

Time (h)	0		3		6		9
Total rainfall (mm/h)		6		10		18	

Calculate the ordinates of the composite hydrograph using the convolution technique.

Solution

The details of computations are given in Table 2.9.

Table 2.9 Derivation of the composite unit hydrograph for Example 2.8 by convolution method

Time (h)	*UH*	0.6 × *UH*	1.0 × *UH*	1.8 × *UH*	*Composite hydrograph*
(1)	(2)	(3)	(4)	(5)	(6)
0	0	0	0	0	0.0
3	10	6	0	0	16.0
6	20	12	10	0	42.0
9	30	18	20	18	86.0
12	25	15	30	36	106.0
15	20	12	25	54	111.0
18	15	9'	20	45	89.0
21	10	6	15	36	67.0
24	0	0	10	27	37.0
27	—	—	—	18	18.0
31	—	—	—	0	0.0

Column (1) gives the ordinates of the unit hydrograph. Columns (3) through (5) represent unit hydrograph ordinates multiplied by precipitation that occurred at 3-h, 6-h and 9-h respectively. Column (6) represents the ordinates of the composite hydrograph in m^3/s.

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2.3 LARGE CATCHMENTS

2.3.1 Characteristics

In this type of catchments the runoff response is due to rainfall varying both in space and time. The temporal and spatial variation of rainfall and runoff demand the use of reservoir and channel routing (beyond the scope of this book). Also, the response of large catchments is 24 Design of Bridge Structures superconcentrated. In practice, large catchments are generally gauged and have long periods of records, hence the application of the principle of frequency analysis of floods is more appropriate here.

2.3.2 Frequency Analysis of Floods

Flood frequency analysis refers to the application of frequency analysis to the study of occurrences of floods. Many probability distributions have been used for this purpose.

Plotting positions

Frequency distributions are plotted on probability papers. One of the scales on a probability paper is the probability scale, the other is either the arithmetic or the logarithmic scale.

Arithmetic probability paper. This type of paper has a normal probability scale and an arithmetic scale and is used for plotting normal and Pearson distributions.

Log probability paper. This type of paper has a normal probability scale and a logarithmic scale and is used for plotting Lognormal and Pearson distributions.

Extreme value probability paper. This paper has an extreme value scale and an arithmetic scale and is used for plotting extreme value distributions like Gumbel distribution.

Several plotting positions formulae [12] are available and are given in Table 2.10.

Table 2.10 Plotting position formulae

Method	*Plotting position*	*Remark*
California	m/n	
Hazen	$(2m - 1)/2n$	In all the formulae, m is the rank of the series and n is the number of values in the series
Beard	$1 - 0.5^{1/n}$	
Weibull	$m/(n + 1)$	
Chegadayey	$(m - 0.3)/(n + 0.4)$	
Blom	$(m - 8)/(n + 0.25)$	
Turkey	$(3m - 1)/(3n + 1)$	

Curve fitting

Once the data have been plotted on a probability paper, the next step is to fit a curve through the plotted points. Curve fitting can be accomplished by any one of the following methods: (1) graphical, (2) least square, (3) moments, and (4) maximum likelihood.

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The graphical method consists of fitting a function visually to the data. This method has the disadvantage that the results are highly dependent on the person fitting the curve. In the least square method, the sum of the squares of the differences between the observed data and the fitted values is minimized. In the case of the method of moments, first the distribution is selected and then the moments of the distribution are calculated. This method provides an exact theoretical fitting provided the chosen probability distribution is correct (Example: Log Pearson and Gumbel distribution). In the method of maximum likelihood, the distribution parameters are estimated in such a way that the product of probabilities is maximized.

Normal distribution

The normal distribution is a symmetrical bell-shaped frequency function, also known as Gaussian distribution or the natural law of errors. Although it often does not perfectly fit the sequence of hydrologic data, it has wide applications. The normal distribution was first used by Horton [13], and shortly thereafter by Fuller [14].

EXAMPLE 2.9

The following are the discharges measured at a stream gauging station:

Year	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
Annual flow (cumecs)	8	10	12	8	7	6	14	21	11	8	17	7	22	15	22	17	10	17	5	25	19	14	20

Calculate the statistical mean, variance, standard deviation, coefficient of variance and skewness factor.

Solution

The details of calculations are shown in Table 2.11.

Table 2.11 Computation of statistical parameters for Example 2.9

Year	*Annual flow* (x_i) (cumecs)	$(x_i - \overline{x})$	$(x_i - \overline{x})^2$	$(x_i - \overline{x})^3$
1	8	–5.7	32.4	–185.193
2	10	–3.7	13.69	–50.653
3	12	–1.7	2.89	–4.913
4	8	–5.7	32.49	–185.193
5	7	–6.7	44.89	–300.763
6	6	–7.7	59.29	–456.533
7	14	0.3	0.09	0.027
8	21	7.3	53.29	389.017
9	11	–2.7	7.29	–19.683
10	8	–5.7	32.49	–185.193
11	17	3.3	10.89	35.937
12	7	–6.7	44.89	–300.763
13	22	8.3	68.89	571.787
14	15	1.3	1.69	2.197
15	22	8.3	68.891	571.787
16	17	3.3	10.89	35.937
17	10	–3.7	13.69	–50.653

(*Contd.*)

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Table 2.11 Computation of statistical parameters for Example 2.9 (*Contd.*)

Year	*Annual flow* (x_i) (cumecs)	$(x_i - \bar{x})$	$(x_i - \bar{x})^2$	$(x_i - \bar{x})^3$
18	17	3.3	10.89	35.937
19	5	–8.7	75.69	–658.503
20	25	11.3	127.69	1442.897
21	19	5.3	28.09	148.877
22	14	0.3	0.09	0.027
23	20	6.3	39.69	250.047
SUM	315	–0.10	780.78	1085.931

Mean $\bar{x} = \Sigma x_i/n = 13.70$ cumecs
Variance $S^2 = [\Sigma(x_i - \bar{x})^2]/(n - 1) = 35.49$
Standard deviation $S = 5.95$ cumecs
Coefficient of variation $C_v = S/\bar{x} = 0.434$
Skewness $a = n[\Sigma(x_i - \bar{x})^3]/(n - 1)(n - 2) = 54.06$
Skewness coefficient $C_s = a/S^3 = 0.256$

EXAMPLE 2.10

For the annual flows given in Example 2.9, fit the normal distribution curve.

Solution

To fit the normal distribution, Weibull formula is used. The details of computations are given in Table 2.12.

Table 2.12 Normal distribution curve fit for Example 2.10

Year	*Annual flow* (m^3/s)	*Ranked flow* (m^3/s)	*Rank* (m)	*% Probability* $p = m/(n + 1)$	*Return period* $T = 1/p$
1	8	25	1	4.16	24
2	10	22	2	8.30	12
3	12	22	3	12.50	8
4	8	21	4	16.60	6
5	7	20	5	20.80	4.8
6	6	19	6	25.00	4
7	14	17	7	29.16	3.42
8	21	17	8	33.33	3
9	11	17	9	37.50	2.6
10	8	15	10	41.66	2.4
11	17	14	11	45.80	2.18
12	7	14	12	50.00	2
13	22	12	13	54.16	1.84
14	15	11	14	58.33	1.71
15	22	10	15	62.50	1.6
16	17	10	16	66.60	1.5
17	10	8	17	70.83	1.41
18	17	8	18	75.00	1.33
19	5	8	19	79.16	1.26
20	25	7	20	83.33	1.20
21	19	7	21	87.50	1.14
22	14	6	22	91.66	1.09
23	20	5	23	95.83	1.04

From Table 2.11, the mean and standard deviation are respectively 13.70 m^3/s and 5.95 m^3/s. The theoretical normal best fit is a straight line through $(\bar{x} - S)$ at 15.9%, $\bar{x}$ at 50% and $(\bar{x} + S)$ at 84.1%. Thus, we have

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Plotting positions (right hand scale)

$(\bar{x} - S) = 13.70 - 5.95 = 7.75$	15.9%
$\bar{x} = 13.70$	50%
$(\bar{x} + S) = 13.70 + 5.95 = 19.65$	84.10%

Lognormal distribution

Many hydrological events exhibit a marked right skewness, parily because of the influence of natural phenomena having values greater than zero, or some other lower limit, and being unconstrained, theoretically in the upper range. In such cases, frequencies will not follow the normal distribution, but fortunately, the variables often are functionally not normal and natural logarithms follow a normal distribution [15].

The frequency curve is derived from the following steps:

1. Compute the mean $\bar{x}$ and standard deviation S from the given values of flood flows.
2. Compute the coefficient of variation C_v.
3. Compute the coefficient of skew C_s as equal to $3C_v + C_v^3$.
4. For a given value of $\bar{x}$, C_s and known probability p, obtain the value of k (frequency factor) from Table 2.13.
5. Compute the value of flood flow as $x = \bar{x} + ks$ for each of the probabilities.

Table 2.13 Chow frequency factors (k)

Coefficient of skew C_s	*Proba-bility* p	*Probability in percentage equal to or greater than the given variate*									*Coefficient of variation* C_v
		99 –	95 –	80 –	50 –	20 +	5 +	1 +	0.1 +	0.01 +	
0	50.0	2.33	1.65	0.84	0	0.84	1.64	2.33	3.09	3.72	0
0.1	49.3	2.25	1.62	0.85	0.02	0.84	1.67	2.40	3.22	3.95	0.033
0.2	48.7	2.18	1.59	0.85	0.04	0.83	1.70	2.47	3.39	4.18	0.067
0.3	48.0	2.11	1.56	0.85	0.06	0.82	1.72	2.55	3.56	4.42	0.100
0.4	47.3	2.04	1.53	0.85	0.07	0.81	1.75	2.62	3.72	4.70	0.136
0.5	46.7	1.98	1.49	0.85	0.09	0.80	1.77	2.70	3.88	4.96	0.166
0.6	46.1	1.91	1.46	0.85	0.10	0.79	1.79	2.77	4.05	5.24	0.197
0.7	45.5	1.85	1.43	0.85	0.11	0.78	1.81	2.84	4.21	5.52	0.230
0.8	44.9	1.79	1.40	0.84	0.13	0.77	1.82	2.90	4.37	5.81	0.262
0.9	44.2	1.74	1.37	0.84	0.14	0.76	1.84	2.97	4.55	6.11	0.292
1.0	43.7	1.68	1.34	0.84	0.15	0.75	1.85	3.03	4.72	6.40	0.324
1.1	43.2	1.63	1.31	0.83	0.16	0.73	1.86	3.09	4.87	6.71	0.351
1.2	42.7	1.58	1.29	0.82	0.17	0.72	1.87	3.15	5.04	7.02	0.381
1.3	42.2	1.54	1.26	0.82	0.18	0.71	1.88	3.21	5.19	7.31	0.409
1.4	41.7	1.49	1.23	0.81	0.19	0.69	1.88	3.26	5.35	7.62	0.436

(*Contd.*)

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Table 2.13 Chow frequency factors (k) (*contd.*)

Coefficient of skew C_s	*Proba-bility* p	*Probability in percentage equal to or greater than the given variate*									*Coefficient of variation* C_v
		99 –	95 –	80 –	50 –	20 +	5 +	1 +	0.1 +	0.01 +	
1.5	41.3	1.45	1.21	0.81	0.20	0.68	1.89	3.31	5.51	7.92	0.462
1.6	40.8	1.41	1.18	0.80	0.21	0.67	1.89	3.36	5.66	8.26	0.490
1.7	40.4	1.38	1.16	0.79	0.21	0.65	1.89	3.40	5.80	8.58	0.597
1.8	40.0	1.34	1.14	0.78	0.22	0.64	1.89	3.44	5.96	8.88	0.544
1.9	39.6	1.31	1.12	0.78	0.22	0.63	1.89	3.48	6.10	9.20	0.570
2.0	39.2	1.28	1.10	0.77	0.23	0.61	1.89	3.52	6.25	9.51	0.596
2.1	38.8	1.25	1.08	0.76	0.24	0.60	1.89	3.55	6.39	9.79	0.620
2.2	38.4	1.22	1.06	0.76	0.24	0.59	1.88	3.59	6.51	10.12	0.643
2.3	38.1	1.20	1.04	0.75	0.25	0.58	1.88	3.62	6.65	10.43	0.667
2.4	37.7	1.17	1.02	0.74	0.25	0.57	1.88	3.65	6.77	10.72	0.691
2.5	37.4	1.15	1.00	0.74	0.26	0.56	1.88	3.67	6.90	10.95	0.713
2.6	37.1	1.12	0.99	0.73	0.26	0.55	1.87	3.70	7.02	11.25	0.734
2.7	36.8	1.10	0.97	0.72	0.26	0.54	1.87	3.72	7.13	11.55	0.755
2.8	36.6	1.08	0.96	0.72	0.27	0.53	1.86	3.74	7.25	11.80	0.776
2.9	36.3	1.06	0.95	0.71	0.27	0.52	1.86	3.76	7.36	12.10	0.796
3.0	36.0	1.04	0.93	0.71	0.27	0.51	1.85	3.78	7.47	12.36	0.818
3.2	35.5	1.01	0.90	0.69	0.28	0.49	1.84	3.81	7.65	12.85	0.857
3.4	35.1	0.98	0.88	0.68	0.28	0.47	1.83	3.84	7.84	13.36	0.895
3.6	34.7	0.95	0.86	0.67	0.29	0.46	1.81	3.87	8.00	13.83	0.930
3.8	34.2	0.92	0.84	0.66	0.29	0.44	1.80	3.89	8.16	14.23	0.966
4.0	33.9	0.90	0.82	0.65	0.29	0.42	1.78	3.91	8.30	14.70	1.000
4.5	33.0	0.84	0.78	0.63	0.29	0.39	1.75	3.93	8.60	15.62	1.081
5.0	32.3	0.80	0.74	0.62	0.30	0.37	1.71	3.95	8.86	16.45	1.155

EXAMPLE 2.11

Carry out the flood frequency analysis for the data given below by the method of lognormal distribution.

Year	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Flood (m^3/s)	40	92	134	147	119	60	80	101	204	54	71	90	57	71	108	99	34

Solution

The computation of statistical parameters is presented in Table 2.14.

Table 2.14 Computation of statistical parameters for Example 2.11

Year	*Flood flow* (m^3/s)	$(x_i - \bar{x})$	$(x_i - \bar{x})^2/100$	$(x_i - \bar{x})^3/1000$
1	40	–51.82	26.85	–39.152
2	92	0.18	0.000	0.000
3	134	42.18	17.79	75.04
4	147	55.18	30.44	168.01
5	119	27.18	7.38	20.08
6	60	–31.82	10.12	–32.21
7	80	–11.82	1.39	–1.65
8	101	9.18	0.842	+0.773
9	204	112.18	125.84	1411.71
10	54	–37.82	14.34	–54.09
11	71	–20.82	4.33	–9.02
12	90	–1.82	0.034	6.02
13	57	–34.82	12.12	–42.21
14	71	–20.82	4.33	–9.02
15	108	+16.18	2.61	4.23
16	99	7.18	0.515	0.370
17	34	–57.82	33.43	–93.30

Mean $\bar{x} = 91.82$ m^3/s

Variance $S^2 = 1829$

Standard deviation $S = 42.76$ m^3/s

Coefficient of variance $C_v = 0.46$

Skewness $a = 84.54 \times 10^3$

Skewness coefficient $C_v = 3C_v + c_v^3 = 1.48$

The lognormal fit calculations are shown in Table 2.15

Table 2.15 Lognormal distribution fit calculations for Example 2.11
$S = 42.76$ m^3/s, $\bar{x} = 91.82$ m^3/s

Probability	*k for* $C_s = 1.48$ *from Table* 2.13	*ks*	$x = \bar{x} + ks$
99	–1.4516	–62.04	29.78
95	–1.256	–53.68	38.14
80	–0.819	–35.00	56.82
50	–0.180	–7.70	91.82
20	0.690	29.50	121.32
5	1.890	80.77	172.52
1	3.330	141.04	232.86
0.1	5.348	228.57	320.39
0.01	7.623	325.80	417.62

Pearson distribution

This distribution has been widely adopted as a standard method for flood frequency analysis in a form known as log Pearson III in which the transformation $y = \log x$ is used to reduce skewness [16, 17]. Although three moments are required to fit the distribution, it is extremely flexible in that a zero skew will reduce the log Pearson III distribution to a lognormal and Pearson III to normal. Tables of cumulative functions are available (see Table 2.16).

Table 2.16 Frequency factors k for Pearson III distribution

	Return period T (y)									
	1.05	1.11	1.25	2	5	10	25	50	100	200
	Probability of exceedence p (per cent)									
C_s	95	90	80	50	20	10	4	2	1	0.5
3.0	–0.665	–0.660	–0.636	–0.396	0.420	1.180	2.278	3.152	4.051	4.970
2.8	–0.711	–0.702	–0.666	–0.384	0.460	1.210	2.275	3.114	3.973	4.847
2.6	–0.762	–0.747	–0.696	–0.368	0.499	1.238	2.267	3.071	3.889	4.718
2.4	–0.819	–0.795	–0.725	–0.351	0.537	1.262	2.256	3.023	3.800	4.584
2.2	–0.882	–0.844	–0.752	–0.330	0.574	1.284	2.40	2.970	3.075	4.444
2.0	–0.949	–0.895	–0.777	–0.307	0.609	1.302	2.219	2.912	3.605	4.398
1.8	–1.020	–0.945	–0.799	–0.282	0.643	1.318	2.193	2.848	3.499	4.147
1.6	–1.093	–0.994	–0.817	–0.254	0.675	1.329	2.163	2.780	3.388	3.990
1.4	–1.168	–1.041	–0.832	–0.225	0.705	1.337	2.128	2.706	3.271	3.828
1.2	–1.243	–1.086	–0.844	–0.195	0.732	1.340	2.087	2.626	3.149	3.661
1.0	–1.317	–1.128	–0.852	–0.164	0.758	1.340	2.043	2.542	3.022	3.489
0.8	–1.388	–1.166	–0.856	–0.132	0.780	1.336	1.993	2.453	2.891	3.312
0.6	–1.458	–1.200	–0.857	–0.099	0.800	1.328	1.939	2.359	2.755	3.132
0.4	–1.524	–1.231	–0.855	–0.066	0.816	1.317	1.880	2.261	2.615	2.949
0.2	–1.586	–1.258	–0.850	–0.033	0.830	1.301	1.818	2.159	2.472	2.763
0.0	–1.645	–1.282	–0.842	0.000	0.842	1.282	1.751	2.054	2.326	2.576
–0.2	–1.700	–1.301	–0.830	0.033	0.850	1.258	1.680	1.945	2.178	2.388
–0.4	–1.750	–1.317	–0.816	0.066	0.855	1.231	1.606	1.834	2.029	2.201
–0.6	–1.797	–1.328	–0.800	0.099	0.857	1.200	1.528	1.720	1.880	2.016
–0.8	–1.839	–1.336	–0.780	0.132	0.856	1.166	1.448	1.606	1.733	1.837
–1.0	–1.877	–1.340	–0.758	0.164	0.852	1.128	1.366	1.492	1.588	1.664
–1.2	–1.910	–1.340	–0.732	0.195	0.844	1.086	1.282	1.379	1.449	1.501
–1.4	–1.938	–1.337	–0.705	0.225	0.832	1.041	1.198	1.270	1.318	1.351
–1.6	–1.962	–1.329	–0.675	0.254	0.817	0.994	1.116	1.166	1.197	1.216
–1.8	–1.981	–1.318	0.643	0.282	0.799	0.945	1.305	1.069	1.087	1.097
–2.0	–1.996	–1.302	–0.609	0.307	0.777	0.895	0.959	0.980	0.990	0.995
–2.2	–2.006	–1.284	–0.574	0.330	0.752	0.844	0.888	0.900	0.905	0.907
–2.4	–2.011	–1.262	–0.537	0.351	0.725	0.795	0.823	0.830	0.832	0.833
–2.6	–2.013	–1.238	0.499	0.368	0.696	0.747	0.764	0.768	0.769	0.769
–2.8	–2.010	–1.210	–0.460	0.384	0.666	0.702	0.712	0.714	0.714	0.714
–3.0	–2.003	1.180	–0.420	0.383	0.836	0.660	0.666	0.666	0.667	0.667

To fit log Pearson III distribution, the following steps are necessary:

1. Assemble the annual flood series x_i
2. Calculate the logarithms of the annual flood series as

$$y_i = \log x_i$$

3. Calculate the mean $\overline{y}$, standard deviation S_y and skew coefficient C_{sy} of the logarithm y_i.
4. Calculate the logarithms of the flood discharges, i.e. log Q_i, for each of the several chosen probability levels p_j, using the following frequency formula:

$$\log Q_j = \overline{y} + k_j \times S_y$$

   where k_j is the frequency factor, a function of the probability p_j and skewness coefficient C_{sy}.

   Table 2.16 shows the frequency factor k for ten selected probability levels in the range from 0.5 to 95% and skewness coefficients in the range from –3.0 to 3.0.
5. Calculate the flood discharge Q_j for each p_i probability level (return period T_j) by taking antilogarithms of the log Q_j values.
6. Plot the flood discharge Q_j versus the probability p_j on a log probability paper with discharges in the log scale and probabilities in the probability scale.

EXAMPLE 2.12

For the data given in Example 2.11, fit the log Pearson III curve.

Solution

The details of computations are given in Tables 2.17(a) and 2.17(b).

Table 2.17(a) Computation of statistical parameters for Example 2.12

Flood flows (x) (m^3/s)	$y = \log(x)$	$(y - \overline{y})$	$(y - \overline{y})^2$	$(y - \overline{y})^3$
40	1.602	–0.3177	0.1009	–0.0320
92	1.963	0.0439	0.0000	0.00008
134	2.127	0.2073	0.0429	0.00890
147	2.163	0.2475	0.0612	0.01510
119	2.075	0.1557	0.0242	0.00370
60	1.778	–0.1416	0.0200	–0.0028
80	1.903	–0.0167	0.0002	–0.0000
101	2.004	0.0845	0.0071	0.00060
204	2.309	–0.3898	0.1519	0.05920
54	1.732	–0.1874	0.0351	–0.0065
71	1.851	–0.0685	0.0046	–0.00032
90	1.954	0.0344	0.0011	0.00004
57	1.755	–0.1639	0.0268	–0.0044
71	1.852	–0.0685	0.0046	0.00032
100	2.000	0.1136	0.0129	0.00140
99	1.995	0.0758	0.0057	0.00043
34	1.531	0.3883	0.1507	–0.0585

Mean $\overline{y}$ = 1.919 | Coefficient of variation C_{vy} = 0.105
Standard deviation S_y = 0.202 | Coefficient of skewness C_{sy} = –0.126

Table 2.17(b) Log Pearson III method for Example 2.12

Return period T (years)	*Probability p (%)*	*Frequency factor k* (C_{sy} = –0.126)	y_i = log *Q* $y_i = (\bar{y} + k \times S_y)$	$x_i = Q$ (m^3/s)
1.05	95	– 1.674	1.580	38
1.11	90	– 1.292	1.658	45
1.25	80	– 0.846	1.748	56
2	50	0.0155	1.915	83
5	20	0.846	2.089	123
10	10	1.270	2.175	150
25	4	1.717	2.265	183
50	2	1.202	2.323	209
100	1	2.256	2.374	234
200	0.5	2.564	2.436	260

2.3.3 Gumbel Method

Gumbel [18] was the first to realize that the annual maximum series (minimum series) are nothing but the extreme values in different years of observations and they should, therefore, follow the extreme value distribution. The following steps are needed to apply this method:

1. Assemble the flood series.
2. Calculate the mean $\bar{x}$ and standard deviation *S* of the flood series.
3. Use Table 2.18 to determine the mean $\bar{y}_n$ and standard deviation σ_n of the Gumbel variate *y* as a function of the record length ***n*.**
4. Select several return periods T_j and the corresponding exceedence probabilities p_j.
5. Calculate the Gumbel variate y_j corresponding to the periods T_j using the equation,

$$y = -\ln \ln T/(T - 1)$$

and calculate the flood discharge $Q_j = x_j$ for each Gumbel variate using the equation,

$$x = \bar{x} + (y - \bar{y}_n)\, S/\sigma_n$$

The values of *Q* are plotted versus *y* or *T* or *p* on Gumbel probability paper and a straight line is fitted through these points. Gumbel probability paper has an arithmetic scale of Gumbel variate *y* as the abscissa and an arithmetic scale of discharge *Q* as the ordinate.

Table 2.18 Mean $\bar{y}_n$ and standard deviation σ_n of Gumbel variate (*y*)

n	$\bar{y}_n$	σ_n	*n*	$\bar{y}_n$	σ_n	*n*	$\bar{y}_n$	σ_n
8	0.4843	0.9043	14	0.5100	1.0095	20	0.5236	1.00628
9	0.4902	0.9288	15	0.5128	1.0206	21	0.5252	1.0696
10	0.4952	0.9497	16	0.5157	1.0316	22	0.5268	1.0754
11	0.4996	0.9676	17	0.5181	1.0411	23	0.5283	1.0811
12	0.5035	0.9833	18	0.5202	1.0493	24	0.5296	1.0864
13	0.5070	0.9972	19	0.5220	1.0566	25	0.5320	1.0915

(*Contd.*)

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Table 2.18 Mean $\bar{y}_n$ and standard deviation σ_n of Gumbel variate (y) (*Contd.*)

n	$\bar{y}_n$	σ_n	n	$\bar{y}_n$	σ_n	n	$\bar{y}_n$	σ_n
26	0.5320	1.0961	47	0.5473	1.1557	76	0.5561	1.1906
27	0.5332	1.1004	48	0.5477	1.1574	78	0.5565	1.1923
28	0.5343	1.1047	49	0.5481	1.1590	80	0.5569	1.1938
29	0.5353	1.1086	50	0.5485	1.1607	82	0.5572	1.1953
30	0.5362	1.1124	51	0.5489	1.1623	84	0.5576	1.1967
31	0.5371	1.1159	52	0.5493	1.1638	86	0.5580	1.1980
32	0.5380	1.1193	53	0.5497	1.1653	86	0.5580	1.1980
33	0.5388	1.1226	54	0.5501	1.1667	90	0.5586	1.2007
34	0.5396	1.1255	55	0.5504	1.1681	92	0.5589	1.2020
35	0.5403	1.1285	56	0.5508	1.1696	94	0.5592	1.2032
36	0.5410	1.1313	57	0.5511	1.1708	96	0.5595	1.2044
37	0.5418	1.1339	58	0.5515	1.1721	98	0.5598	1.2005
38	0.5424	1.1363	59	0.5518	1.1734	100	0.5600	1.2065
39	0.5430	1.1388	60	0.5521	1.1747	150	0.5646	1.2253
40	0.5436	1.1413	62	0.5527	1.1770	200	0.5672	1.2360
41	0.5442	1.1436	64	0.5533	1.1793	250	0.5688	1.2429
42	0.5448	1.1458	66	0.5538	1.1814	300	0.5699	1.2479
43	0.5453	1.1480	68	0.5543	1.1834	400	0.5714	1.2545
44	0.5458	1.1499	70	0.5548	1.1854	500	0.5724	1.2588
45	0.5463	1.1519	72	0.5552	1.1873	750	0.5738	1.2651
46	0.5468	1.1538	74	0.5557	1.1890	1000	0.5745	1.2685

EXAMPLE 2.13

For the data given in Example 2.10, fit the Gumbel curve.

Solution

From Table 2.14 the mean and standard deviation for the given flood flows are

$$\bar{x} = 91.82 \text{ m}^3/\text{s}$$

$$S = 42.76 \text{ m}^3/\text{s}$$

From Table 2.18 for n = 17, the mean and standard deviation of the Gumbel variate are

$$\bar{y}_n = 0.5181$$

$$\sigma_n = 1.0411$$

The results are tabulated in Table 2.19. Columns 1 and 2 show the selected return periods and the corresponding probabilities. Column 3 shows the values of the Gumbel variate calculated from the equation

$$y = -\ln \ln T/(T - 1)$$

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Column 4 shows the flood discharges Q calculated from the equation

$$x = \bar{x} + [\ln \ln T/(T - 1) - y_n]S/\sigma_n$$

The peak flow follows a straight line when plotted on the Gumbel probability paper.

Table 2.19 Gumbel probability fit for Example 2.13

Return period T (years)	*% probability p*	*Gumbel variate y*	*Peak flow Q* (m^3/s)
1.05	95	–1.113	25
1.10	90	–0.838	37
1.25	80	–0.476	51
2	50	0.367	86
5	20	1.500	133
10	10	2.250	163
25	4	3.199	202
50	2	3.902	231
100	1	4.600	260
200	0.5	5.296	288

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Chapter 3

River Channels

3.1 DETERMINATION OF PEAK DISCHARGE

During floods, because of high stages and swift currents, it is not possible to measure the discharges in river channels. The peak flow discharge can be estimated by an indirect method, with the aid of open channel flow formulae, called the *slope-area* method.

For application of the slope-area method, the following data [19] are needed:

1. The length of the reach
2. The difference in water surface elevations through the reach (the fall)
3. The flow area, the wetted perimeter of the cross-section, and the velocity head coefficients at upstream and downstream sections
4. The average value of Manning coefficient *n* for the reach.

3.1.1 Selection of Reach

The following guidelines are prescribed for selection of a suitable reach:

1. The high flood level mark should be easily recognizable.
2. For accurate measurement of the fall, the length of the reach should be sufficiently long.
3. The reach should be relatively straight and stable.
4. The converging reach is preferred over a diverging one.
5. Bridges, channel bends, waterfalls and other features causing non-uniformity of flow should be avoided.

An ideal site should satisfy one or more of the following criteria to improve the accuracy of measurement:

1. The ratio of the length of the reach to hydraulic depth should be greater than 75.

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2. The fall should be more than or equal to 0.15 m.
3. The fall should be greater than either of the velocity heads computed at upstream and downstream sections [20].

The computation of maximum flood flow consists of the following steps:

1. Calculate conveyance both at upstream and downstream sections as follows:

$$K_u = \frac{A_u R_u^{2/3}}{n}$$
$$K_d = \frac{A_d R_d^{2/3}}{n} \tag{3.1}$$

   where
   K is the conveyance
   A is the area of flow
   R is the hydraulic mean radius
   n is Manning coefficient for the reach
   the superscripts u and d denote upstream and downstream respectively.

2. Calculate the reach conveyance, equal to the geometric mean of the upstream and downstream conveyances. That is,

$$K = (K_u K_d)^{1/2} \tag{3.2}$$

   where K is the reach conveyance.

3. Calculate the first approximation to the energy slope:

$$S = \frac{F}{L} \tag{3.3}$$

   where
   F is the fall
   L is the length of the reach
   S is the first approximation to the energy slope.

4. Calculate the first approximation to the peak discharge:

$$Q_i = KS^{1/2} \tag{3.4}$$

   where Q_i is the first approximation to the peak flow.

5. Calculate the velocity heads as follows:

$$h_{vu} = \frac{\alpha_u (Q_i/A_u)^2}{2g}$$
$$h_{vd} = \frac{\alpha_d (Q_i/A_d)^2}{2g} \tag{3.5}$$

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where

h_{vu} and h_{vd} are the velocity heads at upstream and downstream sections, respectively
α_u and α_d are the velocity head coefficients at upstream and downstream sections, respectively g is the acceleration due to gravity.

6. Calculate an updated value of the energy slope:

$$S_i = \frac{F + f(h_{vu} - h_{vd})}{L} \tag{3.6}$$

where

S_i is the updated value of the energy slope
f is the loss coefficient
f = 0.5 for diverging flow ($A_d > A_u$)
f = 1.0 for converging flow ($A_d < A_u$)

7. Calculate an updated value of peak discharge:

$$Q_i = KS_i^{1/2} \tag{3.7}$$

8. By using the above updated values repeat Steps 5 to 7 till the difference between two successive values of peak discharge obtained from Step 7 is negligible.

EXAMPLE 3.1

A bridge is to be constructed across a river. By using the following data, calculate the peak discharge.

Length of the reach = 1250 m, Fall = 0.35 m, Manning coefficient n = 0.028.

	Upstream	*Downstream*
Flow area (m^2)	3522	3259
Wetted perimeter (m)	650	621
Velocity head coefficient	1.17	1.21

Solution

The hydraulic mean radius at upstream

$$R_u = \frac{A_u}{P_u} = \frac{3522}{650} = 5.41 \text{ m}$$

The conveyance at upstream

$$K_u = \frac{A_u R_u^{2/3}}{n} = \frac{(3522)(5.41)^{2/3}}{0.028} = 387,204 \text{ m}^3\text{/s}$$

The hydraulic mean radius at downstream

$$R_d = \frac{A_d}{P_d} = \frac{3259}{621} = 5.24 \text{ m}$$

The conveyance at downstream

$$K_d = \frac{A_d R_d^{2/3}}{n} = \frac{(3259)(5.24)^{2/3}}{0.028} 350,752 \text{ m}^3/\text{s}$$

The first approximation to energy slope

$$S = \frac{F}{L} = \frac{0.35}{1250} = 0.00028$$

The first approximation to peak flow using Eq. (3.4)

$$Q_1 = 6166 \text{ m}^3/\text{s}$$

The velocity heads at upstream and downstream using Eq. (3.5) are

$$h_{vu} = 0.183$$

$$h_{vd} = 0.221$$

$$\text{since } A_u > A_d, f = 1.0$$

The first updated value of energy slope using Eq. (3.6) is

$$S_1 = 0.0002496$$

The second updated value of peak flow using Eq. (3.7) is

$$Q_2 = 5822 \text{ m}^3/\text{s}$$

The remaining calculations are shown in Table 3.1.

Table 3.1 Computation of peak discharge by slope-area method for Example 3.1

Iteration no.	h_{vn} (m)	h_{vd} (m)	*Energy slope*	*Peak discharge* (m³/s)
1			0.00028	6166
2	0.1830	0.221	0.0002496	5822
3	0.1631	0.1970	0.0002520	5863
4	0.1652	0.1995	0.0002520	5860
5	0.1650	0.1993	0.0002523	5860

3.2 HYDRAULIC GEOMETRY

The factors which affect the geometry of river channels are: (1) the discharge, (2) the characteristics of the bed and bank materials, and (3) the sediment transport capacity of channels. No single approach is available to calculate channel geometry applicable to all types of rivers. Many empirical and semi-theoretical formulae are in use, each of which is restricted to channels of particular type. The equations given below are only guidelines to compute channel geometry, as variations in channel slope and sediment transport load may significantly affect the width and depth of flow.

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3.2.1 Sand Bed Channels

For sand bed channels, according to Blench [21], we have the following equations:

$$B = 14\, Q^{0.5}\, D_{50}^{0.25}\, F_s^{-0.5} \tag{3.8}$$

$$y = 0.38\, q^{0.67}\, D_{50}^{-0.17} \tag{3.9}$$

where

B is the mean channel width in m
y is the mean depth of flow in m
Q is the discharge in cumecs
q is the discharge per metre width
D_{50} is the medium size of the bed material in m
F_s is the bank toughness
F_s = 0.1 for sandy loam
= 0.2 for silty clay loam
= 0.3 for cohesive banks.

3.2.2 Gravel Bed Channels

For gravel bed channels, Kellerhals [22] arrived at the following equations:

$$B = 3.26\, Q^{0.5} \tag{3.10}$$

$$y = 0.47\, q^{0.8}\, D_{90}^{-0.12} \tag{3.11}$$

where B, y, Q and q are defined as above and D_{90} is the srze of the bed material in metres, such that 90% of the stones by number are smaller.

3.2.3 Cohesive Bed Channels

The resistance to scour of cohesive materials is more complex than that of cohesionless materials. The only fairly reliable method of estimating scour is to measure soil properties and carry out model tests in the laboratory [23]. The depth of flow in a channel may be calculated assuming that scour continues to occur till the tractive stress approaches the critical value. Thus

$$y = 51.4\, n^{0.86}\, q^{0.86}\, \tau_c^{-0.43} \tag{3.12}$$

where

y = the mean depth of flow in m
n = Manning coefficient
q = discharge per unit width
τ_c = the tractive stress for scour to occur, in N/m^2

The values of τ_c are tabulated in Table 3.2.

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Table 3.2 Critical tractive stress (τ_c)

Type of soil	*Critical tractive stress* (N/m^2)			
	Void ratio			
	2–1.2	1.2–0.6	0.6–0.3	0.3–0.2
Sandy clay	1.9	7.5	15.7	30.2
Heavy clay	1.5	6.7	14.6	27.0
Clay	1.2	5.9	13.5	25.4
Lean clay	1.0	4.6	10.2	16.8

Note: The specific gravity of the material is assumed as 2.64.

3.3 EFFECT OF BRIDGE ON RIVER REGIME

The construction of a bridge across a river channel or flood plain affects both the flow pattern and flow intensity, which may lead to a local geometry change and a new relationship between water and discharge.

Abutments and piers reduce the waterway, increase the discharge, scour and head loss through the bridge openings. Protective works in the flood plain (wing walls, returns, river training works, etc.) interfere with natural drainage and divert the flow from the flood plain to adjacent lands.

3.4 LINEAR WATERWAYS

3.4.1 Streams with Rigid Boundaries

When the banks and the beds of a stream are very rigid, the waterway of the bridge should be made equal to the width of the water surface measured from edge to edge along the design high flood level on the plotted section. However, a certain reduction in the waterway may be possible provided the velocity under the bridge is not severe, thus resulting in tangible savings in the cost of construction of the bridge.

3.4.2 Quasi-Alluvial Streams

In this type of rivers, the waterway should be made equal to the width of the water surface measured from edge to edge along the design high flood level.

3.4.3 Alluvial Streams

The linear waterway of a bridge across a fully alluvial stream should be kept equal to the regime width as given by Lacey.

For regime conditions, Lacey arrived at the following equations:

Regime cross-section

$$P = 4.8Q^{0.5} \tag{3.13}$$

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$$R = 0.473\left(\frac{Q}{f}\right)^{1/3} \tag{3.14}$$

$$A = 2.3\frac{Q^{5/6}}{F^{1/3}} \tag{3.15}$$

Regime velocity and slope

$$v = 0.44Q^{1/6}f^{1/3} \tag{3.16}$$

$$S = 0.0003\ \frac{f^{5/4}}{Q^{1/6}} \tag{3.17}$$

Regime width and depth

$$w = 4.8\sqrt{Q} \tag{3.18}$$

$$d = 0.473\left(\frac{Q}{f}\right)^{1/3} \tag{3.19}$$

Silt factor

$$f = 1.76\sqrt{m} \tag{3.20}$$

where

Q = discharge in cumecs
P = wetted perimeter in m
R = hydraulic mean depth in m
A = cross-sectional area in sq. m
v = velocity of flow in m/s
w = regime width in m
d = regime depth in m
m = mean diameter of particles in m
f = silt factor.

The values of f for different types of bed materials are given in Table 3.3.

Table 3.3 Lacey's silt factor [24]

Type of bed material	*Grain size* (mm)	*f*
Silt		
Very fine	0.0081	0.500
Fine	0.1200	0.600
Medium	0.2330	0.850
Standard	0.3230	1.000
Sand		
Medium	0.5050	1.250
Coarse	0.7250	1.500

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The following guidelines for fixing linear waterways across alluvial streams are recommended:

1. In alluvial rivers meandering over a wide belt, it normally costs less to confine the waterway opening and make the crossing in a combination of embankments than to bridge the full width of the flood plain. A cost comparison of such constructions is produced in Table 3.4 [24].
2. The linear waterway should not be more than the regime width. As the regime depth is not a function of the regime width [Eq. (3.19)], an increase in the span of the bridge will not allow reductions in the depth of foundation.
3. The linear waterway should not be less than the regime width. The cost of savings effected by a decrease in waterway openings will be offset by increased costs of foundations and training works.

Table 3.4 Cost comparison of bridges over flood plains

Name of the river and location of the bridge	*Width of the flood plain* (m)	*Length of the bridge over the confined waterway* (m)	*Length of the guide bank* (m)			*Ratio of the cost of a bridge spanning the flood plain to the cost of a bridge over the confined waterway and associated training works*
			Left	*Right*	*Total*	
Krishna at Vijayawada	1463	1138	823	610	1432	1.12
Chenab at Shershaka	3653	1114	1097	1220	2316	2.94
Ganga at Garhmukteshwar	1707	712	466	661	1128	1.65

3.4.4 Sand Gravel and Cohesive Soil Beds

A trial linear waterway for a sand and gravel bed may be calculated from Eqs. (3.8) and (3.9) respectively. For cohesive soils, the maximum depth measured at the given channel section is substituted in Eq. (3.12) and a trial discharge per unit width is obtained, then by dividing the design flow by the discharge per unit width, a trial linear waterway is computed. The total span of the bridge may be obtained from the water width by making due allowance for the obstruction to flow by piers, abutments and the skew of the bridge.

3.5 ECONOMIC SPAN

Economic span is one for which the total cost of the bridge is minimum. For the most economical span, the cost of the superstructure equals the cost of the substructure, with the following assumptions:

1. The cost of the superstructure is proportional to the square of the span.
2. The spans are of equal length.

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3. The cost of each abutment is the same
4. The cost of each pier is the same
5. The cost of railings, parapet, approach is constant.

Let

A be the cost of each abutment;
B be the cost of each pier;
C be the cost of railings, parapet, etc.;
D be the cost of approach;
T be the total cost of the bridge;
n be the number of spans;
l be the length of each span; and
L be the total span of the bridge.

Therefore, the total cost of the bridge

$$T = A + (n - 1)B + C + D + nkl^2$$

where k is the cost coefficient of the superstructure.

For minimum cost, $dT/dl = 0$.

Differentiating the above equation, w.r.t. l and equating to zero, and by writing $n = L/l$, we get

$$B = kl^2$$

Hence for economic span (l_e), the cost of the superstructure of one span is equal to the cost of the substructure of the same span. That is,

$$l_e = \sqrt{\frac{B}{k}} \tag{3.21}$$

3.6 AFFLUX

It is rarely feasible economically to bridge the river in one span. Normally, piers are located within the main flow channel and embankments encroach into the flood plain. These obstruct the flow and cause the upstream water level to rise above the free discharge level. This heading up of water on the upstream side of the bridge is known as afflux. It is one of the important parameters required to fix the various levels for the bridge. The velocity of flow under the bridge is also governed by afflux. The vertical clearance and freeboard are influenced by afflux as well.

The vertical clearance is the difference between the high flood level (HFL) and the lowest point on the superstructure. The freeboard is the difference between the highest flood level, after allowing for afflux if any, and the formation level of the communication route or the top level of guide banks.

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Some of the formulae used for computation of afflux are listed below:

Molesworth formula

$$x = \left(\frac{v^2}{17.9} + 0.015\right)\left(\frac{A^2}{a^2} - 1\right) \tag{3.22}$$

where

x = the afflux in m
v = the normal velocity of flow in m/s
A = the area of natural waterway in sq. m
a = the area of artificial waterway in sq. m

Marriman's formula

$$x = \frac{v^2}{2g}\left[\left(\frac{A}{Ca}\right)^2 - \frac{A}{A_1}\right] \tag{3.23}$$

where

g = acceleration due to gravity
A_1 = the enlarged area on the upstream of the brldge in sq. m
C = 0.75 + 0.35(a/A) – 0.1 $(a/A)^2$

The definitions of x, v, A, and a are the same as in Molesworth formula.

Drown Weir formula

$$x = \frac{v^2 d^2}{2g(d + x)^2}\left[\frac{L^2}{C^2 L_1^2} - 1\right] \tag{3.24}$$

where

L = natural linear waterway (width of the stream at high flood level)
L_1 = artificial linear waterway (linear waterway under the bridge)
C = discharge coefficient which varies from 0.7 for sharp to 0.9 for bell-mouth entry
d = depth of flow.

As afflux causes an increase in the velocity of flow through the bridge, it is normally limited to 200 to 300 mm. The allowable safe velocities for different types of soils under the bridge are:

Loose clay or fine sand—up to 0.5 m/s
Coarse sand—0.5 to 1.0 m/s
Fine gravel, sandy or silty clay—1.0 to 1.5 m/s
Coarse gravel, rocky soil—1.5 to 2.5 m/s
Boulder and rock—2.5 to 5.0 m/s

In case the velocity goes beyond the permissible safe limits, suitable protective works would be needed.

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3.7 SCOUR

Scour occurs, during the passage of high discharge, when the velocity of the stream exceeds the limiting velocity that can be withstood by the particles of the bed materials.

In the design of piers, abutments, training works, etc. for bridges across rivers, the assessment of amount of scour adjacent to the structures needs a careful consideration: In order to understand the mechanism of scour, the rivers can be classified as follows:

Streams with rigid boundaries. Streams whose both banks and the bed are very rigid are known as streams with rigid boundaries.

Quasi-alluvial streams. Streams flowing between the banks which are made up of rigid rock or mixture of sand and clay, whereas the bed material is composed of 'loose granular material which can be picked up by the current and transported, are known as quasi-alluvial streams. Quasi-alluvial streams never attain the regime condition.

Alluvial streams. Streams flowing between erodible banks and having erodible beds are known as alluvial streams.

3.7.1 Alluvial Streams

Regime condition of alluvial streams

The regime or equilibrium condition may be defined as a stable channel whose geometrical dimensions (width, depth and bed slope) have undergone modifications by silting and scouring as well as have attained an equilibrium state.

Normal scour depth for alluvial streams

The depth of the stream measured at the middle of the channel, when it is in the regime condition, is known as the *normal scour depth*. To attain normal scour depth, a constant discharge should pass through a straight stable reach of an alluvial stream for an indefinite time. Under such conditions, according to Lacey, the scour lines follow a semi-elliptical shape. According to Lacey, the normal scour depth is

$$d = 0.473\left(\frac{Q}{f}\right)^{1/3}$$

where

Q = discharge in cumecs

f = silt factor.

In practice, the flood flow is not indefinite. Due to this, the geometry of the flood section of any natural stream is governed by the magnitude and duration of floods. Observations have shown that [24] the natural scour lines have a sharper curvature in the middle than that of an ellipse, during sustained flows, with the result that Lacey's normal scour depth is an underestimate, when applied to natural rivers during sustained floods.

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3.7.2 Quasi-Alluvial Streams

Normal scour depth

The following cases arise:

When the width of the stream is not very large. Plot the cross-section and the longitudinal section of the stream for the dry weather season. Over the plotted cross-section, superimpose a probable scour line. Estimate the cross-sectional area of scour (A) and compute the hydraulic mean radius (R) of scour. From the longitudinal plot, measure the bed slope (S) of the stream. Then by using the Manning equation, calculate the velocity of flood flow and in turn the flood discharge (Q). If this agrees with the designed discharge, the estimated probable scour line is correct, otherwise repeat the procedure.

When the width of the stream is very large compared to the depth. Three sub-cases arise as follows:

When the velocity is known. If

w = width of the stream
v = velocity of flow
d = depth of flow

then

$$Q = v \times w \times d \tag{3.25}$$

$$d = \frac{Q}{w \times v} \tag{3.26}$$

When the slope is known. From Manning equation, we have

$$Q = \frac{R^{2/3} S^{1/2} wd}{n}$$

But

$$R = \frac{A}{P} = \frac{wd}{w + 2d}$$

For $P \approx w$, we have

$$R = \frac{wd}{w} \approx d$$

$$Q = \frac{d^{2/3} S^{1/2} wd}{n}$$

$$= \frac{d^{5/3} S^{1/2} w}{n} \tag{3.27}$$

Knowing Q, n, S and w, d can be computed.

When both velocity and slope are not known. Let P, R, and Q be the wetted perimeter, hydraulic mean radius and discharge, respectively, for quasi-alluvial rivers, then

$$Q = \frac{(PR) R^{2/3} S^{1/2}}{n} \tag{3.28}$$

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If the stream were to be alluvial and P_1 and R_1 the corresponding wetted perimeter and hydraulic mean radius, then

$$Q = \frac{(P_1 R_1) R_1^{2/3} S^{1/2}}{n} \tag{3.29}$$

Equating Eqs. (3.28) and (3.29), we get

$$\frac{R}{R_1} = \left(\frac{P_1}{P}\right)^{3/5} \tag{3.30}$$

But for alluvial streams, from Lacey's equations, we have

$$w_1 = 4.8\sqrt{Q} \tag{3.31}$$

$$R_1 = 0.473\left(\frac{Q}{f}\right)^{1/3} \tag{3.32}$$

$$P_1 = 4.8\sqrt{Q} \tag{3.33}$$

From Eqs. (3.30) to (3.33), we have

$$\frac{R}{0.473\left(\frac{Q}{f}\right)^{1/3}} = \left(\frac{4.8\sqrt{Q}}{P}\right)^{3/5} \tag{3.34}$$

After simplification, we get

$$R = \frac{1.21Q^{0.63}}{f^{0.33} P^{0.60}} \tag{3.35}$$

For wide rivers

$$P \approx w$$
$$R \approx d$$

Then,

$$d = \frac{1.21Q^{0.63}}{f^{0.33} w^{0.60}} \tag{3.36}$$

It should be noted that in deriving the above equation the Manning coefficient n and the bed slope S for both quasi-alluvial and alluvial streams are assumed to be the same. This condition can be realized for old rivers and materials that are really incoherent [24].

3.7.3 Alluvial and Quasi-Alluvial Streams

Derivation of equations for normal and maximum scour depths

In Fig. 3.1, w is the width, d the normal depth of scour and u the undisturbed normal velocity of flow on the upstream of the bridge. According to Kennedy's [25] theory, when the scour has ceased

$$u = m(d)^{0.61} \tag{3.37}$$

where m is the critical velocity ratio.

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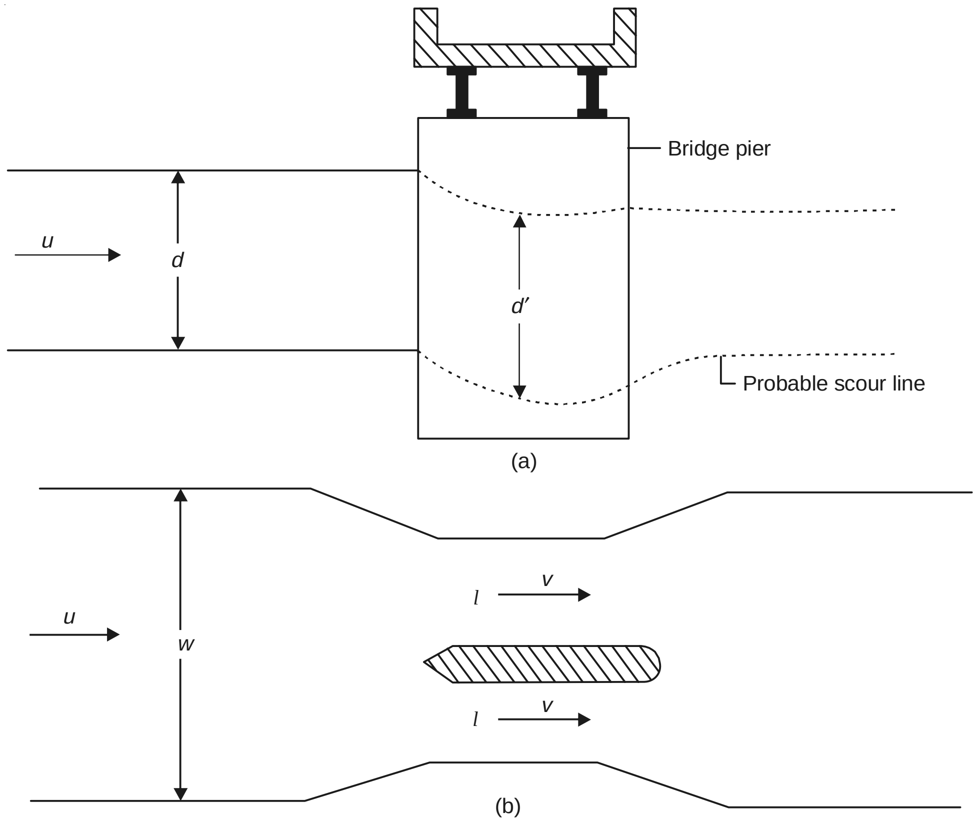

Fig. 3.1 Definition sketch for normal scour depth (alluvial sheam).

As soon as the flow approaches the bridge, the waterway is constricted, the velocity, v (under the bridge) is more than u.

Initially,

$$uwd = vLd$$

$$v = \frac{uw}{L} \tag{3.38}$$

where L is the span of the bridge.

The initial velocity, as the scour progresses, goes on decreasing (because of increase in area) till it reaches an equilibrium stage. Under the equilibrium condition, if d' is the depth of flow, according to Kennedy, the final velocity v' can be written as

$$v' = m(d')^{0.64} \tag{3.39}$$

From Eqs. (3.37) and (3.38), we have

$$v'/u = (d'/d)^{0.64} \tag{3.40}$$

From equation of continuity, we have

$$uwd = v'Ld'$$

or

$$v' = \frac{uwd}{Ld'} \tag{3.41}$$

From Eqs. (3.40) and (3.41), we get

$$d' = d\left(\frac{w}{L}\right)^{0.61} \tag{3.42}$$

Therefore, knowing d, w and L, d' can be calculated.

In natural streams, however, scour is not uniform (Fig. 3.2). At some pockets there may

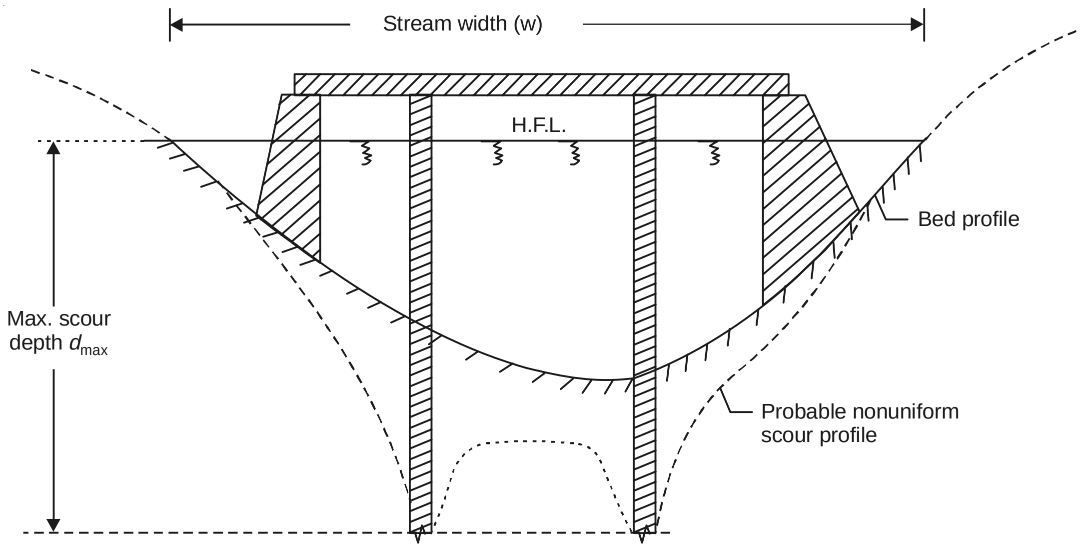

Fig. 3.2 Definition sketch for maximum scour depth.

be deeper scour than that at others. The velocity through these deepened parts does not decrease, because more water rushes into pockets of deeper scour and the initial velocity v does not change. Therefore,

$$v = \frac{wu}{L} \tag{3.43}$$

And as already explained

$$u = m(d)^{0.64} \tag{3.44}$$

According to Kennedy, when the maximum scour depth has developed, there will be equilibrium between v and d_{max}. Hence

$$v = m(d_{max})^{0.64} \tag{3.45}$$

From Eqs. (3.44) and (3.45), we have

$$\frac{v}{u} = \left(\frac{d_{max}}{d}\right)^{0.64} \tag{3.46}$$

From Eqs. (3.43) and (3.46), we get

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56} \tag{3.47}$$

The modified normal scour depth [Eq. (3.42)] and maximum scour depth [Eq. (3.47)] are applicable to both alluvial and quasi-alluvial streams.

3.7.4 Local Scour

The equations derived above for normal scour depths are based on the assumptions that scour depth is uniform across the section and the bed level is lowered across the whole section of the bridge in the form of a smooth curve. However, in practice, the scour does not occur uniformly. At some points, it will be deeper than that at others. Thus, to obtain the maximum scour depth the normal scour depths are modified according to the procedures detailed below:

For a straight reach of the stream and where the bridge is a single-span structure

ALLUVIAL STREAMS

Without constriction

Normal scour depth

$$d = 0.473\left(\frac{Q}{f}\right)^{1/3} \tag{3.48}$$

Maximum scour depth

$$d_{max} = 1.5d \tag{3.49}$$

For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56} \tag{3.50}$$

The larger of the two values obtained from Eqs. (3.49) and (3.50) is chosen as the maximum scour depth.

With constriction

Normal scour depth

$$d' = d\left(\frac{w}{L}\right)^{0.61} \tag{3.51}$$

Maximum scour depth

$$d_{max} = 1.5d' \tag{3.52}$$

For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56} \tag{3.53}$$

The larger of the two values obtained from Eqs. (3.52) and (3.53) is chosen as the maximum scour depth.

QUASI-ALLUVIAL STREAMS

For narrow cross-sections the estimation of maximum scour depth has been explained in the preceding subsection. For wide rivers, one of the following equations is used for estimation of the maximum scour depth:

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Without constriction

Normal scour depth—(a) When the velocity is known

$$d = \frac{Q}{vw} \tag{3.54}$$

(b) When Q, n, S and w are known

$$Q = \frac{wd^{5/3}S^{1/2}}{n} \tag{3.55}$$

(c) When both velocity and slope are not known

$$d = \frac{1.21Q^{0.63}}{f^{0.33}w^{0.60}} \tag{3.56}$$

For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56} \tag{3.57}$$

Maximum scour depth

$$d_{max} = 1.5d \tag{3.58}$$

The larger of the two values obtained from Eqs. (3.57) and (3.58) is chosen as the maximum scour depth.

With constriction

Normal scour depth

$$d' = d\left(\frac{w}{L}\right)^{0.61} \tag{3.59}$$

Maximum scour depth

$$d_{max} = 1.5d' \tag{3.60}$$

For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56} \tag{3.61}$$

The larger of the two values of Eqs. (3.60) and (3.61) is taken as the maximum scour depth.

For bad sites on curves or where diagonal currents exist or where the bridge is a multispan bridge

The procedure for calculating the maximum scour depth remains the same as explained in the preceding section, except that Eqs. (3.49), (3.52), (3.58) and (3.60) are multiplied by a factor 2 instead of 1.5.

Local scour at bridge piers

General methods of assessing scour depths have been described in this chapter. The interaction of flow around a bridge pier and the river bed surrounding it is very complex. The scour around the pier is due to the formation of a vortex flow system. The main vortex initiates the scour holes at the upstream nose of the pier. As bed material is removed by the flow, a spiral roller

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develops within the hole formed. In plan, the vortex system has a horseshoe shape and is generally known as "horseshoe vortex". The scour hole goes on increasing in size till the equilibrium depth is reached.

A number of empirical and semi-empirical formulae are available for estimating local scour around piers for cohesionless and cohesive materials. Some of them are given in Table 3.5 for cohesionless materials. Very little reliable information is available to calculate scour around piers in cohesive soils. Table 3.6 may be used as a guide.

Table 3.5 Equations for predicting local scour at cylindrical piers in cohesionless materials [26]

Scour condition	*Sand or gravel bed*	*F = Froude No.* $= v/\sqrt{(gy)}$	D_{50}	*Equation*
Clear water	Sand	—	—	$d_s = 1.17\ U_0^{0.62} b^{0.62}$
Sediment	Sand	$F < 0.5$	—	$d_s = 1.11\ y_0^{0.5} b^{0.5}$
Sediment	Sand	$F > 0.5$		$d_s = 1.59\ U_0^{0.67} b^{0.67}$, or $d_s = 1.11\ y_0^{0.5} b^{0.5}$
Sediment	Sand	$F < 0.3$	$0.001 < D_{50} < 0.004$	$d_s = 1.8\ y_0^{0.75} b^{0.25} - y_0$ $y_0 = 0.38\ q_0^{0.67} D_{50}^{-0.17}$
Clear water	Gravel			$d_s = Cy_0$ $y_0 = 0.23(s-1)^{-0.43} q_0^{0.86} D_{90}^{-0.29}$
Sediment	Gravel			$d_s = Cy_0$ $y_0 = 0.47\ q_0^{0.8} D_{90}^{-0.12}$

where

d_s = depth of scour measured below the upstream bed level;
b = width of the pier;
U_0 = approach velocity in m/s;
y_0 = depth upstream of pier in m;
q_0 = discharge per metre width, upstream of pier;
D_{50} = median particle size of the bed material in m;
D_{90} = size of the bed material such that 90% of the particles by number are smaller in a metre;
s = specific gravity of the bed material.

Table 3.6 Depth of scour in cohesive soils [27]

Pier shape in plan	*Inclination of pier faces*	*Depth of scour*
Circle	Vertical	1.50*b*
Rectangle	Vertical	2.00*b*
Lenticular	Vertical	1.20*b*
Rectangle with semicircular nose	Vertical. Inclined inwards towards top. Angle more than 20° to vertical.	1.50*b*
Rectangle with semicircular nose	Inclined outwards towards top. Angle more than 20° to vertical.	1.00*b*
Rectangle with semicircular nose		2.00*b*

(b = width of the pier)

Local scour at non-cylindrical piers

Local scour at non-cylindrical piers may be estimated by applying appropriate equations given in Table 3.5. These equations may be extended to circular piers by multiplying them by a shape factor f_2 (Table 3.7) and an oblique factor f_3 (Fig. 3.3). That is,

$$\text{Scour depth at non-cylindrical piers} = d_s\, f_2\, f_3$$

Table 3.7 Pier shape factor [27], f_2

Shape in plan	*Length/width*	f_2
Circular	1.0	1.0
Lenticular	2.0	0.91
	3.0	0.76
	4.0	0.67
	7.0	0.41
Parabolic	—	0.80
Triangular	—	—
60°	—	0.75
90°	—	1.25
Elliptic	2.0	0.91
	3.0	0.83
Ogival	4.0	0.86
Rectangular	2.0	1.11
	4.0	1.40
	6.0	1.11

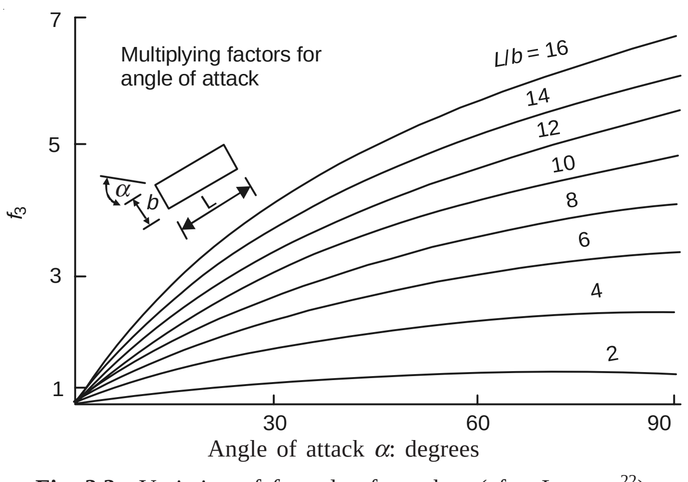

Fig. 3.3 Variation of f_3 angle of attack α (after Laursen[22]).

Local scour at pile groups

Due to availability of limited literature on scour around pile groups, the only way to estimate the scour is to assume that the group effectively becomes a single solid pier of the dimensions formed by the outermost piles in the group.

EXAMPLE 3.2

Determine the waterway for a bridge across a stream with a flood discharge of 225 m^3/s, velocity 1.5 m/s and width of flow at high flood level 60 m, if the allowable veloclty under the bridge is 1.8 m/s.

Solution

Area of the natural waterway

$$A = \frac{Q}{v} = \frac{225}{1.5} = 150 \text{ m}^2$$

Mean depth of flow

$$d = \frac{A}{L} = \frac{150}{60} = 2.5 \text{ m}$$

$$\text{Safe velocity (assumed)} = \frac{90}{100} \times \text{allowable velocity}$$

$$= 0.9 \times 1.8 = 1.62 \text{ m/s}$$

Area of the artificial waterway

$$a = \frac{225}{1.62} = 138.9 \text{ m}^2$$

From Molesworth formula, we have

$$\text{Afflux } x = \left(\frac{1.5^2}{17.9} + 0.015\right)\left(\frac{A^2}{a^2} - 1\right)$$

$$= \left(\frac{1.5^2}{17.9} + 0.015\right)\left(\frac{150^2}{138.9^2} - 1\right) = 0.024 \text{ m}$$

Linear waterway, $$L_1 = \frac{a}{d+x} = \frac{138.9}{2.5 + 0.024} = 55.03 \text{ m}$$

EXAMPLE 3.3

Design a waterway for a bridge over a trapezoidal channel having side slope of 1:1 with a discharge of 25 m^3/s, a bed fall of 1:1000 and a bed width to depth ratio of 6:1. The bed material is sand with a safe velocity of 2.5 m/s. The afflux should not be more than 8 cm. Take Manning coefficient, n = 0.025.

Solution

Area of flow

$$A = b \times d + 2\frac{d^2}{2}$$

$$= d(b + d) = 7d^2 \quad (\because b = 6d)$$

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Wetted perimeter

$$P = b + 2d\sqrt{2} = 8.83d$$

Hydraulic mean depth

$$R = \frac{A}{P} = \frac{7d^2}{8.83d} = 0.80d$$

From Manning's formula, we have

$$v = \frac{R^{2/3}S^{1/2}}{n} = \frac{(0.80d)^{2/3}\left(\frac{1}{1000}\right)^{1/2}}{0.025} = 1.1d^{2/3}$$

The flood discharge

$$Q = v \times A$$

or

$$25 = 1.1d^{2/3}\, 7d^2$$

Therefore,

$$d = 1.55 \text{ m}$$

and

$$P = 8.83d = 13.68 \text{ m}$$

and

$$v = 1.1d^{2/3} = 1.1(1.55)^{2/3}$$

$$= 1.47 \text{ m/s}$$

Since the velocity under the bridge is less than the given safe velocity (2.5 m/s), the design in sound.

From Drown Weir formula, we have

$$\text{Afflux, } x = \frac{v^2d^2}{2g(d+x)^2}\left(\frac{L^2}{C^2L_1^2} - 1\right)$$

or

$$0.08 = \frac{1.47^2 \times 1.55^2}{2 \times 9.81(1.55 + 0.08)^2}\left(\frac{10.88^2}{0.95^2 \times L_1^2} - 1\right)$$

Therefore,

$$L_1 = 8.5 \text{ m}$$

Also, velocity under the bridge

$$v = \frac{Q}{7d^2} = \frac{25}{7 \times (1.55)^2} = 1.49 \text{ m/s}$$

EXAMPLE 3.4

Design the waterway for the cross-section of a stream shown in Fig. 3.4. The average bed fall of the stream is 1:1600. Take Manning coefficient $n = 0.025$.

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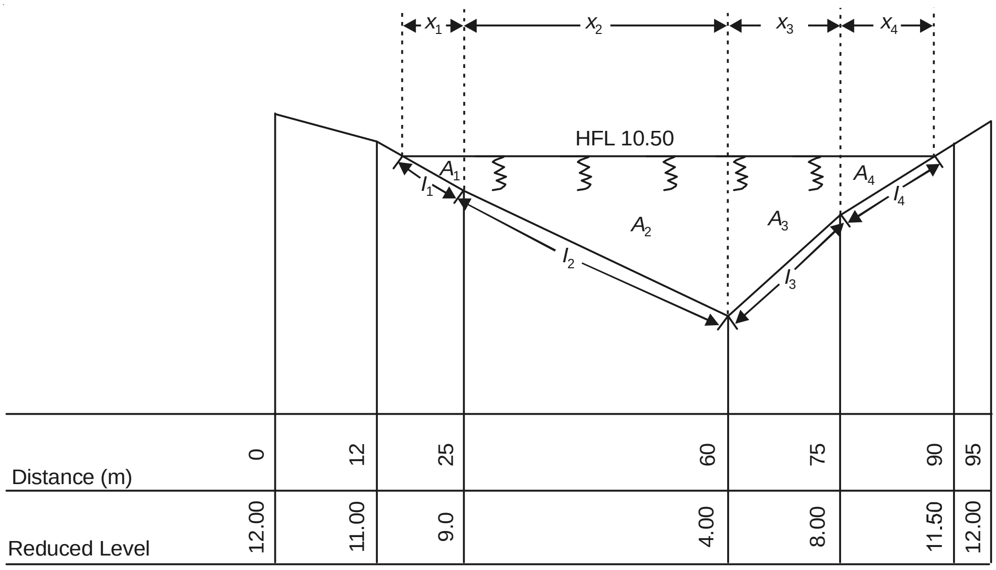

Fig. 3.4 Cross-section of the stream for Example 3.4 (not to scale).

Solution

Draw the HFL line on the given cross-section of the stream (Fig. 3.4). Since x_1 and x_4 are the unknowns, they are calculated by the principle of similar triangles. That is,

$$\frac{x_1}{13} = \frac{1.5}{2} \quad \text{or} \quad x_1 = 9.75 \text{ m}$$

$$\frac{x_4}{15} = \frac{2.5}{3.5} \quad \text{or} \quad x_4 = 10.7 \text{ m}$$

Inclined lengths are given by

$$l_1 = (9.75^2 + 1.5^2)^{0.5} = 9.86 \text{ m}$$
$$l_2 = (35^2 + 5^2)^{0.5} = 35.35 \text{ m}$$
$$l_3 = (15^2 + 4^2)^{0.5} = 15.52 \text{ m}$$
$$l_4 = (10.7^2 + 2.5^2)^{0.5} = 11.0 \text{ m}$$

$$\text{Wetted perimeter } P = 71.73 \text{ m}$$

Individual areas are given by

$$A_1 = 0.5 \times 9.75 \times 1.5 = 7.31 \text{ m}^2$$
$$A_2 = 35 \times 1.5 + 0.5 \times 35 \times 5 = 140.0 \text{ m}^2$$
$$A_3 = 15 \times 2.5 + 0.5 \times 15 \times 4 = 67.50 \text{ m}^2$$
$$A_4 = 0.5 \times 10.7 \times 2.5 = 13.38 \text{ m}^2$$

$$\text{Total area } A = 228.19 \text{ m}^2$$

Hydraulic mean depth

$$R = \frac{A}{P} = \frac{228.19}{71.73} = 3.18 \text{ m}$$

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From Manning's formula, we have

$$v = \left(\frac{1}{0.025}\right)(3.18)^{2/3}\left(\frac{1}{1600}\right)^{1/2} = 2.16 \text{ m/s}$$

Flood discharge

$$Q = Av = 228.19 \times 2.16$$
$$= 492.90 \text{ m}^3\text{/s}$$

Velocity of flow under the bridge

$$= \frac{120 \times 2.16}{100} = 2.59 \text{ m/s}$$

Artificial waterway area

$$a = \frac{492.90}{2.59} = 190.30 \text{ m}^2$$

From Molesworth formula, we have

$$x = \left(\frac{2.16^2}{17.9} + 0.015\right)\left(\frac{228.19^2}{190.30^2} - 1\right) = 0.12 \text{ m} = 12 \text{ cm}$$

Normal depth

$$d = \frac{A}{L} = \frac{228.19}{70.46} = 3.24 \text{ m}$$

Span of the bridge

$$L_1 = \frac{a}{d + x} = \frac{190.30}{3.24 + 0.12} = 56.63 \text{ m}$$

EXAMPLE 3.5

A bridge has a linear waterway of 120 m constructed across a stream whose natural waterway is 200 m. If the flood discharge is 1000 m^3/s and the mean depth of flow is 3 m, calculate the afflux under the bridge.

Solution

Area of the natural waterway

$$A = 200 \times 3 = 600 \text{ m}^2$$

Normal velocity of flow

$$v = \frac{1000}{600} = 1.67 \text{ m/s}$$

From Drown Weir formula, we have

$$x = \frac{1.67^2 \times 3^2}{2 \times 9.81(3 + x)^2}\left(\frac{200^2}{0.95^2 \times 120^2} - 1\right)$$

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or

$$x^3 + 6x^2 + 9x - 2.65 = 0$$

By trial and error the solution for the cubic equation is

$$x = 0.252 \text{ m}$$

EXAMPLE 3.6

A two-span plate girder bridge is to be provided across a river having the following data:

Flood discharge	100 m^3/s
Bed width	30 m
Side slope	1:l
Bed level	50.00
HFL	52.50
Maximum allowable afflux	15 cm

Calculate the span of the bridge.

Solution

Area of flow $A = 30 \times 2.5 + 2(0.5 \times 2.5 \times 2.5)$

$= 81.25 \text{ m}^2$

Normal velocity of flow $v = \dfrac{100}{81.25} = 1.23 \text{ m/s}$

From Molesworth formula, we have

$$0.15 = \left(\frac{1.23^2}{17.9} + 0.015\right)\left(\frac{A^2}{a^2} - 1\right)$$

or

$$2.50 = \frac{81.25^2}{a^2}$$

Therefore,

$$a = 51.38 \text{ m}^2$$

Span of the bridge $L_1 = \dfrac{a}{d + x} = \dfrac{51.38}{2.5 + 0.15} = 19.39 \text{ m}$

EXAMPLE 3.7

A Tee-beam and slab bridge needs to be provided across a river having the following data:

Distance (m)	0	11.28	24.4	52.4	68.3	80.5	85.3
RL	10.97	9.75	4.27	2.44	5.49	10.36	10.67
HFL	9.15						
Manning coefficient		0.03					

Maximum allowable velocity under the bridge = 1.2 × natural velocity
Slope of the river bed 1/1650

Calculate the span of the bridge (see Fig. 3.5)

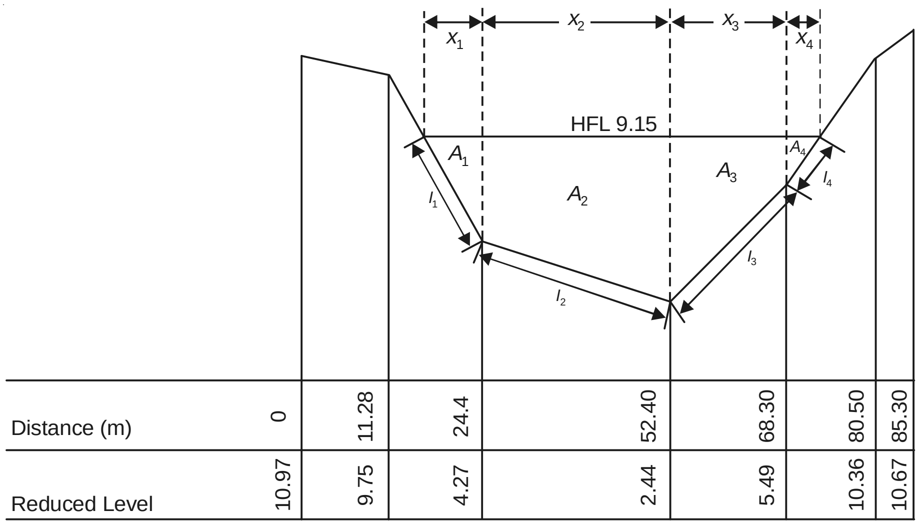

Fig. 3.5 Cross-section of the stream for Example 3.7 (not to scale).

Solution

From similar triangles (for details refer Example 3.4), we have

$$x_1 = 11.68 \text{ m and } x_4 = 9.16 \text{ m}$$

Area of the natural waterway

$$A = (0.5 \times 11.68 \times 4.88) + (28 \times 4.88 + 0.5 \times 28 \times 1.83) + (15.9 \times 3.66 + 0.5 \times 15.9 \times 3.05) + (0.5 \times 9.16 \times 3.66)$$
$$= 289.96 \text{ m}^2$$

Inclined lengths are given by

$$l_1 = (11.68^2 + 4.88^2)^{0.5} = 12.65 \text{ m}$$
$$l_2 = (28^2 + 1.83^2)^{0.5} = 28.05 \text{ m}$$
$$l_3 = (15.9^2 + 3.05^2)^{0.5} = 16.19 \text{ m}$$
$$l_4 = (9.16^2 + 3.66^2)^{0.5} = 9.86 \text{ m}$$
$$\text{Wetted perimeter } P = 66.75 \text{ m}$$

Hydraulic mean depth

$$R = \frac{289.96}{66.75} = 4.34 \text{ m}$$

Normal velocity of flow, using the Manning's formula

$$v = \left(\frac{1}{0.03}\right)(4.34)^{2/3}\left(\frac{1}{1650}\right)^{1/2} = 2.18 \text{ m/s}$$

Flood discharge

$$Q = 2.18 \times 289.96 = 632.2 \text{ m}^3\text{/s}$$

Velocity under the bridge

$$= 1.20 \times 2.18 = 2.60 \text{ m/s}$$

Area of the artificial waterway

$$a = \frac{632.2}{2.60} = 243.2 \text{ m}^2$$

Afflux

$$x = \left(\frac{2.18^2}{17.9} + 0.015\right)\left(\frac{289.96^2}{243.2^2} - 1\right)$$

$$= 0.10 \text{ m} = 10 \text{ cm}$$

Normal depth of flow

$$d = \frac{289.96}{64.74} = 4.47 \text{ m}$$

Span of the bridge

$$L_1 = \frac{243.2}{4.47 + 0.10} = 53.2 \text{ m}$$

EXAMPLE 3.8

The approximate costs of one superstructure and one pier for a multispan bridge are given below. Estimate the economic span.

Span (m)	12	18	21
Superstructure cost (Rs)	34,000	80,000	150,000
Substructure cost (Rs)	50,000	54,000	48,000

Solution

The average cost coefficient is calculated as shown in the following table. This calculation is based on the assumption that the cost of the superstructure is proportional to the square of the span.

Span	*Cost coefficient*	*Avg. cost coefficient*
12 m	$34{,}000/12^2 = 236.1$	$\frac{1}{2}(236.1 + 246.9 + 340) = 274.3$
18 m	$80{,}000/18^2 = 246.9$	
21 m	$150{,}000/21^2 = 340.0$	

Average cost of one pier (Rs)

$$= \frac{1}{3}(50,000 + 54,000 + 48,000)$$

$$= 50,666$$

Economic span

$$l_e = \left(\frac{B}{k}\right)^{0.5} = \left(\frac{50,666}{274.3}\right)^{0.5} = 13.6 \text{ m}$$

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EXAMPLE 3.9

The flood discharge under a bridge is 300 m^3/s. If the river bed has a deep layer of coarse sand, determine the maximum depth of scour under piers and abutments.

Solution

Normal depth of scour

$$d = 0.473\left(\frac{Q}{f}\right)^{1/3}$$

$$= 0.473\left(\frac{300}{1.5}\right)^{1/3} \quad (\because f = 1.5 \text{ from Table 3.3})$$

$$= 2.76 \text{ m}$$

Depth of scour under piers

$$= 2 \times 2.76 = 5.52 \text{ m}$$

Depth of scour under abutments

$$= 1.5 \times 2.76 = 4.14 \text{ m}$$

EXAMPLE 3.10

A stream with hard banks has a width of 80 m. Its bed is alluvial (f = 1.1) and discharge through the section is 500 m^3/s. Calculate the maximum scour depth under the bridge having a single span of 50 m.

Solution

Since the velocity and depth of flow are not known, the normal depth of scour with constriction is given by:

$$d' = d\left(\frac{w}{L}\right)^{0.61}$$

where

$$d = \frac{1.21Q^{0.63}}{f^{0.33}w^{0.6}}$$

$$= \frac{1.21 \times 500^{0.63}}{1.1^{0.33} \times 80^{0.6}} = 4.25 \text{ m}$$

Maximum depth of scour

$$d_{max} = d\left(\frac{w}{L}\right)^{0.61} \times 1.5$$

$$= 4.25\left(\frac{80}{50}\right)^{0.61} \times 1.5 = 8.49 \text{ m}$$

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For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56}$$

$$= 4.25\left(\frac{80}{50}\right)^{1.56} = 8.84 \text{ m}$$

Hence we adopt d_{max} = 8.84 m, being the larger of the two values.

EXAMPLE 3.11

Calculate the maximum scour depth for a bridge of two spans of total linear waterway of 60 m. The stream flows between quasi-alluvial soil with hard banks and alluvial bed (f = 1.1). The flood discharge is 450 m^3/s and the width of flow is 70 m.

Solution

Normal depth of scour

$$d = \frac{1.21Q^{0.63}}{f^{0.33}w^{0.6}}$$

$$= \frac{1.21 \times 450^{0.63}}{1.1^{0.33} \times 70^{0.6}} = 4.30 \text{ m}$$

Maximum depth of scour

$$d_{max} = 1.5 \times d'$$

$$= 1.5 \times 4.30\left(\frac{70}{60}\right)^{0.61} = 7.1 \text{ m}$$

For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56}$$

$$= 4.30\left(\frac{70}{60}\right)^{1.56} = 5.47 \text{ m}$$

Hence we adopt d_{max} = 7.1 m, being the larger of the two values.

EXAMPLE 3.12

A bridge needs to be constructed across an alluvial stream having a discharge of 500 cumecs. Calculate the depth of maximum scour when the bridge consists of:

(i) three spans of 15 m each
(ii) two spans of 30 m each, and
(iii) four spans of 30 m each

Take f = 1.1

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Solution

Regime width

$$w = 4.8 \times Q^{1/2} = 4.8(500)^{1/2} = 107.3 \text{ m}$$

Regime depth

$$d = 0.473\left(\frac{Q}{f}\right)^{1/3}$$

$$= 0.473\left(\frac{500}{1.1}\right)^{1/3} = 3.63 \text{ m}$$

Case 1

Maximum depth of scour

$$d_{max} = 2 \times d\left(\frac{w}{L}\right)^{0.61} = (2 \times 3.63)\left(\frac{107.3}{45}\right)^{0.61} = 12.33 \text{ m}$$

For non-uniform scour

$$d_{max} = d\left(\frac{w}{L}\right)^{1.56} = 3.63\left(\frac{107.3}{45}\right)^{1.56} = 14.08 \text{ m}$$

Hence we adopt d_{max} = 14.08 m, being the larger of the two values.

Case 2

$$d_{max} = 2 \times 3.63\left(\frac{107.3}{60}\right)^{0.61} = 10.3 \text{ m}$$

For non-uniform scour

$$d_{max} = 3.63\left(\frac{107.3}{60}\right)^{1.56} = 8.98 \text{ m}$$

Hence, we adopt d_{max} = 10.3 m, being the larger of the two values.

Case 3

Since $L > w$, we have

$$d_{max} = 2 \times d = 2 \times 3.63 = 7.26 \text{ m}$$

EXAMPLE 3.13

Calculate the depth of scour from the following data:

(i) Depth of flow on the upstream = 3.2 m/s
(ii) Approach velocity = 0.9 m/s
(iii) Median particle size = 0.78 mm
(iv) Length of the pier = 6.4 m
(v) Width of the pier = 4.4 m
(vi) Angle of attack = 10°
(vii) Critical approach velocity = 0.3 m/s

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Solution

Froude number

$$F = \frac{U_0}{(gy_0)^{0.5}} = \frac{0.9}{(9.81 \times 3.2)^{0.5}} = 0.16$$

From Table 3.5, local scour at cylindrical pier is

$$d_s = 1.11\ y^{0.5}\ b^{0.5}$$
$$= 1.1 \times 3^{0.5} \times 4.4^{0.5}$$
$$= 4.00 \text{ m}$$

From Table 3.7, for L/B = 2.0, the shape factor f_2 = 1.11,
From Fig. 3.3, for angle of attack of 10°, factor f_3 = 1.3
Hence, scour depth

$$= d_s \times f_2 \times f_3$$
$$= 4.00 \times 1.1 \times 1.3$$
$$= 5.72 \text{ m}$$

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Questions

1. What is the importance of hydraulic data in bridge design?
2. How do hydraulic factors influence the design of bridges?
3. Briefly explain the different phases of bridge design.
4. Explain the importance of peak flood flow in bridge design.
5. What are the limitations of empirical methods in estimation of peak flood flow? Give three examples.
6. What is an envelope curve? How is it useful in estimation of maximum flood discharge?
7. How is catchment scale useful in classification of catchments?
8. What do you understand by a small catchment? What are its characteristics?
9. Explain the terms: Runoff concentration, Concentration time, and Composite catchments.
10. Describe the methodology involved in the use of rational method for computation of maximum flood discharge from small catchments.
11. When do you prefer the rational method for analysis of runoff response from small catchments? What are its limitations?
12. What do you understand by: Concentrated catchment, subconcentrated catchment and superconcentrated catchment?
13. What are the characteristics of midsize catchments?
14. How are runoff curve numbers useful in estimation of peak flood flow from midsize catchments?
15. What is unit hydrograph?
16. Explain the steps involved in the derivation of unit hydrograph.
17. What are the methods available to change the duration of the unit hydrograph?
18. What do you understand by large catchments?
19. What is meant by frequency analysis of floods?

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20. What are plotting positions?
21. What are the methods available for fitting mean curve through flood data?
22. Explain the applications of Normal, Lognormal, Gumbel and Pearson Distributions in flood flow analysis.
23. When do you use the slope-area method for computation of peak flood flow?
24. Enumerate the data needed for application of the slope-area method.
25. Write down the guidelines required to be followed in selection of reach for the slope area method.
26. Describe in detail the steps involved in computation of flood flow by the slope-area method.
27. Distinguish between alluvial and quasi-alluvial streams.
28. What is meant by economical span of a bridge? Derive the equation for economical span. List the assumptions made.
29. Explain afflux. List and explain the different formulae used for estimation of afflux.
30. Differentiate between vertical clearance and freeboard.
31. What is the difference between the normal and maximum scour depths?
32. How do you estimate the normal scour depth of a quasi-alluvial stream? Derive the respective equations subjected to different constraints.
33. Clearly describe the method of estimation of maximum scour depth for alluvial and quasi-alluvial streams.

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Problems

1. Rainfall falls on a 160 ha catchment with an intensity of 2.5 cm/h and duration of 2.5 h. Use the rational method to calculate the peak runoff, assuming runoff coefficient $C = 0.7$ and concentration time as 1.5 h.
2. Using Kirpich and Hathaway formulae, estimate the concentration time for a catchment with the following characteristics: $L = 0.8$ km, $S = 0.05$.
3. Rain falls on a 250 ha composite catchment which drains two subareas as follows: (1) Subarea X, steep, draining 30% with concentration time 10 min and $C = 0.75$ and (2) Subarea Y, mild, draining 70% with concentration time 60 min and $C = 0.35$. Calculate the peak runoff corresponding to 25-year frequency. Use the following IDF function:

$$I = \frac{100\, T^{0.2}}{(t_r + 20)^{0.7}}$$

   where I = rainfall intensity in mm/h, T = return period in years, and t_r = rainfall duration in minutes. Assume linear concentration at the catchment outlet.
4. An agricultural watershed has the following hydrologic characteristics: (1) a subarea in fallow, with bare soil group B, covering 30%, and (2) a subarea planted with row crops, contoured and terraced in good hydrologic conditions, soil group C, covering 70%. Determine the runoff in centimetres, for 10 cm rainfall. Assume AMCII antecedent moisture conditions.
5. The total runoff from a catchment area of 65 sq. km due to an effective rainfall duration of 6 h is tabulated below. Derive a 6-h unit hydrograph. Assume a constant base flow of 12 cumecs.

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First day		*Second day*	
Time (h)	*Total runoff* (cumecs)	*Time* (h)	*Total runoff* (cumecs)
0500	13	0200	31
0800	34	0500	26
1100	64	0800	22
1400	83	1100	19
1700	76	1400	17
2000	59	1700	15
2300	41	2000	14

6. The ordinates of a 2-h unit hydrograph for a basin are as follows. Determine the ordinates of the 6-h unit hydrograph.

Time (h)	*Ordinates of the* 2-h *UH* (cumecs)	*Time* (h)	*Ordinates of the* 2-h *UH* (cumecs)
0	0	14	90
2	62	16	80
4	90	18	70
6	150	20	65
8	120	22	62
10	105	24	00
12	95	—	—

7. The following data represents the ordinates of a 6-h unit hydrograph. Obtain the 5-h unit hydrograph.

Time (h)	*Ordinates* (cumecs)	*Time* (h)	*Ordinares* (cumecs)
0	0	11	41
1	6	12	34
2	36	13	27
3	66	14	23
4	91	15	17
5	106	16	13
6	93	17	9
7	79	18	6
8	68	19	3
9	58	20	1.5
10	49	21	0

8. By using the slope-area method, calculate the peak discharge for the following data: Length of reach = 1200 m, Fall = 0.40, Manning coefficient n = 0.022.

	Upstream	*Downstream*
Flow area (sq. m)	3260	3500
Wetted perimeter (m)	620	650
Velocity coefficient	1.2	1.1

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9. Determine the waterway for a bridge across a stream with a flood discharge of 200 cumecs, velocity 1.5 m/s, and width of flow at HFL 55 m, if the allowable velocity under the bridge is 1.8 m/s.
10. Design the waterway for a bridge over a trapezoidal channel having the side slope of 1:1 with a discharge of 30 cumecs, bed fall of 1:1200 and a bed width to depth ratio of 5:1. The bed material can withstand a safe velocity of 2.5 m/s. The afflux is limited to 10 cm. Take Manning coefficient n = 0.025.
11. A bridge has a linear waterway of 110 m constructed across a stream, whose natural waterway is 190 m. If the flood flow is 950 cumecs and the mean depth of flow is 2.75 m, calculate the afflux under the bridge.
12. A bridge has to be constructed with the following data: Flood discharge = 90 cumecs, bed width = 27 m, side slope = 1:1, bed level = 100.0, HFL = 102.5, and maximum allowable afflux = 12 cm. Calculate the span of the bridge.
13. A plate girder bridge has to be constructed across a river with the following data:

Distance (m)	0	15	24	60	72	84	96
RL	90	89	87	82	86	89.5	90

HFL = 86.5, Manning coefficient n = 0.03, maximum allowable velocity is 1.2 times the normal velocity and the bed slope is 1:1600. Calculate the span of the bridge.

14. A stream has a regime width of 25 m and discharge of 450 cumecs. Calculate the maximum scour depth, if the bridge has a single span of 50 m. Take f = 1.1.
15. Calculate the maximum scour depth for a bridge of two spans of total linear waterway of 62 m. The stream flows between quasi-alluvial soil with hard banks and alluvial bed (f = 1.1). The flood discharge is 400 cumecs and the width of flow is 65 m.

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Appendix

Computer Programs

Program 1

```
#include<stdio.h>
#include<math.h>
#include<conio.h>
FILE *fp2;
/* To run this program refer Examples 2.9 and 2.10 */
main()
  {
    float q[200],p[200];
    float avr,std,cs,cv,a,a2,a3,temp,a1;
    int i,j,n;
    clrscr();
    fp2=fopen(“file2”,“r”);
fscanf(fp2,“%d\n”,&n);
                    for(i=0;i<=n–l;i++)
               { fscanf(fp2,“%f0.2”,&q[i]);
                 p[i]=(double)(i+l)/(n+l);
                 p[i]=l00*p[i];
                 } fclose(fp2);
                 avr=0.0;
                 std=0.0;
                  a=0.0;
                  for(i=0;i<=n–l;i++)
                  avr+=q[i];
                  avr=avr/n;
                  printf(“FREQUENCY ANALYSIS-NORMAL DISTRIBUTION\n\n”);
                  printf(“average=%fcumecs”,avr);
                          for(i=0;i<=n–l;i++)
               { al=q[i]–avr;
                 a2=pow(a1,2.0);
                 a3=pow(a1,3.0);
                 std=std+a2;
                 a=a+a3;}
```


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```
        std=std/(n–1);
        std=pow(std,0.5);
        printf("\n standard deviation=%fcumecs\n",std);
        a=a*n/((n–l)*(n–2));
        cs=a/pow(std,3.);
        cv=std/avr;
        printf("coefficient of variation=%f\n", cv);
        printf("coefficient of skewness =%f\n", cs);
          for(i=0;i<=n–l;i++)
          { for(j=i+l;j<=n;j++)
            if(q[i]<q[j])
            {temp=q[i];
             q[i]=q[j];
             q[j]=temp;}}
      printf("\n\nCOMPUTATIOlN OF PLOTTING POSITIONS\n\n");
      printf("Rank    Ranked values       Probability\n");
      printf("                                   (m/n+1)\n");
      printf("  (m)        (cumecs)                    (%)\n");
      for(i=0;i<=4;i++)
      {printf("%–4d         %5.2f         %2.2f\n",i,q[i],p[i];}
```

Program 2

```
#include<stdio.h>
#include<math.h>
#include<conio.h>
/*To use this program refer Example 2.11*/
FILE*fwl,*ft2;
float draw (float[],float[],int,float);
main()
{float q[200],p[200],t[200],acs[38][9],bs[38],xb[38]xx[38];
float f[10],fq[10],xy[38],bcs[38];
float avr,std,cs,cv,a,al,a2,a3;
int n,m,l,it,i,il,ii,ir,j;
printf("enter the total no. of values in the flood series = ");
scanf("%d", &n);
fw1=fopen("file3","r");
fscanf(fw1,"%d\n%d\n",&l,&m); printf("%d\n%d\n",l,m);
for(i=0;<=m–l; i++)
fscanf(fw1,"%f",&t[i]);
for(i=0;i<m–l;i++)
fscanf(fwl,"%f"&p[i]);
for (i=0;i<=l–1;i++)
{for(j=0;j<=m–1;++j) /*acs[]-columns 3–9 of Table 2.13 */
fscanf(fw1,"%f\n",&acs[i][j]);}
for(i=0;i<=l–l ;i++l)
fscanf(fw1,"%f\n",&bcs[i]); /*bcs[]-skew coefficients, column-1, Table 2.13*/
fclose(fw1);
ft2=fopen("file5","r'');
for(i=0;i<=n–1;i++)
{fscan(ft2,"%f\n",&q[i]);/*q[i]-flood series*/
}fclose(ft2);
for(i=0;i<=l–1;i++)
{for(j=0;j<=m–l;j++)
```

```
avr=0;
std=0;
a=0;
for(il=0;il<=n–1;il++)
avr+=q[il];
avr/=n;
printf("FREQUENCY ANALYSIS-LOGNORMAL METHOD");
printf("average=%f cumecs\n",avr);
for(il=0;il<=n–l;i++)
{al=q[il]–avr;
a2=pow(a1,2);
a3=pow(a1,3);
std+=a2;
a+=a3;}
std/=(n–1);
std=pow(std,0.5);
printf("standard deviation =%fcumecs\n",std);
a*=n/((n–l)*(n–2));
cv=std/avr;

cs=3*cv+pow(cv,3);
printf("coefficient of variation =%f\n",cv);
printf("coefficient of skewness=%f\n\n",cs);
printf("COMPUTATI0N OF FLOOD FLOWS");
printf("PROBABILITY RETURN PERIOD FREQUENCY FACTOR FLOOD FLOWS\n");
printf("   (%)   (years)   (k)   (cumecs)   ");
for(ii=0;ii<=m–l;ii++)
{for(il=0;il<=m–l;il++)
{for(ir=0;ir<=l–1;ir++)
        {xx[ir]=acs[ir][il];}
        xy[il]=(double)draw(bcs,xx,l,cs);}
        f[ii]=(double)draw(t,xy,m,t[ii]);
        fq[ii]=(double)(avr+f[ii]*std);}
        for(ii=0;ii<=m–1;++ii)
printf("\n\n%6.3f %8.2f %7.3f %8.3f\n",p[ii],t[ii],f[ii],fq[ii]);

}

/*function draw interpolates the values of frequency factors k from Table 2.13*/
float draw (float aa[38],float b[38],int n, float ax)
{ int k,j,m,l;
  float ay;
  if(ax<aa[0])
  {ay=((b[l]–b[0])*ax+(b[0]*aa[l]–b[l]*aa[0]))/(aa[l]–aa[0]);
  return(ay);}
  if(ax>+aa[n–1])
  {k=n–2;
  ay=((b[n–1]–b[k])*ax+(b[k]*aa[n–1]–b[n–l]*aa[k]))/(aa[n–l]–aa[k]);
  return(ay);}
  for(j=l;j<=n–1;j++)
  if(ax>=aa[j–1]&&ax<aa[j])
  {m=j;
  l=m–1;
  ay=((b[m]–b[l]*ax+(b[l]*aa[m]–b[m]*aa[l]))/(aa[m]–aa[l]);
  return(ay);}
  }
```


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Program 3

```
#include<stdio.h>
#include<conio.h>
#include<math.h>
/*To use this prograrn refer Example 2.12 */
FILE*frl,*ft3;
float draw(float[],float[],int,float);
main()
{clrscr();
 float q[200],p[200],t[200],acs[38],[10],bcs[38],xx[38];
 float xb[38],xy[38],br[38][10],bs[38],f[38],fq[38];
 float avr,std,cv,cs,a,al,a2,a3;
 int ij,ii,il,it,ir,l,m,n;
 printf("enter the total no. of values in the flood series = ");
 scanf("%d", &n);
 frl=fopen("file4","r");
 fscanf(fr1,"%d\n%d\n",&l,&m);
    for(i=0;i<=m–l ;i++)
 fscanf(fr1,"%f",&t[i]);
 for(i=0;i<=m–l;i++)
    fscanf(fr1,"%f",&p[i]);
 for(i=0;i<=l–l ;i++)
 for(j=0;j<=m–l;j++)
 fscanf(frl,"%f", &acs[i][j]);
    for(i=0;i<=l–l;i++)
 fscanf(fr1,"%f\n",&bcs[i]);
 fclose(frl);
    ft3=fopen("file6","r");
 for(i=0;i<=n–l;i++)
 fscanf(ft3,"%f\n",&q[i]);
 fclose(ft3);
    avr=0.0;
 std=0.0;
 a=0.0;
    for(il=0;il<=n–l;il++)
    avr+=(double)log10(q[il]);
 printf("FREQUENCY ANALYSIS-LOG PEARSON-TYPEIII METHOD\n\n");
 avr=avr/n; printf("average(log)=%f\n",avr);

 for(il=0;il<=n–1;il++)
   {al=(double)log10(q[il]–avr;
       a2=pow(a1,2);
       a3=pow(a1,3);
       std+=a2;
       a+=a3;}
       std= std/(n–1);
       std=pow(std,0.5);
       printf("standard deviation(1og) = %f\n",std);
       a=a*n/((n–l)*(n–2));
       cv=std/avr;
       11=1–1;
       cs=a/pow(std,3); printf("skewness factor= %f\n\n",cs);
```

```
   printf("COMPUTATI0N OF FLOOD FLOWS\n\n");
 printf("Probability      Return period      Frequency factor flood flows\n");
 printf("   (%)          (years)          (k)          (cumecs)\n");
        for(it=0;it<=l–1;it++)
        { bs[it]=bcs[l1–it];
        for(ii=0;ii<=m–l;ii++)
        br[it][ii]=acs[l1–it][ii];)
        for(ii=0;ii<=m–l;ii++)
        {for(il=0;il<=m–l;it1++)
        {for(ir=0;ir<=l–1;ir++)
        {xx[ir]=br[ir][il];}
           xy[i1]=draw(bs,xx,l,cs);}
        f[ii]=draw(t,xy,m,t[ii]);
        fq[ii]=avr+f[ii]*std;
        fq[ii]=(double)pow(10,(double)fq[ii]); }
        for(i=0;i<=m–l;i++)
 printf("%6.3f      %8.2f        %7.3f        %8.2f\n",p[i],t[i],f[i],fq[i]);
        }

 /*function draw interpolates the values from Table 2.16*/
 float draw(float aa[30],float b[30],int n, float ax)
        {
        int k,j,m,l;
        float ay;
        if(ax<aa[0])
 (ay=((b[l]–b[0])*ax+(b[0]*aa[l]–b[l]*aa[0]))/(aa[l]–aa[0]);
        return(ay);}
        if(ax>=aa[n–1])
        {k=n–2;
 ay=((b[n–1]–b[k])*ax+(b[k]*aa[n–1]–b[n–l]*aa[k]))/(aa[n–l]–aa[k]);
        return(ay); }
        for(j=l;j<=n–l;j++)
        if(ax>=aa[j–1]&&ax<aa[j])
        { m=j;
        l=m–l;
 ay=((b[m]–b[l])*ax+(b[l]*aa[m]–b[m]*aa[i]))/(aa[m]–aa[l]);
        return(ay);)
        }
```

Program 4

```
#include<stdio.h>
#include<conio.h>
#include<math.h>
FILE*f3;
/*To use this program refer Example 2.13 */
main()
{
 float q[200],p[10],t[10];

 double avr,std,cv,cs,anr,dnr,a,al,a2,a3,pl,p2,fq[l0];
```

```
int m,n,i,nl;
     clrscr();
     avr=0.0;
     std=0.0;
     a=0.0;
        f3= fopen("file1","r");
     fscanf(f3,"%d,%d\n",&m,&n);
        for(i=0;i<=m;i++)
        {fscanf(f3,"%f0.2",&t[i]);
        p[i]=((t[i]–l.)/t[i])*100;
        }
           nl=n+l;
           for(i=0;i<=n;i++)
           fscanf(f3,"%f0.2",&q[i]);
           for(i=0;i<=n;i++)
           avr+=q[i];
              avr=avr/nl;
        printf("FREQUENCY ANALYSIS-GUMBEL METHOD\n\n");
        printf(" ( ANALYTICAL SOLUTION ) \n\n ");
                 printf("average=%4.2fcumecs\n",avr);
                 for(i=0;i<=n;i++)
                 {a1=(q[i]–avr);
                 a2=pow(a1, 2);
                 a3=pow(a1, 3);
                 std=std+a2;
                 a=a+a3; }
                 std=std/(nl–1.);
                 std=pow(std,0.5);
                 printf("standard deviation = %fcumecs\n",std);
                 a=(a*nl)/((nl–l)*(nl–2));
                 a=a/pow(std,3.);
                 cv=std/avr;
                 printf("coefficient of variation=%f\n\n",cv);
                 anr=avr–0.45*std;
                 dnr=0.7797*std;
printf("COMPUTATION OF FLOOD FLOWS\n\n");
printf("Probability   Return period   Flood flows\n");
printf("   (%)          (years)          (cumecs)    \n\n");
        for(i=0;i<=m;i++)
        {p2=(t[i]–1)/t[i];
        p1=–(double)log(double (p2));
        pl=(double) log(double (pl));
        fq[i]=anr-dnr*pl;
     printf("%7.3f       %–7.3f       %7.3f\n",  p[i],t[i],fq[i]);
        }
     }
```


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Program 5

```
#include<stdio.h>
#include<conio.h,
#include<math.h>
main()
{
/* To use this program refer Example 3.1*/
 int i;

float  rl,fall,an,au,tu,uvelco,ad,
       dvelco,p,s,f,
       ku,kd,k;
float  ru,pd,pu,rd,q,hvu[100],hvd[100],si[100],qi[100];

       printf("\n\n\n");
       printf("C0MPUTATION OF PEAK FLOOD FLOW BY SLOPE AREA METHOD\n");
       printf("****************************************************\n").
       clrscr();

       printf("enter reach length in metres:");
       scanf("%f",&rl);
          print("enter fall:");
       scanf("%f",&fall);
       printf("enter mannings n:");
       scanf("%f",&an);
       printf("enter upstream area in sq. m:");
       scanf("%f",&au);
       printf("enter upstream wetted perimeter in m:");
       scanf("%f",&pu);
       printf("enter u/s vel coefficient:");
       scanf("%f",&uvelco);
       printf("enter d/s area in sq. m:");
       scanf("%f",&ad);
       printf("enter d/s wetted perimeter in m:");
       scanf("%f",&pd);
       printf("enter d/s vel coeff:");
       scanf("%f",&dvelco);
          ru=au/pu;
       rd=ad/pd;

          p=2./3.;
       ku=au*(pow(ru,p))/an;
          kd=ad*(pow(rd,p))/an;
          k=(ku*kd);
          k=pow(k,0.5);
          s=fall/rl;
          s=pow(s,0.5);
          q=k*s;
          i=0;
          clrscr();
```

```
    printf("\nPEAK DISCHARGE BY SLOPE AREA METHOD\n',n");
    printf(" .........................................................\-n");
    printf(" hvu hvd si          peak discharge\n");
    printf(" (m) (m)             (cumecs) \n");
    printf(" ......................................................... \n");

    start:
    hvu[i]=(uvelco*(q/au)*(q/au));
    hvu[i]=hvu[i]/(2*9.81);
    hvd[i]=(dvelco*(q/ad)*(q/ad));
    hvd[i]=hvd[i]/(2.*9.81);
    if(ad>au)f=0.5;
    if(au>ad)f=l.0;
    si[i]=(fall+r*(hvu[i]-hvd[i]))/rl;
    si[i]=pow(si[i],0.5);
    qi[i] =k*si[i];

printf("\n%l.3f   %1.3f              %1.5f%5.0f",hvu[i],hvd[i],si[i],qi[i]);
    if (abs(q–qi[i])<=0.001)
    goto stop;
    else (q=qi[i];i+=l;goto start;)

stop:
 }
```


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Section II

Structural Design

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Chapter 4

Design Loads for Bridges

4.1 INTRODUCTION

The design of superstructure or for that matter any other component of a bridge, is based on a set of loading conditions which the component must withstand. These loads may vary depending on duration (permanent or temporary), direction of action, type of deformation, and nature of structural action (shear, bending, torsion, etc.). In order to form a consistent basis for design, the Indian Road Congress (IRC) has developed a set of standard loading conditions, which are taken into account while designing a bridge. Other nations maintain their own set of design loads such as

- BS 5400 loads—United Kingdom
- Ontario Highway Bridge Design Code (OHBDC)—Canada
- American Association of State Highway and Transportation Officials (AASHTO)—USA

4.2 DESIGN LOADS

4.2.1 Dead Load

The dead load on a superstructure is the aggregate weight of all superstructure elements (elements above bearings) such as the deck, wearing coat, railings, parapets, stiffeners and utilities. It will be seen in design that the first step is to calculate the dead load of all the elements. The IRC 6 provides a table where the dead load unit weights of various constrnction materials are listed.

4.2.2 Vehicle Live Load

The term live load means a load that moves along the length of the span. By this definition, a man walking on the bridge is also a live load. But a highway bridge is designed to withstand

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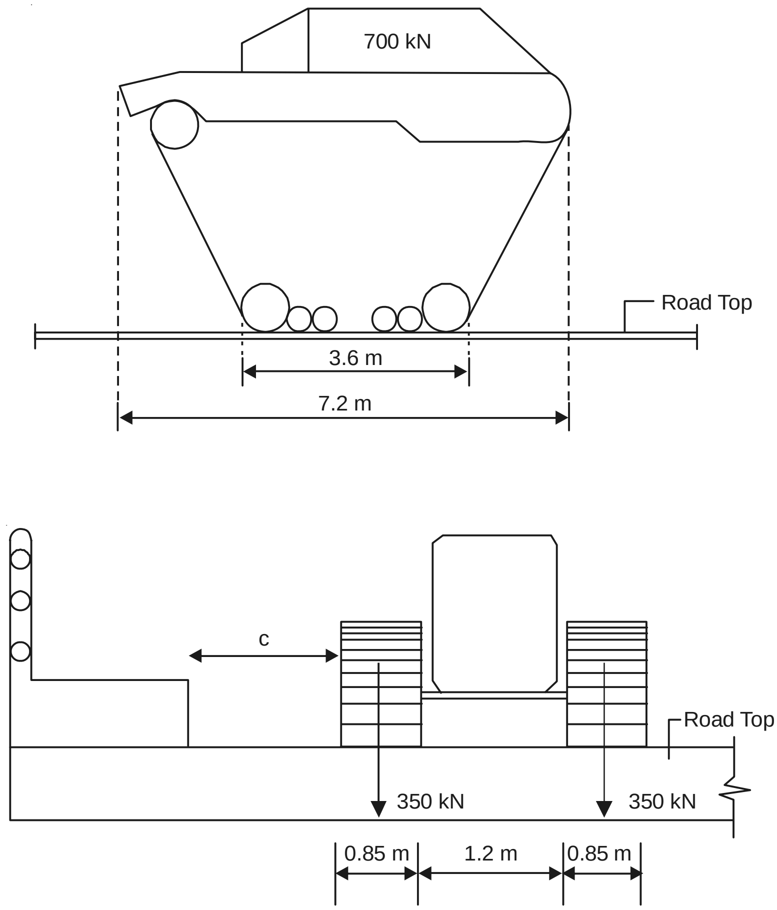
much more than just pedestrian loading. To give the designers the ability to accurately model the live load on a structnre, hypothetical vehicles were evolved by IRC long ago in 1946. The loads are categorized based on their configuration and intensity. They are explained below.

IRC Class AA loading

This is treated as heavy loading and is meant to be used for bridges for construction in certain industrial areas and other specified areas and highways. It is necessary that the bridges designed for IRC Class AA are checked for Class A loading as well. The IRC Class AA loadings have two patterns: (a) tracked type, and (b) wheeled type. The details of their geometry are shown in Fig. 4.1.



(a) Tracked Vehicle

Fig. 4.1 (Contd.)

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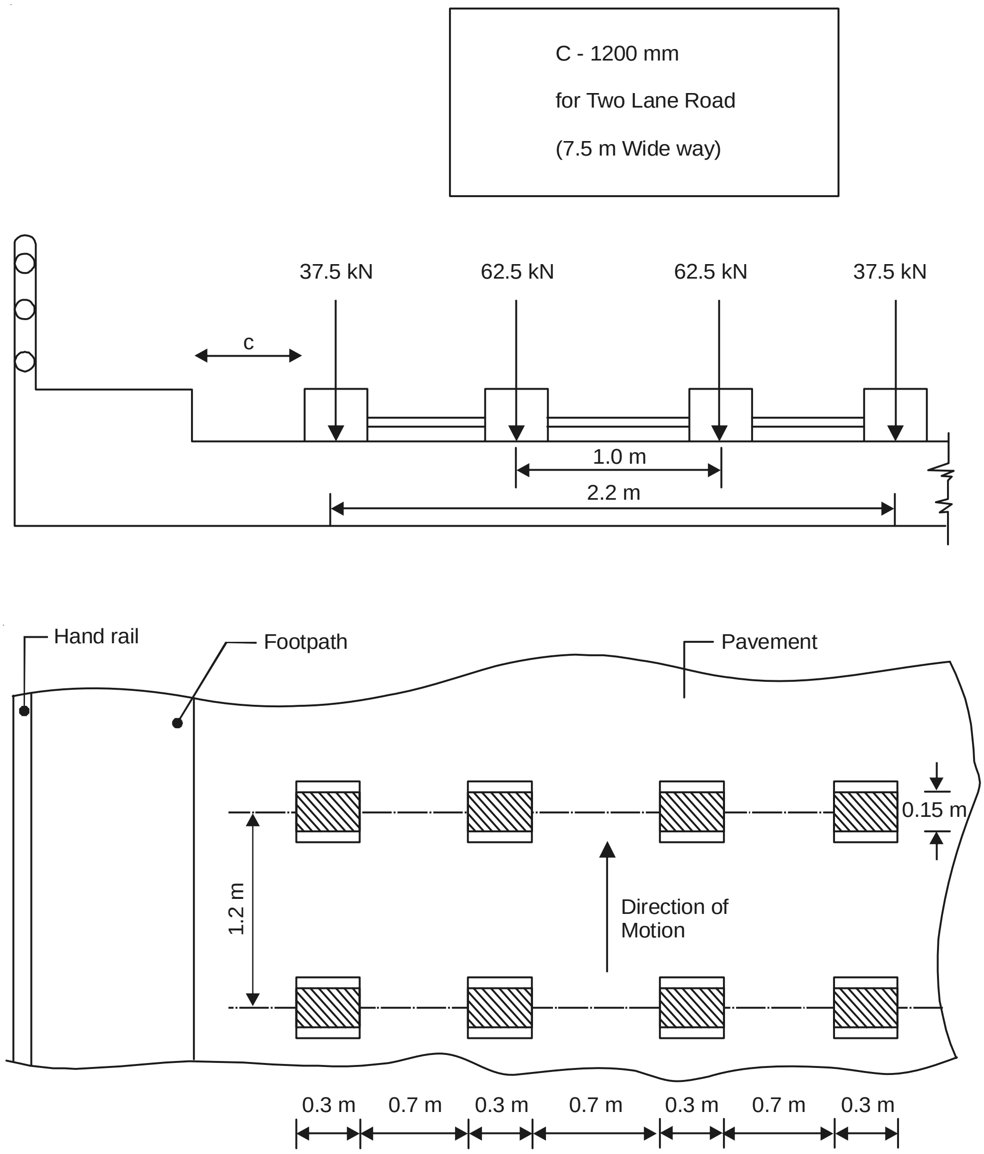
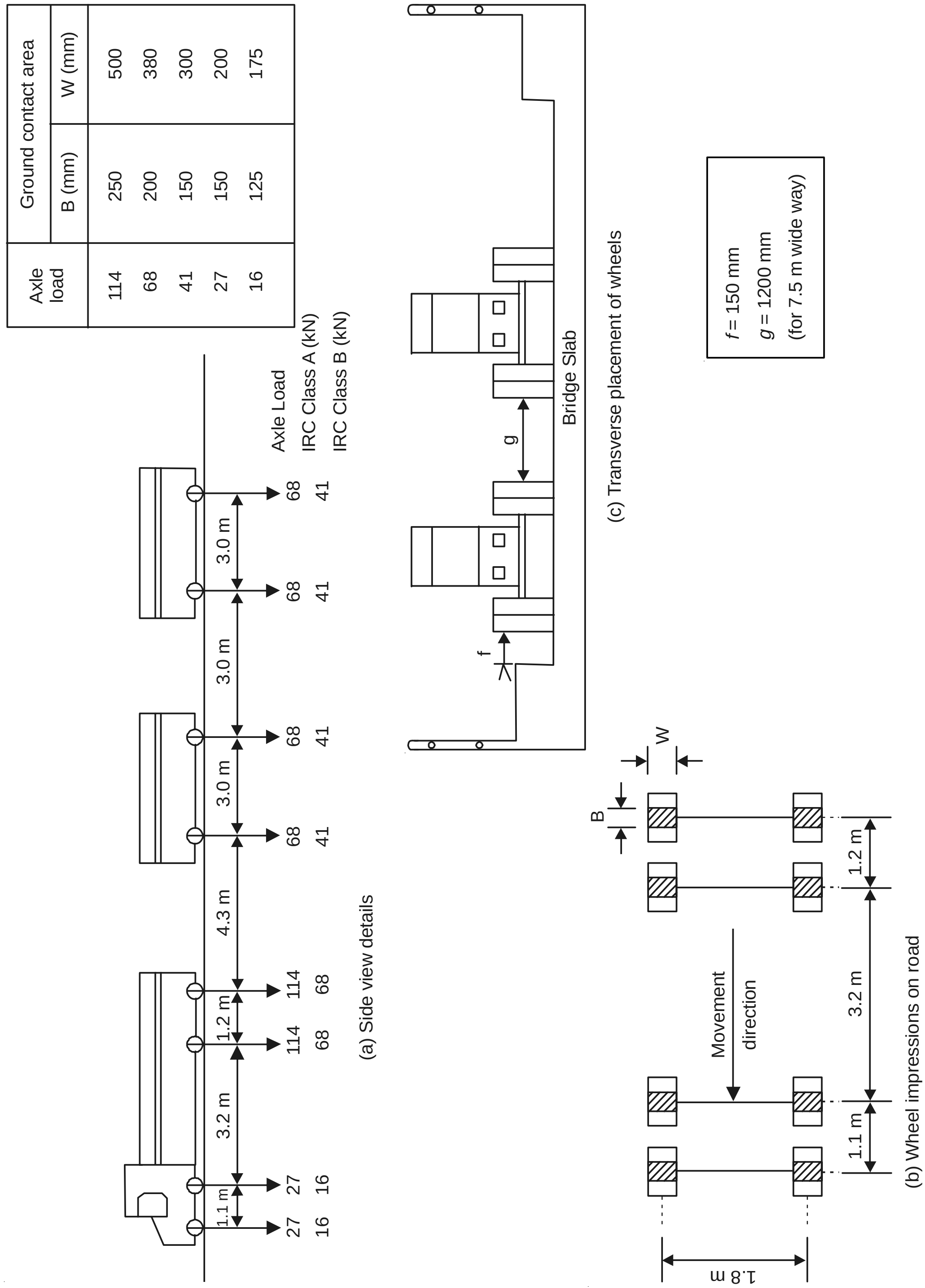


(b) Wheeled vehicle

Fig. 4.1 IRC Class AA loadings.

IRC Class A loading

This is treated as standard loading. It is considered for all permanent bridges in general. This loading has eight axles with a total length of about 25 m. The loading configuration is displayed in Fig. 4.2.



Axle load	Ground contact area	
	B (mm)	W (mm)
114	250	500
68	200	380
41	150	300
27	150	200
16	125	175

Fig. 4.2 IRC Class A and Class B loadings.

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IRC Class B loading

This is considered light loading (Fig. 4.2) and is used in the design of temporary bridges (timber bridges). In addition to the above classes of loading, Class 70R is also specified for use in lieu of IRC Class AA loading. This loading is a little different from Class AA and is shown in Fig. 4.3. It has been reported [31] that IRC loading is severe for a single lane bridge, but less severe when compared with French, West German, Japanese and British standards for a two-lane bridge.

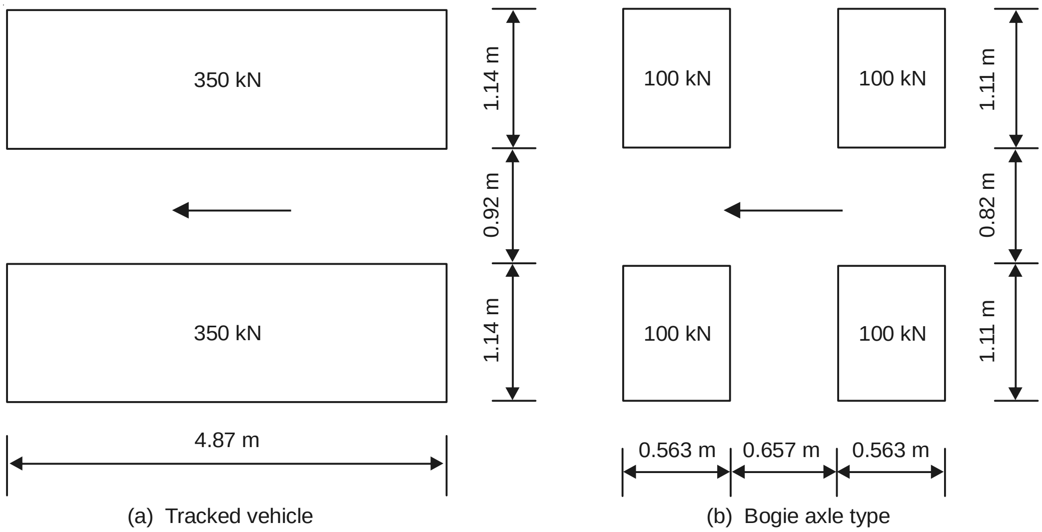

Fig. 4.3 Class 70R loadings.

4.2.3 Impact Effect

In order to account for the dynamic effects of the sudden loading of a vehicle on to a bridge structure, an impact factor is used as a multiplier for loads on certain structural elements. From basic dynamics we know that a load that moves across a member introduces larger stresses than those caused by a standstill load. However, the basis of impact factors predicted by IRC is not fully known. It has been felt by researchers [6] that the impact factor to a large extent depends on weight of the vehicle, its velocity, as well as surface characteristics of the road. It is pertinent to note that the live load increases on account of the consideration of the impact effect. For example, a span which is 9 m long would yield an impact factor of 0.10 (10%) and an impact multiplier of 1.10. The IRC specifications for impact factors are computed as mentioned below.

For IRC Class A or Class B loading

$$I_f = \frac{A}{B + L} \tag{4.1}$$

where

I_f = impact factor
A = constant, 4.5 for RCC bridges, 9.0 for steel bridges
B = constant, 6.00 for RCC bridges, 13.50 for steel bridges
L = span in m.

For IRC Class AA and 70R loading

1. Spans < 9 m

(a) *Tracked vehicle.* 25% for spans up to 5 m linearly reducing to 10% for spans up to 9 m.

(b) *Wheeled vehicle.* 25% for spans up to 9 m.

2. Spans > 9 m

(a) *Tracked vehicle.* For RCC bridges, 10% for spans up to 40 m and as per graph for spans > 40 m. For steel bridges, 10% for all spans.

(b) *Wheeled vehicle.* For RCC bridges, 25% for spans up to 12 m, and in accordance with graph for spans > 12 m. For steel bridges, 25% for spans up to 23 m and as per graph (IRC 6) for spans exceeding 23 m.

Appropriate impact factors as mentioned below need be considered for substructures as well.

- At the bottom of the bed block: 0.5
- For the top 3 m of the substructure: 0.5 to 0.0
- For portion of the substructure > 3 m below the block: 0.0

4.2.4 Wind Loading

Wind loading offers a complicated set of loading conditions, which must be idealized in order to provide a workable design. The modelling of wind forces is dynamic one, with winds acting over a given time interval; these forces can be approximated to a static load uniformly distributed over the exposed region of the bridge. The exposed region of a bridge is taken as the aggregate surface areas of all elements (both superstructure and substructure) as seen in elevation (perpendicular to the longitudinal axis of the bridge). The wind forces may be selected from Ref. [21].

4.2.5 Longitudinal Forces

These forces result from vehicles braking or accelerating while travelling on a bridge. As a vehicle brakes, the load of the vehicle is transferred from its wheels to the bridge deck. The IRC specifies a longitudinal force of 20% of the appropriate lane load. This force is applied at 1.2 m above the level of the deck. The effect of longitudinal forces on the superstructure is inconsequential; substructure elements, however, are affected more significantly. In general, the more stiff or rigid the structure is, the more severe the effects of longitudinal forces will be [30].

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4.2.6 Centrifugal Forces

For bridges on horizontal curves, the effects of the centrifugal force must also be calculated. Like longitudinal loading, centrifugal loading results from a vehicle travelling on a bridge and, in this instance, following a curvilinear path. This force is applied at 2 m above the level of the deck, and is defined as

$$C = \frac{WV^2}{127R} \tag{4.2}$$

where

C = centrifugal force in kN, without impact
W = live load in kN
V = design speed in km/h
R = radius of the curve in m.

4.2.7 Buoyancy

Bridges with components which are submerged underwater (e.g. piers) can sometimes suffer from effects of buoyancy. This is generally a problem only for very large structures. Buoyancy can have undermining effects on pier footings and piles. The forces of buoyancy should be considered depending on the extent of submergence.

4.2.8 Water Current Forces

Horizontal forces are exerted on submerged portions of substructures because of water current. The intensity of pressure is maximum at the top surface and linearly reduces to zero at the bed level. It is given by

$$P = KW\left[\frac{V^2}{2g}\right] \tag{4.3}$$

where

P = intensity of pressure in kN/m^2 because of water current
W = unit weight of water in kN/m^3
V = velocity of water current in m/s
g = acceleration due to gravity in m/s^2
K = a constant dependmg on the shape of the pier. The value of K is
1.5 for square ended piers,
0.66 for circular cut and ease water, and
0.9 for triangular cut water.

When water current has an oblique approach, it is resolved along the pier and across the pier. To allow for a possible variation in direction, current direction of 20° is normally considered.

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4.2.9 Thermal Forces

The effects of temperature on a structure are significant and should not be underestimated by the designer. Thermal forces are caused by fluctuations in temperature. If one side of a structure is continually exposed to the sun while the other side is shaded, this differential in temperature can cause high thermal forces. These forces generally have an adverse impact on bearings and deck joints. Temperature stresses are tensile stresses. Since concrete is not proficient in handling tension, these stresses can cause cracks in concrete structures. To abate this, added reinforcement is provided in the concrete element. This reinforcement, known as temperature reinforcement, is laid perpendicular to the main reinforcement.

4.2.10 Deformation and Horizontal Forces

Deformation loads are induced by both internal and external changes in properties of materials or geometry of members. The effects of deformations such as creep and shrinkage in concrete induce stresses on a member. Horizontal forces on bridges are basically of two types—self-induced type and applied type. The self-induced forces are due to creep, elastic shortening of the deck, temperature changes and shrinkage. The applied forces are due to braking, earthquake and wind. The distribution of these horizontal forces is affected by the horizontal deformation of bearings, swaying of supports and rotation of foundations.

For simply supported spans with rocker (fixed type) and rocker roller (free) bearings, the horizontal forces shall be as follows:

At rocker bearings. $\{F_h - \mu(R_g + R_q)\}$ or $\{F_h/2 + \mu(R_g + R_q)\}$ whichever is greater.

At free bearings. $\mu(R_g + R_q)$

where

F_h = applied horizontal force on the deck on the span under consideration
R_g = reaction at the free end owing to dead load
R_q = reaction at the free end owing to live load.

Table 4.1 below gives the coefficient of friction at the free bearings.

Table 4.1 Friction coefficients for different types of bearings

Type of bearing	*Friction coefficient*
Steel roller bearing	0.03
Concrete roller bearing	0.40
Sliding bearings	
Steel on steel	0.50
Cast iron on cast iron	0.40
Concrete over concrete	0.60
Teflon on steel	0.05

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4.2.11 Erection Stresses

It is possible that, during erection, various members of a structure come under loading conditions that are induced by construction equipment or other types of loads. If this is foreseen during the design process, the designer should take such additional loads into account and provide the necessary bracing or support structures on the plans.

4.2.12 Seismic Forces

These forces depend on the geographic location of the bridge. Like the live loads of vehicles, seismic forces are temporary loads on a structure which act for a short duration. An earthquake exerts forces on a bridge which are defined as a function of the following factors:

- Dead load of the structure
- Ground motion
- Period of vibration
- Nature of soil

The seismic force acts as a horizontal force equal to a fraction appropriate to the region (zones) as given in IRC 6. This horizontal force is given by

$$F = \alpha_h W \tag{4.4}$$

where

F = horizontal force owing to earthquake
α_h = seismic coefficient for the region
W = weight of the dead and live loads acting above the section.

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Chapter 5

Masonry Arch Bridges

5.1 INTRODUCTION

Masonry arch bridges were probably the first category of bridges to be evolved. These bridges are aesthetically superior to slab bridges. An arched bridge consists of a solid barrel between two face walls that are mounted on the arch ring. The arch ring is the major load-carrying member of the bridge. The ring resists the superimposed load essentially as a compressive thrust. Since the naturally available stones are strong in bearing compressive loads, the masonry arch bridges that are still existing have withstood the loads that are more than what the bridges are designed for. It is reported that these bridges are taking 40% more than their designed load. The various types of arch rings are shown in Fig. 5.1. The salient parts of an arch ring are

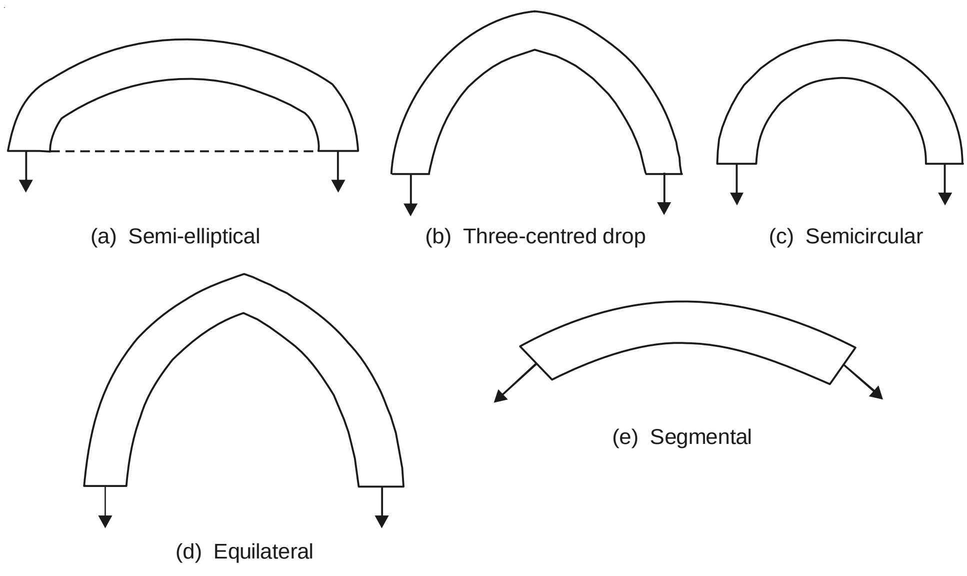

Fig. 5.1 Types of arch rings.

shown in Fig. 5.2. The arch ring is made up of stones cut to the shape of a wedge arranged in radial joints. The central key stone is a little bigger than the rest and it is placed at the end of the construction of the arch. Arch in its natural tendency tries to open up at the springing level because of self and superimposed loads. To prevent this, some extra weight is placed on the haunch of the arch by laying lean concrete that is sloped tangential to arch extrados. The haunch filling is retained by the curtain wall (back wall).

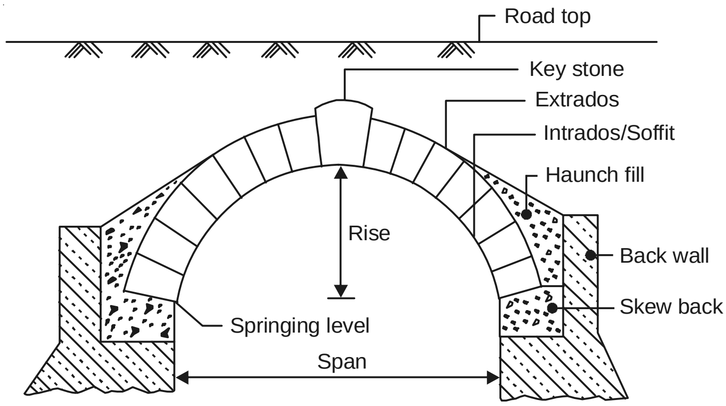

Fig. 5.2 Salient parts of an arch ring.

It is worth considering arch bridges for spans up to 30 m. However, the span is restricted to 6 m in seismic zones. Masonry arch bridges are preferable to modem types of bridges in situations where supplies of materials such as cement and steel, skilled labour and heavy machinery are inadequate. However, nowadays, masonry bridges have become almost obsolete as the construction of these bridges is labour intensive and uneconomical.

The design details of an arch bridge and the drawing details for a Roadway and Railway bridge are presented in this chapter. Figure 5.3 shows a complete view of an arch bridge with various details furnished therein.

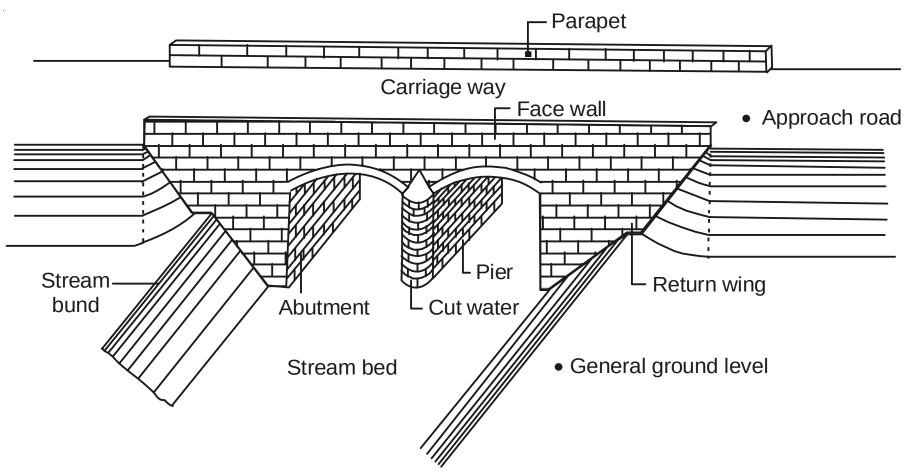

Fig. 5.3 A view of an arch bridge.

5.2 DESIGN DETAILS

The design of an arch bridge involves, arriving at dimensions of various components of the bridge superstructure such as rise of the arch, radius of the arch ring, thickness of the arch ring, depth of haunch filling, etc. by using empirical formulae.

5.2.1 Rise of the Arch

The rise of the arch is decided based on the rise to span ratio. Thus, the rise varies from 1/2 to 1/10 of spans. Too large a rise results in a big headroom and too small a rise results in increased lateral thrust. For segmental arches, a rise of 1/3 to 1/4 of span is found to be strong and economical.

5.2.2 Radius of the Arch

This can be calculated from the geometry of the circle. The radius R of the intrados is given by

$$R = \frac{S^2 + 4r^2}{8r} \tag{5.1}$$

where

S = span of the arch
r = rise of the arch.

5.2.3 Thickness of the Arch Ring

Several empirical formulae are available to calculate the thickness of the arch ring (Trautwyne, Hurst, Rankine, French) but the most commonly used is one suggested by Trautwyne, wherein the thickness is given by

$$t = \frac{\sqrt{R + 0.5S}}{7} + 0.06 \tag{5.2}$$

It is suggested that for large spans (more than about 15 m), the thickness of the arch ring at the springing level should be 25% to 50% more than that at the crown level (usually central one-third of the span). Further, if the arch is of brick masonry the thickness so calculated must be increased by about 35%.

5.2.4 Depth of Haunch Filling

This depth is measured from the springing level, and is given by

$$d = \frac{r + t}{2} \tag{5.3}$$

The filling is done tangential to the arch extrados, or laid in a slope of 1 in 6. Sometimes this filling is taken up to the crown level. This filling serves a dual purpose, i.e. it serves as an additional weight and also provides a working platform for construction of the face wall.

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5.3 DIMENSIONING OF SUBSTRUCTURES

5.3.1 Abutment

The top width of the abutment is fixed by the following empirical formula given by Trautwyne.

$$a = 0.6 + \frac{2}{10}R + \frac{1}{10}r \tag{5.4}$$

Further, the back batter is given by

$$b = \frac{S}{24r} \tag{5.5}$$

The length of the abutment should match with the width of the superstructure.

5.3.2 Pier

The top width of the pier should not be less than twice the width of the bearings or arch skewbacks, with clearance between them. The following rules of thumb are available for calculating the top width of the pier.

Top width = $2t + 0.3$
or
$\frac{1}{6}$ to $\frac{1}{7}$ of span

} The higher of the two values is chosen.

Straight sections without batter may be provided for heights less than 3 m. For heights more than 3 m, a side batter of 1 in 12 to 1 in 30 may be given. This is for stability and aesthetics. A batter of 1 in 24 is common.

5.3.3 End Connectors

End connectors are the structures of masonry built together with the abutment. The purpose of end connectors is

- ➢ to anchor the bridge to its approach road,
- ➢ to pave a confined way for smooth movement of water beneath, and
- ➢ to retain the earthen embankment and prevent it from spilling into or interfering with the flow of stream.

Two types of wing walls are commonly in use as end connectors. Their pattern is explained below.

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Return type wing wall

The return type wing walls emerge at right angles to the abutment and are prolonged towards the road bund. Their top is kept at the embankment formation level. A cavity is thus created, which is filled with soil up to formation level. This type of wing wall is preferred if approaches are in cuttings, or for small embankments of height less than 4 m. This type is also preferred if the banks are steep and the soil is firm. The length of the return wall is defined by the profile of the site (stream bund slope, embankment slope, safe margin from edge of water, etc.). The general arrangement of a return type wing wall is shown in Fig. 5.4.

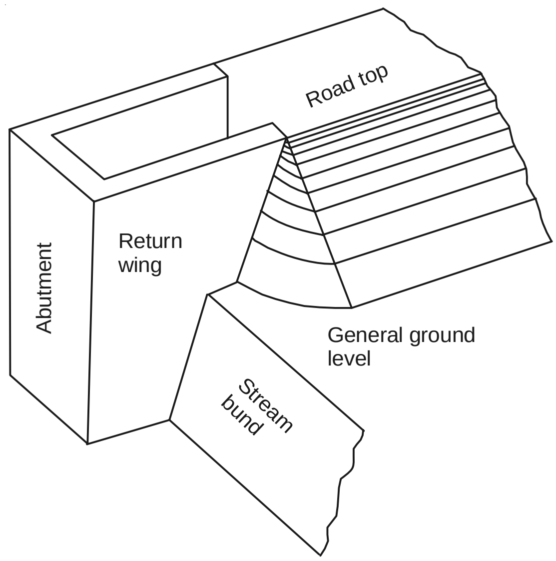

Fig. 5.4 Return type wing wall.

Splayed type wing wall

This type of wing wall is built at an angle to the abutment. The inclination of the wing wall with respect to the face of the abutment is called the *splay*. For bridges it is normally 45°. In this type, the top of the wing wall slopes down from the top level of the embankment to general ground level at the site. Thus, this type of wing wall allows the embankment to rest with its natural slope. The wing wall ends up in a small pedestal or newel of size 60 cm or more. The top width of the wing wall is generally 0.5 m. The front face of the wing wall is kept vertical for small heights (less than 5 m), and for walls of heights more than 5 m a batter of 1 in 10 to 1 in 30 is provided for stability. The bottom width is obtained by considering a back batter of 1 in 6. However, the base width should not be less than 0.4 times the height of the wall. These types of wing walls are found to be effective in case of high embankments and loose soils. The configuration of a this type of wing wall is sketched in Fig. 5.5.

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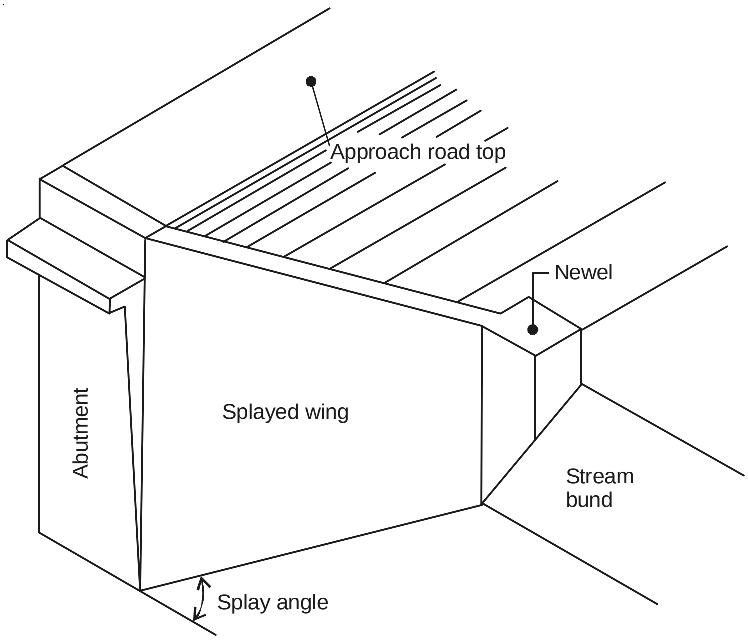

Fig. 5.5 Splayed type wing wall.

EXAMPLE 5.1

The following data pertains to a site, where a masonry arch bridge is proposed to be constructed. Design the arch bridge and its components.

Span length: 10 m
No of spans: two
HFL of the stream: 108 m
Bed level: 105 m
Stream bund top level: 108.75 m
Road top level: 113.00 m
Stream bed width: 25 m
Slope of the stream bund: 1:1
Slope of the road bund: 2:1
Road width: two lane (7.5 m), with 600 mm wide kerbs.

Design of the Superstructure

Rise of the arch intrados

$$r = \text{span}/4 = 10/4 = 2.5 \text{ m}$$

Radius of the arch intrados

$$R = \frac{S^2 + 4r^2}{8r} = \frac{10^2 + 4 \times 2.5^2}{8 \times 2.5} = 6.25 \text{ m}$$

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Thickness of the arch ring as given by Trautwynes formula is

$$t = \frac{\sqrt{R + 0.5S}}{7.0} + 0.06 = \frac{\sqrt{6.25 + 0.5 \times 10}}{7.0} + 0.06 = 0.539 \approx 0.55 \text{ m}$$

Depth of the haunch filling

$$d = \frac{r + t}{2} = \frac{2.5 + 0.55}{2} = 1.525 \approx 1.5 \text{ m}$$

The height is with respect to the springing level. The filling is made tangential to the arch extrados.

Design of the Abutment

The springing level is assumed to be 1 m above HFL. The top width of the abutment at the springing level is (see Fig. 5.6) given by Trautwynes formula

$$a = 0.6 + \frac{2}{10}R + \frac{1}{10}r$$
$$= 0.6 + \frac{2 \times 6.25}{10} + \frac{2.5}{10} = 2.1 \text{ m}$$

The front face of the abutment is made vertical while the back is provided with a batter to keep the resultant within the middle-third of the base.

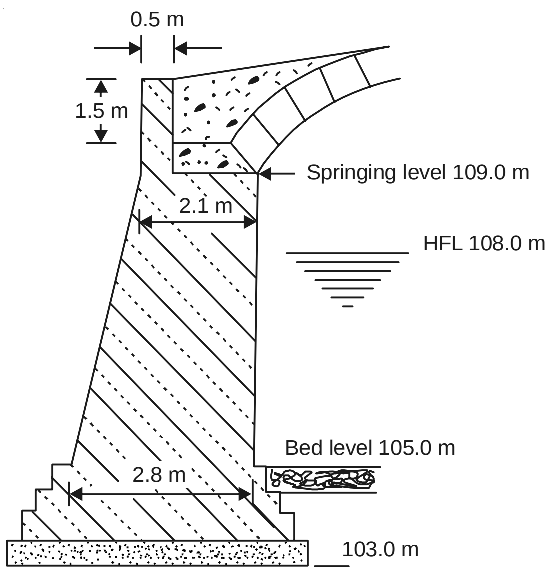

Fig. 5.6 Cross-section of the abutment (Example 5.1).

Back batter $b = \frac{S}{24r} = \frac{10}{24 \times 2.5} = \frac{1}{6}$

Base width at bed level = Top width + $\frac{1}{6}$(height)

$$= 2.1 + \frac{1}{6}(109 - 105) = 2.76 \approx 2.8$$

Design of the Pier

As the height of the pier is more than 3 m, a batter of 1 in 24 has been provided to both sides of the pier (see Fig. 5.7).

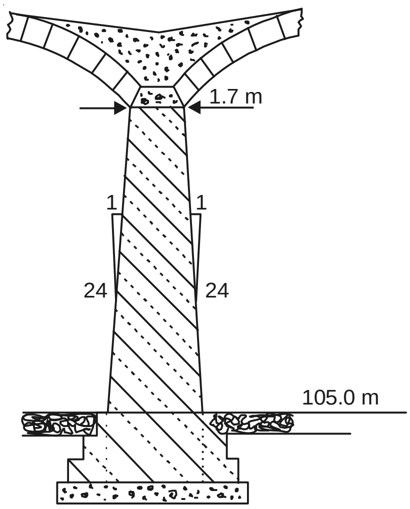

Fig. 5.7 Cross-section of the pier (Example 5.1).

Top width = $\frac{1}{7}$ of span = $\frac{1}{7} \times 10 = 1.4$ m

Bottom width = $1.4 + \frac{1}{24} \times 4 \approx 1.6$ m

Pier length = Road width + 2 (kerb width) = 7.5 + 2 × 0.6 = 8.7 m

The pier is provided with a semicircular cut and ease water.

Design of the End Connectors

The return type of wing wall is proposed here. The top width of the wing wall is 0.5 m.

Bottom width = 0.25(Height of the wing) = 0.25(113 – 105) = 2.0 m

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The return length can be calculated considering the profile of the stream bund and road bund.

L = (Projected width of the stream bund slope) + clearance
+ (projected length of the road bund slope)

or L = 3.75 + 1.00 + 2(4.25) = 13.25 m

The end connection details are shown in Plate 1.
Plate 1 shows the drawing details of the bridge.

EXAMPLE 5.2

A semicircular masonry arch bridge is to be constructed across a stream to carry a single broad gauge railway. The salient details available are given below. Design the arch bridge.

Span: 6 m
No of spans: three
Bed level: 100 m
HFL of the stream: 102.50 m
General ground level: 103.20 m
Springing level: 103.50 m
Railway formation level: 109 m
Stream bed width: 26 m
Slope of the stream bund: 1: 1.5
Slope of the railway bund: 1:2

Design of the Superstructure

Rise of the arch

$$r = \frac{\text{span}}{2} = \frac{6}{2} = 3 \text{ m}$$

Radius of the arch

$$R = \frac{S^2 + 4r^2}{8r} = \frac{6^2 + 4 \times 3^2}{8 \times 3} = 3 \text{ m}$$

Thickness of the arch ring

$$t = \frac{\sqrt{R + 0.5S}}{7.0} + 0.06 = \frac{\sqrt{3.0 + 0.5 \times 6}}{7.0} + 0.06 = 0.40 \text{ m}$$

Depth of the haunch filling

$$d = \frac{r + t}{2} = \frac{3 + 0.4}{2} = 1.7 \text{ m}$$

Design of the Abutment

$$\text{Top width} = 0.6 + \frac{2}{10}R + \frac{1}{10}r$$

$$= 0.6 + \frac{2}{10}(3) + \frac{3}{10} = 1.5 \text{ m}$$

$$\text{Back batte} = \frac{S}{24r} = \frac{6}{24 \times 3} = \frac{1}{12}$$

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Since the reaction at the springing level of the arch is vertical, it is preferable to slope the front face of the abutment at 1 in 6. Thus, the base width at the bed level will be

$$= 1.5 + \frac{(1)(103.5 - 100)}{6} = 2.08 \approx 2 \text{ m}$$

Length of the abutment = Formation width + 2 kerbs

= 6 + 2 × 0.5 = 7 m

In Plate 2, the details of the railway embankment have been sketched.

Design of the Pier

Height of the pier = 103.5 – 100 = 3.5 m

A straight section may be adopted

$$\text{Width of the pier} = \frac{1}{6}(\text{span}) = \frac{6}{6} = 1 \text{ m}$$

Triangular cut water and ease water with an apex angle of 90° is provided.

Design of the Wing Walls

Splayed wing walls are provided at a splay angle of 45° (Fig. 5.8). The top width of the wing wall is 0.5 m. The top is located at the formation level of the embankment (109 m). The top

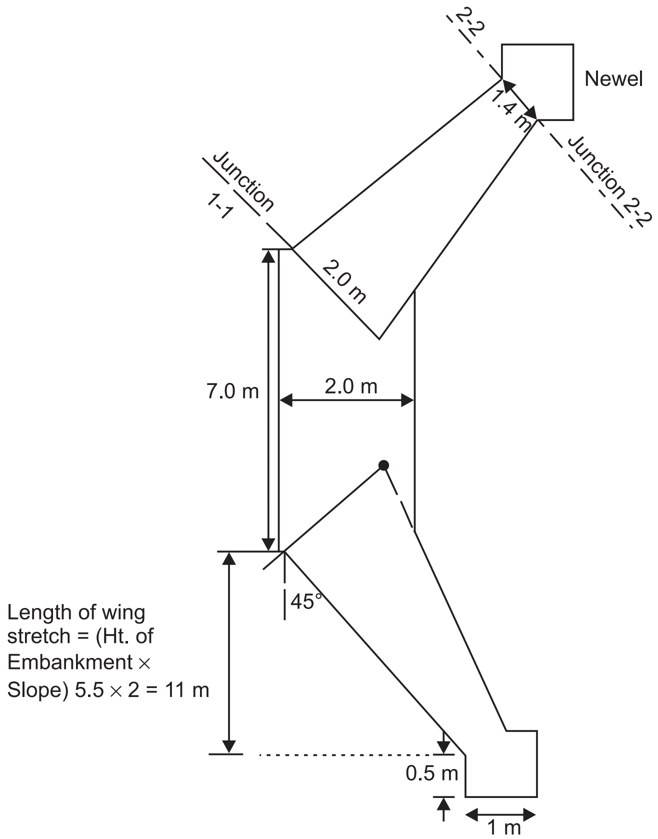

Fig. 5.8 Foundation plan of the splayed wing (Example 5.2).

gradually slopes down and joins a newel whose top is located 30 cm above the general ground level. The base width of the wing wall is the same as that of the abutment at junction 1-1 (Fig. 5.8). At junction 2-2, the height of the wing wall becomes 3.50 m (103.50 – 100).

Base width at junction 2-2 = 0.4 × 3.50 = 1.4 m

Plate 2 shows the drawing details of the bridge.

DESIGN PROBLEM

Design a segmental arch bridge to be constructed across a stream. The available data is given below:

Span length: 8 m
No. of spans: two
Road width: 7.5 m
Kerbs: 0.6 m
Top level of the road: 120 m
Bed level of the stream: 115 m
Top level of the stream bund: 116.75 m
Bed width of the channel: 22 m
Adopt splayed type of wing walls.

Also, draw to a suitable scale:

Half front elevation and half longitudinal section.
Half plan at foundation level and half top view.
Transverse section.

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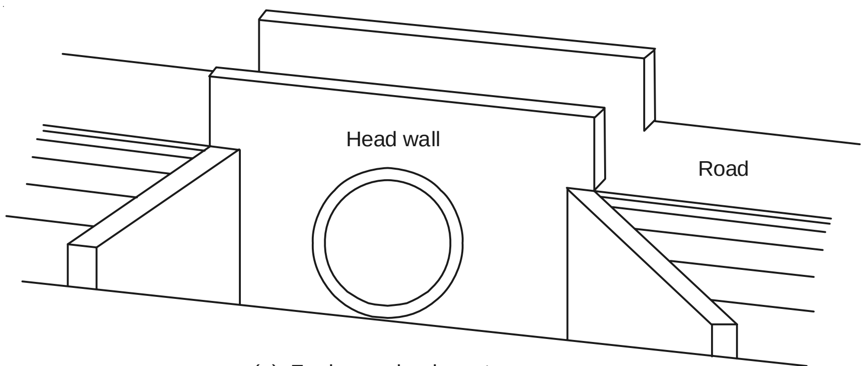
Chapter 6

Pipe Culverts

6.1 INTRODUCTION

When cross drainage flows on a relatively flat terrain with not so well-defined channel or gorge and if the discharge is limited, then pipe culverts are the best choice. At present, RCC pipes are in vogue owing to their negligible maintenance costs and ease of large-scale production of quality pipes using the spinning process.

A pipe culvert consists essentially of a pipe barrel (conveyance part) under the embankment with protection works at the entry and exit. Basically, two types of pipe culverts are constructed—pipe culverts for high embankments and pipe culverts for low embankments. In case of high embankment culverts, minimum protection works are built allowing the embankment to stand with its natural slopes. On the other hand, a low embankment pipe culvert will have two head walls on either side to envelop the embankment. Schematic views of these two types are presented in Fig. 6.1



(a) For low embankment

Fig. 6.1 (Contd.)

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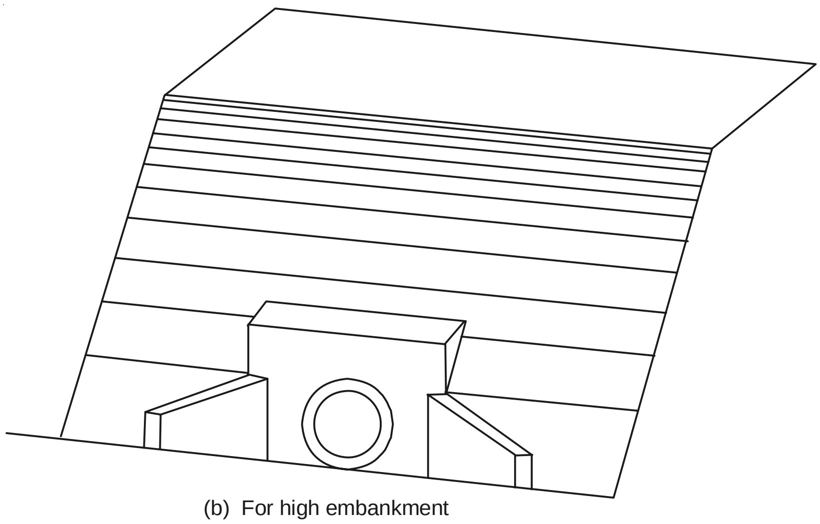

(b) For high embankment

Fig. 6.1 Types of pipe culverts.

6.2 FLOW PATTERNS IN PIPE CULVERTS

It is natural that the culvert acts as a constriction and creates a backwater effect to the approach flow, causing a pondage of water above the culvert entrance. The flow within the barrel (pipe) itself may have a free surface with subcritical or supercritical flow conditions depending on length, roughness, gradient and upstream and downstream water levels of the culvert. The various flow types that can exist in the pipe barrel of a culvert are shown in Table 6.1. The hydraulic design of the culvert is based on the characteristics of the barrel flow. The notations used are indicated in Fig. 6.2.

Table 6.1 Types of flow in the barrel of culverts

Type	*H/D*	*Exit depth* (Y_2)	*Flow type*	*Length*	*Slope(s)*	*Control*	*Remarks*
Submerged entrance							
1	> 1.0	< D	Full	Any	Any	Outlet	Pipe flow
2	> 1.2	< D	Full	Long	Any	Outlet	Pipe flow
3	> 1.2	< D	Part full	Short	Any	Outlet	Orifice
Free entrance							
4	< 1.2	< D	Part full	Any	Mild	Outlet	Subcritical
5	< 1.2	< D	Part full	Any	Mild	Outlet	Subcritical
6	< 1.2	< D	Part full	Any	Steep	Inlet	Supercritical

H = Height of the embankment

D = External diameter of the pipe

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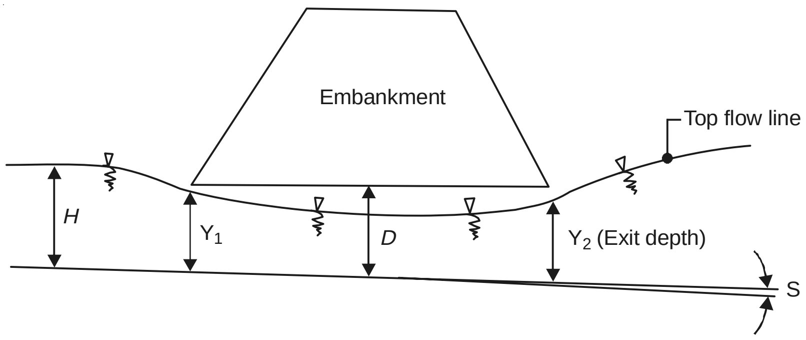

Fig. 6.2 Flow pattern in the barrel of culvert.

6.3 CULVERT ALIGNMENT

To minimize the head losses, it is necessary that the barrel (pipe) follows the natural drainage alignment and its gradient. Such an arrangement may lead to a long skew culvert which will require more complex head walls and end walls. However, it is sometimes more economical to place the culvert perpendicular to the highway with certain acceptable changes in channel alignment.

6.4 CULVERT ENTRANCE STRUCTURES

To prevent erosion of the banks and to improve the hydraulic characteristics of the culvert, the various types of recommended entrance structures (end walls and wing walls) are shown in Fig. 6.3.

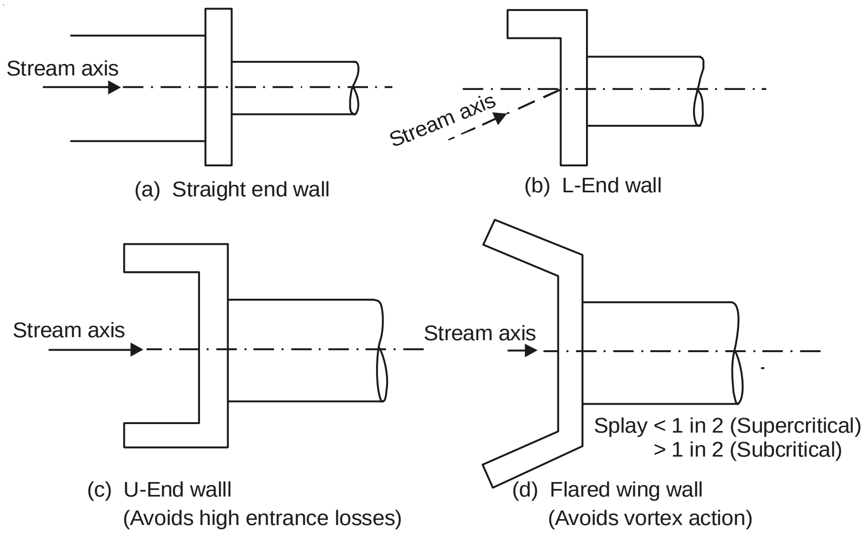

Fig. 6.3 Culvert entrance structures.

6.5 HYDRAULIC DESIGN OF PIPE CULVERTS

The hydraulic design of a pipe culvert involves working out the number of pipes required and their size. The diameter of the pipe, required to pass the given discharge, can be found by trial and error. Pipe culverts are assumed to flow full with sufficient head causing flow. The difference between the upstream water level and the downstream water level is taken as the head. The discharge through the pipe flowing full is given by the equation

$$Q = AK\sqrt{2gh} \tag{6.1}$$

where

Q = discharge in cu.m/s

A = area of the pipe in m^2

g = acceleration due to gravity, 9.81 m/s^2

h = the driving head in m; in the absence of any data it may be assumed to be 0.25 m

K = a constant (conveyance factor) that depends on the type of entry, roughness of the pipe and length of the pipe. It is given by

$$K = \frac{1}{\sqrt{1 + K_e + K_f}}$$

where

K_e = coefficient of head loss at entry

= 0.08 ≫ for bell-mouthed entry

= 0.51 ≫ for sharp-edged entry

K_f = coefficient of head loss due to friction

= 0.0033 $L/(R)^{1.3}$, for concrete pipes, where

L = length of the pipe in m

R = hydraulic mean depth in m.

6.6 STRUCTURAL DESIGN OF PIPE CULVERTS

A pipe laid beneath an embankment has to sustain the load of the earth fill, and the live load caused by the movement of vehicles. This load expressed per metre run on the pipe will be the design force for the pipe. The supporting strength of the pipe is related to the standard three edge bearing strength as follows:

$$\text{Supporting strength} = (\text{strength factor} \times \text{three edge bearing strength}) \tag{6.2}$$

The strength factor is a function of the type of bedding. The different types of beddings and their respective strength factors are given in Table 6.2. The bedding patterns are shown in Fig. 6.4.

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Table 6.2 The types of beddings and their strength factors

Type of bedding	*Strength factor*
Earth bedding	2.0
First class bedding	2.3
Concrete cradle bedding	3.7

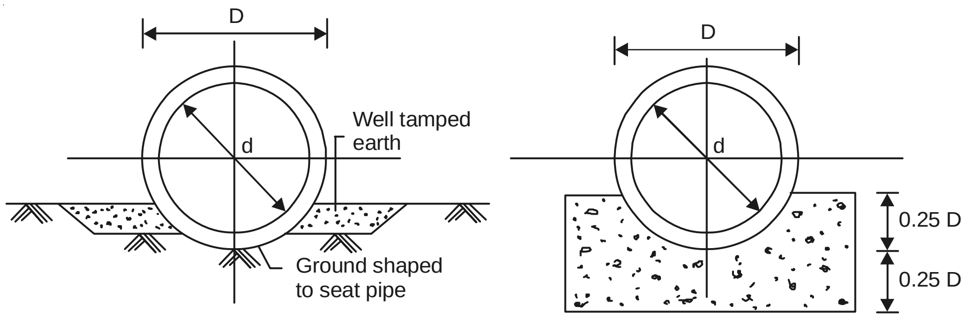

Fig. 6.4 Types of beddings.

Further, the type of bedding is so chosen that the system satisfies the following equation:

$$\frac{\text{Three edge bearing strength (kN/m)}}{\text{Factor of safety}} = \frac{\text{Weight of the filling material (kN/m)}}{\text{Strength factor}} + \frac{\text{Surface live load}}{\text{Factor of safety}}$$

The load on the pipe owing to filling material (earth) is given by the equation

$$W = C_e w D^2 \tag{6.3}$$

where

W = vertical external load in kN/m on pipe owing to embankment material

C_e = coefficient which depends on the ratio of height of the embankment to the external diameter of the pipe. These coefficients are given in IS 783–1959

w = density of the embankment material in kN/m^3

D = external diameter of the pipe in m.

Also, the load transferred on the pipe owing to concentrated wheel load on highways is given by

$$W = 4C_s IP \tag{6.4}$$

where

W = vercical load in kN/m on pipe owing to concentrated surface load

C_s = influence coefficient depending on D and H as given in Table 6.3

I = impact factor (recommended as 1.5 for highways)

P = concentrated wheel load in kN.

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Table 6.3 Influence coefficient C_s (For NP3 pipes)

Internal diameter (mm)	*External diameter* (mm)	*Height of embankment above the pipe* (m)									
		0.1	0.2	0.3	0.4	0.6	0.8	1.0	2.0	3.0	4.0
500	560	0.246	0.228	0.198	0.169	0.117	0.083	0.060	0.017	0.008	0.005
600	770	0.247	0.234	0.210	0.182	0.131	0.094	0.068	0.022	0.010	0.006
700	870	0.247	0.236	0.215	0.186	0.140	0.102	0.075	0.024	0.010	0.006
800	990	0.249	0.240	0.220	0.196	0.149	0.110	0.083	0.027	0.013	0.007
900	1100	0.249	0.241	0.225	0.202	0.156	0.117	0.089	0.029	0.014	0.008
1000	1230	0.249	0.242	0.228	0.205	0.162	0.123	0.095	0.032	0.015	0.010
1200	1440	0.249	0.242	0.230	0.209	0.171	0.131	0.104	0.036	0.020	0.011

6.7 CLASSIFICATION OF RCC PIPES

Depending on the application, RCC pipes are categorized as non-pressure and pressure types. The standards for these pipes are laid down in IS 458–1988. The utility of each category is highlighted in Table 6.4. The NP3 pipes are generally used for road culverts. The standard length of these pipes is 2.5 m. If the overall length required is not a multiple of the standard length, fractional pipe lengths may be ordered from the manufacturer.

Table 6.4 Types of pipes and their utility (IS 458–1988)

Pipe designation	*Utility*
NP1: Unreinforced concrete non-pressure pipes	For drainage and irrigation above ground or in shallow trenches.
NP2: Reinforced concrete light duty non-pressure pipes	For drainage and irrigation and for culverts carrying light traffic.
NP3: Reinforced medium duty non-pressure pipes	For drainage and irrigation and for culverts carrying heavy traffic (roads)
NP4: Reinforced heavy duty non-pressure pipes	For drainage and irrigation use and for culverts carrying heavy traffic (railways)

6.8 REINFORCEMENT IN PIPES

The pipes are reinforced with longitudinal reinforcements and hoop reinforcements. These reinforcements are designed for the field load. However, the quantity of steel used in a pipe for longitudinal and hoop reinforcements should comply with IS 458–1988. The reinforcement requirements and the ultimate three edge bearing strength for NP3 pipes are listed in Table 6.5. As per IS 458–1988, the pitch of the circumferential reinforcement shall not be more than the following:

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- ➢ 200 mm for pipes of nominal diameter from 80 to 150 mm
- ➢ 150 mm for pipes of internal diameter from 200 to 350 mm
- ➢ 100 mm for pipes of internal diameter from 400 mm and above.

Table 6.5 Reinforcement requirements for NP3 pipes (IS 458–1988)

Internal diameter (mm)	*Longitudinal steel with permissible stress of* 125 MPa (kg/m)	*Spiral reinforcement with permissible stress of* 140 MPa (kg/m)	*Ultimate three edge bearing strength* (kg/m)
350	0.78	2.95	25.16
400	0.78	3.30	28.74
450	0.78	3.79	32.34
500	0.78	4.82	35.93
600	1.18	7.01	43.11
700	1.18	10.27	50.30
800	2.66	13.04	57.48
900	2.66	18.30	64.67
1000	2.66	21.52	71.85
1100	2.66	27.99	79.00
1200	3.55	33.57	86.22

EXAMPLE 6.1

A pipe whose edge bearing resistance is 43 kN/m and internal diameter 600 mm is chosen to be used for a culvert. The earth cover on the pipe is 2 m. Two IRC Class A vehicles are to be considered for live load. If $C_s = 0.022$, $C_e = 2$ and the unit weight of soil is 18 kN/m^3, choose the bedding for the pipe.

Solution

Loading on the pipe owing to wheel load

$$\begin{aligned} W &= 4C_s IP \\ &= 4 \times 0.022 \times 1.5 \times 2(114) \qquad \text{(see Fig. 4.2)} \\ &= 30.14 \text{ kN/m} \end{aligned}$$

Loading on the pipe owing to earth fill

$$W = C_e W D^2$$

External diameter for 600 mm pipe is 770 mm (Table 6.3). Therefore

$$\begin{aligned} W &= 2 \times 18 \times 0.77^2 \\ &= 21.34 \text{ kN/m} \end{aligned}$$

The equation to be satisfied is

$$\frac{\text{Edge bearing strength}}{\text{Factor of safety}} = \frac{\text{Loading owing to earth fill}}{\text{Strength factor (SF)}} + \frac{\text{Loading owing to wheel load}}{\text{Factor of safety}}$$

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or

$$\frac{43}{1.5} = \frac{21.34}{\text{SF}} + \frac{31}{1.5}$$

Therefore,

$$\text{Strength factor (SF)} = 2.67$$

Concrete cradle bedding may therefore be chosen (see Table 6.2).

EXAMPLE 6.2

Design a pipe culvert through a road embankment of height 6 m. The width of the road is 7.5 m and the formation width is 10 m. The side slope of the embankment is l.5:1. The maximum discharge is 5 m^3/s. The safe velocity is 3 m/s. Class AA tracked vehicle is to be considered as live load. Assume bell-mouthed entry. Given C_e = 1.5, C_s = 0.010 and the unit weight of the soil = 20 kN/m^3.

Solution

Hydraulic design

Discharge through the pipe

$$Q = KAV$$

where

$$K = \frac{1}{\sqrt{1 + K_e + K_f}}$$

Now,

$$K_f = 0.0033 \frac{L}{(R)^{1.3}}$$

where L is the length of the pipe, which is equal to the base width of the embankment. Therefore

$$L = 10 + (2 \times 1.5 \times 6) = 28 \text{ m}$$

Assuming 1 m diameter pipe, we have

$$R = \frac{A}{P} = \frac{\frac{\pi}{4} D^2}{\pi D} = \frac{D}{4} = \frac{1}{4} = 0.25$$

Therefore,

$$K_f = \frac{0.0033 \times 28}{(0.25)^{1.3}} = 0.56$$

and

$$K_e = 0.08 \text{ for bell-mouthed entry}$$

Therefore, we have

$$\text{Conveyance factor} = \frac{1}{\sqrt{1 + 0.08 + 0.56}} = 0.78$$

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Hence,

$$5 = A \times 0.78 \times 3$$

or

$$A = 2.13 \text{ m}^2$$

Area provided by each pipe

$$= \frac{\pi D^2}{4} = \frac{\pi \times 1^2}{4} = 0.785 \text{ m}^2$$

Therefore the no. of pipes required

$$= \frac{2.13}{0.785} = 2.71 \approx 3$$

Bedding for the pipes

From Table 6.3, for a pipe of internal diameter 1 m, the external diameter is 1.23 m. Therefore, Height of the embankment over the pipe = (6 – 1.23) = 4.8 m

As C_e = 1.5, therefore, the load on the pipe owing to earth fill

$$C_e w D^2 = 1.5 \times 20 \times 1.23^2$$
$$= 45.4 \text{ kN/m}$$

and load on the pipe owing to wheel load

$$4 C_s I P = 4 \times 0.010 \times 1.5 \times 700 \qquad \text{(see Fig. 4.1)}$$
$$= 42 \text{ kN/m}$$

Bedding is chosen based on the strength factor. Referring to IS 458–1988, three edge bearing strength for a NP3 pipe of 1000 mm internal diameter is 72 kN/m. Hence the equation to be satisfied is

$$\frac{\text{Three edge bearing strength (kN/m)}}{\text{Factor of safety}} = \frac{\text{Load owing to earth fill (kN/m)}}{\text{Strength factor (SF)}} + \frac{\text{Load owing to wheel load}}{\text{Factor of safety}}$$

or

$$\frac{72}{1.5} = \frac{45.4}{\text{SF}} + \frac{42}{1.5}$$

Therefore,

$$\text{SF} = 2.30$$

Hence concrete cradle bedding may be provided (see Table 6.2).

Reinforcements

The minimum reinforcements to be provided in the pipe according to IS 458–1988 (Table 6.5) are:

Spiral reinforcement = 21.52 kg/m

Longitudinal reinforcement = 2.66 kg/m

Weight of the 12 mm spiral (diameter = 1.1 m)

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$$= \frac{\pi \times 0.012^2 \times 7850}{4} \ (\pi \times 1.1) = 3.068 \text{ kg/m}$$

Providing 30 kg/m of spiral, no. of spirals = $\frac{30}{3.068} = 9.77 \approx 10$

c/c distance = 1000/10 = 100 mm

Providing 6 mm dia mild steel bars as longitudinal steel and providing 4 kg/m of run,

Weight of a single bar = $\frac{\pi \times 0.006^2 \times 1 \times 7850}{4} = 0.22$ kg/m

Providing at 4 kg/m, no. of bars = $\frac{4}{0.22}$ = 18.18

Spacing = $\frac{\pi \times 1100}{18.18}$ = 190.08 mm ≈ 150 mm c/c

The details of reinforcements are shown in Fig. 6.5.

The drawing of the pipe culvert is presented in Plate 3.

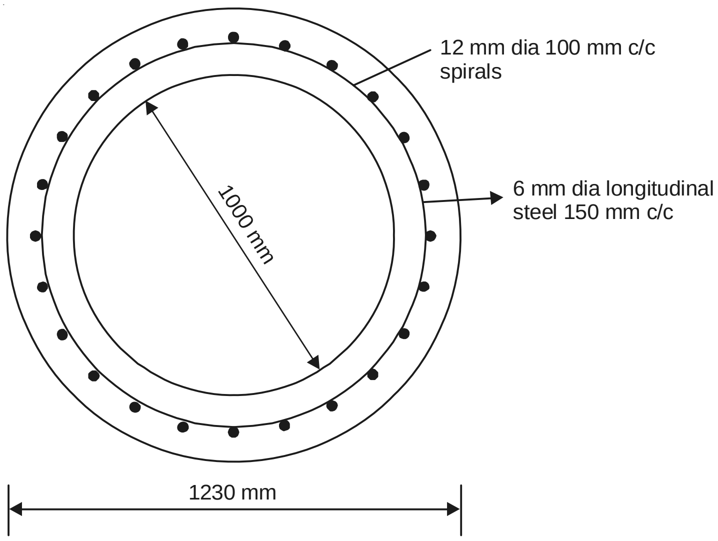

Fig. 6.5 Reinforcements details (Example 6.2).

DESIGN PROBLEMS

1. Design a pipe culvert using the following data. Also, draw the longitudinal section of the culvert.

   Discharge in the stream: 4 m^3/s

   Road top level: 106 m

   Road bottom level: 101.5 m

   Road width: 7.5 m with 1 m wide shoulders

   Density of the soil at site: 18 kN/m^3

   Loading: IRC Class A

   $C_e = 1.8$

2. Design a pipe culvert to be provided for a highway embankment across a small valley having the following data:

   Top width of the embankment: 8 m

   Height of the embankment: 4 m

   Side slope: 1.5 H: 1 V

   Maximum discharge: 6 cu·m/s

   Slope of the valley: 1/100

   Loading: IRC Class AA

   Also, draw a longitudinal section along the pipe.

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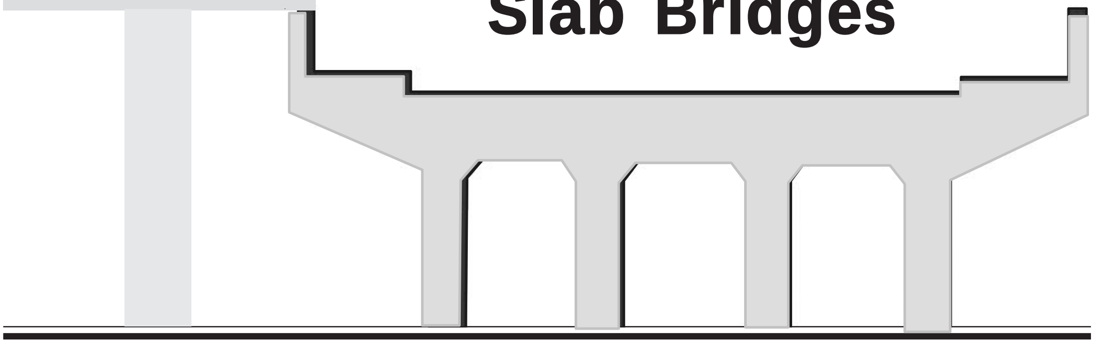
Chapter 7

Slab Bridges



7.1 INTRODUCTION

A deck slab bridge is the simplest type of construction, adopted mostly for small bridges and culverts. The span should not exceed 8 m for the bridge in order to be built at minimum cost. Though the thickness of the slab will be considerably high, its construction is simpler and the cost of the form work is also less.

7.2 WHEEL LOAD ON SLABS

A wheel load is practically considered as a concentrated load on the slab (supporting media). This load will get dispersed with its effects along spanwise and widthwise directions. Thus, the load will get distributed along a particular length (spanwise) and width (widthwise) of the slab. A mathematical model for analysis of moments and shears in respect of steel plates subjected to concentrated loads was developed by Navier. His methods were improved further. However, as these methods were not applicable to concrete slabs, semi-empirical methods were suggested by the researchers in this field. There are three methods available for analysis of slabs subjected to concentrated loads.

1. Effective width method
2. Piegeaud’s coefficient method
3. Westergaards method

The first method is applicable to one way slabs which are supported on two opposite edges, i.e. to bridges considered in this chapter. The second method is used for two way slabs which are supported on all four edges. This method is discussed in Chapter 9. Westergaards method is cumbersome and therefore rarely adopted for slab designs.

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7.3 EFFECTIVE WIDTH METHOD

As mentioned earlier, this method is applicable where one way action prevails. For this, the slab needs to be supported on only two edges, however, a very long slab may be supported on all four edges. This method is based on the observation that, it is not only the strip of the slab immediately below the load that participates in taking the load but also a certain width of the slab. This width of the slab over which the action of the load prevails is known as the *effective width of dispersion*. The extent of the effective width depends on the location of the wheel load with reference to support and dimensions of the slab. Thus, the concentrated load virtually transforms into a uniformly distributed load—distributed along some length (dispersed length along the span) and width (effective width). This is shown in Fig. 7.1. The IRC 21 recommends formulae for computing the effective width for two types of slabs, namely

1. Simply supported slabs (supported on opposite edges)
2. Cantilever slabs

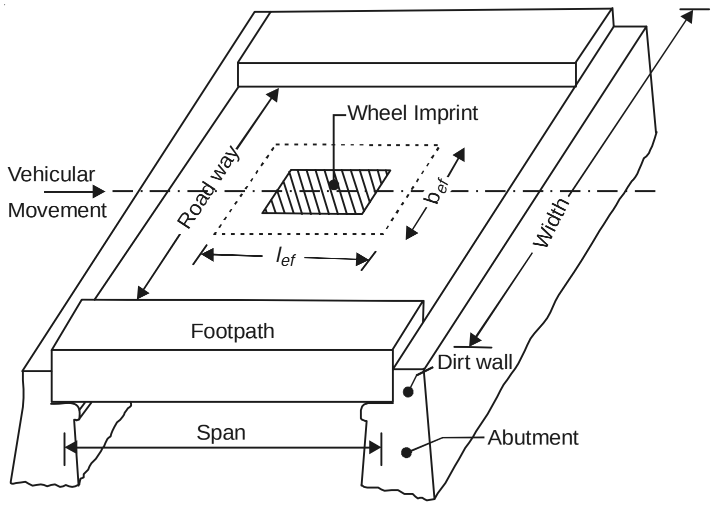

Fig. 7.1 Load dispersion on slab.

7.3.1 Slab Supported on Two Edges (Simply Supported Slabs)

For the slab supported on two edges and carrying concentrated loads, the maximum live load bending moment is calculated by considering the effective width of the slab. This effective width also called the effective width of dispersion is measured parallel to the supporting edges of the slab (Fig. 7.2). The effective width of dispersion can be estimated by using the following formula:

$$b_{ef} = \alpha x\left(1 - \frac{x}{l}\right) + b_1 \tag{7.1}$$

where

b_{ef} = width of the slab over which the load is effective

l = effective span of the simply supported slab (clear span in case of continuous slabs)

x = distance of the centre of gravity of the concentrated load from the nearest support
α = a constant having values depending on B/l values (Appended in Table 7.1)
b_1 = width of the dispersion area of the wheel load on the slab through the wearing coat. This is given by $(w + 2h)$, where h is the thickness of the wearing coat, w is the contact width of the wheel on the slab perpendicular to the direction of movement.

Table 7.1 Values of α for slabs (IRC 21)

B/l	*α for sss**	*α for cs**	*B/l*	*α for sss*	*α for cs*
0.1	0.40	0.40	1.1	2.60	2.28
0.2	0.80	0.80	1.2	2.64	2.36
0.3	1.16	1.16	1.3	2.72	2.40
0.4	1.48	1.44	1.4	2.80	2.48
0.5	1.72	1.68	1.5	2.84	2.48
0.6	1.96	1.84	1.6	2.88	2.52
0.7	2.12	1.96	1.7	2.92	2.56
0.8	2.24	2.08	1.8	2.96	2.60
0.9	2.36	2.16	1.9	3.00	2.60
1.0	2.48	2.24	2.0 and above	3.00	2.60

* sss = simply supported slab, cs = continuous slab

It is obvious that the maximum value of the effective width will be equal to the width of the slab. For two or more concentrated loads in a line, in the direction of the span, the net effective width should be calculated. A closer view of this width along span and across span is shown in Fig. 7.2.

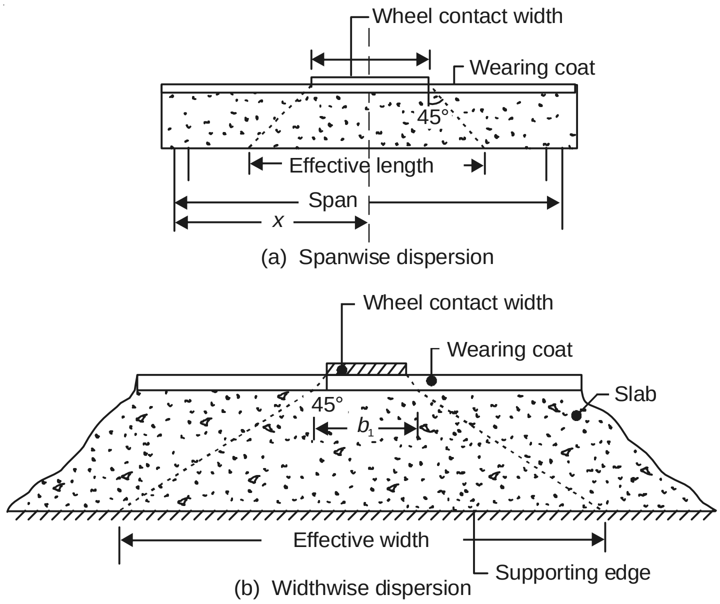

Fig. 7.2 Load dispersions.

7.3.2 Cantilever Slabs

If a cantilever slab carries wheel loads, the maximum live load bending moment is assumed to be resisted by an effective width measured along the supported edge. The formula for effective width in case of a single concentrated load is as follows:

$$b_{ef} = 1.2x + b_1 \tag{7.2}$$

where

x = distance of the centre of gravity of concentrated wheel load from the face of the fixed support

b_1 = breadth of the dispersion area of the wheel load glven by = $w + 2h$

For two or more concentrated loads, the net effective width should be calculated.

7.4 DISPERSION LENGTH

Dispersion of the wheel load along the span is known as the *effective length* of dispersion. It is also called the *dispersion length*. It can be calculated as shown below:

Dispersion length = Length of the tyre contact + (2 × overall thickness of the deck including the thickness of the wearing coat)

The other design particulars of the slab like the longitudinal reinforcement, distribution reinforcement, cover requirements, etc. are listed in IRC 21 (2000).

EXAMPLE 7.1

An RCC deck slab bridge is to be constructed over a trapezoidal channel of 6 m base width and side slopes 1:l laid at a bed slope of 0.2 m/km. The following details are available. Design the slab bridge.

Chezy's constant: 60
Bed level of the stream: 100 m
Full supply level: 101.4 m
Bottom slab level: 103.0 m
Materials: M25 concrete, Fe 415 steel
Loading: IRC Class AA (tracked vehicle)
Road width: 7.5 m
Footpath: 600 mm on either side
Wing walls: Splayed type

Computation of Linear Waterway

$$\text{Cross-sectional area} = 6 \times 1.4 + 2 \times 0.5 \times 1.4 \times 1.4 = 10.36 \text{ m}^2$$

$$\text{Wetted perimeter } (P) = 6 + 2\sqrt{1.4^2 + 1.4^2} = 9.96 \text{ m}$$

Hydraulic mean depth $R = A/P = 10.36/9.96 = 1.04$ m

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Using Chezy's formula, the velocity of normal flow in stream,

$$= c\sqrt{mi}$$

$$= 60\sqrt{\frac{0.2 \times 1.04}{1000}}$$

$$= 0.86 \text{ m/s}$$

where m is the hydraulic mean depth and i the bed slope.

The linear waterway under the bridge can be calculated by limiting the afflux to 20 cm (should be less than 30 cm). The expression for afflux is

$$x = \frac{v^2}{2g}\left[\frac{l^2}{C^2 l_1^2} - 1\right]$$

where

x = afflux in water level
v = velocity of normal flow in channel
g = acceleration due to gravity
l = width of the stream at HFL = 8.8 m
l_1 = linear waterway under the bridge
C = discharge coefficient = 0.9

Substituting the above values, we have

$$0.2 = \frac{0.86^2}{2 \times 9.81}\left(\frac{8.8^2}{0.9^2 l_1^2} - 1\right)$$

From which, we get

$$l_1 = 3.9 \text{ m} \approx 4 \text{ m}$$

Design of the Slab

Design constants

For M30 concrete and Fe 415 steel, taking basic stress values from IRC 21 (2000), we have

m = modular ratio = 10

n = neutral axis constant = $\dfrac{10 \times 10}{10 \times 10 + 200} = 0.333$

j = lever arm constant = 1 – 0.333/3 = 0.88
q = moment of resistance constant = 0.5 × 10 × 0.88 × 0.333 = 1.465 ≈ 1.5

Depth of the slab and effective span

Assume the thickness of the slab to be 80 mm per metre span of the bridge deck.

[*Note:* Thickness may be assumed in the range of 80–90 mm per metre span of the deck. The larger depth is needed for small spans in order to satisfy the shear criteria.]

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Overall depth of the slab = 4 × 80 = 320 mm

Using 20 mm diameter bars with a clear cover of 40 mm, we have

Effective depth of the slab d = 320 – 40 – 10 = 270 mm

Bearing width of 300 mm is taken if the span is less than or equal to 3 m, and bearing width of 400 mm is taken if the span is more than 3 m.

Effective span is the least of

(i) Clear span + Effective depth = 4 + 0.27 = 4.3 m

(ii) Clear span + Bearing width = 4 + 0.4 = 4.4 m

Effective span is therefore taken as 4.3 m.

Dead load bending moment and shear force

A wearing coat of 80 mm thickness is assumed.

Dead load of the slab = 0.32 × 24 = 7.68 kN/m^2

Dead load of the wearing coat = 0.08 × 22 = 1.76 kN/m^2

9.44 kN/m^2

$$\text{Dead load bending moment} = \frac{Wl^2}{8}$$

$$= \frac{9.44 \times 4.3^2}{8} = 21.81 \approx 22 \text{ kN} \cdot \text{m}$$

$$\text{Dead load shear force} = \frac{9.44 \times 4.3}{2} = 20.3 \text{ kN}$$

Live load bending moment and shear force

Impact factor is 25% for spans up to 5 m, linearly reducing to 10% for a span of 9 m.

For 4.3 m span, the impact factor is 25%.

To obtain the maximum bending moment, the wheels of the IRC AA vehicle are symmetrically placed on the slab as shown in Fig. 7.3. The load disperses through the slab in the direction of the span at an angle of 45°. Therefore,

Dispersed wheel load length = Length of the contact (see Fig. 4.3) + 2 (overall thickness of the slab)

= 3.6 + 2(0.36 + 0.08) = 4.5 m

As load dispersion has gone out of the slab span by 0.2 m, the dispersion length can be taken equal to the span of the slab. The intensity of the load actually transmitted to the slab should be reduced proportionally as the slab does not provide complete dispersion of the load.

$$\text{Proportional load to be considered} = \frac{4.3 \times 700}{4.5} \text{ (see Fig. 7.3)} = 684 \text{ kN}$$

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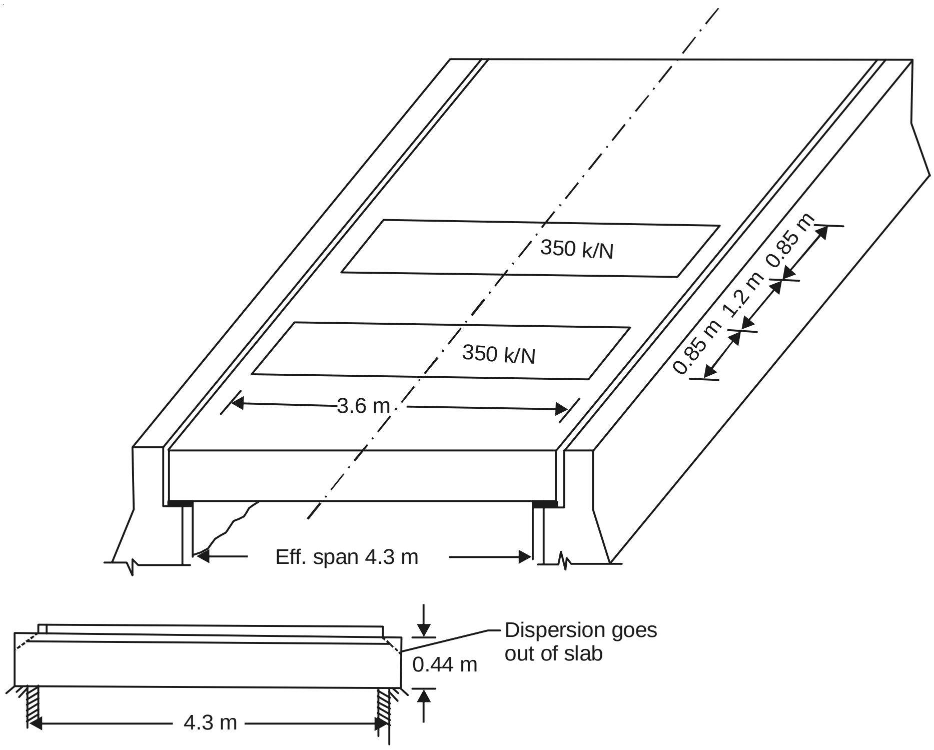

Fig. 7.3 Arrangement of IRC Class AA wheels (Example 7.1).

The effective width of dispersion is

$$b_{ef} = \alpha x\left(1 - \frac{x}{l}\right) + b_1$$

Width of the slab (B) = 7.5 + 2 × 0.6 = 8.7 m

Therefore,

$$k = B/l = 8.7/4.3 = 2.02$$

From Table 7.1, we have

$$\alpha = 3.00$$

$$x = 4.3/2 = 2.15 \text{ m}$$

$$b_1 = 0.85 + 2 \times 0.08 = 1.01 \text{ m} \qquad \text{(see Fig. 7.3)}$$

Therefore,

$$b_{ef} = 3.00 \times 2.15\left(1 - \frac{2.15}{4.3}\right) + 1.01 = 4.24 \text{ m}$$

This is for a single wheel only. As there are two wheels to be positioned widthwise, the net effective width must be calculated (Fig. 7.4). The net effective width for two wheels is given by

$$b_{ef} = 2.12 + 2.05 + 2.12 = 6.3 \text{ m}$$

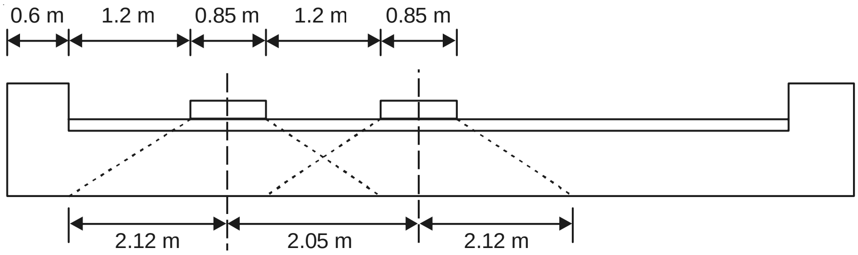

Fig. 7.4 Net effective width for two wheels (Example 7.1).

Therefore, the wheel load will have a dispersed area = 4.3 m × 6.3 m

Intensity of loading including impact factor

$$= \frac{1.25 \times 684}{4.3 \times 6.3}$$

$$= 31.56 \text{ kN/m}^2$$

Maximum live load bending moment at centre of the slab

$$= \frac{31.56 \times 4.3^2}{8}$$

$$= 72.94 \text{ kN·m}$$

Design bending moment = Dead load B.M. + Live load B.M.

= 22 + 72.94 = 94.94 ≈ 95 kN·m

To obtain maximum shear force, the load is so placed that the dispersion just touches the support, thus bringing the concentrated load nearer to the support. As the dispersion length is more than the span length in this case, the load spreads all along the span.

Therefore, the effective length is 4.3 m

For two wheels the net effective width is 6.3 m

Intensity of loading is 31.56 kN/m^2

Live load shear force 31.56 × 4.3/2 = 67.85 kN

Design shear force = Dead load shear force + Live load shear force

= 20.3 + 67.85 = 88.15 kN

Slab

$$\text{Effective depth required} = \sqrt{\frac{95 \times 10^6}{1.5 \times 1000}} = 251.66 \text{ mm}$$

Effective depth provided = 270 mm

$$\text{Area of longitudinal reinforcement} = \frac{95 \times 10^6}{200 \times 0.9 \times 270} = 1955 \text{ mm}^2$$

$$\text{The c/c distance of this reinforcement} = \frac{314.2 \times 1000}{1955} = 160.71 \text{ mm}$$

However, the c/c distance of 160 mm can be adopted. Alternate bars are bent up near the support.

Actual steel area of the steel provided = 1963 mm^2

Distribution steel should be designed for a bending moment given by

$$= 0.3 \times \text{live load moment} + 0.2 \times \text{dead load moment}$$

$$= 0.3 \times 72.94 + 0.2 \times 22$$

$$= 26.28 \text{ kN·m} \approx 26.3 \text{ kN·m}$$

Effective depth available in the widthwise direction with 12 mm diameter rods,

$$= 270 - 10 - 6 = 254 \text{ mm}$$

$$\text{Area of distribution steel} = \frac{26.3 \times 10^6}{200 \times 0.9 \times 254} = 575 \text{ mm}^2$$

$$\text{The c/c distance of distribution steel} = \frac{113.1 \times 1000}{575} = 196.7 \text{ mm}$$

Distribution steel at c/c distance of 180 mm is provided.

Check for shear stress

The check for shear stress in the slab is done as per the guidelines stipulated by IRC 21 (2000).

$$\text{Nominal shear stress } \tau_v = \frac{\text{SF}}{bd}$$

$$= \frac{88.15 \times 10^3}{1000 \times 270} = 0.33 \text{ MPa}$$

As per IRC 21 (2000), for solid slabs the permissible shear in concrete shall be $k\tau_c$, where k is a factor that depends on concrete grade.

$$\rho \text{ is \% of steel} = \frac{100\,A_s}{pd} \quad [A_s \text{ is } \left(\frac{1963}{2}\right) \text{ as half the bars are bent up}]$$

$$= \frac{100 \times 982}{1000 \times 270} = 0.363$$

From table 12B of IRC 21 (2000), for

ρ = 0.363, and M30,

τ_c is 0.28 N/mm^2

k is 1.00 (table 12c)

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As per IRC 21 (2000), the permissible shear stress value τ_c for M30 grade concrete is 0.28 MPa. Therefore,

τ_c = 1.00 × 0.28 = 0.28 MPa which is slightly less than τ_v, therefore it is ok.

Design of the Kerb

The kerb may be designed for a live load of 4 kN/m^2. The minimum height of the kerb may be taken 225 mm above the road level.

The total depth of the kerb = 360 + 80 + 225
= 665 mm

Live load/metre run of the kerb = 0.6 × 1 × 4 = 2.4 kN/m

Dead load of the kerb = 0.655 × 0.6 × 24 = 9.6 kN/m

Weight of the railings = 0.5 kN/m

12.5 kN/m

$$\text{Bending moment} = \frac{12.5 \times 4.3^2}{8} = 28.89 \text{ kN.m}$$

As the kerb is also a part of the deck slab, the vehicular load will have influence in generating bending moment in the kerb. This bending moment is normally taken as 50% of the live load bending moment obtained for the slab. Therefore,

Live load bending moment generated in the kerb is = 0.5 × 72.94 = 36.47 kN·m

Therefore,

Design bending moment = Dead load B.M. + Live load B.M.
= 28.89 + 36.47
= 65.36 kN·m

$$\text{Hence, effective depth required} = \sqrt{\frac{65.36 \times 10^6}{600 \times 1.5}} = 269 \text{ mm}$$

Total depth provided = 665 mm

Using 16 mm diameter steel, the available depth = 665 – 30 – 8 = 627 mm

$$\text{Area of steel} = \frac{65.36 \times 10^6}{200 \times 0.9 \times 627} = 579 \text{ mm}^2$$

$$\text{No. of bars of 16 mm diameter} = \frac{579 \times 4}{\pi \times 16^2} = 2.87 = 3$$

Two legged stirrups of 12 mm diameter are provided at a nominal spacing of 300 mm. The longitudinal section and cross-section of the deck slab are shown in Fig. 7.5.

Design of the Substructures

The abutment and wing wall configurations are selected as per IRC standards and the details are sketched in Fig. 7.6. The detailed drawing of the bridge is shown in Plate 4. Details of the handrails for slab bridges are presented in Fig. 7.6A.

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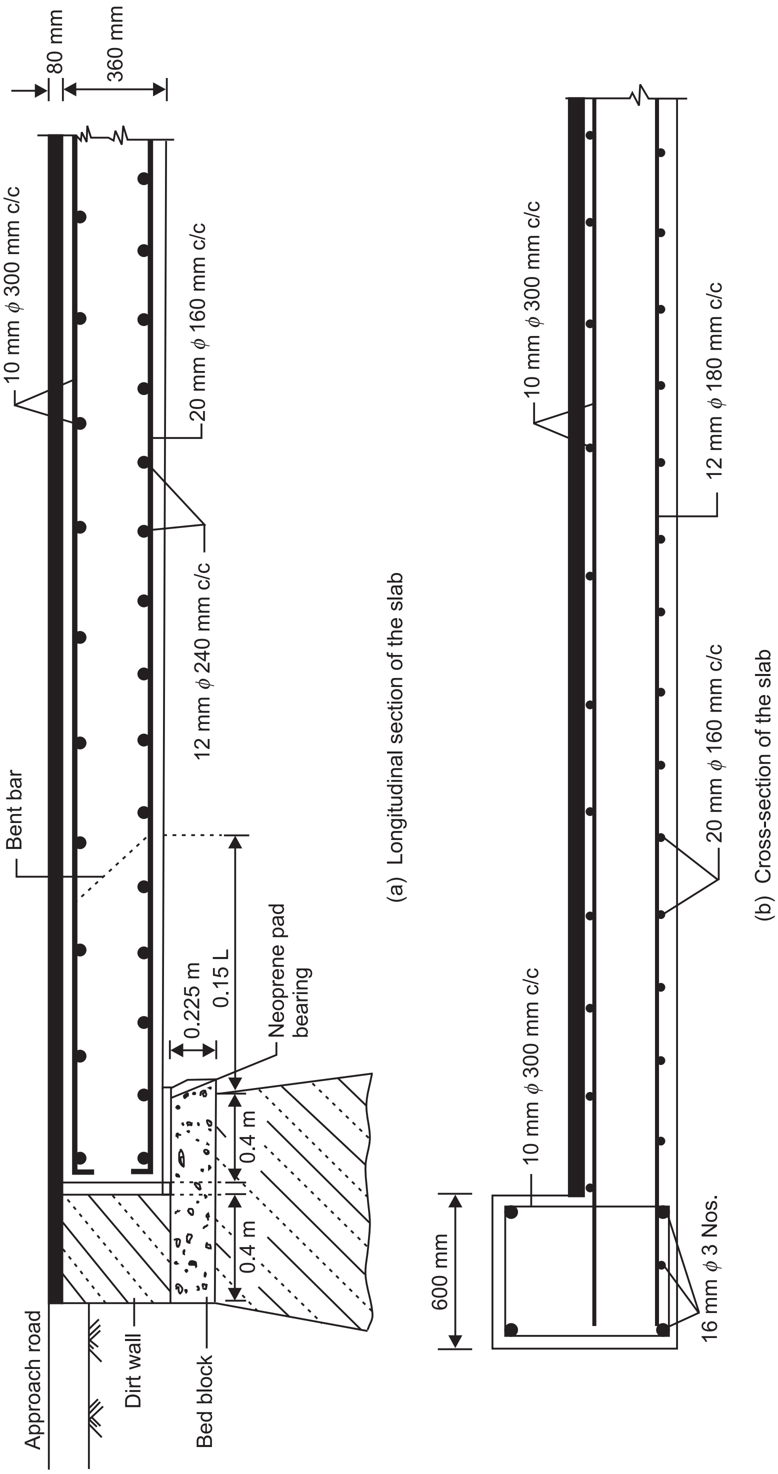

Fig. 7.5 Sectional details of the slab (Example 7.1).

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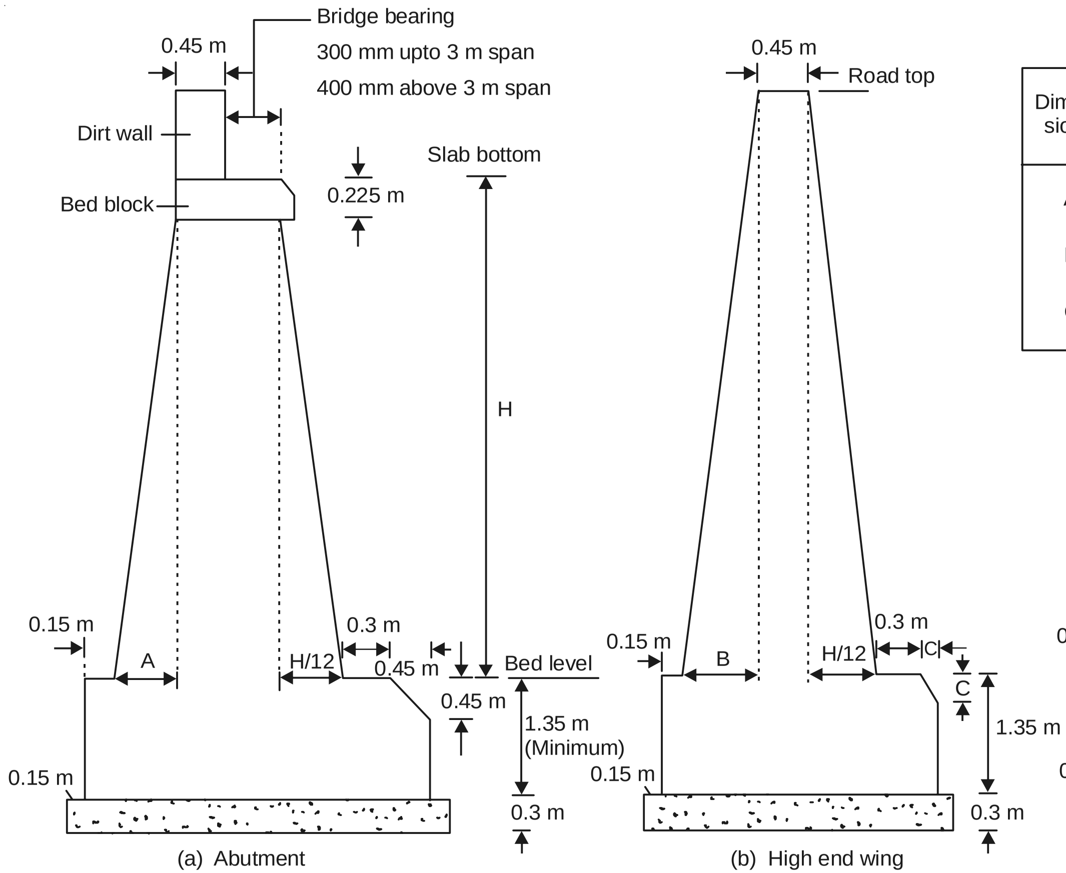

Typical Dimensions

Dimen-sions	Value of H in metres				
	1.5	2.0	2.5	3.0	3.5
A	0.75	0.90	1.20	1.65	2.60
B	0.20	0.30	0.45	0.65	1.06
C	0.45	0.45	0.45	0.45	1.45

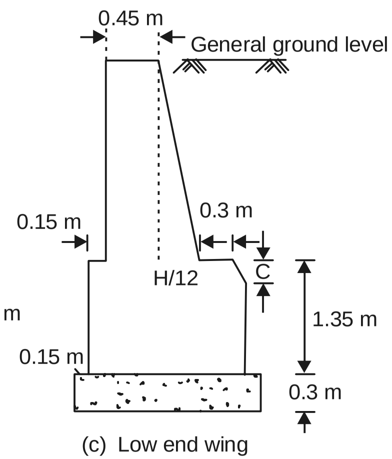

Fig. 7.6 Standard section of abutment and wing walls (Example 7.1).

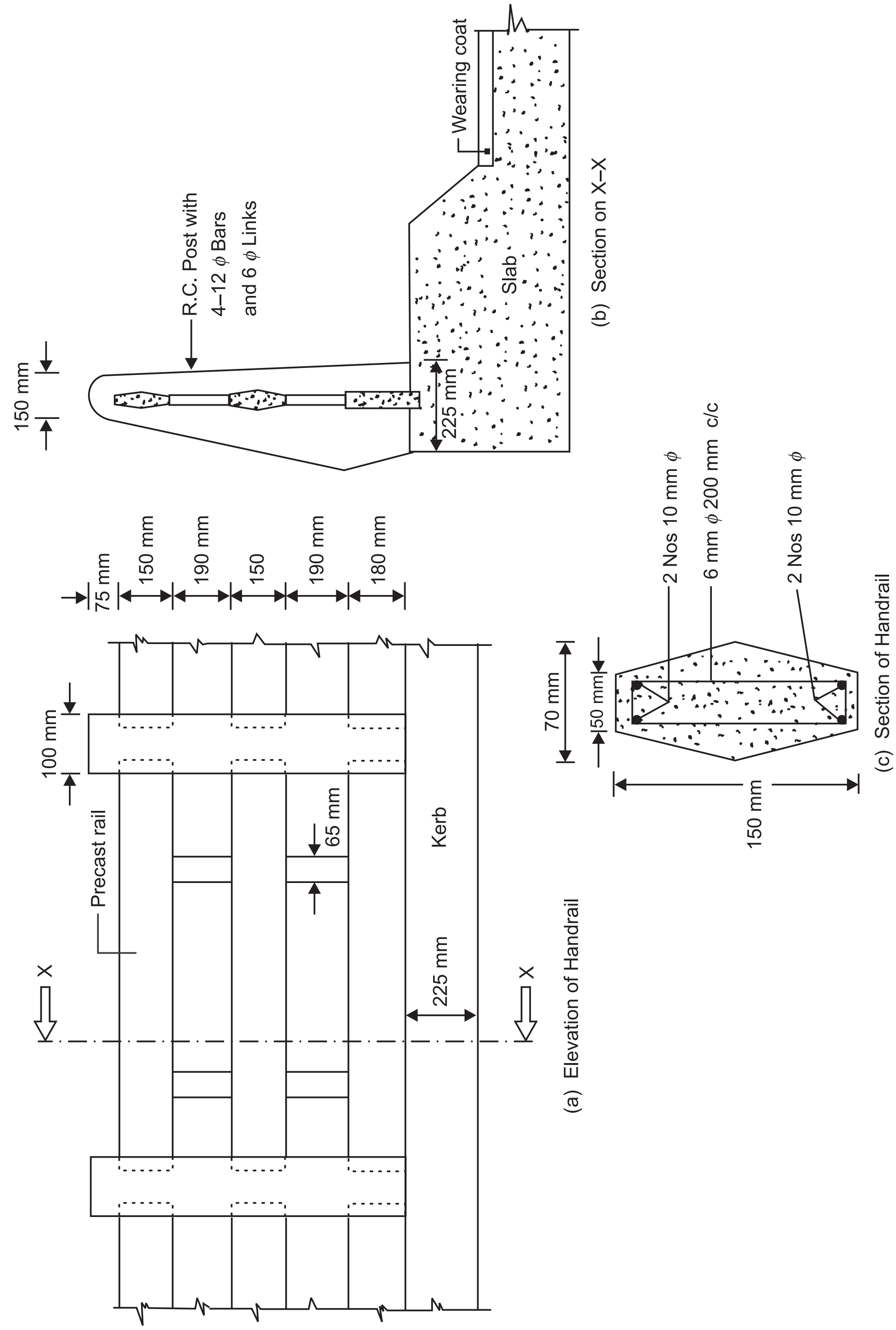

Fig. 7.6A Details of typical handrail for road bridges (Example 7.1).

EXAMPLE 7.2

Design a deck slab for the following particulars:

Clear span: 5.5 m
Width of the footpath: 1 m on either side
Wearing coat: 100 mm
Loading: IRC Class AA (Tracked)
Materials: M35 concrete and Fe 415 steel.

Design of the Slab

Design parameters

$m = 10$

$$n = \frac{10 \times 11.67}{10 \times 11.67 + 200} = 0.368$$

$j = 1 - 0.368/3 = 0.877$
$q = 0.5 \times 11.67 \times 0.877 \times 0.368 = 1.88$

Dead load bending moment and shear force

The overall thickness of the slab is assumed to be 80 mm per metre span of the deck.

Overall depth of the slab = 80 × 5.5 = 440 mm

Using 25 mm diameter bar and a clear cover of 40 mm, we have

Effective depth of the slab = 440 – 12.5 – 40 = 387.5 mm

Effective span is the least of

(i) Clear span + Effective depth = 5.5 + 0.387 = 5.887 m = 5.9 m
(ii) Clear span + Bearing width = 5.5 + 0.400 = 5.900 m

Effective span is therefore taken as 5.9 m.

Dead load of the slab = 0.44 × 24 = 10.56 kN/m^2
Dead load of the wearing coat = 0.1 × 22 = 2.20 kN/m^2
12.76 N/m^2

$$\text{Dead load bending moment} = \frac{Wl^2}{8}$$

$$= \frac{12.76 \times 5.9^2}{8} = 55.52 \text{ kN·m}$$

$$\text{Dead load shear force} = \frac{12.76 \times 5.9}{2} = 37.64 \text{ kN}$$

Live load bending moment and shear force

Width of the deck slab (B) = 7.5 + 2 × 1 = 9.5 m

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Therefore,

$$k = B/l = 9.5/5.9 = 1.61$$

From Table 7.1, $\alpha = 2.88$

Centre of gravity distance x of the wheel from the support = 5.9/2 = 2.95 m

$$b_1 = w + 2h$$
$$= 0.85 + 2 \times 0.1 = 1.05 \text{ m}$$

Effective width of dispersion for single wheel,

$$b_{ef} = \alpha x\left(1 - \frac{x}{l}\right) + b_1$$

$$= 2.88 \times 2.95\left(1 - \frac{2.95}{5.9}\right) + 1.05$$

$$= 5.3 \text{ m}$$

Effective width of dispersion for two wheels can be calculated by referring to Fig. 7.7. Thus, the net effective width for two wheels is

$$= 2.625 + 2.05 + 2.70 = 7.375 \text{ m} \approx 7.38 \text{ m}$$

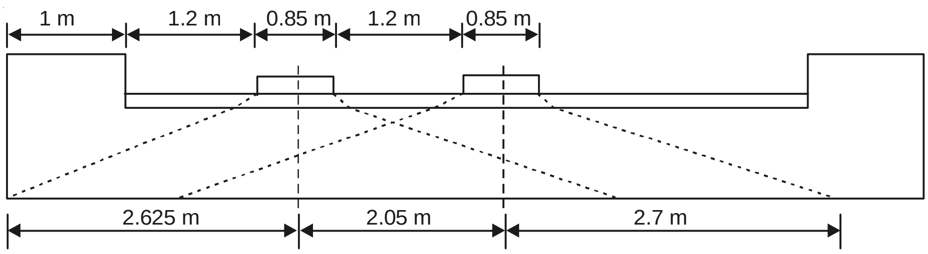

Fig. 7.7 Effective width of two wheels (for B.M.) (Example 7.2).

Effective length of dispersion = 3.6 + 2(0.44 + 0.1) = 4.68 m

Impact factor is calculated by interpolation. (Impact factor is 25% for spans up to 5 m and linearly reduces to 10% for a span of 9 m). Therefore,

$$I_f = 10 + \frac{15}{9-5} \times (9 - 5.9) = 21.63\% \approx 21.7\%$$

$$\text{Intensity of loading} = \frac{1.217 \times 700}{4.68 \times 7.38} = 24.66 \text{ kN/m}^2$$

Maximum live load bending moment occurs at the centre of the slab (Fig. 7.8)

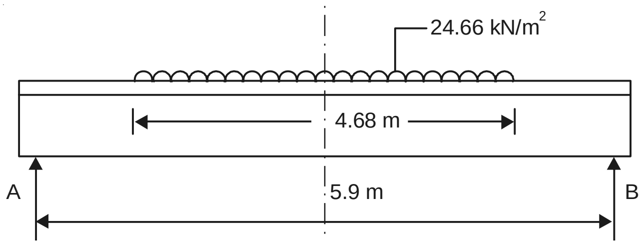

Fig. 7.8 Loading for B.M. (Example 7.2).

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Maximum live load B.M. = $\dfrac{24.66 \times 4.68}{2} \times \dfrac{5.9}{2} - 24.66 \times \dfrac{4.68}{2} \times \dfrac{4.68}{4}$ = 102.71 kN·m

Design bending moment = dead load B.M. + live load B.M.

= 55.52 + 102.71 = 158.23 kN·m

Effective depth required = $\sqrt{\dfrac{158.23 \times 10^6}{1.88 \times 10^3}}$ = 284.55 mm

Effective depth actually provided = 397.5 mm

Area of the main reinforcement

$$A_{st} = \frac{118.45 \times 10^6}{200 \times 0.9 \times 397.5} = 1655 \text{ mm}^2$$

25 mm diameter bars are spaced at 200 mm c/c.

Actual steel provided = 2448 mm^2

Bending moment for distribution steel = 0.3 × 62.93 + 0.2 × 55.52 = 29.98 kN·m

Assuming 10 mm diameter bars, the depth available in the widthwise direction

= 397.5 – 12.5 – 5 = 380 mm

$$\text{Area of distribution steel} = \frac{29.98 \times 10^6}{200 \times 0.9 \times 3.80} = 438.30 \text{ mm}^2$$

$$\text{Spacing of bars} = \frac{78.5}{438.30} \times 1000 = 179 \text{ mm say at 170 mm c/c.}$$

Cheek for shear stress

The loading should be arranged as shown in Fig. 7.9.

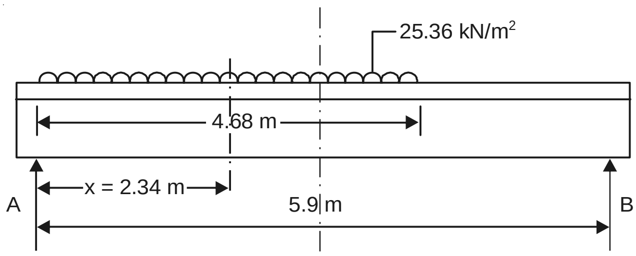

Fig. 7.9 Loading for S.F. (Example 7.2).

Distance of the centre of gravity of the concentrated load from the nearest support A

x = 4.68/2 = 2.34 m

Effective width of dispersion = 2.88 × 2.34 $\left(1 - \dfrac{2.34}{5.9}\right)$ + 1.05 = 5.12 m

Effective width for two wheels (Fig. 7.10) = 2.56 + 2.05 + 2.56 = 7.17 m

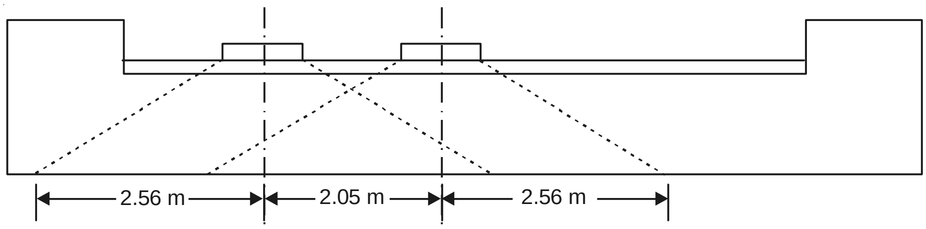

Fig. 7.10 Effective width for two wheels (For S.F.) (Example 7.2).

Intensity of loading = $\dfrac{1.217 \times 700}{4.68 \times 7.17}$ = 25.36 kN/m^2

Maximum shear force is the reaction at A (Fig. 7.9). Thus,

$$\text{Live load shear force} = \frac{(25.36 \times 4.68)(5.9 - 2.34)}{5.9} = 71.63 \text{ kN}$$

$$\text{Design shear force} = 37.64 + 71.63 = 109.3 \text{ kN}$$

$$\text{Nominal shear stress, } \tau_v = \frac{109.3 \times 1000}{1000 \times 397.5} = 0.28 \text{ N/mm}^2$$

Permissible shear stress, $\tau_c = k\tau_c$

For arriving at τ_c, we need ρ (% of steel) = $\dfrac{100 \times 1224}{1000 \times 397.5} = 0.308.$

for ρ = 0.308 and M35 concrete τ_c from table 12B of IRC 21 is 0.28 N/mm^2

τ = 1 × 0.28 = 0.28 N/mm^2, which is same as τ_v ∴ Safe.

Hence the slab is just alright against shear.

EXAMPLE 7.3

Compare bending moment and shear force values considering IRC Class A loading. The other details of the slab are the same as those of Example 7.1.

Live load bending moment

For analysing the slab for B.M. and S.F., two vehicles of IRC Class A are used. The heavier wheels are placed symmetrically with respect to the centre as shown in Fig. 7.11.

The impact factor for Class A loading is given by

$$I_f = \frac{4.5}{6 + L} = \frac{4.5}{6 + 4.3} = 0.44$$

Length of dispersion needs to be calculated for two wheels:

$$\begin{aligned} \text{Dispersion length for one wheel} &= 0.25 + 2(0.36 + 0.08) \\ &= 1.13 \text{ m} \end{aligned}$$

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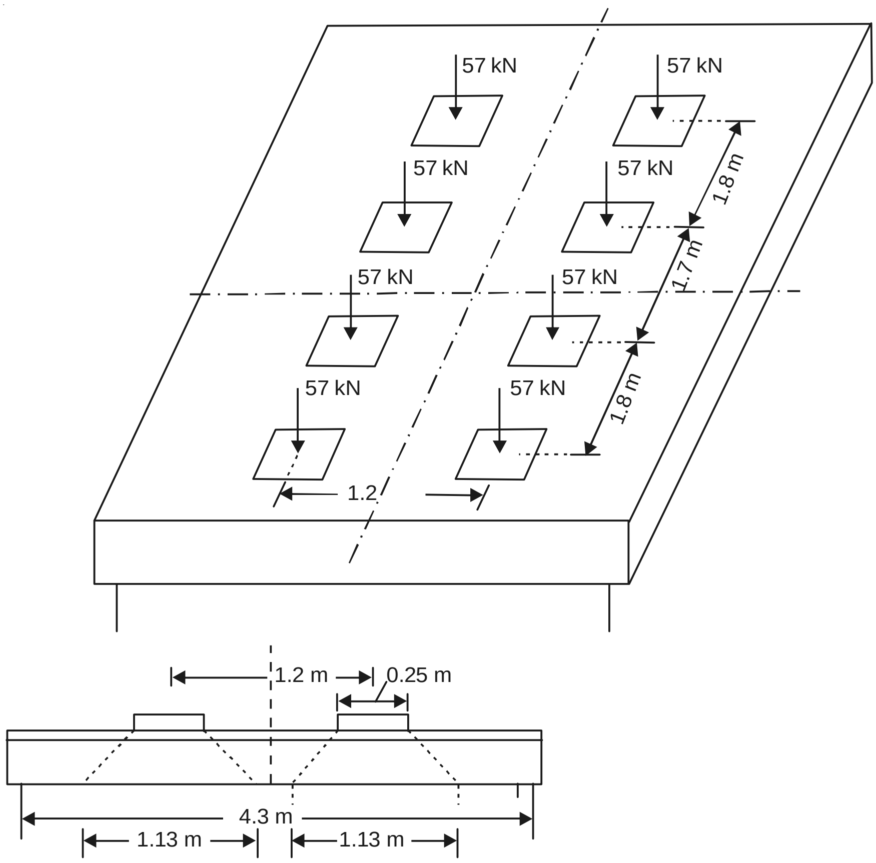

Fig. 7.11 Arrangement of IRC Class A and effective length of dispersion (Example 7.3).

Dispersion length for two wheels = $1.2 + \dfrac{1.13}{2} + \dfrac{1.13}{2} = 2.33$ m

Effective width of dispersion for a single wheel

$$= 3.0 \times 1.55\left(1 - \frac{1.55}{4.3}\right) + 0.5 + 2(0.08) = 3.63 \text{ m}$$

Effective width of dispersion for four wheels arranged along the width of the deck (see Fig. 7.12).

$$= 1.00 + 1.80 + 1.70 + 1.80 + \frac{3.63}{2} = 8.115 \text{ m}$$

Intensity of distributed load = $\dfrac{1.44 \times 4 \times 114}{8.115 \times 2.33} = 34.72$ kN/m^2

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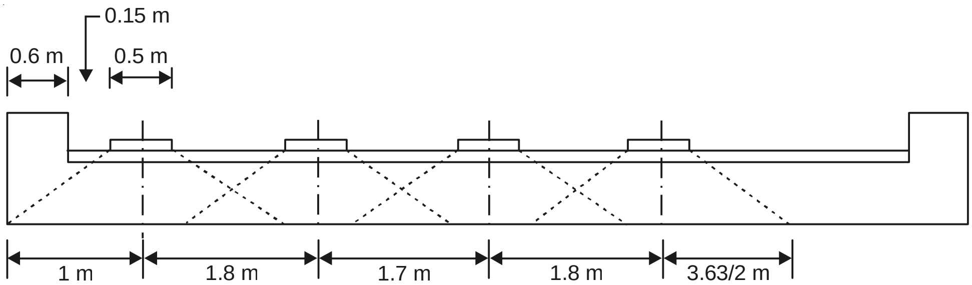

Fig. 7.12 Effective width for four IRC Class A wheels (Example 7.3).

Maximum live load bending moment

$$= \frac{34.72 \times 2.33 \times 4.3}{4} - \frac{34.72 \times 1.165 \times 1.165}{2} = 63.40 \text{ kN} \cdot \text{m}$$

This compares well with the value obtained for IRC Class AA loading.

Shear force

To obtain maximum shear force, the wheels are adjusted in such a way that the dispersion edge just touches the support (see Fig. 7.13).

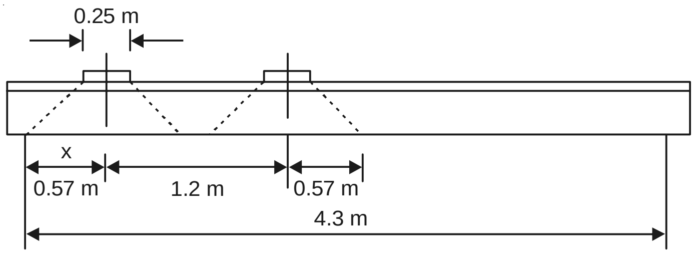

Fig. 7.13 Class A loads for maximum B.M. (Example 7.3).

Effective width of dispersion for single wheel

$$= 3 \times 0.655 \left(1 - \frac{0.655}{4.3}\right) + 0.66 = 2.325 \text{ m}$$

Effective width for four wheels (see Fig. 7.14)

$$= 1.00 + 1.80 + 1.70 + 1.80 + \frac{2.325}{2} = 7.463 \text{ m}$$

Intensity of loading

$$= \frac{1.44 \times 4 \times 114}{7.463 \times 2.33} = 37.76 \text{ kN/m}^2$$

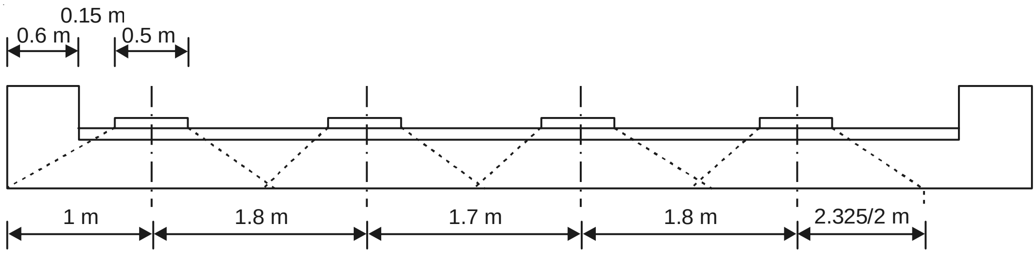

Fig. 7.14 Effective width for four IRC Class A wheels (For S.F.) (Example 7.3).

Maximum shear force which is the support reaction at A (see Fig. 7.9) is

$$= \frac{37.76 \times 2.33\,(4.3 - 0.5 \times 2.33)}{4.3} = 64.14 \text{ kN}$$

This is low compared to the shear force value of 70.95 kN for IRC Class AA loading.

DESIGN PROBLEMS

1. Design a deck slab bridge for the following data:

   Road: National Highway (Two lanes)
   Kerbs: 600 mm on either side
   Span: 6.5 m clear
   Loading: IRC70 R (Tracked wheel)
   Materials: M35 concrete, Fe 415 steel
   Abutment: Standard sections
   Wing wall: Retum type
   Bed level of the stream: 110 m
   Hard soil for foundation is available at: 107 m
   Maximum water level: 111.75 m
   General ground level at the bridge site: 113 m
   Road top level: 115.50 m

2. The following data pertains to a deck slab bridge:

   Clear distance between abutments: 6.7 m
   Road: National Highway (two lane)
   Footpath: 1 m on either side
   Wearing coat: 80 mm (average)
   Loading: IRC Class AA (tracked)
   Materials: M40 concrete, Fe 415 grade steel

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Chapter 8

Box Culverts

8.1 INTRODUCTION

If the discharge in a drain or channel crossing a road is small, and if the bearing capacity of the soil is low, then a box culvert is an ideal bridge structure. A box culvert consists of an RCC box of square or rectangular opening with span generally restricted to 4 m. The top of the box may be at road level or it may be at a depth below the road level if the road is in embankment. If the design discharge is considerable, a single box culvert becomes uneconomical because of the higher thicknesses of the slab and walls. In such cases, more than one box is cast side-by-side monolithically. A typical view of a box culvert is presented in Fig. 8.1.

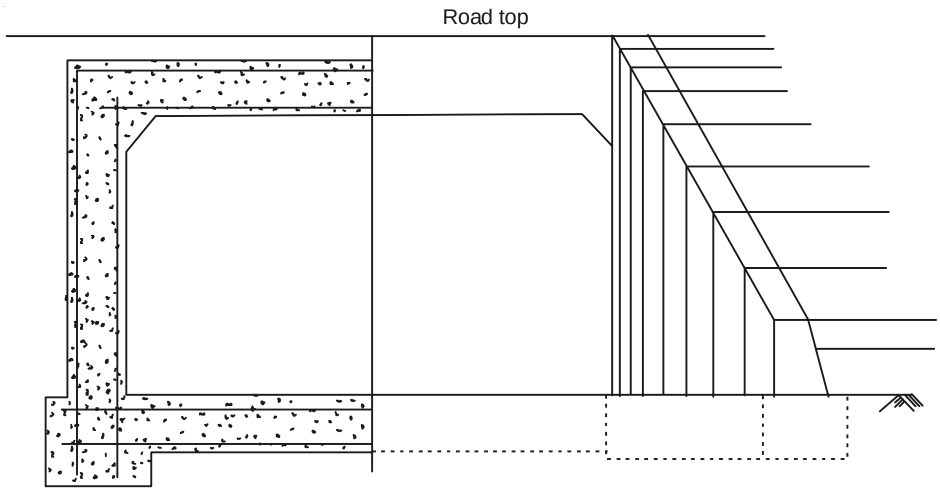

Fig. 8.1 Box culvert—half section and half elevation.

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Box culverts are economical for the reasons mentioned below:

- ➢ The box is a rigid frame structure and both the horizontal and vertical members are made of a solid slab, which is very simple in construction.
- ➢ In case of high embankments, an ordinary culvert will require very heavy abutments that will not only be expensive but also transfer heavy loads to the foundations.
- ➢ The box type of structure is suitable for non-perennial streams where scour depth is not significant but subgrade soil is weak.
- ➢ The dead load and superimposed load are distributed almost uniformly over a wider area as the bottom slab serves as a raft foundation, thus reducing pressure on soil.

8.2 DESIGN METHOD

The design of a single box culvert is done by treating the culvert as a rigid frame. The moment distribution method is generally adopted for determination of final moments at joints of the frame. The culvert is analysed for critical loading conditions. The following three conditions of loading are deemed to be critical:

1. Live load, dead load and earth pressure being present and no water pressure from inside (no flow in the drain).
2. Live load, dead load and earth pressure acting from outside and water pressure from inside.
3. Live load and dead load acting on the top slab and water pressure acting from inside and no lateral pressure due to live load.

It becomes very tedious to use the same procedure for designing a multiple box culvert. For designing multiple box culverts, the interpolation formula may be used as this is also accepted by the Ministry of Shipping and Transport (Road Wing). In this method, the design moments for any box culvert with a span range of 3–9 m can be evaluated with reference to some known values of loads and moments for box culverts of standard or known dimensions. The design of the single box culvert is presented in this chapter.

EXAMPLE 8.1

Design a box culvert having inside dimensions of 3 m × 3 m. This culvert is subjected to a dead load of 14,000 N/m^2 and a live load of IRC Class AA tracked vehicle. Assume the unit weight of soil to be 18,000 N/m^3 The angle of repose of soil is 30°. Use M25 concrete and F 415 steel. Road width is 7.5 m. Span is 3.3 m.

Design

For our design purpose, 1 m length of box culvert is considered. Analysis is done for the three critical loading cases.

Thicknesses of slab and walls are assumed as 300 mm (this thickness is normally taken as 1/10th to 1/15th of the span). Wearing coat thickness is taken as 80 mm. For analysis, the central line of the frame is considered. The frame dimensions work out to be 3.3 m × 3.3 m.

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Case 1. Dead load and live loads acting from outside, while no water pressure from inside. Concentrated vertical load owing to wheel load

$$W = \frac{PI}{b_{ef}}$$

where

P = wheel load

I = impact factor

b_{ef} = effective width of dispersion.

Effective width of dispersion for a single wheel load is

$$b_{ef} = \alpha x\left(1 - \frac{x}{l}\right) + b_w$$

Here

$$B = 7.5 \text{ m}, \quad l = 3.3 \text{ m}, \quad B/l = 7.5/3.3 = 2.27$$

From Table 4.1, IRC 21, for this B/l value, α = 2.60. Therefore,

$$b_{ef} = 2.60 \times 1.65\left(1 - \frac{1.65}{3.30}\right) + 0.85 + 2 \times 0.08 = 3.155 \text{ m}$$

Net effective width of dispersion for the tracks, after allowing for the overlap is

$$= 1.2 + 0.425 + 2.05 + \frac{3.155}{2} = 5.2525 \text{ m} \approx 5.26 \text{ m}$$

Effective length of dispersion is

$$l_{ef} = 3.6 + 2(0.3 + 0.08) = 4.36 \text{ m}$$

This length is greater than the actual span available, therefore, the intensity of live loading needs to be reduced proportionately.

Reduced load = $\dfrac{700 \times 3.3}{4.36}$ = 530 kN

Load with impact effect = 1.25 × 530 = 662.27 kN

Intensity of live load on the slab = $\dfrac{662.27}{3.3 \times 5.26}$ = 37 kN/m^2 = 37,000 N/m^2

Loads and reactions

Dead load of the top slab = 0.30 × 1 × 1 × 25,000 = 7500 N/m^2

Total load on the culvert = Dead load + Live load

= 14,000 + 37,000 = 51,000 N/m^2

Therefore,

Total design load on the top slab = 51,000 + 7500 = 58,500 N/m^2

Weight of each wall (Side Wall) = 3.3 × 0.3 × 25,000 = 24,750 N/m

Upward soil reaction at base = $\dfrac{(58,500 \times 3.3) + (2 \times 24,750)}{3.3 \times 1}$ = 73,500 N/m^2

Lateral pressure

Coefficient of active earth pressure is

$$k_a = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = \frac{1}{3}$$

Lateral pressure owing to dead and live loads = Total vertical load × k_a

$$= 51,000 \times \frac{1}{3} = 17,000 \text{ N/m}^2$$

Lateral pressure owing to soil = $k_a wh$ = $\dfrac{1}{3}$ × 18,000 = 6000 N/m^2

Lateral intensity at top = 17,000 N/m^2

Lateral intensity at bottom = 17,000 + (6000 × 3.3) = 36,800 N/m^2

Moments and shear force calculation

On account of symmetry, it is enough to consider half the frame AEFD (see Fig. 8.2) for moment distribution. As all the members are of uniform thickness and have the same dimensions, their moments of inertia are equal.

Relative stiffness of members are:

$$K_{AD} = 1, K_{AE} = K_{DF} = \frac{1}{2}$$

Distribution factors are:

$$D_{AD} = D_{DA} = \frac{1}{1 + \frac{1}{2}} = \frac{2}{3}$$

$$D_{AB} = D_{DC} = \frac{\frac{1}{2}}{1 + \frac{1}{2}} = \frac{1}{3}$$

Fixed end moments are:

$$MF_{AB} = -\frac{Wl^2}{12} = -\frac{58,500 \times 3.3^2}{12} = -53,088.75 \text{ N·m}$$

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$$MF_{DC} = +\frac{Wl^2}{12} = \frac{73{,}500 \times 3.3^2}{12} = 66{,}701{,}25 \text{ N·m}$$

$$MF_{AD} = +\frac{pl^2}{12} + \frac{Wl}{15} = \frac{17{,}000 \times 3.3^2}{12} + \frac{3.3 \times 0.5 \times 3.3 \times 19{,}800}{15} = 22{,}615 \text{ N·m}$$

$$MF_{DA} = -\frac{pl^2}{12} - \frac{Wl}{10} = \frac{17{,}000 \times 3.3^2}{12} - \frac{3.3 \times 0.5 \times 3.3 \times 19{,}800}{10} = -26{,}208.6 \text{ N·m}$$

Table 8.1 Moment distribution table (Case 1)

Joint	*D*		*A*	
Member	*DC*	*DA*	*AD*	*AB*
Distribution factor	1/3	2/3	2/3	1/3
Fixed end moments	66,701.25	–26,208.6	22,615	–53,088.75
Balance	–13,497.55	–26,995.1	20,315.83	10,157.91
Carry over		10,157.91	–13,497.55	
Balance	–3385.97	–6772	8998.33	4499.16
Carry over		4499.16	–3385.97	
Balance	–1500	–3000	2257.3	1128.65
Carry over		1128.65	–1500	
Balance	–376.21	–752.43	1000	500
Carry over		500	–376.21	
Balance	–167	–333	250.80	125.40
Carry over		125.40	–167	
Balance	–41.80	–83.60	111.33	55.66
Carry over		55.66	– 41.80	
Balance	–18.55	–37.11	28.00	14.00
Final end moments	47,715	–47,715	36,608	–36,608

For horizontal slab *AB*, carrying udl at 58,500 N/m^2, the vertical reactions at *A* and *B* are

$$= \frac{1}{2} \times 58{,}500 \times 3.3 = 96{,}525 \text{ N}$$

Similarly, for bottom slab *DC* carrying udl at 73,500 N/m^2, the vertical reactions at *D* and *C* are

$$= \frac{1}{2} \times 73{,}500 \times 3.3 = 121{,}275 \text{ N}$$

For vertical member *AD*, the horizontal reaction h_A at *A* is found by taking moments about *D*. Thus,

$$-h_A \times 3.3 + 36.608 - 57{,}715 + 17{,}000 \times 3.3 \times \frac{3.3}{2} + \frac{(36{,}800 - 17{,}000) \times 3.3 \times 3.3}{3 \times 2} = 0$$

or $$h_A = 35{,}574.24 \text{ N}$$

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Therefore, the horizontal reaction at the other end *D* is

$$h_D = \frac{(17,000 + 36,800)}{2} \times 3.3 - 35,574.24 = 53,195.76 \text{ N}$$

Free bending moment at mid-point $E = \dfrac{58,500 \times 3.3^2}{8} = 79,633$ N·m

Net bending moment at $E = 79,633.125 - 36,608 = 43,025.125$ N·m

Free bending moment at $F = \dfrac{73,500 \times 3.3^2}{8} = 100,051$ N·m

Net bending moment at $F = 100,051 - 47,715 = 52,337$ N·m

For vertical member *AD*, simply supported bending moment at mid-span is

$$\frac{17,000 \times 3.3^2}{8} + \frac{(36,800 - 17,000) \times 3.3^2}{16} = 36,617.625 \text{ N·m}$$

$$\text{Net bending moment} = \frac{47.715 + 36,608}{2} - 36,617.625 = 5544 \text{ N·m}$$

The loadings on the components of the culvert are shown in Fig. 8.2. The bending moments, shear forces and axial forces are shown in Fig. 8.3.

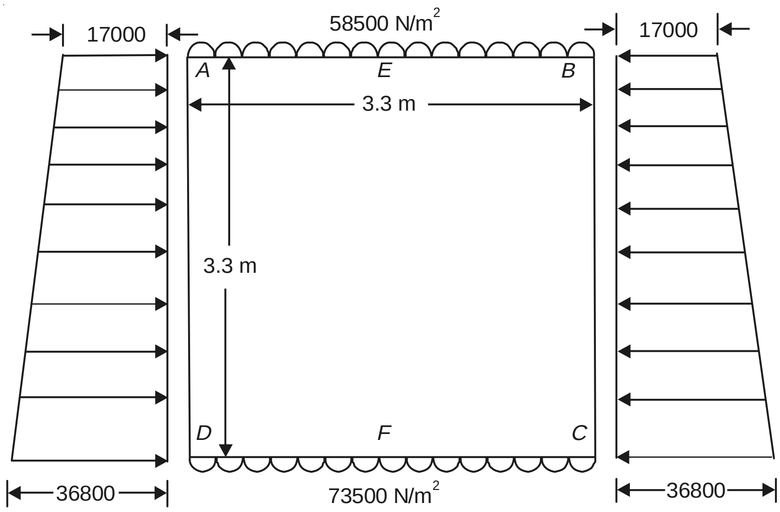

Fig. 8.2 Forces on box culvert (Case 1).

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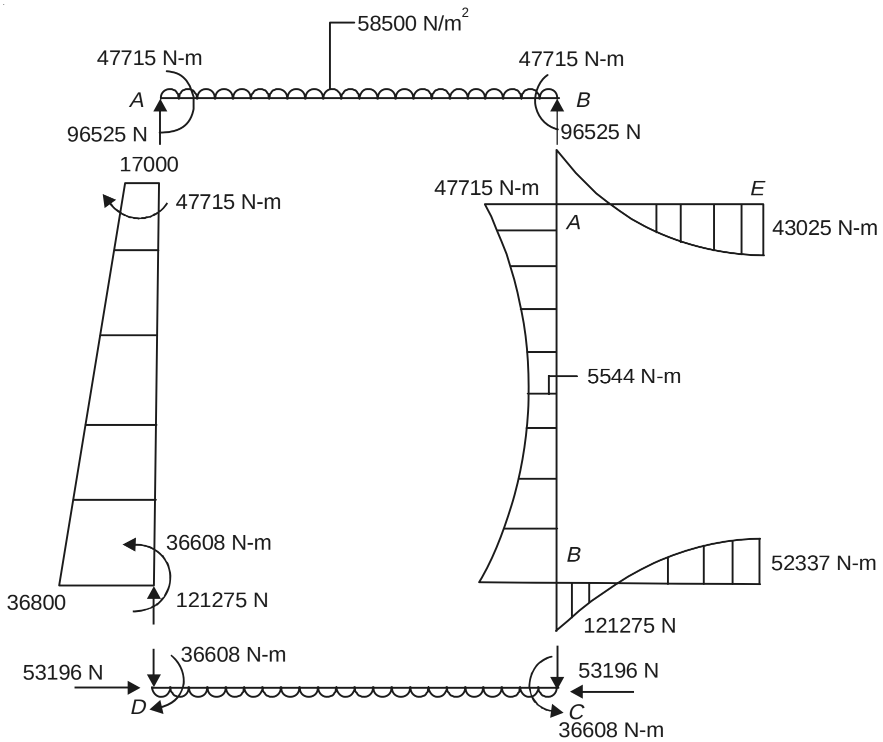

Fig. 8.3 Bending moments, shear forces and axial forces (Case 1).

Case 2. Dead load and live loads acting from outside, while water pressure acting from inside. Water pressure is zero at *A*. At *D* it has the peak value. The intensity of this pressure at *D* is

$$wh = 9800 \times 3.3 = 32{,}340 \text{ N/m}^2$$

$$MF_{AB} = –53{,}088.75 \text{ N·m}$$

$$MF_{DC} = 66{,}701.25 \text{ N·m}$$

$$MF_{AD} = \frac{4460 \times 3.3^2}{12} + \frac{12{,}540 \times 3.3 \times 3.3}{2 \times 10} = -10{,}875.48 \text{ N·m}$$

$$MF_{DA} = -\frac{4461 \times 3.3^2}{12} - \frac{14{,}440 \times 3.3 \times 3.3}{2 \times 15} = -9289.17 \text{ N·m}$$

For horizontal slab *AB*, carrying udl at 58,500 N/m^2, the vertical reactions at *A* and *B* are 96,525 N. For bottom slab *DC* carrying udl at 73,500 N/m^2, the vertical reactions at *D* and C are 121,275 *N*. For vertical member *AD*, the horizontal reaction h_A at *A* is found by taking moments about *D*. Thus,

$$-h_A \times 3.3 + 30{,}086 - 39{,}900 + \frac{4460 \times 3.3^2}{2} + \frac{12{,}450 \times 3.3 \times 2 \times 2.3}{2 \times 3} = 0$$

Table 8.2 Moment distribution table (Case 2)

Joint	*D*		*A*	
Member	*DC*	*DA*	*AD*	*AB*
Distribution factor	1/3	2/3	2/3	1/3
Fixed end moments	66,701.25	–9289.17	10,875.48	–53,088.75
Balance	–19,137.36	–38,274.42	28,142.18	14,071.09
Carry over		14,071.09	–19,137.21	
Balance	–4690.36	–9380.72	12,758.14	6379.07
Carry over		6379.07	–4690.36	
Balance	–2126.35	–4252.71	3126.90	1563.45
Carry over		1563.45	–2126.35	
Balance	–521.15	–1042.3	1417.57	708.785
Carry over		708.78	–521.15	
Balance	–236.26	– 472.52	347.43	173.71
Carry over		173.71	–236.36	
Balance	–57.90	–115.81	157.50	78.75
Carry over		78.75	–57.90	
Balance	–26.25	–52.50	38.60	19.30
Carry over		19.30	–26.25	
Balance	–6.43	–12.86	17.50	8.75
Final end moments	39,900	–39,900	30,086	–30,086

On simplification, we get

$$h_A = 18{,}179 \text{ N}$$

$$\text{Horizontal reaction at } D = h_D = \frac{17{,}000 + 4460}{2} \times 3.3 - 18{,}179 = 17{,}230 \text{ N}$$

The loading on the components of the culvert for this case is sketched in Fig. 8.4

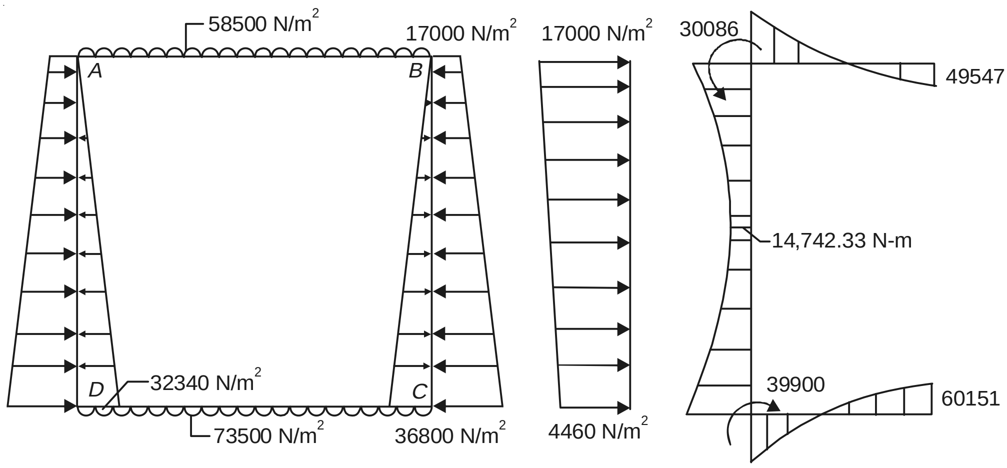

Fig. 8.4 Pressure distribution and bending moment diagram (Case 2).

Net bending moment at mid-point E = 79,633.125 – 30,086 = 49,547 N·m

Net bending moment at mid-point F = 100,051 – 39,900 = 60,151.0 N·m

For AD, simply supported bending moment at mid-span is

$$= \frac{4560 \times 3.3^2}{8} + \frac{12,540 \times 3.3^2}{16} = 14,742.33 \text{ N·m}$$

$$\text{Net bending moment} = \frac{39,900 + 30,086}{2} - 14,742 = 20,251 \text{ N·m}$$

***Case* 3.** Live load and dead load acting on top of the slab, water pressure acting from inside and no lateral pressure due to live load.

In this case no consideration is made for the lateral effect of the live load on the walls. Thus, the top of the slab is subjected to a loading of 58,500 N/m^2. The bottom of the slab has a loading of 73,500 N/m^2.

Lateral pressure owing to dead load = 14,000/3 = 4667 N/m^2

Lateral pressure owing to earth is = (18,000 × 3.3)/3 = 19,800 N/m^2

Total lateral pressure at bottom = 4667 + 19,800 = 24,467 N/m^2

In addition to these, the veritical wall will be subjected to water pressure of zero intensity at top and 32,340 N/m^2 (9800 × 3.3) at bottom. This pressure acts from inside.

Fixed end moments are:

$$MF_{AB} = -53,088.75 \text{ N·m}$$

$$MF_{DC} = 66,701.25 \text{ N·m}$$

$$MF_{AD} = \frac{pl^2}{12} - \frac{Wl}{15} = \frac{4667 \times 3.3^2}{12} - \frac{12,540 \times 3.3 \times 3.3}{2 \times 15} = -316.71 \text{ N·m}$$

$$MF_{DA} = \frac{-4667 \times 3.3^2}{12} - \frac{12,540 \times 3.3 \times 3.3}{2 \times 10} = 2592 \text{ N·m}$$

For vertical member AD, the horizontal reaction h_A at A is found by taking moments about D. Thus,

$$h_A \times 3.3 + 24,415 - 34,054 + \frac{4667 \times 3.3^2}{2} - \frac{12,540 \times 3.3 \times 3.3}{2 \times 3} = 0$$

On simplification, we get

$$h_A = 2117.30 \text{ N}$$

The horizontal reaction at D is

$$h_D = \frac{12,450 \times 3.3}{2} - 4667 \times 3.3 - 2117.30 = 5241.7 \text{ N}$$

Table 8.3 Moment distribution table (Case 3)

Joint	*D*		*A*	
Member	*DC*	*DA*	*AD*	*AB*
Distribution factor	1/3	2/3	2/3	1/3
Fixed end moments	66,701.25	2592	–317	–53,088.75
Balance	–23,097.75	–46,195.5	35,603.833	17,801.91
Carry over		17,801.91	–23,097.75	
Balance	–5934	–1186.8	15,398.5	7699.25
Carry over		7699.25	–5934.00	
Balance	–2566.41	–5132.83	3956	1978
Carry over		1978	–2566.41	
Balance	–659.33	–1318.66	1710.94	855.472
Carry over		855.472	–659.33	
Balance	–285.15	–570.31	439.55	219.77
Carry over		219.77	–285.15	
Balance	–73.25	–146.51	190.10	95.05
Carry over		95.05	–73.25	
Balance	–31.68	–63.36	48.83	24.40
Final end moments	34,054	–34,054	24,415	–24,415

Free bending moment at mid-point E = 79,633.125 N·m

Net bending moment at E = 79,633.125 – 24,415 = 55,218.125 N·m

Free bendmg moment at F= 100,051 N·m

Net bending moment at F = 100,051 – 34,054 = 65,997 N·m

For vertical member *AD*, the simply supported bending moment at mid-span is

$$= \frac{-4667 \times 3.3^2}{8} - \frac{12,540 \times 3.3^2}{16} = 2182 \text{ N·m}$$

Net bending moment = (34,054 + 24,415)/2 + 2182 = 31,416.5 N·m

The forces, moments and pressure distributions for this case of loading are shown in Fig. 8.5. Let the maximum hogging bending moment occur at a distance x below A. Therefore, we have

$$M_x = 24,415 + 2117.3x + \frac{4667x^2}{2} - \frac{12,540x}{3.3}\left(\frac{x}{2}\right)\left(\frac{x}{3}\right)$$

$$= 24,415 + 2117.3x + 2333.5x^2 - 633.33x^3$$

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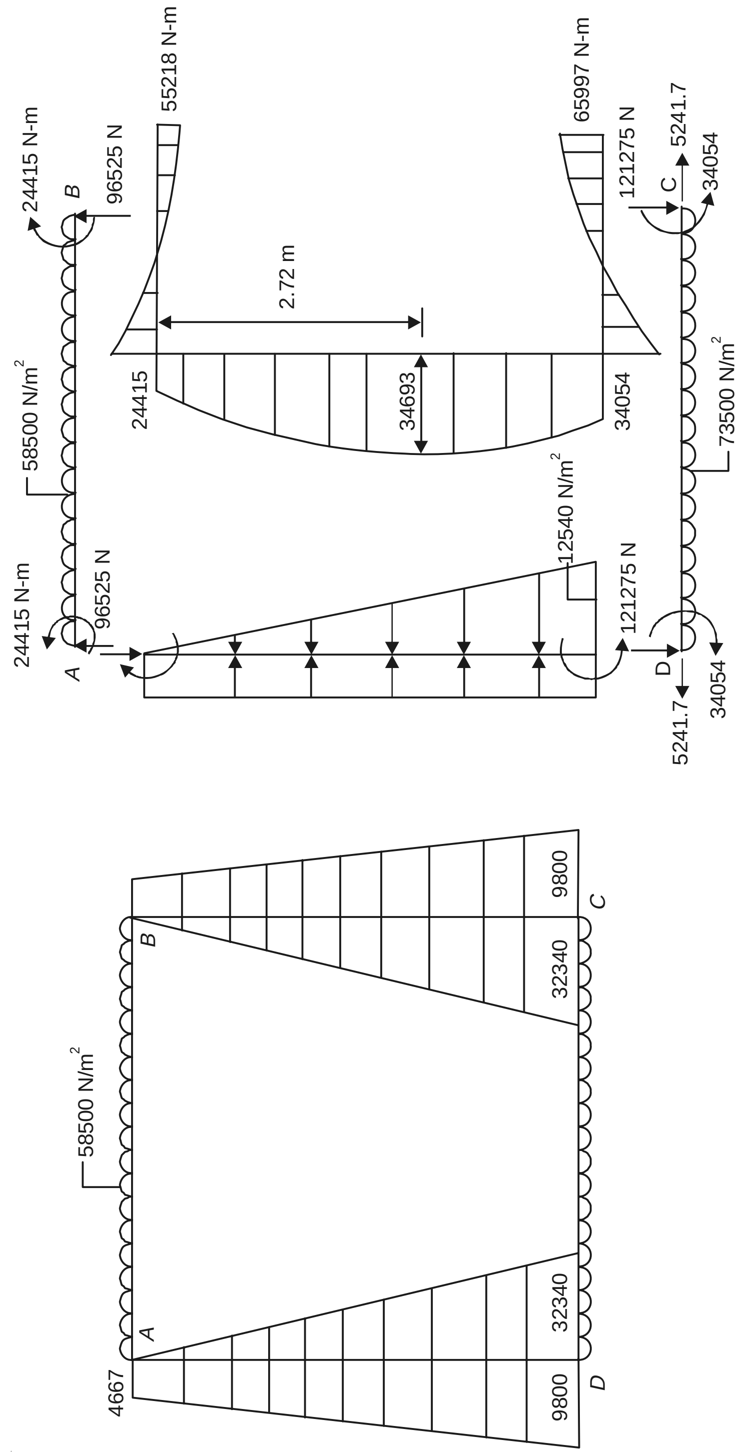

Fig. 8.5 Pressure distribution and bending moment diagram (Case 3).

For maximum hogging bending moment, we have

$$\frac{\partial M_x}{\partial x} = 0$$

or

$$2117.3 + 4667x - 3(633.33)x^2 = 0$$

or

$$1900x^2 - 4667x - 2117.3 = 0$$

or

$$x = 2.72 \text{ m}$$

Therefore,

Maximum bending moment, $M_x = 24{,}415 + 2117.3(2.72) + 2333.5(2.72)^2 - 633.33(2.72)^3$

$= 34{,}693.31$ N·m

Top slab

The top slab is subjected to the following values of bending moments and direct forces under different loading conditions (Table 8.4).

Table 8.4 Top slab design

Case	*Bending moment at centre* (N·m)	*Bending moment at ends* (N·m)	*Direct force* (N)
1	43,025.125	36,608.23	35,574.24 (C)
2	49,546.127	30,086.42	18,179.00 (C)
3	55,218.125	24,415.03	2117.30 (T)

Section has to be designed for the maximum bending moment of 55,218.125 N.m induced at the centre.

The direct tensile force of 2117.30 N is small and therefore neglected.

Overall depth = 30 mm

Effective cover = 50 mm

Effective depth = 300 – 50 = 250 mm

For M25 concrete and Fe 415 steel, $j = 0.9$, $q = 1.098$

Effective depth required = $\sqrt{\dfrac{55{,}218.125 \times 10^3}{1.098 \times 1000}} = 274.42 \text{ mm}^2$

Therefore, the depth provided is adequate.

Area of steel, $A_{st} = \dfrac{55{,}218.125 \times 10^3}{200 \times 0.9 \times 250} = 1224.34 \text{ mm}^2$

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Provide 20 mm diameter bars at 250 mm c/c.
These bars are bent alternatively near supports at a distance of $L/5$ = 3.3/5 = 0.66 m from edges.
Area of distribution steel is taken as 0.2% of the gross sectional area.

$$A_{st} = \frac{300 \times 1000 \times 0.2}{100} = 600 \text{ mm}^2$$

Area on each face is = 300 mm^2

8 mm diameter bars may be provided at a c/c distance of 130 mm on each face.
At support points additional reinforcements are required in order to cater to bending moment at the joints.
Bending moment at support = 36,608.23 N·m

$$\text{Area of steel required} = \frac{36{,}608.23 \times 1000}{200 \times 0.90 \times 250} = 811.71 \text{ mm}^2$$

Area available from bars bent up from middle section = 1256/2 = 628 mm^2
Provide additional bars of 8 mm diameter at 200 mm c/c.

Bottom slab

Moments and direct forces for the bottom slab obtained in different loading cases are tabulated below (Table 8.5).

Table 8.5 Bottom slab design

Case	*Bending moment at centre* (N·m)	*Bending moment at ends* (N·m)	*Direct force* (N)
1	52,337	47,715	53,195.76 (C)
2	60,151	39,900	17,230.00 (C)
3	65,997	34.054	5241.70 (T)

Section has to be designed for the maximum bending moment of 65,997 N·m induced at the centre. The direct tensile force of 5241.70 N may be neglected, being small.

$$\text{Effective depth required} = \sqrt{\frac{65{,}997 \times 1000}{1000 \times 1.098}} = 245.32 \text{ mm}$$

Depth provided is therefore adequate.

$$\text{Area of steel, } A_{st} = \frac{65{,}997 \times 10^3}{200 \times 0.9 \times 250} = 1466.6 \text{ mm}^2$$

20 mm diameter bars may be provided at 200 mm c/c (Actual area = 1570 mm^2). Bars are bent towards the outer face near the support. Distribution steel may be the same as that of the top slab, i.e. 8 mm diameter bars at 130 mm c/c.
Additional reinforcement is to be placed at the joints.

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$$\text{Additional area} = \frac{47{,}715 \times 1000}{200 \times 0.90 \times 250} = 1057.98 \text{ mm}^2$$

$$\text{Area available from bent bars} = \frac{1570}{2} = 785 \text{ mm}^2$$

Additional area = 1057.98 – 785 = 272.98 mm^2

8 mm diameter bars may be provided at 200 mm c/c throughout the bottom.

Side walls

The moments and direct forces at the ends of the walls are listed in Table 8.6.

Table 8.6 Design of side walls

Case	*Bending moment at centre* (N·m)	*Bending moment at ends* (N·m)	*Direct force* (N)
1	5544	44,715	121,275
2	20,251	39,900	121,275
3	31,416	34,054	121,275

The side walls can be designed using Sp 16.

Here E_u = –47,710 N·m N_u = 121,275.0 N

= 47.1 kN·m = 121.2 kN

Now

$$\frac{M_u}{f_{ck} b D^2} = \frac{47.1 \times 1.5 \times 10^6}{25 \times 1000 \times 300^2} = 0.039$$

$$\frac{N_u}{f_{ck} b D} = \frac{121.2 \times 1.5 \times 10^3}{25 \times 1000 \times 300} = 0.030$$

where

M_u = ultimate moment, in kN·m

N_u = ultimate axial thrust, in kN

f_{ck} = characteristic strength of concrete

b = unit width, in m

D = depth of wall/slab, in m

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For the ratios 0.039 and 0.030, refer to the interaction curve of Sp 16 (Design Aids—for reinforced concrete—to IS 456–1978). From this curve (p. 116, Sp 16), we get

$$\frac{P}{f_{ck}} = 0.02$$

where P is the percentage of steel, f_{ck} = 25 N/mm^2, therefore,

$$P = 0.02 \times 25$$

Also, d'/D = 50/300, where d' is effective cover for reinforcement.

Area of steel, $$A_{st} = \frac{0.5\,PbD}{100} = \frac{0.5 \times 0.02 \times 25 \times 1000 \times 300}{100} = 750 \text{ mm}^2$$

Area of steel = 2 × 750 = 1500 mm^2 (for two faces)

But the minimum reinforcement is 0.8%. Therefore,

$$A_{st} = \frac{0.8 \times 300 \times 1000}{100} = 2400 \text{ mm}^2$$

Provide 20 mm diameter bars at 300 mm c/c on both faces in the vertical walls. Distribution steel of 8 mm diameter may be placed at 300 mm c/c on both the faces. Figure 8.6 shows the details of reinforcements in the component parts of the box.

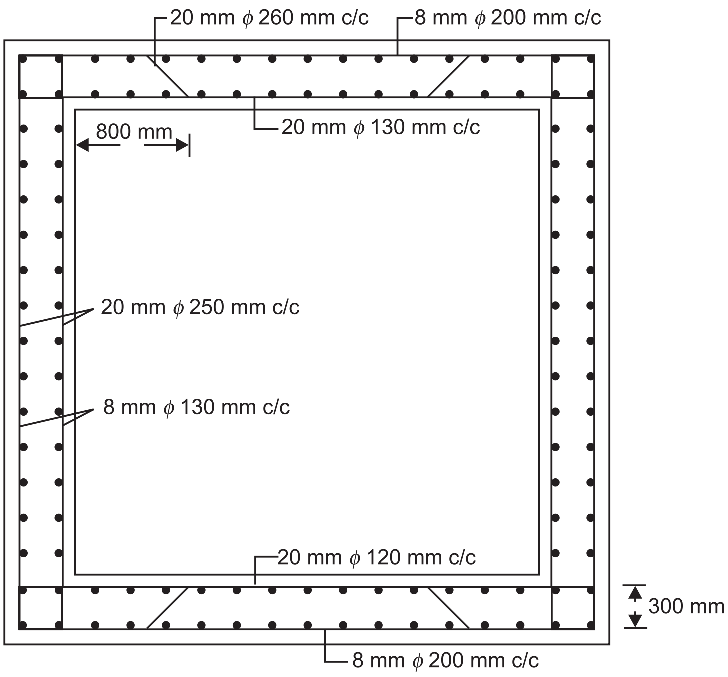

Fig. 8.6 Transverse section of the box.

DESIGN PROBLEMS

1. Design a box culvert with the following particulars:
   Inside dimensions: 3.5 m × 3.5 m
   Live load: IRC Class A
   Density of soil: 18 kN/m^3
   Angle of repose = 30°
   Materials: M25 concrete, Fe 415 grade steel.
2. Design a box culvert with the following particulars:
   Inside dimensions: 2.5 m × 3.0 m
   Loading: IRC Class AA (tracked)
   Materials: M30 concrete and Fe 415 grade steel
   Type of the road: one lane
   Height of the embankment above the box: 1.2 m
   Unit weight of soil: 16 kN/m^3
   Type of stream: Non-perennial

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Chapter 9

Beam and Slab Bridges

9.1 INTRODUCTION

A beam and slab bridge or T-beam bridge is constructed when the span is between 10–20 m. The bridge deck essentially consists of a concrete slab monolithically cast over longitudinal girders so that the T-beam effect prevails. To impart transverse stiffness to the deck, cross girders or diaphragms are provided at regular intervals. The number of longitudinal girders depends on the width of the road. Three girders are normally provided for a two-lane road bridge. A complete design of a T-beam deck would involve the design of the interior panel of the slab, longitudinal girders and cross girders. The general arrangement of the bridge components is shown in Fig. 9.1.

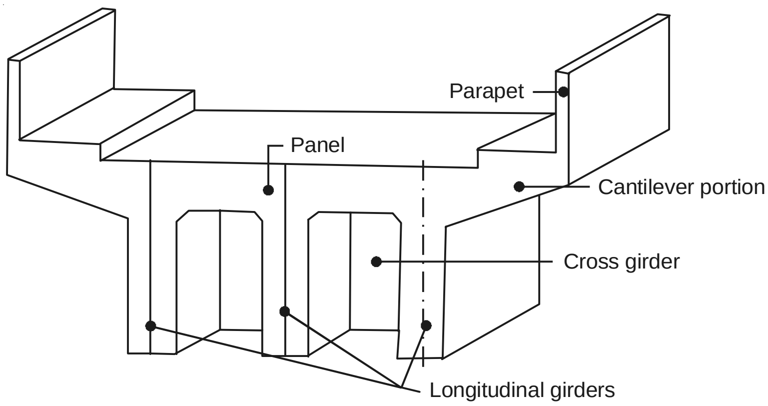

Fig. 9.1 Components of a T-beam bridge.

9.2 DESIGN OF INTERIOR PANEL OF SLAB

In a T-beam bridge deck with cross beams, the slab may be regarded as supported on all the four edges and continuous over the beams. Many methods are available for analysis of such two way slabs subjected to concentrated loads. Among them are:

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1. Rankine-Grashoff method
2. Diagonals method
3. Westergaards method
4. Pigeauds method.

The first two methods are approximate methods. The first method divides the slab into a number of strips criss-crossing each other and determines the load distribution on the basis that the deflections at the point of crossing of two strips are equal. The diagonals method assumes that the diagonals of the slab approximately define the critical sections. It further suggests that the two way reinforcement should be placed perpendicular to the diagonals. It is felt by the designers that these two methods are inconvenient because continuity action and Poisson effect are completely ignored. Westergaards method recommends moment coefficients, which have formed the basic part of the American Concrete Institutes Code. However, this method is not used in India. Pigeauds method is widely used for design of two way slabs subjected to concentrated loads.

9.3 PIGEAUDS METHOD

In this method, the short span and long span bending moment coefficients are read from curves developed by M. Pigeaud. These curves are used for slabs supported along four edges with restrained comers and subjected to symmetrically placed loads distributed over some well-defined area. These curves were developed for thin plates using the elastic flexural theory. However, their use has been extended to concrete slabs too. Poisson's ratio of 0.15 is considered. Before using the graph, certain dimensionless ratios need to be considered. The method is explained with reference to Fig. 9.2. This figure shows a wheel load being centrally placed on a panel of dimensions: B (short span) $\times$ L (long span). If b and l are the wheel contact dimensions imprinted on the slab, and if t is the thickness of the wearing coat, and as the load disperses

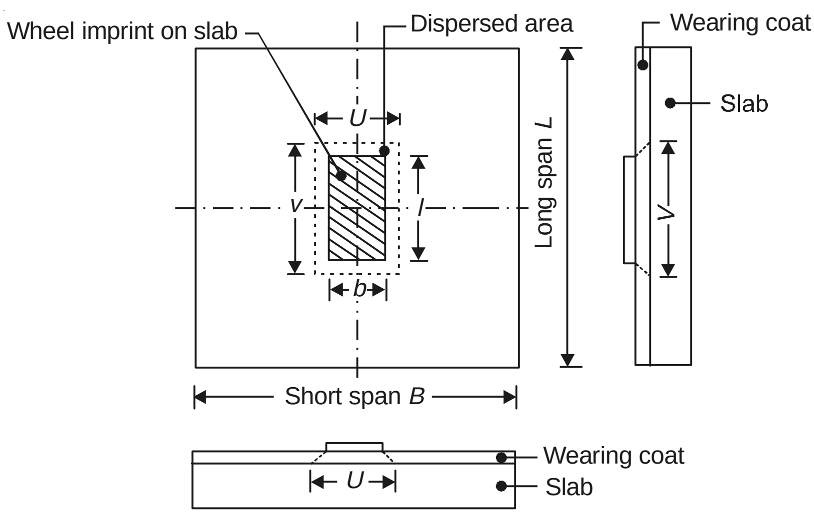

Fig. 9.2 Dispersed dimensions of wheel contact on the slab.

through the wearing coat media at an angle of 45°, the dispersed dimensions of the wheel on the slab become,

$$V = l + 2t \text{ —— along long span}$$
$$U = b + 2t \text{ —— along short span}$$

The values of moment coefficients m_1 (for short span), and m_2 (for long span) for the slab are given by Pigeauds curves for various values of the aspect ratios $(K = B/L)$ and ratios U/B and V/L. The bending moments in the slabs can then be calculated using the following equations:

$$\text{Short span B.M., } Mb = W(m_1 + 0.15m_2)$$
$$\text{Long span B.M., } Ml = W(m_2 + 0.15m_1)$$

Figures 9.3 through 9.10 give the moment coefficients for the respective K values. When the slab is supported on two edges only (no cross beams), B/L may be taken as zero and Fig. 9.10 can then be used. For square panels (B/L = 1), Fig. 9.9 can be used.

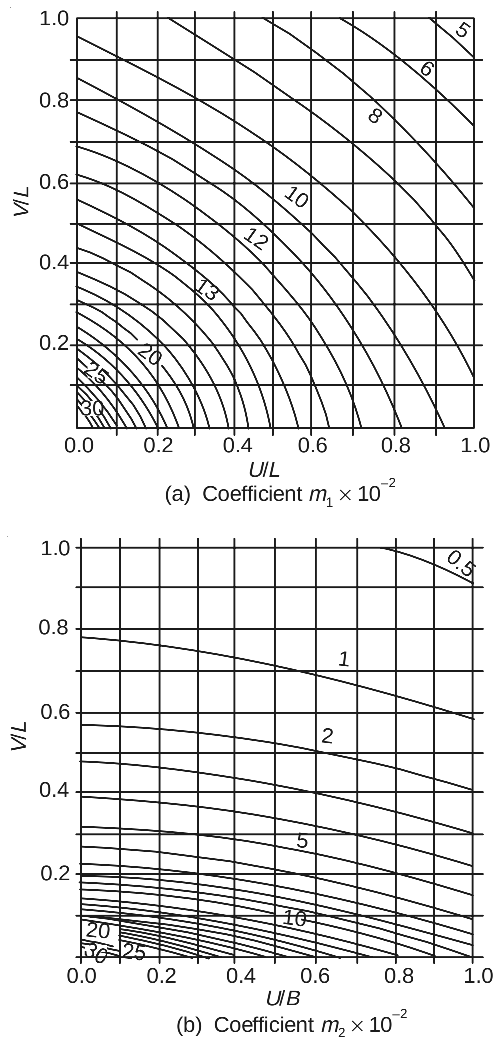

Fig. 9.3 Moment coefficients m_1 and m_2 for K = 0.4

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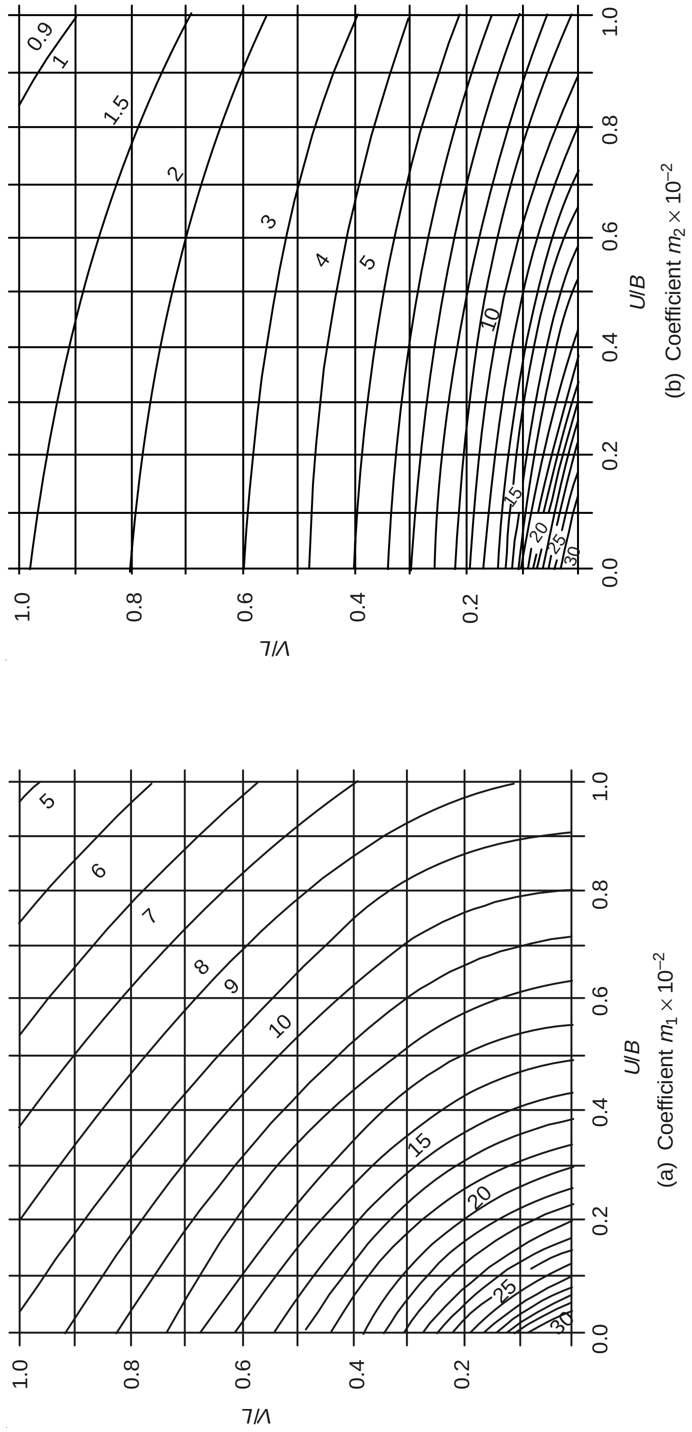

Fig. 9.4 Moment coefficients m_1 and m_2 for $K = 0.5$.

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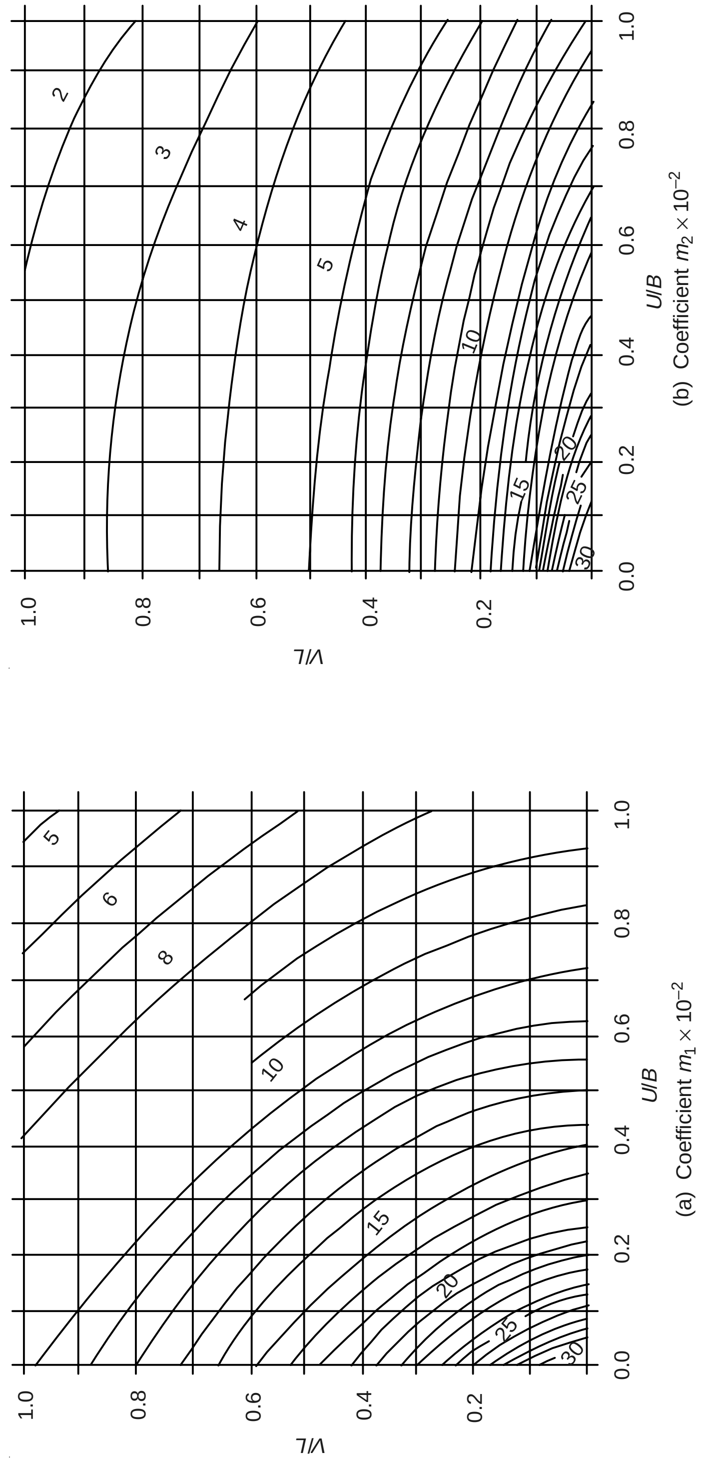
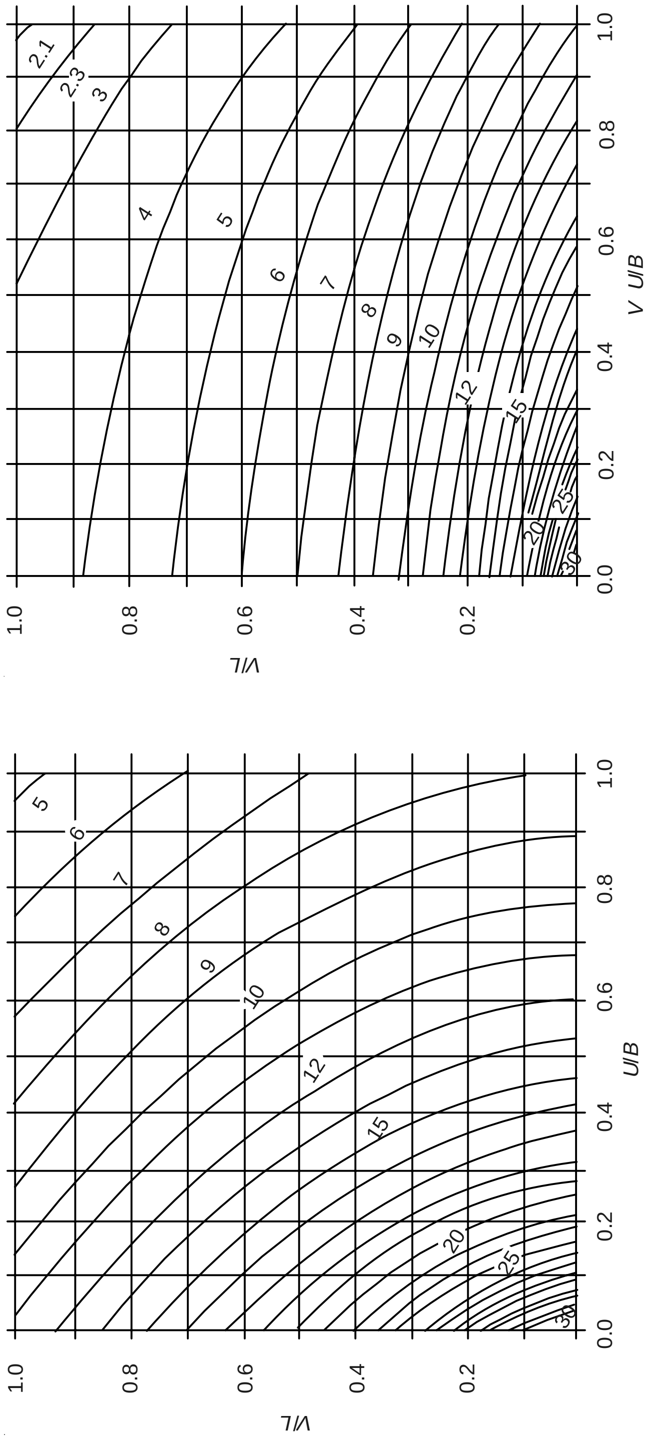


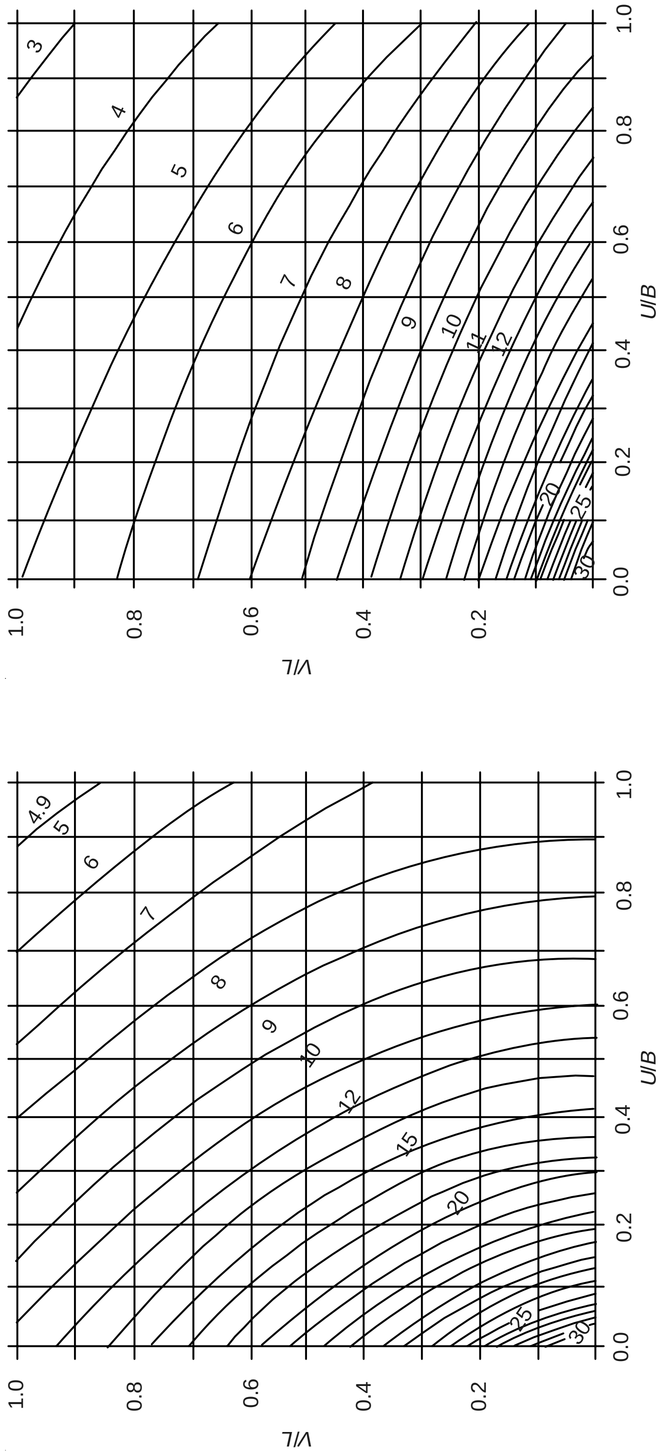
Fig. 9.5 Moment coefficients m_1 and m_2 for $K = 0.6$.



(a) Coefficient $m_1 \times 10^{-2}$

(b) Coefficient $m_2 \times 10^{-2}$

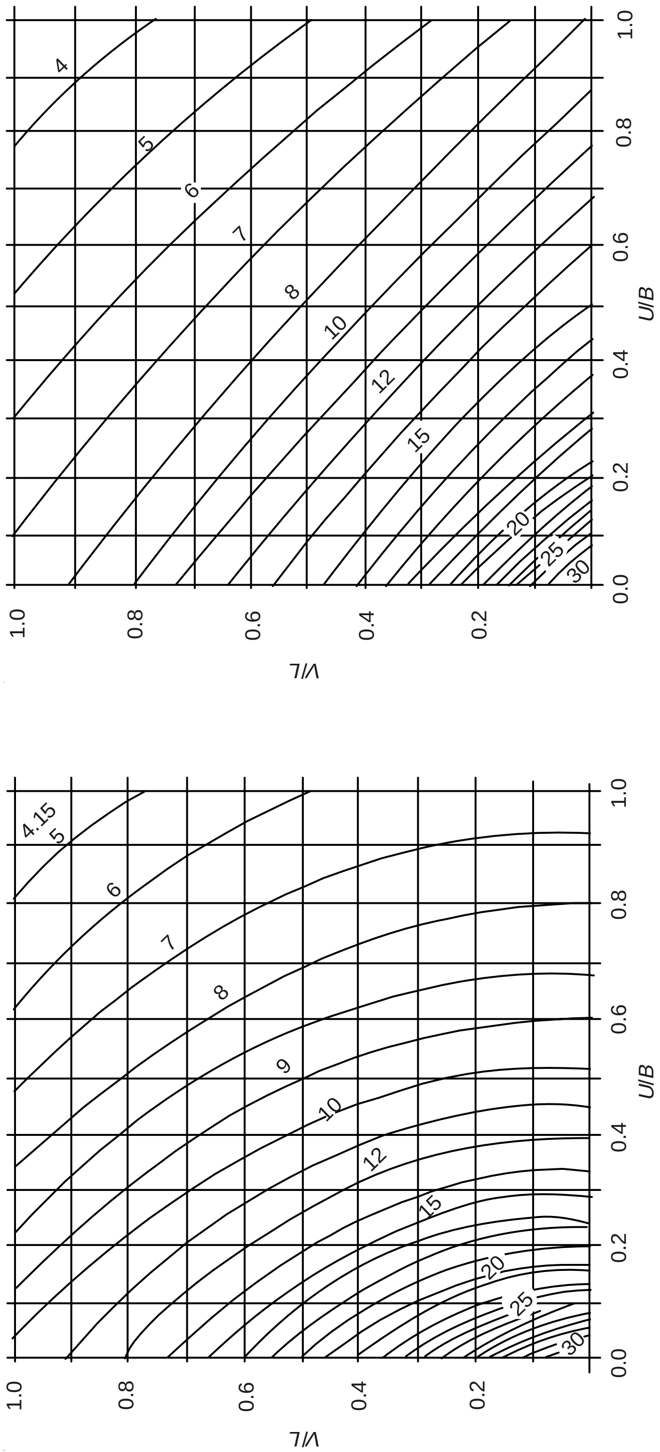
Fig. 9.6 Moment coefficients m_1 and m_2 for $K = 0.7$.



(b) Coefficient $m_2 \times 10^{-2}$

(a) Coefficient $m_1 \times 10^{-2}$

Fig. 9.7 Moment coefficients m_1 and m_2 for $K = 0.8$.



(b) Coefficient $m_2 \times 10^{-2}$

(a) Coefficient $m_1 \times 10^{-2}$

Fig. 9.8 Moment coefficients m_1 and m_2 for K = 0.9.

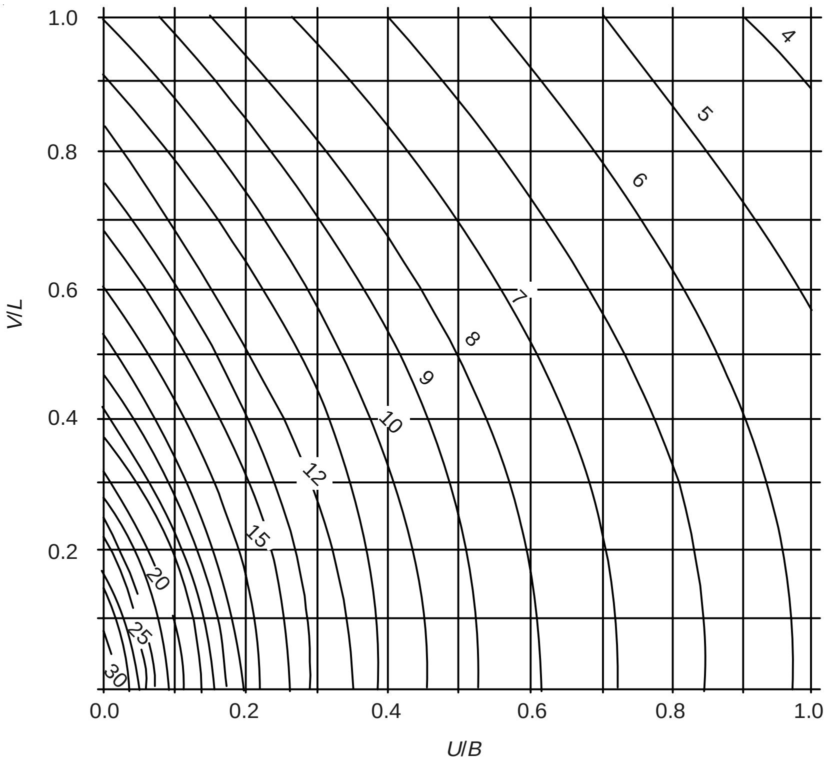


Fig. 9.9 Moment coefficients m_1 and m_2 for $K = 1.0$.

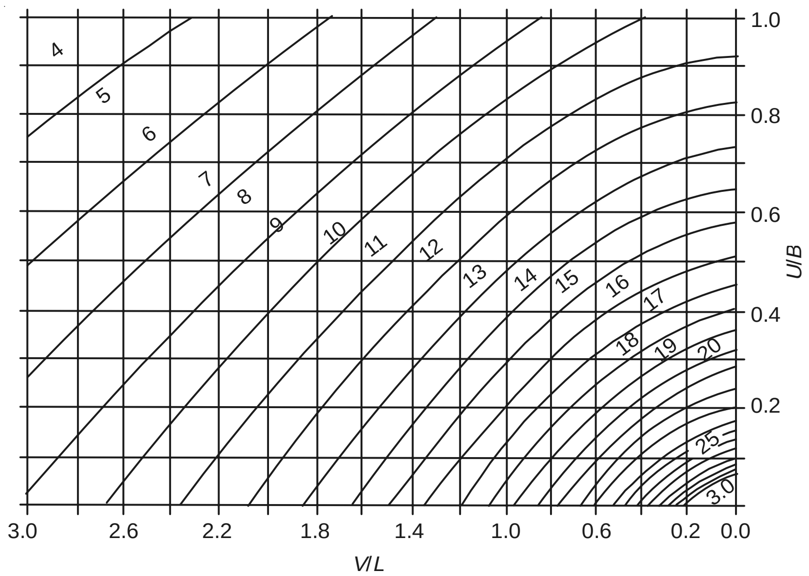


Fig. 9.10 (Contd.)

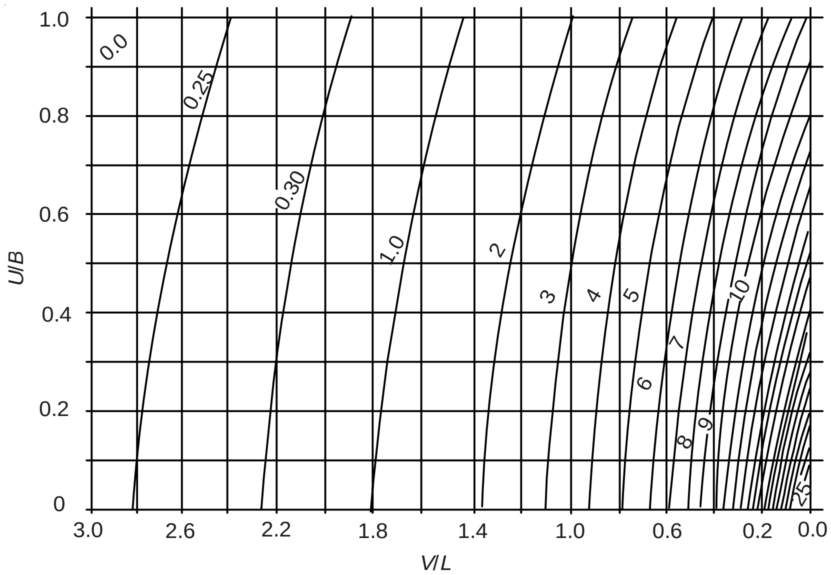


Fig. 9.10 Moment coefficients m_1 and m_2 for $K = 0$.

9.4 DESIGN OF LONGITUDINAL GIRDERS

It is known that the bridge loads are transmitted from the deck to the superstructure and then to the supporting substructure elements. It is rather difficult to imagine how these loads get transferred. If a vehicle is moving on the top of a particular beam, it is reasonable to say that, this particular beam is resisting the vehicle or truckload. However, this beam is not alone, it is connected to adjacent members through the slab and cross girders. This connectivity allows different members to work together in resisting loads, though it is logical to assume that, this specific beam is carrying most of the load. As a result of being connected to other members, the adjacent members will also assist in carrying part of the load. The supporting girders share the live load in varying proportions depending on the flexural stiffness of the deck and the position of the live load on the deck. For determining the fraction of the load carried by the longitudinal girders, several methods have been suggested. Among them, the rational ones are:

- ➢ Guyon–Massonet method
- ➢ Hendry–Jaegar method
- ➢ Courbon's method

9.5 GUYON-MASSONET METHOD

This method is based on orthotropic plate analysis. The bridge deck is assumed to be an orthotropic plate which possesses the same longitudinal and transverse stiffness as that of the actual deck. The longitudinal and transverse stiffness of the deck is related by flexural parameter θ given by

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$$\theta = \frac{b}{2a}\left[\frac{i_L}{i_T}\right]^{0.25} \tag{9.1}$$

where

$2b$ = effective width of the bridge

$2a$ = span of the bridge

i_L = moment of inertia of the plate per unit width

i_T = moment of inertia of the plate per unit length

θ lies between 0.3 to 1.0, for most of the cases.

The relation between longitudinal and transverse torsional stiffness is related by torsional parameter α, given by

$$\alpha = \frac{G(J_L + J_T)}{2E\sqrt{i_L i_T}} \tag{9.2}$$

where

J_L = torsional inertia of the longitudinal beam

J_T = torsional inertia of the transverse beam

G = modulus of rigidity

E = elastic modulus.

The value of α will be less than 1 for T-beam bridges. The distribution coefficient is affected only by the transverse location of the load and is the same for different positions of the load along the span with same eccentricity with respect to the central line of the bridge. Massonet presented tables giving values of distribution coefficient K versus values of torsional parameter α, i.e. K_0 (for $\alpha = 0$) and K_1 (for $\alpha = 1$). As the distribution coefficient is dependent on widthwise allocation of the load, the width $2b$ of the deck is divided into eight equal sections as shown in Fig. 9.11. These nine points are called the standard positions or reference stations.

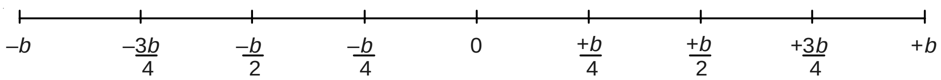

Fig. 9.11 Standard positions or reference stations.

The curves developed by Massonet are, however, slated for only two values of α, i.e. $\alpha = 0$ (K_0) and $\alpha = 1$(K_1). The distribution coefficient for a specific value of α may be obtained by interpolation as follows:

$$K_\alpha = K_0 + (K_1 - K_0)\sqrt{\alpha} \tag{9.3}$$

9.5.1 Calculation of Longitudinal Moment

To begin with, the maximum average longitudinal moments in the bridge are found, considering the load to be equally divided in all the beams. The actual loads should be changed to equivalent loads acting at standard positions on the width *2b* of the deck. This is done by considering the equivalent load as the reaction from a beam of span $b/4$. The load distribution coefficients K_0 and K_1 are tabulated for a given value of θ. The value of K_α is then computed using Eq. (6.3). From these tables, the value of K_α is calculated for various load positions ($-b$, $3b/4$, 0, $+b/4$, $+b/2$, etc.). These coefficients are then multiplied by loads W transferred at standard load positions to get Q, where $Q = K_\alpha W$ at each load position. The summation of Q and that of W is found. A new coefficient K' is then found using the expression, $K' = \Sigma Q/\Sigma W$ for each girder. The final moment is then given by

$$M_x = 1.1\ K' M_{av} \tag{9.4}$$

9.6 HENDRY–JAEGAR METHOD

In this method, the variables affecting the behaviour of interconnected girder bridges are grouped as follows:

1. Variables relating to mechanical properties of the material, i.e. modulus of rigidity and elasticity.
2. Variables relating to geometry of the structure, i.e. number of longitudinal beams, spacing, span of beams.
3. Variables relating to stiffness of the bridge, i.e. flexural stiffness (EI), torsional stiffness. (J), flexural stiffness of transverse deck ($E_T I_T$).
4. Variables relating to loading, i.e. longitudinal position of the load, number of wheel loads and their transverse position on the bridge.

The conditions of analysis are such that the variables mentioned above do not appear separately but are combined with the following dimensionless parameters:

$$\eta = \frac{I_1}{I}$$

$$\alpha = \frac{12}{\pi^4}\left(\frac{L}{h}\right)^2 \frac{E_T I_T}{EI} \tag{9.5}$$

$$\beta = \frac{\pi^2 h}{2L}\,\frac{CJ}{E_T I_T} \tag{9.6}$$

where

I_1 = moment of inertia of the outer girder
I = moment of inertia of the inner girder
L = span length of longitudinal beams
$E_T I_T$ = flexural stiffness of the transverse system
E_I = flexural stiffness of longitudinal beams
C = rigidity modulus
J = torsional stiffness factor.

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The moment coefficients for different numbers of longitudinal girders (up to six) and for values of $\beta = 0$, $\beta = \infty$ have been presented by Hendry and Jaegar. Coefficients for intermediate values of α may be had by interpolation from the equation

$$m_F = m_0 + (m_\infty - m_0)\sqrt{\frac{\beta\sqrt{\alpha}}{3 + \beta\sqrt{\alpha}}} \tag{9.7}$$

9.7 COURBON'S THEORY

Courbon, a French Engineer, assumed the cross girders to be infmitely rigid and worked out the proportions of live loads on these girders based on that consideration. According to his theory, no flexure of transverse deck is possible because of the presence of infinitely rigid diaphragms, and a concentric load, instead of one pushing down only nearby girders, causes equal deflection of all the girders. In case of an eccentric load, the deflection profile of the girders assumes the form as shown in Fig. 9.12. Since the load carried by the girders depends on the magnitude of the deflection, the load distribution proportions also vary linearly.

The deck with infinite transverse flexural stiffness may be compared with a stiff pile cap having a single row of piles. The method of evaluating the load coming on each pile owing to concentric or eccentric load on the pile cap may be applied to determine the load on the girders.

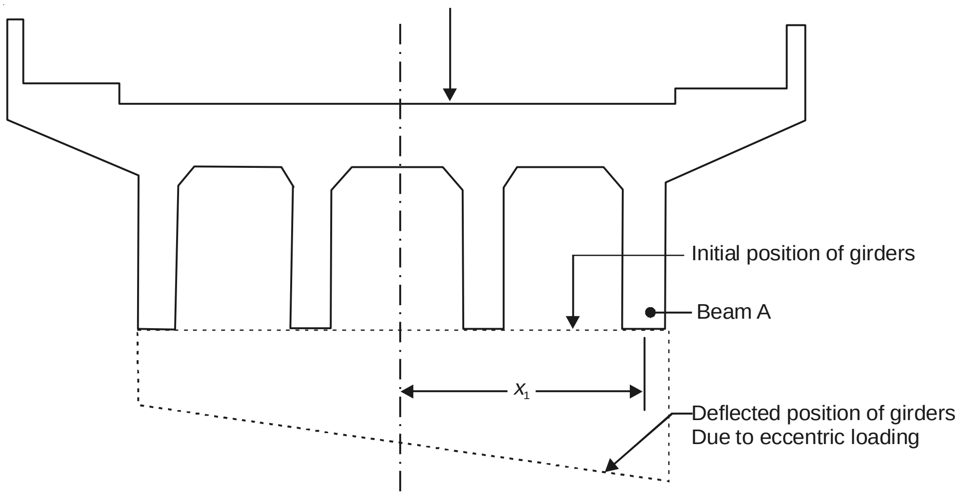

Fig. 9.12 Deflection of girders owing to an eccentric load.

Thus in Fig. 9.12

$$\text{Load on beam A} = \left[\frac{W}{n} + \frac{WeX_1}{\Sigma X^2}\right] \tag{9.8}$$

$$\text{Distribution coefficient } (K) = \frac{\text{Load carried by beam A}}{\text{Average load per beam}} = \frac{W\left[\dfrac{1}{n} + \dfrac{eX_1}{\Sigma X^2}\right]}{\dfrac{W}{n}} \tag{9.9}$$

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If a number of wheels are positioned on the transverse deck, then K is

$$K = \frac{\Sigma W}{n}\left[1 + \frac{neX_1}{\Sigma X^2}\right] \tag{9.10}$$

For bridge decks in which the girders have different moments of inertia, the distribution coefficients are given by

$$K = \frac{\Sigma W}{m}\left[1 + \frac{neI_1X_1}{\Sigma IX^2}\right] \tag{9.11}$$

where in Eqs. (9.9) – (9.11)

W = the eccentric concentrated load

n = the number of longitudinal girders

e = the eccentricity of the wheel load from the centre line of the deck

X_1 = the distance of the girder under consideration from the central axis of the deck

ΣX^2 = the sum of the distances of longitudinal girders from the centre line of the deck

I_1 = the moment of inertia of the girder under consideration

I = the moment of inertia of n girders.

Courbon's theory is applicable when the following conditions are satisfied.

1. The span–width ratio is greater than 2 and less than 4.
2. At least five symmetrical cross girders connecting the longitudinal girders are present.
3. The depth of the cross girders is at least 3/4th of the depth of longitudinal girders.

Courbon's method is used in this text for finding distribution coefficients or reaction factors for longitudinal girders. For the use of other methods, the reader is advised to go through the References [21, 22].

EXAMPLE 9.1

A T-beam bridge has to be provided across a channel having the following data. Design the bridge deck.

Flood discharge: 30 m^3/s

Bed width: 12 m

Side slope: 1 : l

Bed level: 50 m

HFL: 51.25 m

Maximum allowable afflux: 1.5 cm

General ground level: 52 m

Hard rock available at: 48 m

Road formation level: 54 m

Road: National Highway (2 lanes)

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Footpath: 1 m wide on either side
Loading: IRC Class AA
Materials: M40 concrete, Fe 415 steel
No. of longitudinal girders: 3

Design of the Linear Waterway

With reference to Fig. 9.13, we have
A (area of the natural waterway) = $(b + d)d$ = (12 + 1.25)1.25 = 16.56 m^2
Normal velocity of flow (V) = Q/A = 30/16.56 = 1.81 m/s

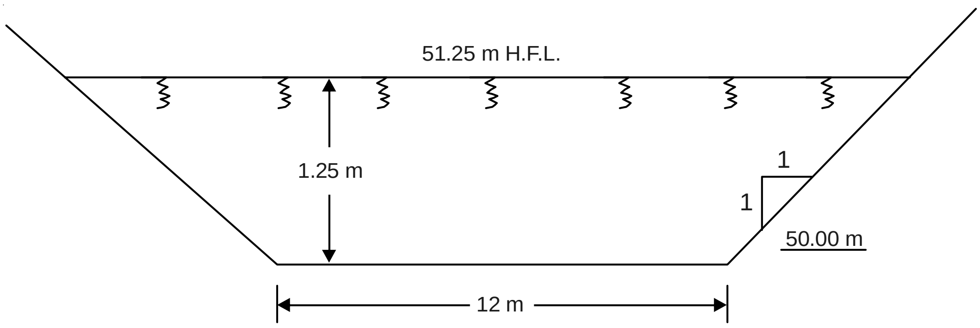

Fig. 9.13 Cross-section of the channel (Example 9.1).

Afflux is given by Molesworth's formula:

$$x = \left(\frac{V^2}{17.9} + 0.015\right)\left(\frac{A^2}{a^2} - 1\right)$$

Therefore,

$$0.015 = \left(\frac{1.81^2}{17.9} + 0.015\right)\left(\frac{16.56^2}{a^2} - 1\right)$$

or

$$a = 15.97 \text{ m}^2$$

But

$$a = L_1(d + x)$$

Therefore,

$$15.97 = L_1(1.25 + 0.015)$$

or

$$L_1 = 12.62 \text{ m}$$

A span of 14 m is therefore assumed. This span is amenable to practical spacing of cross girders. Also, a little higher value of linear waterway would always reduce the percentage of constriction.

Preliminary Dimensions

Let us provide three longitudinal beams at c/c spacing of 3 m. Let the rib width of each beam be 300 mm and the thickness of the slab 220 mm. Cantilever slab is 230 mm thick at its fixity which is reduced to 120 mm at the free end. Let the footpath be 200 mm above the wearing coat. The cross girders are placed at 3.5 m c/c. The cross-section of the deck is shown in Fig. 9.14.

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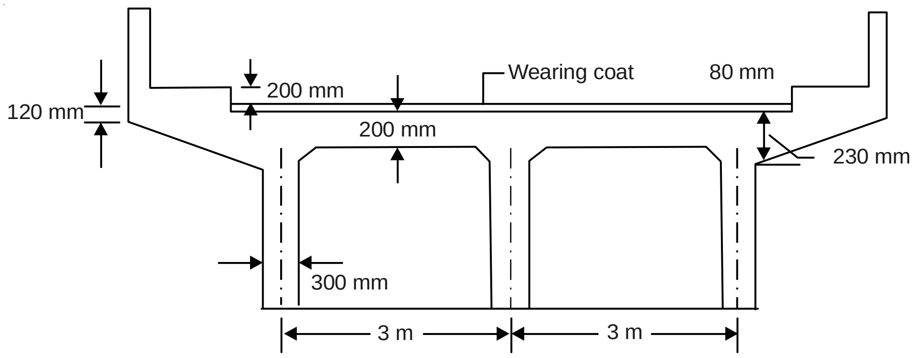

Fig. 9.14 Cross-section of the deck—Preliminary dimensions (Example 9.1).

Design Parameters

$$n = \frac{10 \times 13.33}{10 \times 13.33 + 200} = 0.4$$

$$J = 1 - 0.4/3 = 0.87$$

$$q = 0.5 \times 0.87 \times 0.4 \times 13.33$$

$$= 2.31$$

Design of the Cantilever Slab Portion

Dead load shear force and bending moment with reference to Fig. 9.15 are tabulated below:

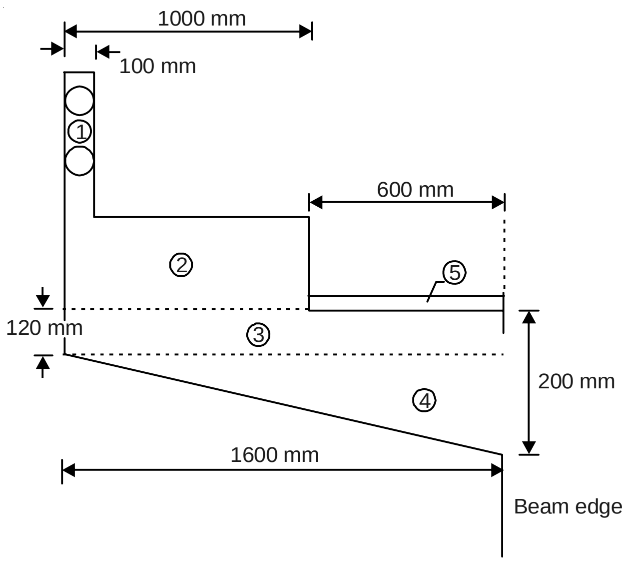

Fig. 9.15 Cantilever portion of the deck for dead load (Example 9.1).

Component	*Dead load shear force/ metre run* (kN/m)	*Distribution of c.g.* (m)	*Bending moment* (kN·m)
Railing (1)	0.7	1.55	1.08
Footpath (2)	0.32 × 1 × 24 = 7.7	1.10	8.50
Slab rectangular (3)	0.12 × 1.6 × 24 = 4.60	0.80	3.70
Slab triangular (4)	0.5 × 0.08 × 1.6 × 24 = 1.60	0.53	0.85
Wearing coat (5)	0.08 × 0.6 × 22 = 1.06	0.30	0.32
	Total 15.66		14.45

Live load bending moment and shear force

The minimum clearance distance of Class AA wheel edge from the footpath edge is 1.2 m. As the available clear distance of the cantilever is only 600 mm, Class AA loading cannot be considered on the cantilever portion. However, Class A loading may be considered as it can be accommodated. For calculating the effective length and the effective width of dispersion, an average thickness of 200 mm of the slab may be considered. With reference to Fig. 9.16, we have

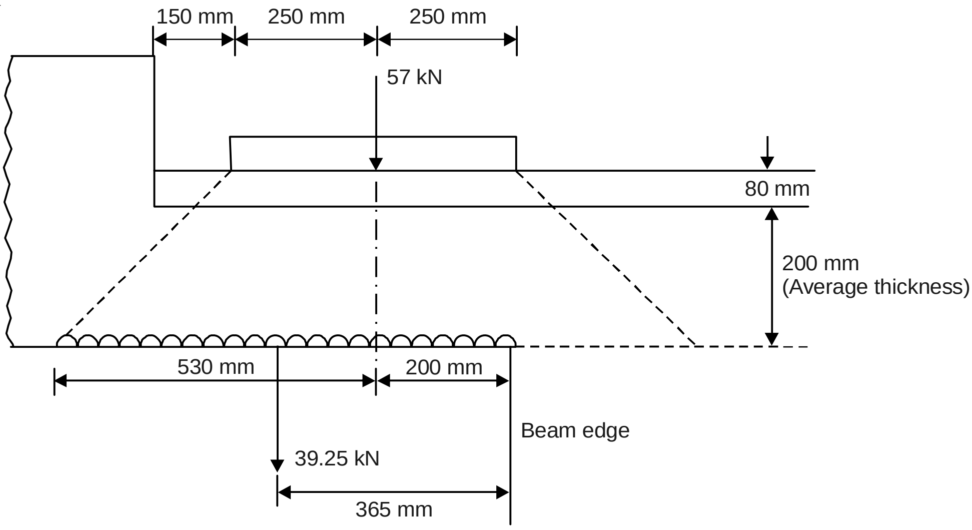

Fig. 9.16 Live load on cantilever portion (Example 9.1).

Distance of the c.g. of the wheel from the edge of the cantilever = 600 – (150 + 250)

= 200 mm

Dispersed length of the load along the span = 500 + 2 × 280 = 1060 mm

Out of this length, only 730 mm (1060/2 + 200) will be covered by load effect. Hence, the actual load effective on the cantilever portion is

$$= \frac{114}{2} \times \frac{730}{1060} = 39.25 \text{ kN}$$

Effective width of dispersion is given by

$$b_{ef} = 1.2x + b_1$$

where $b_1 = 0.25 + 2(0.08) = 0.41$ m

Therefore,

$$b_{ef} = \frac{1.2 \times 0.730}{2} + 0.41 = 0.85 \text{ m}$$

$$\text{Impact factor} = \frac{4.5}{6 + L} = \frac{4.5}{6 + 1.6} = 0.59$$

However, the maximum value of the impact factor is taken as 0.50

$$\text{Bending moment owing to live load} = 1.5 \times \frac{39.25}{0.85} \times \frac{0.730}{2} = 25.28 \text{ kN·m}$$

Design moment = 14.45 + 25.28 ≃ 40 kN·m

Design shear = 15.66 + 1.5 × 39.25 = 74.5 kN

$$\text{Effective depth required} = \sqrt{\frac{40 \times 10^6}{2.31 \times 10^3}} = 131.6 \text{ mm}$$

With 14 mm diameter bars and a clear cover of 25 mm, the effective depth actually provided is

$$= 200 - 40 - 7 = 153 \text{ mm}$$

$$\text{Area of main steel reinforcement} = \frac{40 \times 10^6}{200 \times 0.9 \times 153} = 1452.4 \text{ mm}^2$$

$$\text{Spacing of 14 mm bars} = \frac{1000 \times 153.9}{1452.4} = 105.96 \text{ mm}$$

Therefore, we provide 14 mm bars at 100 mm c/c. Distribution steel is provided for a moment, which is given by

= 0.3 × live load moment + 0.2 × dead load moment

= 0.3 × 25.28 + 0.2 × 14.5 ≅ 10.5 kN·m

Effective depth available, using 8 mm diameter rods = 153 – 7 – 4 = 142 mm

$$\text{Area of distribution reinforcement} = \frac{10.5 \times 10^6}{200 \times 0.9 \times 142} = 410.8 \text{ mm}^2$$

$$\text{Spacing of 8 mm diameter bars} = \frac{1000 \times 50.3}{410.8} = 122.44\text{, say 120 mm c/c.}$$

As the calculation of live load shear force is approximate, check for shear is not required.

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Design of the Interior Panel

The interior panel dimensions are 3 m × 3.5 m.

Dead load bending moment and shear force

Dead load bending moment is determined using Pigeauds curves.

Dead weight of the slab = (1 × 0.20 × 24) = 4.8 kN/m^2

Dead weight of the wearing coat = (0.08 × 22) = 1.76 kN/m^2

Total dead load = 6.56 kN/m

Total dead load on the panel = (3.5 × 3 × 6.56) = 68.88 kN

Since the dead load spreads uniformly on the entire slab, we have

$$U/B = 1,\ V/L = 1 \text{ and } K = B/L = 3.0/3.5 = 0.857,\ 1/K = 1.16$$

From Pigeauds curve (Fig. 15.16), we get

$$m_1 = 0.038 \text{ and } m_2 = 0.031$$

$$Mb = 68.88(0.038 + 0.15 \times 0.031) = 2.93 \text{ kN·m}$$

$$Ml = 68.88(0.031 + 0.15 \times 0.038) = 2.52 \text{ kN·m}$$

Continuity effect on the slab is accounted for by a continuity factor which is taken as 0.80. Therefore, bending moment values after effecting continuity are given by

$$Mb = 0.80 \times 2.93 = 2.34 \text{ kN·m}$$

$$Ml = 0.80 \times 2.52 = 2.01 \text{ kN·m}$$

Dead load shear force = (6.56 × 2.7)/2 = 8.86 kN

Live load bending moment and shear force

As the live load is IRC Class AA, one wheel can be accommodated centrally on the panel so as to produce maximum bending moment (see Fig. 9.17).

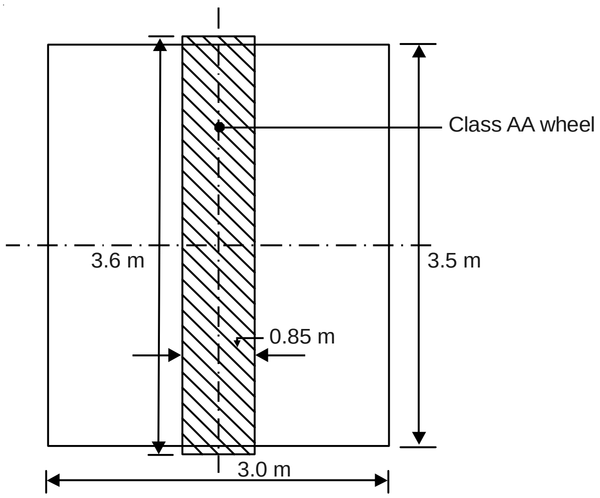

Fig. 9.17 Interior panel with live load (Example 9.1).

Here $U = 0.85 + 2 \times 0.08 = 1.01$ m

$$V = 3.60 + 2 \times 0.08 = 3.76 \text{ m}$$

$$U/B = 1.01/3 = 0.34,\ V/L = 3.76/3.5 = 1.07 \text{ and } K = B/L = 3/3.5 = 0.85$$

Referring Pigeauds curve for $K = 0.9$, we get

$$m_1 = 0.08 \text{ and } m_2 = 0.059$$

$$Mb = 350(0.08 + 0.15 \times 0.059) = 31.09 \text{ kN·m}$$

$$Ml = 350(0.059 + 0.15 \times 0.08) = 24.85 \text{ kN·m}$$

Impact factor and continuity effects are to be considered.

Impact factor = 25%

Continuity factor = 0.80

Actual live load moments are given by

$$Mb = 1.25 \times 0.80 \times 31.09 = 31.09 \text{ kN·m}$$

$$Ml = 1.25 \times 0.80 \times 24.85 = 24.85 \text{ kN·m}$$

Design bendidg moments are given by

$$\text{Along short span, } Mb = 31.09 + 2.34 = 33.43 \text{ kN·m}$$

$$\text{Along long span, } Ml = 24.85 + 2.01 = 26.86 \text{ kN·m}$$

Live load shear force is calculated by the effective width method as Pigeauds method is not applicable. To obtain maximum shear force, the load is positioned such that the dispersion edge just touches the edge of the slab (Fig. 9.18).

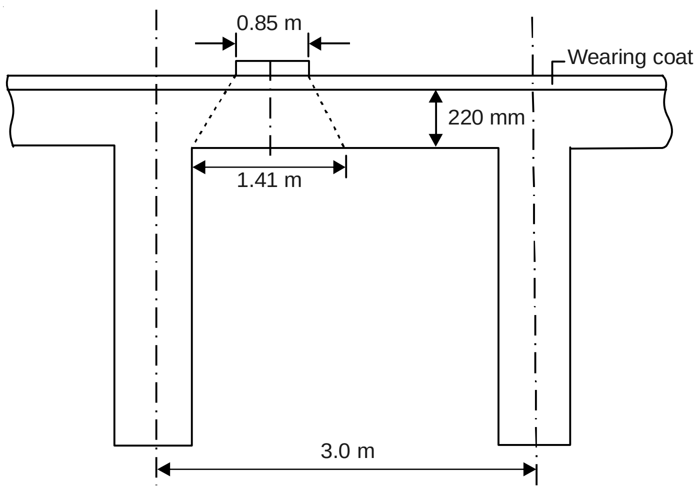

Fig. 9.18 Placement of wheel for maximum shear force (Example 9.1).

Load dispersion length along short span = $0.85 + 2(0.08 + 0.2) = 1.41$ m

Effective width of dispersion = $\alpha x(1 - x/L) + b_1$

Clear panel dimensions = 2.7 m × 3.2 m

And

$$K = B/L = 2.7/3.2 = 0.84$$

From Table 7.1 for K for continuous slab, $\alpha = 2.08$. Therefore,

Effective width of dispersion $= 2.08 \times 0.705 \left(1 - \frac{0.705}{2.7}\right) + 3.6 + (2 \times 0.08) = 4.84$ m

Load per metre width = 350/4.84 = 72.31 kN/m

Shear force = 72.31(2.7 – 0.705)/2.7 = 53.42 kN

Live load shear force after considering impact and continuity effect

$$= 1.25 \times 0.80 \times 53.42 = 53.42 \text{ kN}$$

Design shear force = 53.42 + 8.86 = 62.3 kN

Design of the Slab

Effective depth of the slab $= \sqrt{\frac{33.43 \times 10^6}{2.31 \times 1000}} = 120.3$ mm

Effective depth actually provided (with 14 mm diameter steel and 40 mm clear cover) is

$$= 200 - 40 - 7 = 153 \text{ mm}$$

Area of steel along short span $= \frac{33.43 \times 10^6}{200 \times 0.9 \times 153} = 1213.87 \text{ mm}^2$

c/c distance = 1000 × (153.9/1213.87) = 126.78, say 120 mm

Effective depth along long span using 10 mm diameter bars = 153 – 7 – 5 = 141 mm

Area of steel along long span $= \frac{26.86 \times 10^6}{200 \times 0.9 \times 141} = 1058.3 \text{ mm}^2$

These bars are arranged at 90 mm c/c.

Check for shear

Design shear force = 62.3 kN

Nominal (τ_v) shear $= \frac{62.3 \times 10^3}{1000 \times 153} = 0.40$ MPa

As per IRC 21, we have

Permissible shear stress is,

$\tau = k\tau_c$, $k = 1.2$ for slab thickness of 200 m

Percentage of steel $= \frac{100 \times 1213.87}{1000 \times 153} = 0.79$

for $\rho = 0.79$ and M40 grade concrete

$$\tau_c = 0.38 \text{ MPa}$$

$$\therefore \quad \tau = 1.2 \times 0.38 = 0.46 \text{ MPa}$$

As $\tau_v < \tau$, slab is safe against shear stresses.

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Therefore,

$$\tau_c = 1.008 \times 1 \times 0.40 = 0.403 \text{ MPa}$$

Therefore, the slab is safe against shear stresses.

Design of Longitudinal Girders

Before we design the longitudinal girders, the proportions of the live load shared by internal and external girders need to be established. This fraction (distribution coefficient) is also called the *reaction factor*. The IRC Class AA vehicle is arranged on the deck with stipulations of code (IRC 6) being satisfied. The arrangement is shown in Fig. 9.19.

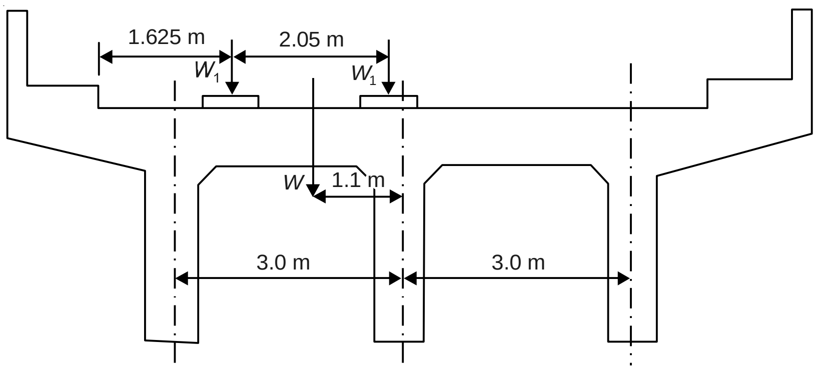

Fig. 9.19 Transverse placement of wheels of IRC Class AA vehicle (Example 9.1).

Using Courbon's method, the reaction factor for the outer girder is

$$= \frac{2W_1}{3}\left(1 + \frac{neX_1}{\Sigma X^2}\right)$$

$$= \frac{2W_1}{3}\left(1 + \frac{3 \times 1.1 \times 3}{2 \times 3^2}\right)$$

$$= 1.03W_1 = 1.03W/2 = 0.52W$$

Similarly, the reaction factor for the inner girder is

$$= \frac{2W_1}{3}\left(1 + \frac{3 \times 1.1 \times 0}{2 \times 3^2}\right)$$

$$= 0.667W_1 = 0.667W/2 = 0.33W$$

Dead load bending moment and shear force

Loading from cantilever portions = 2 × 15.66 = 31.32 kN

Loading from deck = 6.56 × 6.3 = 41.32 kN

Total dead load = 72.65 kN

This dead load is assumed to be taken equally by three girders. Therefore, we have

Load per girder = 72.65/3 = 24.2 kN/m

The self-weight of the girder is calculated assuming the depth of the girder to be 100 mm per metre of span. Therefore,

Depth of the girder = 100 × 14 = 1400 mm

Depth of the rib = 1400 – 200 = 1200 mm

Wtdth of the rib = 300 mm

Weight of the rib per metre length = 0.3 × 1.2 × 24 = 8.64 kN/m

The weight of the cross girder also acts on the longitudinal girders in the form of a concentrated load. Assuming the same dimensions for the cross girder as well, we have

Weight of the cross girder (excluding the length portion covered by the rib of the main girder)

= 8.64 × 2.7 × 2 = 46.65 kN

This load is also taken equally by three girders. Therefore,

Load per girder = 46.65/3 = 15.55 kN

Total udl on each girder = 24.2 + 8.64 = 32.8, say 33 kN/m

Maximum dead load bending moment occurs at the centre of the span and maximum dead load shear force occurs at supports (Fig. 9.20).

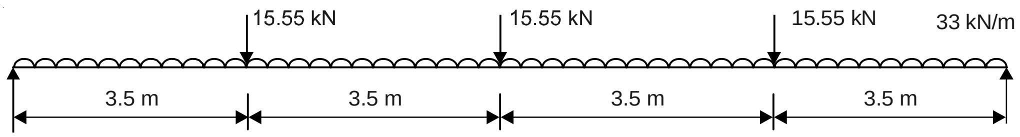

Fig. 9.20 Dead load on girder (Example 9.1).

$$\text{Maximum dead load bending moment} = \frac{33 \times 14^2}{8} + \frac{3 \times 15.55}{2} \times 7 - (15.55 \times 3.5)$$

$$= 917.35 \text{ kN·m}$$

$$\text{Dead load shear force} = \frac{33 \times 14}{2} + 15.55 + \frac{15.55}{2} = 254.33 \text{ kN}$$

Live load bending moment

The maximum live load bending moment will occur when IRC Class AA vehicle is centrally placed on the girder (Fig. 9.21).

$$\text{Live load bending moment} = 2\left(\frac{3.5 + 2.6}{2}\right)\left(\frac{700 \times 1.8}{3.6}\right) = 2135 \text{ kN·m}$$

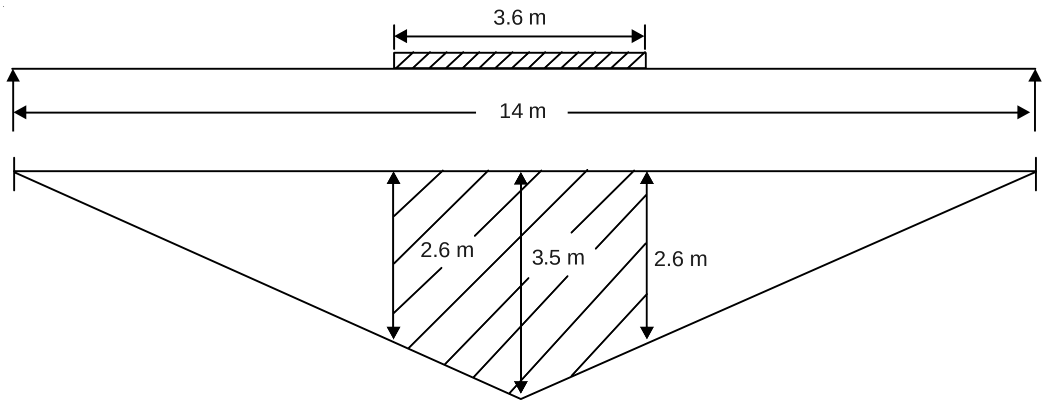

Fig. 9.21 I.L.D. for maximum live load bending moment (Example 9.1).

Bending moments including impact factor (10%) and reaction factor are:

For inner girder = 1.1 × 0.333 × 2135 = 782.05 kN·m

For outer girder = 1.1 × 0.52 × 2135 = 1221.22 kN·m

Live load shear force

Maximum shear force will be developed in the girder when the live load is near the girder. This load is to be placed between the support and the exterior girder. Shear force will be found as a reaction developed by longitudinal girders. The placement of the wheels for maximum shear force is as shown in Fig. 9.22.

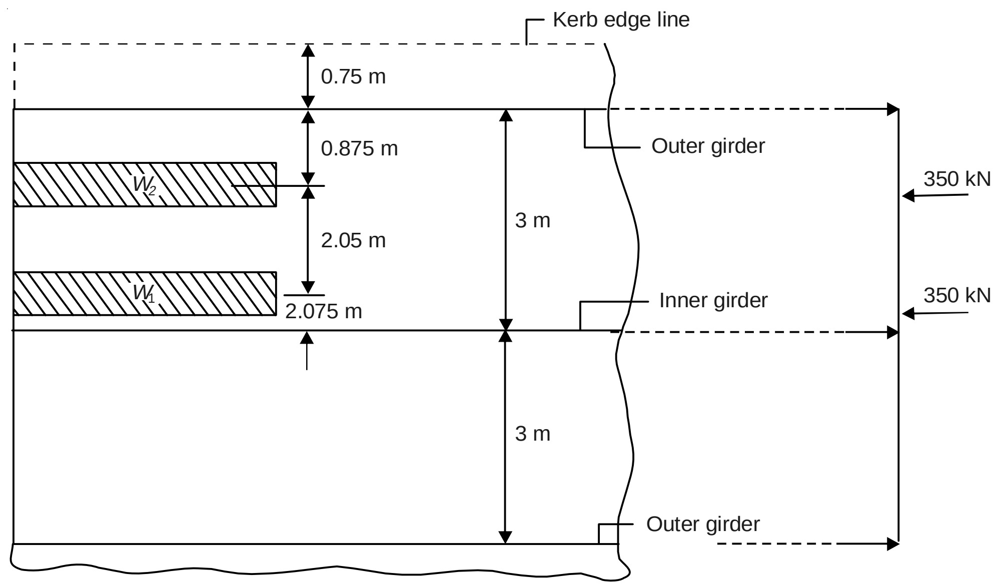

Fig. 9.22 Placement of wheels for maximum shear force (Example 9.1).

Reaction on the outer girder is given by

$$\text{that owing to } W_1 = \frac{350 \times 2.125}{3} = 247.91 \text{ kN}$$

$$\text{that owing to } W_2 = \frac{350 \times 0.075}{3} = 8.75 \text{ kN}$$

$$\text{Total} \simeq 256 \text{ kN}$$

Reaction on the inner girder is given by

$$\text{that owing to } W_1 = \frac{350 \times 0.875}{3} = 102.08 \text{ kN}$$

$$\text{that owing to } W_2 = \frac{350 \times 2.925}{3} = 341.25 \text{ kN}$$

$$\text{Total} \simeq 443 \text{ kN}$$

These reactions are taken as live loads on the girders as shown in Fig. 9.23.

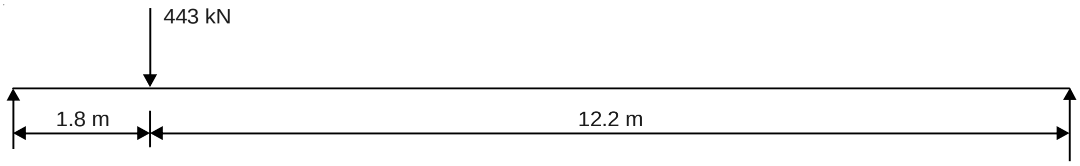

Fig. 9.23 Live load on inner girder for maximum shear force (Example 9.1).

$$\text{Maximum shear force on the outer girder} = \left(\frac{256 \times 12.2}{14}\right) 1.1 = 245.4 \text{ kN}$$

$$\text{Maximum shear force on the inner girder} = \left(\frac{443 \times 12.2}{14}\right) 1.1 = 424.64 \text{ kN}$$

However, maximum values of bending moment and shear force are considered for design. Therefore, we have

Maximum live load bending moment = 1221.22 kN·m

Maximum dead load bending moment = 917.35 kN·m

= 2138.57 kN·m

Maximum live load shear force = 424.64 kN

Maximum dead load shear force = 254.33 kN

= 678.97 kN ≈ 679 kN

Design of section

The beam is designed as a T-beam with an effective cover of 100 mm.

Effective depth of the girder = 1400 – 100 = 1300 mm

Lever arm (approximate) = 1300 – (200/2) = 200 mm

$$\text{Area of steel} = \frac{2138.57 \times 10^6}{200 \times 1200} = 8910.70 \text{ mm}^2$$

No. of bars of 36 mm diameter = 8.75, say 9

Nine bars of 36 mm diameter are provided in three rows.

Reinforcements should also be designed to take up the critical shear stress at supports. Nominal shear stress

$$\tau_v = \frac{V}{bd} = \frac{679 \times 10^3}{300 \times 1300}$$

$$= 1.74 \text{ MPa}$$

[As per IRC 21 (2000), the *m* shear stress in beam shall not exceed τ_{max}. For M40, τ_{max} = 25 MPa]

$\therefore \qquad \tau_v < \tau_{max}$

Three bars of 36 mm diameter are bent at supports to take up shear stress at supports. For

$$\rho = \frac{100A_{st}}{bd} = \frac{100 \times 6 \times 1018}{300 \times 1300} = 1.56$$

from Table 12B of IRC 21, for M40 concrete and ρ = 1.56, τ_c = 0.50 N/mm^2

$$\text{Shear stress taken by concrete} = \frac{0.5 \times 300 \times 1300}{1000} = 195 \text{ kN} \approx 195 \text{ kN}$$

Shear stress resisted by bent-up bars is given by

$$V_s = \sigma_{sv} \times A_{sv} \times \sin \alpha$$

$$= \frac{150 \times 3 \times 1018 \times 1}{1000\sqrt{2}} = 323.92 \text{ kN} \approx 324 \text{ kN}$$

Balance shear is = 679 – 195 – 324 = 160 kN

Using 10 mm diameter 3-legged stirrups, their spacing is

$$= \frac{\sigma_{sv} A_{sv} d}{V} = \frac{150 \times 3 \times 79 \times 1300}{160 \times 1000} = 288 \text{ mm}$$

However, stirrups are provided at a spacing of 250 mm. As the shear stress decreases towards the centre, the spacing could be increased to 300 mm c/c (maximum spaing as per IRC 21).

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Design of Cross Girders

Cross-sectional dimensions of the cross girders are maintained the same as those of the longitudinal girders.

Dead load bending moment and shear force

A cross girder derives dead load from two triangular portions of the slab as shown in Flg. 9.24.

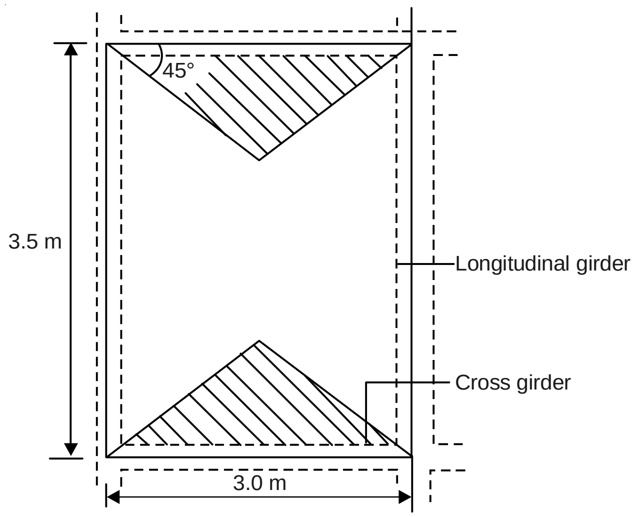

Fig. 9.24 Dead load on cross girder (Example 9.1).

Here

$$\text{Dead load from slab} = \frac{2 \times 3 \times 1.5 \times 6.56}{2} = 29.5 \text{ kN}$$

$$\text{Converting this to udl} = 29.5/3 = 9.83 \text{ kN/m}$$

$$\text{Total dead load} = 8.64 + 9.83 = 18.47 \simeq 18.5 \text{ kN/m}$$

As an approximation, the reaction on each cross girder is (Fig. 9.25) given by

$$= \frac{18.5 \times 6}{3} = 37 \text{ kN}$$

The dead load B.M. is computed at the location where the live load B.M. is maximum.

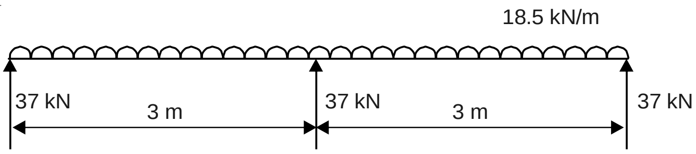

Fig. 9.25 Reactions on cross girder (Example 9.1).

Live load bending moment and shear force

Maximum live load bending moment and S.F. for Class AA loading occur for the position of the load as shown in Fig. 9.26.

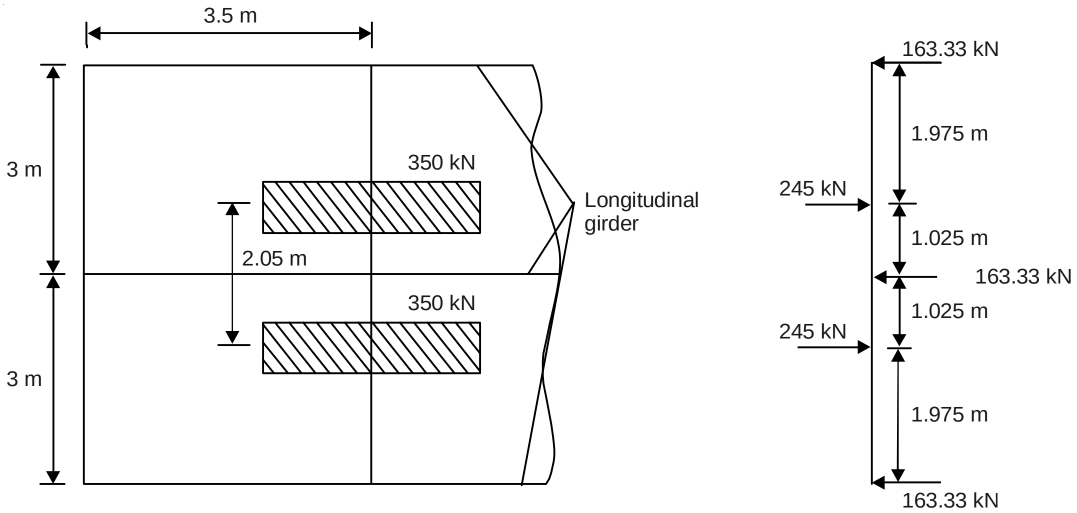

Fig. 9.26 Live loads on cross girder (Example 9.1).

The maximum load transferred to the cross girder = $\dfrac{2 \times \dfrac{350}{2} \times 2.1}{3}$ = 245 kN

Assuming equal sharing, the reaction from longitudinal girders = $\dfrac{245 \times 2}{3}$ = 163.33 kN

Live load shear force = 1.1 × 163.33 = 179.66 kN

Maximum live load in the cross girder will be developed below the load itself. Live load B.M. including impact factor is

$$= (163.33 \times 1.975)\ 1.1 = 354.83 \text{ kN·m}$$

Dead load B.M. (at the location where the live load B.M. is maximum) is

$$= 37 \times 1.975 - \frac{19 \times 1.975^2}{2} = 36 \text{ kN·m}$$

Finally,

$$\text{Design B.M.} = 354.83 + 36 \approx 391 \text{ kN·m}$$

and

$$\text{Design S.F.} = 179.66 + 36 \approx 216 \text{ kN·m}$$

Design of section

Effective depth = 1400 – 100 = 1300 mm

Area of steel = $\dfrac{391 \times 10^6}{200 \times 0.9 \times 1300}$ = 1671 mm^2

We provide 6 bars of 36 mm diameter (Actual area of steel = 1885 mm^2).

Shear stress = $\dfrac{216 \times 10^3}{300 \times 1300}$ = 0.558 N/mm^2

For

$$\rho = \frac{100\ A_s}{bd} = \frac{100 \times 1680}{300 \times 1300} = 0.43$$

from Table 12B of IRC 21, τ_c = 0.31 N/mm^2

Shear stress taken by concrete = $\dfrac{0.31 \times 300 \times 1300}{1000}$ = 121 kN

Balance shear = 216 – 121 = 95 kN

Using 10 mm diameter 2-legged stirrups, their spacing is

$$= \frac{150 \times 2 \times 79 \times 1300}{95 \times 1000} = 324 \text{ mm}$$

However, a maximum spacing of 300 mm is adopted. The spacing may be increased to 400 mm near the centre. The cross-section of the deck, longitudinal section of the girder, and longitudinal section of the cross girder are shown in Figs. 9.27, 9.28, 9.29, respectively.

Details of Substructures

Abutments. Abutment dimensions can be derived as follows:

Top width = Bearing + Clearance + Dirt Wall
= 0.5 + 0.3 + 0.5 = 1.3 m

Bottom width = 0.4 × height from the road formation level
= 0.4(54 – 50) = 1.6 m

Toe projection for the abutment = 0.4 (Depth up to founding level)
= 0.4 (50 – 48) = 0.80 m

End connectors. Splayed wing walls (45°) are provided. A top width of 50 cm is provided for the wing. The wing wall top slopes down to a level just above the general ground level by 50 cm. The wing wall base width is equal to the abutment basement width at the junction with the abutment. At the ends, the wing wall height reduces, and therefore the base width can also be reduced.

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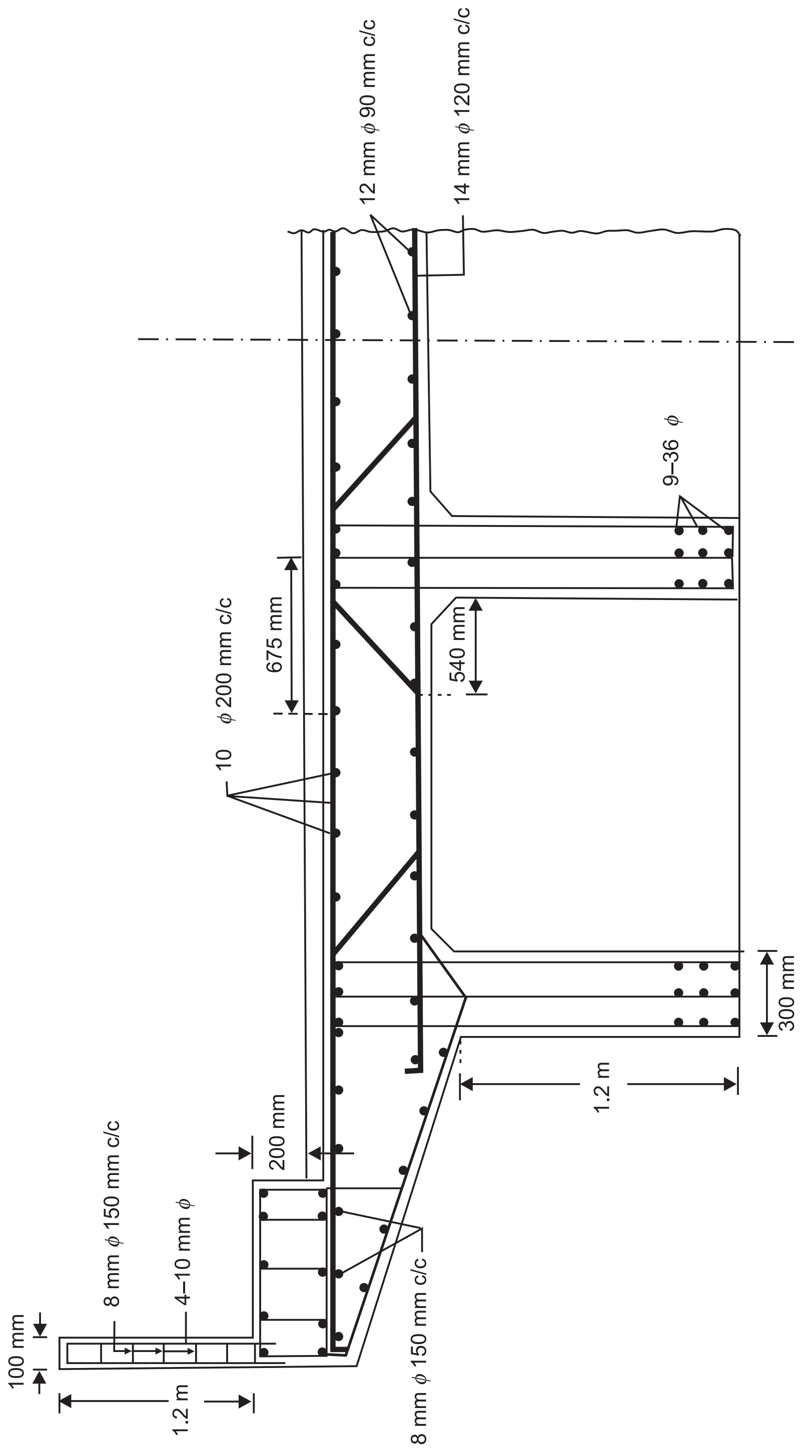

Fig. 9.27 Cross-section of the deck (Example 9.1).

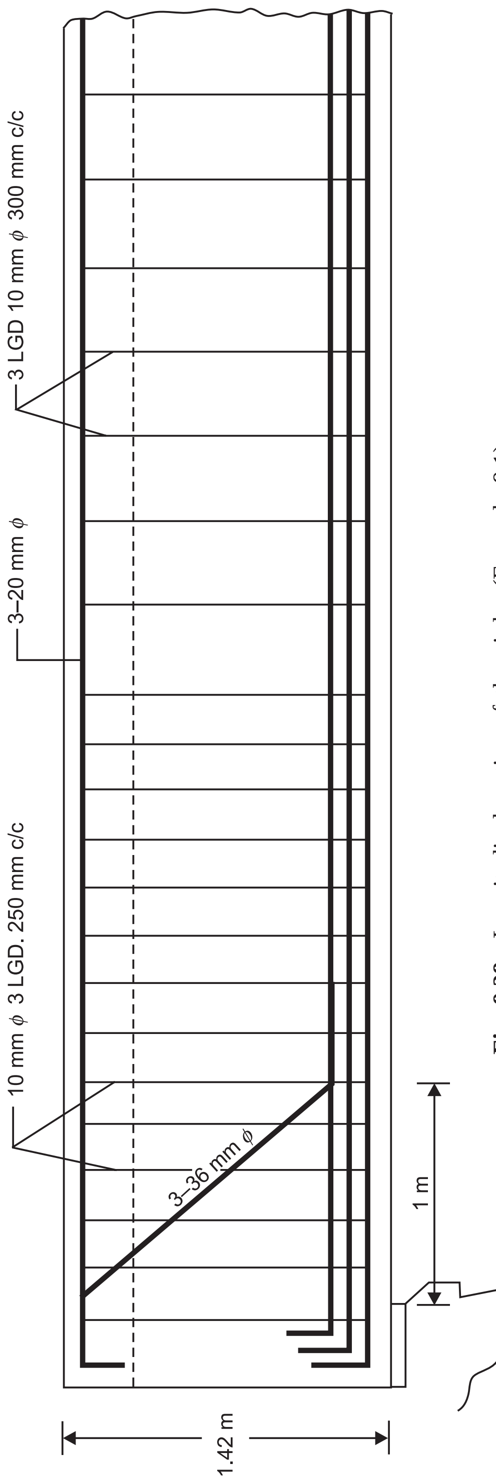

Fig. 9.28 Longitudinal-section of the girder (Example 9.1).

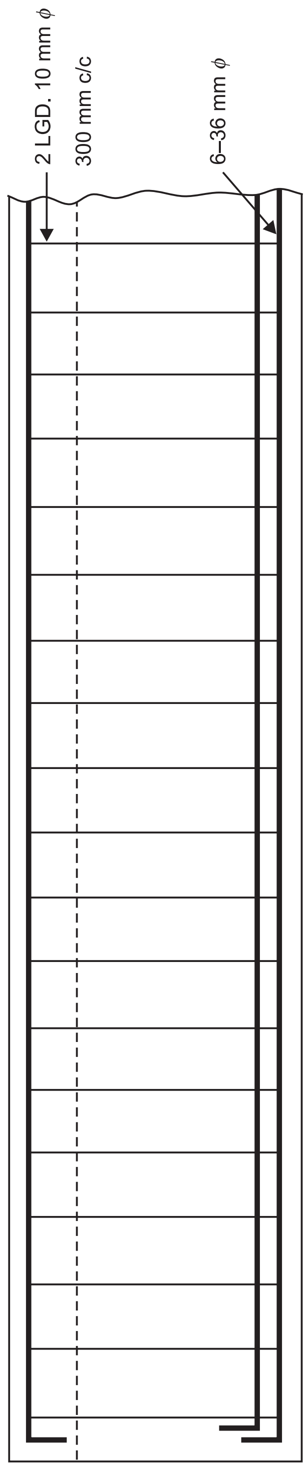

Fig. 9.29 Longitudinal-section of the cross girder (Example 9.1).

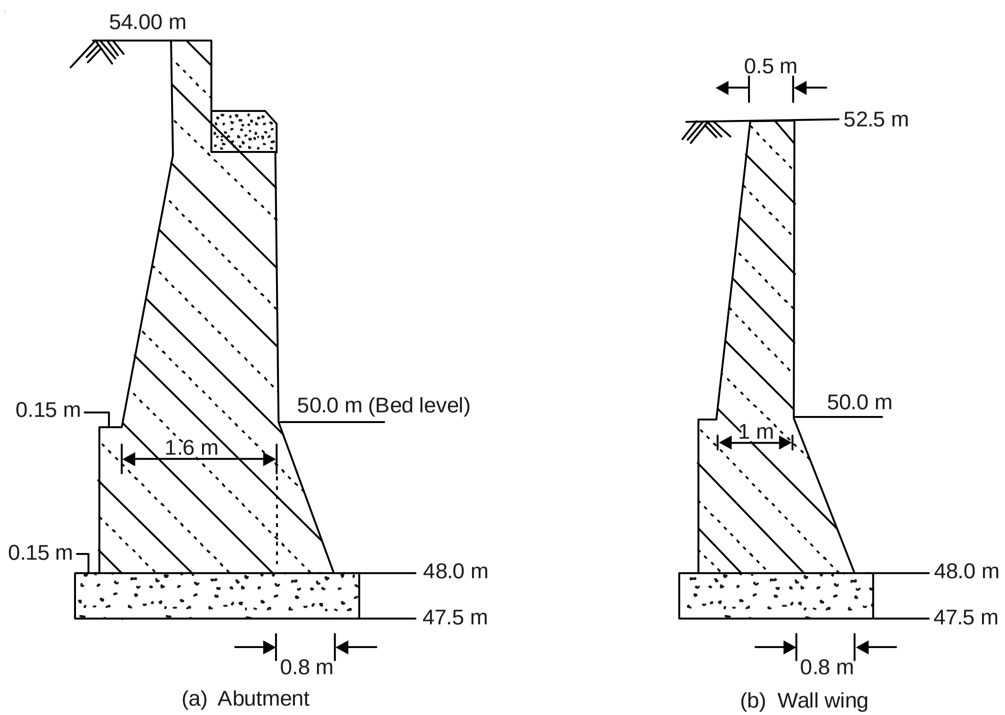

Fig. 9.30 Sectional details of abutment and wing wall (Example 9.1).

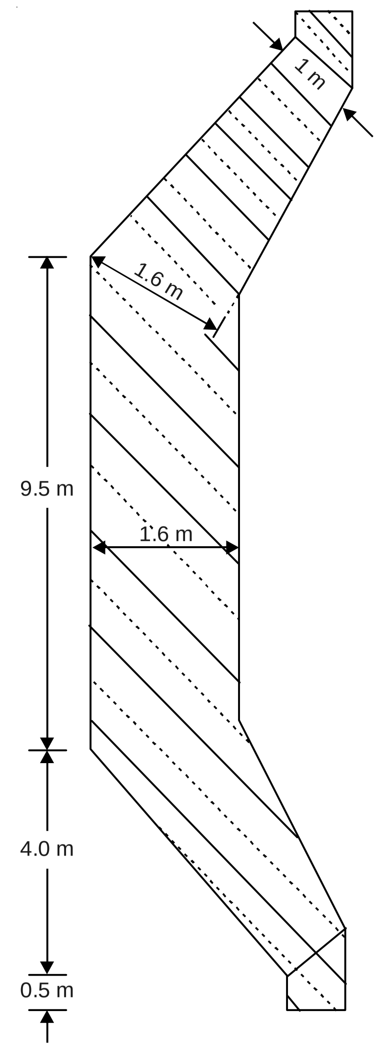

Fig. 9.31 Plan at foundation level (Example 9.1).

$$\text{Wing wall base width at the ends} = 0.4 \text{ (height of the wing wall)}$$
$$= 0.4\,(52.50 - 50) = 1.00 \text{ m}$$

The wing wall is connected to an end block.

The sectional details of the deck, abutments, wing walls are shown in Figs. 9.30 and 9.31 respectively.

The detailed drawing of the bridge is shown in Plate 5.

EXAMPLE 9.2

Obtain the values of short span and long span bending moments in case of an interior panel of a T-beam bridge having the following details:

Dimensions of the panel: 3 m × 3.5 m

Loading: IRC Class A

Loading pattern:

Case (i): Two wheels (each of 57 kN) adjusted symmetrically with respect to centre of the panel

Case (ii): One wheel (57 kN) at centre of the panel.

Case (i)

The wheels are placed as shown in Fig. 9.32. As Pigeauds method is applicable exclusively for a centrally placed concentrated load, some adjustments need to be made so that the two wheels placed become symmetric as shown in Fig. 9.33.

The effect of the actual arrangement is equivalent to the effect of the arrangement as in Fig. 9.33(a) minus the effect of the arrangement as in Fig. 9.33(b).

Moments for the arrangement shown in Fig. 9.33(a)

$$\text{Intensity of wheel load on a small area of contact} = \frac{57}{0.66 \times 0.41} = 210.64 \text{ kN/m}^2$$

Centrally placed concentrated load for Fig. 9.33(a) is

$$= 210.64 \times 0.66 \times 1.61 = 223.14 \text{ kN}$$

Here

$U/B = 0.66/3 = 0.22$, $V/L = 1.61/3.5 = 0.46$ and $K = 3/3.5 = 0.85$

From Pigeauds curves (for $K = 0.9$), $m_1 = 0.145$ and $m_2 = 0.115$

Short span and long span moments are:

$$Mb_a = 223.14(0.145 + 0.15 \times 0.115) = 36.20 \text{ kN·m}$$

$$Ml_a = 223.14(0.115 + 0.15 \times 0.145) = 30.51 \text{ kN·m}$$

Moments for the arrangement shown in Fig. 9.33(b)

Here

$$U/B = 0.66/3 = 0.22, \quad V/L = 0.79/3.5 = 0.225, \quad \text{and} \quad K = 0.85$$

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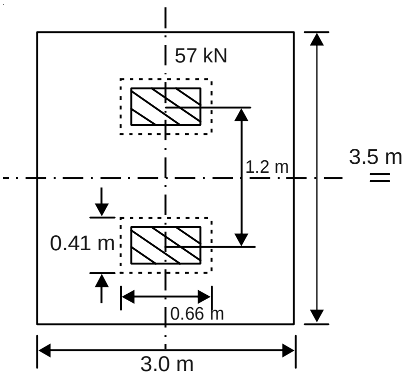

Fig. 9.32 Two symmetric wheels (Example 9.2).

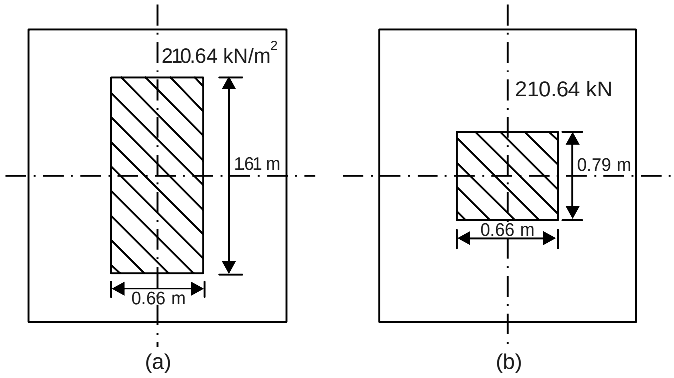

Fig. 9.33 Equivalent arrangement for Fig. 9.32 (Example 9.2).

Again from Pigeaud curves; m_1 = 0.17 and m_2 = 0.15

Equivalent concentrated load (centrally placed) = 210.64 × 0.66 × 0.79 = 109.83 kN

Short span and long span moments are

$$Mb_b = 109.83(0.17 + 0.15 \times 0.15) = 21.14 \text{ kN·m}$$

$$MI_b = 109.83(0.15 + 0.15 \times 0.17) = 19.27 \text{ kN·m}$$

Therefore, the actual short span and long span moments are

$$Mb = 36.20 - 21.14 = 15.06 \text{ kN·m}$$

$$Ml = 30.51 - 19.27 = 11.24 \text{ kN·m}$$

Case (ii)

If one wheel of IRC Class A is placed at the centre of the panel, then the other wheel shall just follow the front wheel and it will occupy the position as shown in Fig. 9.34(a). However, as one of the wheels is centrally placed, the moments owing to this load can be directly calculated. But the other load becomes eccentric. To calculate moments owing to this eccentric load, adjustments as shown in Figs. 9.34(b) and 9.34(c) need to be made.

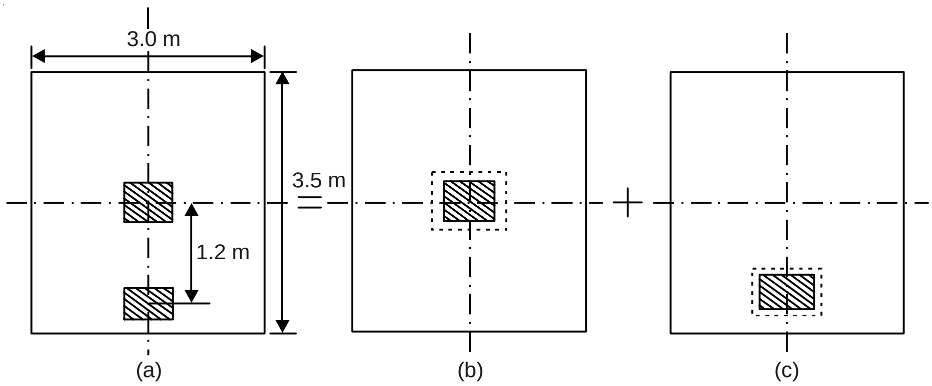

Fig. 9.34 Two wheels of IRC Class A (Example 9.2).

Moments for the centrally placed load (W_1)

Here $U/B = 0.22,\ V/L = 0.41/3.5 = 0.117,\ K = 0.85$

From curves, $m_1 = 0.198$ and $m_2 = 0.168$. Therefore,

$$Mb_1 = 210.64(0.198 + 0.15 \times 0.168) = 47.01 \text{ kN·m}$$

$$Ml_1 = 210.64(0.168 + 0.15 \times 0.198) = 41.64 \text{ kN·m}$$

For the asymmetric loads two cases arise as depicted in Fig. 9.35.

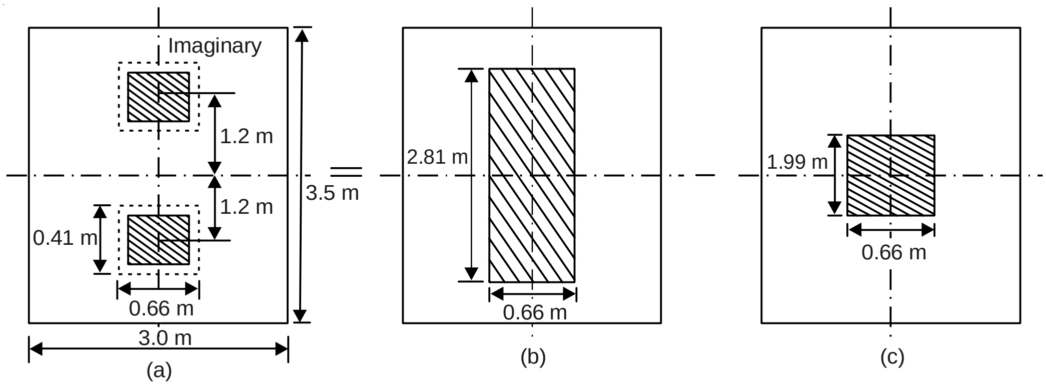

Fig. 9.35 Adjustments for eccentric wheel (for Fig. 9.34c).

For Fig. 9.35(b)

$$U/B = 0.22,\ V/L = 2.81/3.5 = 0.80,\ K = 0.9,\ m_1 = 0.12 \quad \text{and} \quad m_2 = 0.078$$

Concentrated load = 210.64 × 0.66 × 2.81 = 390.65 kN

$$Mb_{2a} = 390.65(0.12 + 0.15 \times 0.078) = 51.44 \text{ kN·m}$$

$$Ml_{2a} = 390.65(0.078 + 0.15 \times 0.12) = 37.50 \text{ kN·m}$$

For Fig. 9.35(c)

$$U/B = 0.22,\ V/L = 1.99/3.5 = 0.56,\ m_1 = 0.13 \text{ and } m_2 = 0.096$$

Concentrated load = 210.64 × 0.66 × 1.99 = 276.65 kN

$$Mb_{2b} = 276.65(0.13 + 0.15 \times 0.096) = 39.94 \text{ kN·m}$$

$$Ml_{2b} = 276.65(0.096 + 0.15 \times 0.13) = 31.95 \text{ kN·m}$$

Moments owing to actual loading are:

$$Mb = Mb_1 + 1/2(Mb_{2a} - Mb_{2b})$$

$$= 47.01 + \frac{1}{2(51.44 - 39.94)} = 52.76 \text{ kN·m}$$

$$Ml = Ml_1 + 1/2(Ml_{2a} - Ml_{2b})$$

$$= 41.64 + \frac{1}{2(37.50 + 31.95)} = 44.41 \text{ kN·m}$$

Therefore, the loading pattern represented by Case (ii) produces more bending moment compared with Case (i).

EXAMPLE 9.3

Obtain Courbon's reaction factor and the maximum bending moment in case of a T-beam bridge having the following details:

Roadway: 2 lanes
Loading: IRC Class A
No. of main girders: 3, c/c spacing = 2.6 m
Span of the bridge: 16 m
Kerb width: 600 mm on either side

Courbons Reaction Factor

The transverse disposition of the load is as shown in Fig. 9.36.

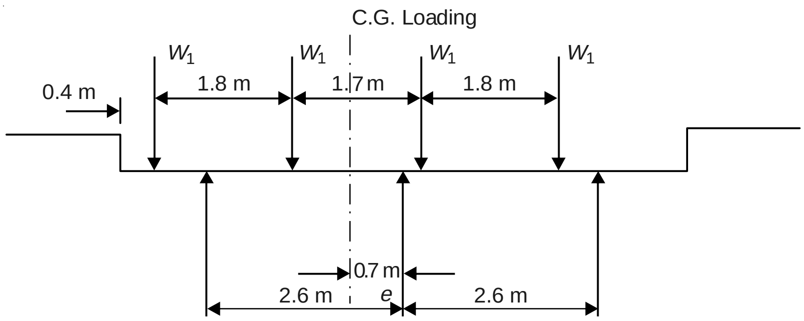

Fig. 9.36 Arrangement of IRC Class A vehicles (Example 9.3).

$$\Sigma x^2 = 2.6^2 + 0^2 + 2.6^2 = 13.52$$

Reaction factor for the exterior girder is

$$= \frac{4W_1}{3}\left(1 + \frac{3 \times 0.8 \times 2.6}{13.52}\right) = 1.948W_1 = 0.974W$$

Reaction factor for the interior girder is

$$= \frac{4W_1}{3}(1 + 0) = 1.33W_1 = 0.667W$$

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Maximum Bending Moment

The absolute maximum value of bending moment always occurs under the wheel load and not anywhere between the wheel loads. It occurs at a section near the centre of the span, under the heavier load, which is near the c.g. of the loading system considered. For this, the centre of the span is midway between the c.g. of the loading system and the maximum wheel load near the c.g. To accommodate as many wheels as possible on the span, consider 6 wheel loads having a total magnitude of 418 kN and a total length of 12.8 m. Taking moment about the outer 27 kN load we get, centre of gravity of loading as

$$x = \frac{1}{418}[(27 \times 1.1) + (114 \times 4.3) + (114 \times 5.5) + (68 \times 9.8) + (68 \times 12.8)]$$
$$= 6.42 \text{ m}$$

The c.g. of the loading lies at a distance of 6.42 – (1.1 + 3.2 + 1.2) = 0.92 m from the fourth load of 114 kN. This load should be placed such that the midpoint of the beam is halfway between this load and c.g. of the entire loading system. The arrangement is shown in Fig. 9.37.

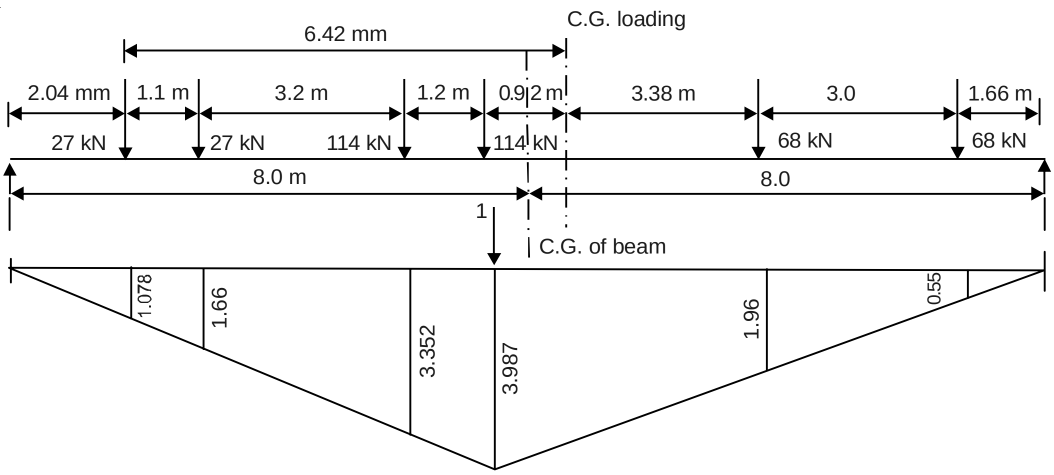

Fig. 9.37 Longitudinal placement of IRC Class A hain and I.L.D. for maximum bending moment (Example 9.3).

By comparing similar triangles, influence ordinates can be computed.

B.M. = (27 × 1.078) + (27 × 1.66) + (114 × 3.352) + (114 × 3.987) + (68 × 1.96) + (68 × 0.55)

= 1080.98 kN·m

Impact factor = 4.5/(6 + 16) = 0.204

B.M. for the outer girder (including reaction factor and impact factor)

= 1.204 × 0.974 × 1080.98 = 1267.66 kN·m

B.M. for the inner girder

= 1.204 × 0.667 × 1080.98 = 868.10 kN·m

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DESIGN PROBLEMS

1. Design a T-beam superstructure for a bridge on a national highway. The following details are available:

   Effective span: 18 m

   Live load: IRC Class AA (Tracked)

   Materials: M40 concrete, Fe 415 steel

   Spacing of cross girders: 3 m

   Sketch the reinforcement details in the component parts of the deck

2. Design a T-beam and slab bridge to span a river whose cross section is given below:

Distance from left bank (m)	0	10	20	30	40	50	60
Ground level (m)	110.2	109.4	108.6	107.2	101.5	100	100.8
	70	80	90	100			
	101.5	102.9	107.9	109.9			

The rock level is 2 m below the ground level at each point.

The other particulars are as follows:

Maximum flood level: 108.6 m

Road formation level: 112.50 m

The slope of the valley: 1 in 1500

Rugosity coefficient: 0.04

Maximum afflux at the site should be limited to 15 cm.

Also, draw the following views to a suitable scale:

1. Half longitudinal section and half longitudinal elevation
2. Transverse section of the bridge at mid span.
3. Half top view and half plan at top.

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Chapter 10

Plate Girder Bridges

10.1 INTRODUCTION

We have seen that when it comes to the question of design of steel beams, predefined rolled beams can be selected (once the design moments and shear forces are calculated) from a variety of sizes set forth in the IS specifications. It has been felt by bridge engineers that for long-span bridges in general, and longitudinally continuous bridges in particular, a greater amount of economy can be achieved by using plate girders. A plate girder is simply a girder (beam) made up of steel plates which are connected by rivets or welds. Plate girders are popular in the construction of railway bridges. Earlier, the elements of a plate girder used to be rivetted together using high strength rivets, which in turn have given way to the welded plate girder. Plate girders can be used for spans from 20 m to 30 m. Figure 10.1 illustrates the important components of a plate girder.

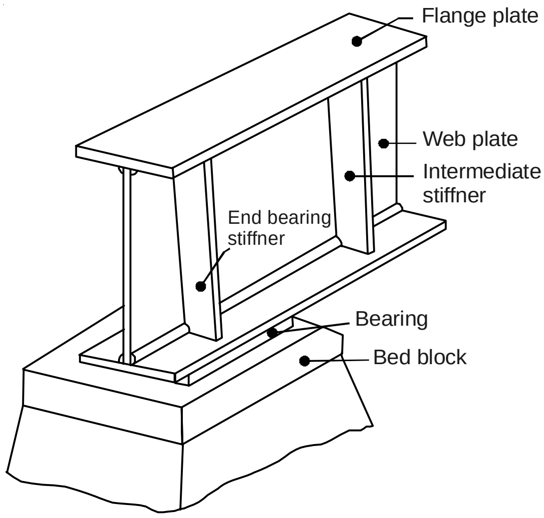

Fig. 10.1 Components of a plate girder.

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A hybrid girder is a plate girder which is made up of plates of different strengths, for example, high strength steel for top and bottom flanges and low strength steel for the web. A homogeneous girder will have steel plates of same strength throughout.

10.2 ELEMENTS OF A PLATE GIRDER AND THEIR DESIGN

As plate girders are preferred for construction of railway bridges, the loadings for these bridges follow the stipulations as laid down by the concerned authority in the Railways.

The loads recommended by Indian Railway Standards for each kind of track are called Equivalent Uniformly Distributed Loads (EUDL). These are listed in Table 10.1.

Table 10.1 Equivalent uniformly distributed loads (EUDL) (For broad gauge railway tracks)

Span (m)	*Total load* (kN, for B.M.)		*Total load* (kN, for S.F.)	
	Main line	*Branch line*	*Main line*	*Branch line*
1	458	346	458	346
2	458	346	524	396
3	469	354	655	495
4	592	448	788	594
5	748	567	903	683
6	852	644	1049	786
7	952	715	1152	871
8	1056	797	1239	935
9	1140	862	1324	1014
10	1210	941	1406	1083
12	1409	1084	1557	1191
15	1606	1232	1806	1382
20	2027	1580	2224	1726
25	2410	1880	2627	2037
30	2800	2165	3023	2375
40	3590	2820	3877	3048
50	4380	3470	4713	3702
60	5148	4052	5528	4310
70	5918	4610	6322	4882
80	6670	5130	7109	5438
90	7420	5680	7898	5978
100	8200	6160	8686	6512
110	8970	6685	9456	7042
120	9730	7195	10253	7564
130	10485	7700	11133	8804

10.2.1 Web

The web of a girder can be of constant height or of varying height. The girder with varying depth of web is called the *haunched girder*. The focus of our study this chapter will be on girders with constant depth of web. In designing the web of a plate girder, we compute its depth (which is dependent on maximum bending moment) and its thickness (which is dependent on

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shear stress). The depth of the web can be decided based on 'Economical depth' of the plate girder. It is given by

$$D = 5\sqrt[3]{\frac{M}{f_b}} \tag{10.1}$$

where

M = design bending moment after incorporating impact effect as given by the impact factor

f_b = permissible bending stress in steel which is taken as $0.66f_y$ (f_y is the yield stress of steel).

The thickness of the web can be calculated based on the shear stress criterion. The thickness should also provide the necessary bearing area. A minimum thickness of 8 mm (IRC 24) is adopted to provide for wear caused by corrosion. Inadequate dimensioning of a web may lead to web buckling. The minimum recommended thicknesses of web plates for different values of yield stress are given in IS 800 (1984).

10.2.2 Flanges

A flange should preferably be a single plate unless a plate of suitable thickness is not available. The width of the plate depends on the span to width ratio which ranges from 40 to 45. The flanges should be connected to the web by welds to transmit the horizontal shear force combined with any vertical loads which are directly applied. The thickness of the flange plate may be calculated based on the approximate requirements of the flange area. The area of a flange is given by

$$A_f = \frac{M}{f_b d} - \frac{A_w}{6} \tag{10.2}$$

where

d = depth of the web

A_w = area of the web.

However, the outstand of the flange should not be greater than 20 times the thickness of the plate. The section so devised (section means web and flanges) must be checked for critical stresses as stipulated by IS 800 (1984).

10.2.3 Intermediate Stiffeners

In order to avoid web failures (diagonal buckling) as well as to comply with lower web thickness, the web must be adequately supported laterally by stiffeners. There are two general types of stiffeners, namely:

- ➢ Vertical stiffeners located over the length of the span.
- ➢ Bearing stiffeners located at the supports of the span.

The arrangement of stiffeners is depicted in Fig. 10.1.

10.2.4 Vertical Stiffeners

The vertical stiffeners are provided at spacing not greater than $1.5d$ and not less than $0.33d$,

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where d is the depth of the web. The web panel dimension between two stiffeners should not be greater than 270 times the thickness of the web. The length of outstanding leg of the vertical stiffener may be taken as 12 times the thickness of the web. These vertical stiffeners should provide moment of inertia, which should not be less than

$$I = \frac{1.5d^3t^3}{c^2} \tag{10.3}$$

where

I = moment of inertia of the pair of stiffeners about the centre of the web or that of a single stiffener about the face of the web

t = thickness of the web

c = clear distance between vertical stiffeners

d = depth of the web.

These stiffeners are connected to the web plate, so as to withstand the shear force at the interface between the web and the stiffener which is given by

$$F = \frac{125t^2}{h} \tag{10.4}$$

where

F = shear force in kN/m

i = thickness of the web in mm

h = outstand of the stiffener in mm.

10.2.5 End Bearing Stiffeners

End bearing stiffeners are provided at the points of supports. The end bearing stiffeners strengthen the web and transmit heavy reactive forces to the flanges of the plate girders. The end bearing stiffeners are designed as columns. The sectional-area of an end bearing stiffener consists of the stiffener together with some length of the web (20 times the thickness of the web) on either side of the centre line of stiffeners. This area is used to determine the radius of gyration and to check the column stresses. Finally, the load bearing capacity of the stiffener as a column should be greater than the applied load or reaction.

10.2.6 Lateral Bracing for Plate Girders

Lateral bracing is a system of cross frames located in the horizontal plane and installed for connecting flanges in order to resist lateral deformation. Lateral deformation is induced by wind loads, which act normal to the centre line of the web. Lateral bracing is required if the span exceeds 20 m. Since the plate girders of railway bridges are considerably deep, the need for lateral bracing is justified. This increased depth creates a larger surface area of the web over which the wind forces can act.

EXAMPLE 10.1

Design a welded plate girder bridge for a broad guage railway line (single track), with splayed type wing walls across a stream having the following cross-sectional details.

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Distance (m)	0.00	7.50	15.00	22.50	30.00	37.50	45.0	52.50	60.0
Ground level (m)	188.7	188.5	181.5	180.5	180.0	181.0	182.2	188.8	189.0
Hard rock level (m)	181.2	181.0	179.0	178.5	177.0	178.0	179.0	180.0	180.6

Formation level of the embankment: 189.50 m
Discharge in the stream: 140 m^3/s
Maximum flood level: 186.00 m
Permissible afflux: 15 cm
Permissible bending stress in steel: 165 N/mm^2
Permissible shear stress in steel: 100 N/mm^2
Permissible bearing stress: 187.5 N/mm^2
Permissible shear stress in weld: 102.50 N/mm^2

Design of the Linear Waterway

With reference to Fig. 10.2, the length of the natural linear waterway at maximum flood level is

$$= 4.82 + 7.5 + 7.5 + 7.5 + 7.5 + 3.34 = 38.16 \text{ m}$$

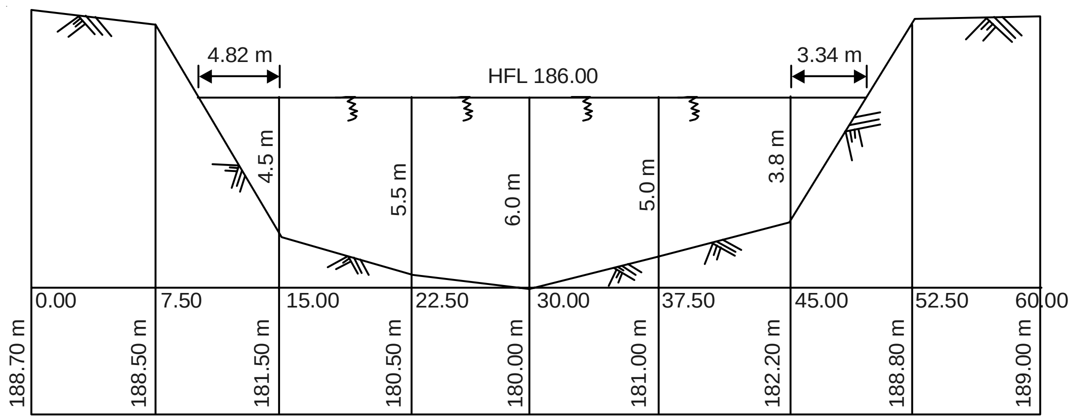

Fig. 10.2 Linear waterway under the bridge (Example 10.1).

Area of the natural waterway

$$= (0.5 \times 4.82 \times 4.5) + 0.5 \times 7.5\ (4.5 + 5.5) + 0.5 \times 7.5\ (5.5 + 6)$$
$$+ 0.5 \times 7.5\ (6 + 5) + 0.5 \times 7.5\ (5 + 3.8) + 0.5 \times 3.34 \times 3.34 \times 3.8$$
$$= 172.1 \text{ m}^2$$

Natural velocity in the stream = 140/172.1 = 0.81 m/s

Assuming an increase of 20% in velocity because of constriction of the waterway, velocity under the bridge = 1.20 × 0.81 = 0.97 m/s. Therefore,

Area of the artificial waterway = 140/0.97 = 144.32 m^2. Thus,

$$144.32 = L\ (x + d) = L\ (0.15 + 6)$$

Therefore,

$$L = 23.46 \text{ m, say } 25 \text{ m}$$

A span of 25 m may thus be adopted.

Dead Load and Live Load B.M. and S.F.

Dead load of the track: 7.5 kN/m

Self-weight of the girder = (0.2L + 1) = (0.2 × 25 + 1) = 6.0 kN/m

Total dead load: 7.5 + 6.0 = 13.5 kN/m

Live load (EUDL) for B.M. per track (Table 10.1) = 2410/2 = 1205 kN

Live load (EUDL) for S.F. per track (Table 10.1) = 2627/2 = 1313.5 kN

Impact factor = 20/(14 + L) = 20/(14 + 25) = 0.51 = 0.50

B.M. owing to dead load = 13.5 × (25^2/8) = 1054.68 kN·m

B.M. owing to live load = 1.5(1205 × 25)/8 = 5648.44 kN·m

Design bending moment = 6703.12 kN·m

Dead load S.F. = 13.5 × 25/2 = 168.75 kN

Live load S.F. = (1.5 × 1313.5)/2 = 985.12 kN

Design S.F. = 1153.87 kN

Dimensioning of the Plate Girder Section

$$\text{Economical depth } D \text{ of the girder } = 5\sqrt[3]{\frac{M}{f_b}}$$

$$= 5\sqrt[3]{\frac{6703.12 \times 10^6}{165}} = 1718.83 \text{ mm}$$

A web thickness of 12 mm may be assumed. The minimum depth of the web required, based on permissible shear stress of 100 N/mm^2, is obtained from

$$d = \frac{1153.87 \times 10^3}{100 \times 2} = 961.55 \text{ mm}$$

Looking into the above two values of depths (i.e. 1718.83 mm and 961.55 mm), the dimension of 1400 mm × 12 mm may be adopted for the web.

Flange Plates

Approximate flange area required is given by

$$A_f = \frac{M}{f_b d} - \frac{A_w}{6} = \frac{6703.12 \times 10^6}{165 \times 1400} - \frac{1400 \times 12}{6} = 26{,}218 \text{ mm}^2$$

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Flange width may be taken at $L/40$ to $L/45$ = 25000/40 to 25000/45
= 625 to 555.55 mm

Flange width of 600 mm is adopted.

Therefore, the thickness of the plate required = 26,218/600 = 43.69 mm, say 50 mm. The cross-section of the girder is shown in Fig. 10.3.

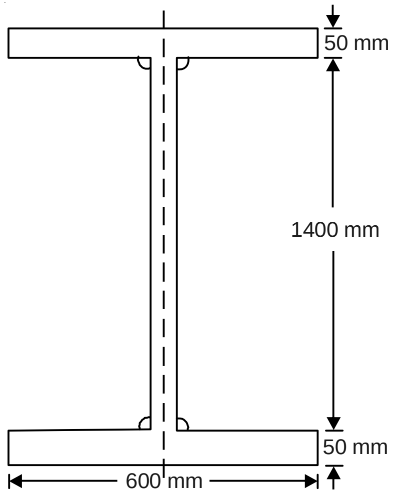

Fig. 10.3 Cross-section of the girder (Example 10.1).

Check for the cross-section adopted

The checking is done by verifying the bending stresses developed in the flange and comparing them with the permissible stress.

Moment of inertia of the section with respect to the neutral axis

$$I_{xx} = \begin{bmatrix} \text{M.I. of the outer} \\ \text{rectangle} \\ 600\text{ mm} \times 1500\text{ mm} \end{bmatrix} - \begin{bmatrix} \text{M.I. of the two rectangle,} \\ \text{each of dimensions} \\ 294\text{ mm} \times 1400\text{ mm} \end{bmatrix}$$

$$= \frac{0.6 \times 1.5^3}{12} - \frac{2(0.294 \times 1.4^3)}{12} = 0.034\text{ m}^4 = 3.42 \times 10^{10}\text{ mm}^4$$

Moment of inertia with respect to y–y axis

$$I_{yy} = \begin{bmatrix} \text{M.I. of two flanges} \\ \text{w.r.t. } y - y \end{bmatrix} + \begin{bmatrix} \text{M.I. of the web} \\ \text{w.r.t. } y - y \end{bmatrix}$$

$$= \frac{2 \times 0.05 \times 0.6^3}{12} + \frac{1.4 \times 0.012^3}{12} = 1.8 \times 10^9\text{ mm}^4$$

Area of cross-section, $A = 2 \times 50 \times 600 + 1400 \times 12 = 76{,}800 \text{ mm}^2$

$$\text{Radius of gyration, } r_{yy} = \sqrt{\frac{I_{yy}}{A}} = \sqrt{\frac{1.8 \times 10^9}{76{,}800}} = 153.1 \text{ mm}$$

The girders are provided with cross frames at 5 m c/c. Thus the length of the girder between the cross frames = 5 m. Overall depth of the girder (D) = 1.5 m, thickness of the flange (T) = 50 mm. Then the elastic critical stress can be calculated by finding X and Y. For this, the ratios, $D/T = 1500/50 = 30$, and $L/r_{yy} = 5000/153.1 = 32.65$ are matched in Table 6.5 of IS: 800, and from this table, we thus get $X = 1728$ and $Y = 1656$.

Elastic critical stress (f_{cb}) is calculated using the formula (IS: 800)

$$f_{cb} = k_1 (X + k_2 Y)(c_1/c_2)$$

where

k_1 = a coefficient whose value depends on Ψ, which is the ratio of total area of flanges at the point of the least moment to the corresponding area at the point of the greatest moment. In this case, $\Psi = 1$, from Table 6.3 of IS: 800, $k_1 = 1.00$.

k_2 = a coefficient to allow inequality of flanges, whose value depends on w, which is the ratio of M.I. of the compression flange to that of sum of M.Is. of flanges. In this case $w = 0.5$, for which $k_2 = 0$.

The c_1, c_2 are respectively the smaller and larger distances from the section neutral axis to the extreme fibres. In this case

$$c_1 = c_2, \; c_1/c_2 = 1$$

Therefore,

$$f_{cb} = 1(1728 + 0)1 = 1728 \text{ N/mm}^2$$

From Table 6.2 of IS: 800 (1984), for $f_{cb} = 1728$ and for $f_y = 250 \text{ N/mn}^2$, permissible compressive stress,

$$\sigma_{bc} = 157 \text{ N/mm}^2$$

Maximum bending compressive stress is

$$\sigma_{bc} = \frac{My}{I} = \frac{6703.12 \times 10^6 \times 750}{3.42 \times 10^{10}} = 146.9 \text{ N/mm}^2 < 157 \text{ N/mm}^2$$

$$\text{Average shear stress in the web} = \frac{1153.87 \times 10^3}{1400 \times 12} = 68.7 \text{ N/mm}^2$$

To obtain permissible average shear stress from the code, d/t and centre to centre spacing of stiffeners are required to be known.

$$d/t = 1400/12 = 116.66$$

Assuming the spacing to be $0.8d$, i.e. $0.8 \times 1400 = 1120$ mm or say 1100 mm, from Table 6.6A of IS: 800, we get

$$\tau_{va} = 100 \text{ N/mm}^2$$

Hence the average shear stress is within limits.

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Connection between the Flange and Web

The connection should be designed to take up the horizontal shear force developed at the junction of the web and flange. This is given by

$$\tau = \frac{VAY}{I} = \frac{1158.87 \times 10^3 \times 600 \times 50 \times 750}{3.42 \times 10^{10}} = 759.12 \text{ N/mm}$$

This shear force is to be taken by the weld. Providing continuous weld on either side, the strength of the weld of size S is given by

$$= 2 \times 0.7 \times S \times 102.5$$

Therefore, $2 \times 0.7 \times S \times 102.5 = 759.12$, which gives $S = 5.29$ mm

6 mm fillet weld may be provided on either side.

Design of Intermediate Stiffeners

As the ratio $d/t = 1400/12 = 116.6$ is greater than 85, vertical stiffeners need to be provided. Spacing (c) of stiffeners is taken to be 1100 mm. However, this spacing should not exceed 270 times the thickness of the web, i.e. $270 \times 12 = 3240$ mm.

Vertical stiffeners of 10 mm thickness with an outstand of 120 mm (this should not be more than $12 \times t = 12 \times 10 = 120$ mm) are adopted.
Moment of inertia provided by the stiffener (w.r.t. face of the web) = $10 \times 120^3/3 = 576 \times 10^4$ mm^4.
But, as per the code, the minimum moment of inertia, to he supplied by the stiffener (Fig. 10.4) is

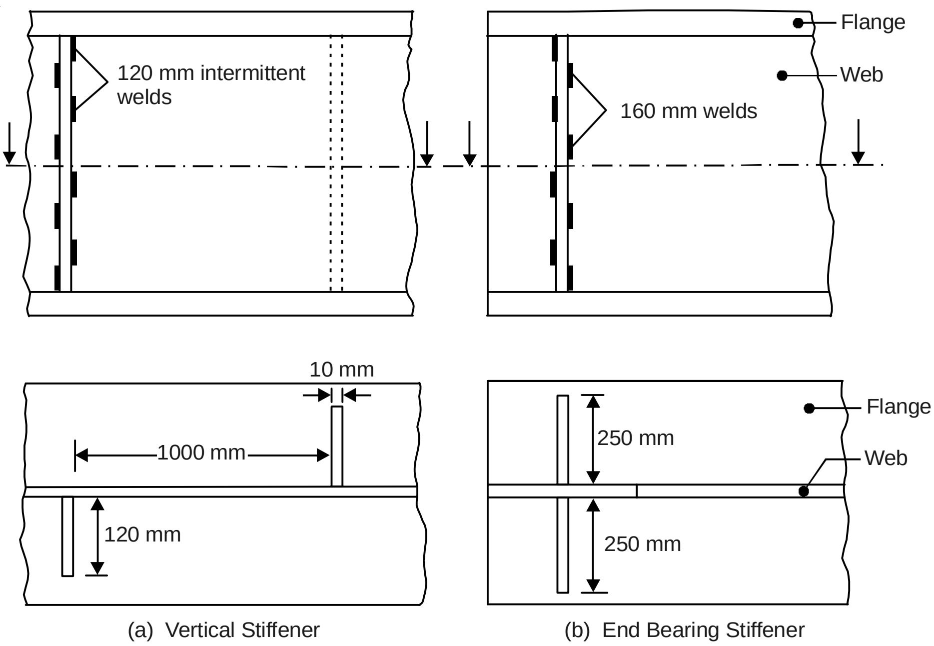

Fig. 10.4 Stiffeners (Example 10.1).

$$I = 1.5\, d^3 t^3 / c^2$$

$$= \frac{1.5 \times 1400^3 \times 12^3}{1100^2} = 494 \times 10^4 \text{ mm}^4$$

Therefore, the moment of inertia provided is more than the requirement.

Vertical Stiffener to Web Connection

The welds connecting the vertical stiffener and the web are supposed to bear the shear force at the interface. This shear force is given by

$$\text{S.F.} = 125 \frac{t^2}{h}$$

$$= \frac{125 \times 10^2}{120} = 150 \text{ kN/m}$$

Size of the weld is given by

$$S \times 0.7 \times 102.50 = 150$$

or

$$S = \frac{150}{0.7 \times 102.50} = 2.09 \text{ mm}$$

Use 120 mm (should not be less than $10t$ = 10 × 12 = 120 mm) long and 5 mm (minimum) sized intermittent welds. The welds may be alternatively placed on either side of the stiffner at 120 mm c/c.

End Bearing Stiffener

The end bearing stiffener should be designed as a column to bear the reaction of the girder as an axial load.

Reaction = 1153.87 kN

The ratio h/t (outstand/thickness) for the stiffener should not be more than 12. If the outstand is 250 mm, then, t = 250/12 = 20.83 mm, say 20 mm.

Bearing area actually needed = 1153.87 × 10^3/187.5 = 6153.97 mm^2

If two plates are used (one plate on each side of the web), then the bearing area actually provided = 2 × 250 × 20 – 10,000 mm^2

However, the adequacy of the bearing area provided should be checked based on the allowable axial compressive stress for the column.

It is assumed that along with the bearing stiffener plates, a certain length of the web plate also participates in taking compressive stresses. This length may be taken as 20 times the thickness of web, i.e. 20 × 12 = 240 mm. The moment of inertia of the section with respect to the central axis of the web (Fig. 10.4) is

$$I = \frac{20 \times 512^3}{12} + \frac{2 \times 230 \times 12^3}{12} = 2.237 \times 10^8 \text{ mm}^4$$

$$A = 512 \times 20 + 230 \times 2 \times 12 = 15{,}760 \text{ mm}^2$$

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$$\lambda = \text{slenderness ratio} = L/r$$

$$r = \sqrt{\frac{I}{A}} = \sqrt{\frac{2.237 \times 10^8}{15,760}} = 119.13\ \text{mm}$$

Effective length of the stiffener = 0.65 × 1400 = 910 mm

λ = 910/119.13 = 7.63. From Table 5.1 of IS: 800, we have σ_{ac} = 150 N/mm^2

Therefore, the area required = (1153.87 × 10^3)/150 = 7693 mm^2

Design of Lateral Bracings

Two lateral bracings can be provided at each end of the girder to take up wind force, racking force and centrifugal force. A wind force of intensity 2 kN/m^2 may be assumed to prevail. This wind force will act on the exposed area of the girder. The wind force on the other girder (leeward girder) may be taken as 25% of the force in the windward direction. Refer Fig. 10.5.

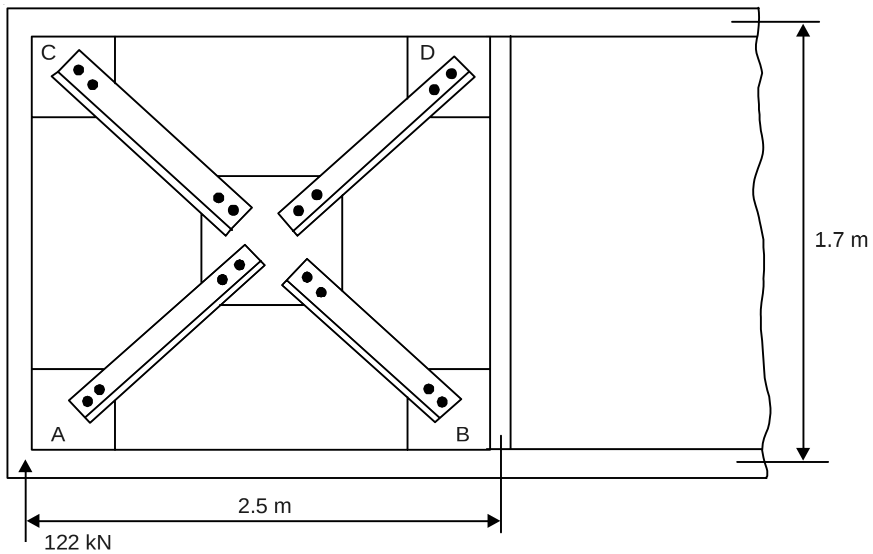

Fig. 10.5 Lateral bracing (Example 10.1).

Wind force on the girder (windward) = 2 × 1.5 × 25 = 75 kN

Force on the leeward girder = 0.25 × 75 = 18.75 kN

Lateral load owing to racking is assumed at 6 kN/m.

Total racking force = 6 × 25 = 150 kN

Total design load = 150 + 75 + 18.75 = 243.75 kN ≈ 244 kN

This total force acts in such a way that half of it is at one end. This loading creates diagonal tension in the member BC.

Tensile force in the member BC is given by

$$= \frac{244}{2} \text{ cosec } \theta = \frac{122\sqrt{2.5^2 + 1.7^2}}{1.7} = 217 \text{ kN}$$

Area of cross-section required for this tension member

$$= \frac{217 \times 10^3}{0.6 \times 250} = 1447 \text{ mm}^2$$

Angle ISA 90 mm × 90 mm × 10 mm could be tried.

Area provided = 2019 mm^2, r_y = 27.1 mm

Effective length = 0.65 × 3.02 = 1.965 m

Slenderness ratio, $\lambda = L/r_y$ = 1.965 × 1000/27.1 = 72.50

From Table 5.1 of IS: 800, permissible stress σ_{ac} = 109.25 N/mm^2

Safe load on the member = 109.25 × 2019 = 220.57 kN

This load is more than 217 kN.

Maximum compressive stress develops in the member AC which is equal to reaction = 122 kN

Effective length = 0.65 × 1.7 = 1.105

Design of Cross Frames (Fig. 10.6)

In order to maintain the alignment provided by girders and supplement additional lateral stability, cross frames are provided. Cross frames are provided for every 5 m distance.

The lateral reactive force (owing to wind and racking) creates tension in one of the diagonal members [AD].

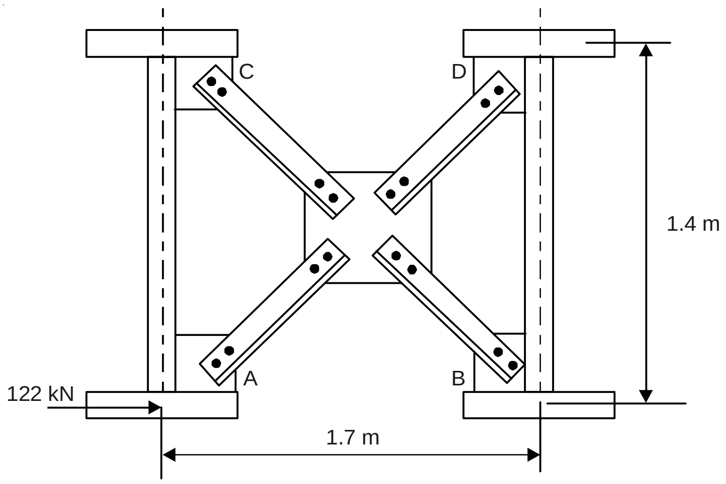

Fig. 10.6 Cross frames (Example 10.1).

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The force in the diagonal member AD is

$$= \frac{122}{\cos\theta} = \frac{122 \times 2.88}{1.676} = 158.70 \text{ kN}$$

$$\text{Area required} = 158.7 \times 10^3/(0.6 \times 250) = 1057.91 \text{ mm}^2$$

ISA 90 mm × 60 mm × 10 mm angles may be selected. The area provided by these angles is 1401 mm^2.

Dimensioning of Substructures

Looking into ground profile and hard rock profile, the ground level at the location of the abutment can be taken as 181 m and hard rock level as 179 m.

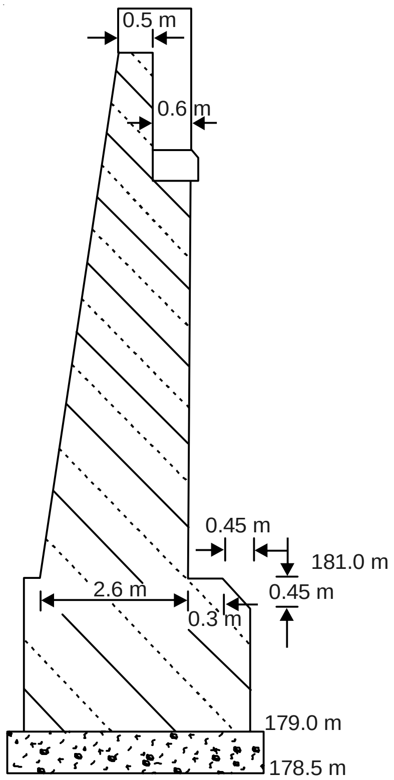
Height of the abutment up to the ground level = 189.50 – 181.00 = 8.50 m

The front face of the abutment may be made vertical, and a batter of 1 in 6 may be provided at the back.

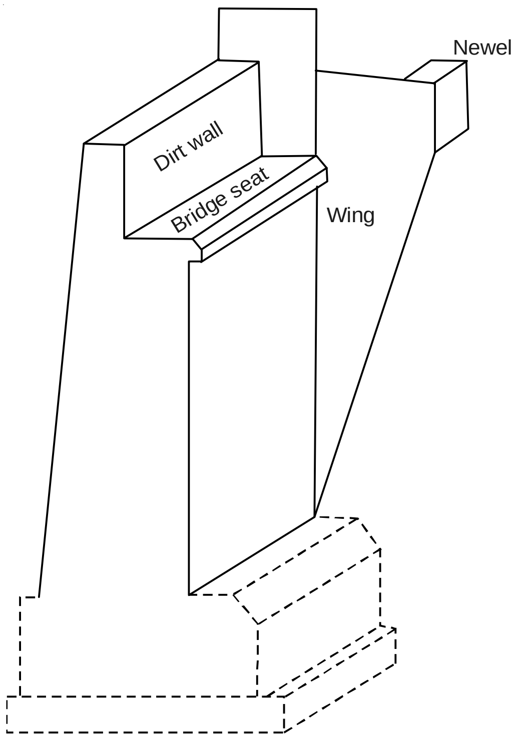
Top width of the abutment = bearing + clearance + dirt wall
= 0.5 + 0.1 + 0.5 = 1.1 m

$$\text{Bottom width of the abutment} = \text{top width} + \frac{1}{6}\ (\text{height}) = 1.1 + \frac{1}{6}(8.5) = 2.5 \text{ m}$$

The abutment is taken further down, till the rock level is met. The cross-section of the abutment is sketched in Fig. 10.7.



(a) Cross-section



(b) Configuration

Fig. 10.7 Abutment details (Example 10.1).

The wing wall will have top width of 0.5 m and it slopes down to a level, which is 50 cm above the general ground level. The wing wall shall have a splay of 45°. The wing wall at the junction with the abutment will have the same foundation width as that of the abutment. At the end, the wing wall will have its top level at 187.60 m. The hard rock level is at 179 m.

$$\text{Bottom width of the wing at the end} = 0.5 + \frac{1}{6}(187.6 - 184.5) \approx 1.00 \text{ m}$$

The plan showing the arrangement of the abutment and the wing wall is shown in Fig. 10.8. The drawing details of the bridge are shown in Plate 6.

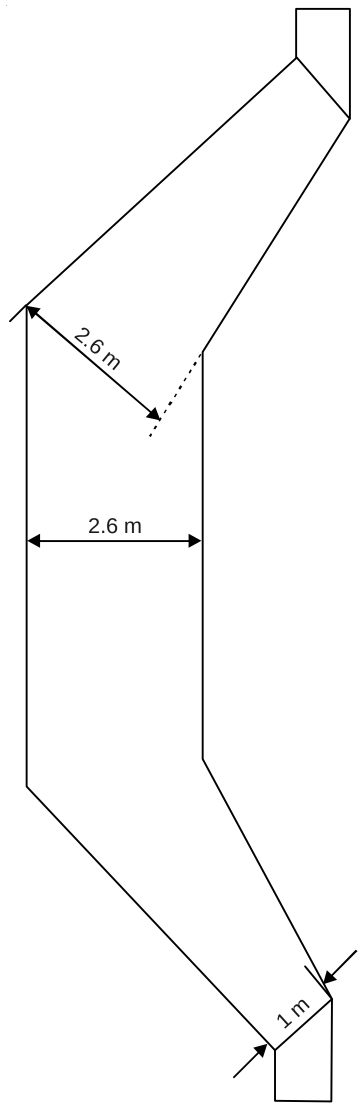


Fig. 10.8 Abutment and wing wall at foundation level (Example 10.1).

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DESIGN PROBLEMS

1. Using the following particulars to design a plate girder bridge for a broad gauge track.

   Span: 20 m

   Top level of the railway embankment: 115 m

   Bed level of the stream: 100 m

   Ground level suitable for foundation: 98 m

   Stream bund top level: 101.50 m

2. Using the following particulars, design a plate girder bridge with suitable abutment and wing walls for a broad gauge track.

   Span: 12 m

   Top level of the rail: 111.20 m

   Bed level of the stream: 100 m

   High flood level: 104 m

   Foundation level: 98 m

   Also, draw the following views of the bridge:

   1. Half-longitudinal section
   2. Transverse section

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Chapter 11

Composite Bridges

11.1 INTRODUCTION

A composite bridge is one whose decking system consists of a concrete slab and which in conjunction with steel girders resists moving loads on the bridge. This type of bridge is found to be economical for spans of 10 to 20 m. A composite bridge offers:

- More efficient use of materials, since the size of the steel member can be significantly reduced owing to incorporation of the deck into the resisting cross-section, i.e. into the compression zone.
- Greater vertical clearance by effecting reduction in beam depth.
- Enhanced stiffness, which in turn makes the deck sustain greater vehicle loading.

Because of the above advantages, composite construction may be utilised whenever possible in an effort to maximise the performance and use of not only different materials but also different structural elements.

11.2 COMPOSITE ACTION

It is said that composite construction has its roots in the mid-nineteenth century. However, the composite bridge construction did not take effect until about late 1940s. To understand how composite construction brings in economy of materials, we have to look back at the basic strength of materials.

From the bending theory, the maximum bending stress in a beam subjected to pure bending is given by

$$f = \frac{My}{I} \tag{11.1}$$

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where

f = bending stress in the beam in MPa

M = bending moment in N·mm

y = distance of the extreme fibre from the neutral axis in mm

I = moment of inertia of the resisting section in mm^4

The above equation can be modified to

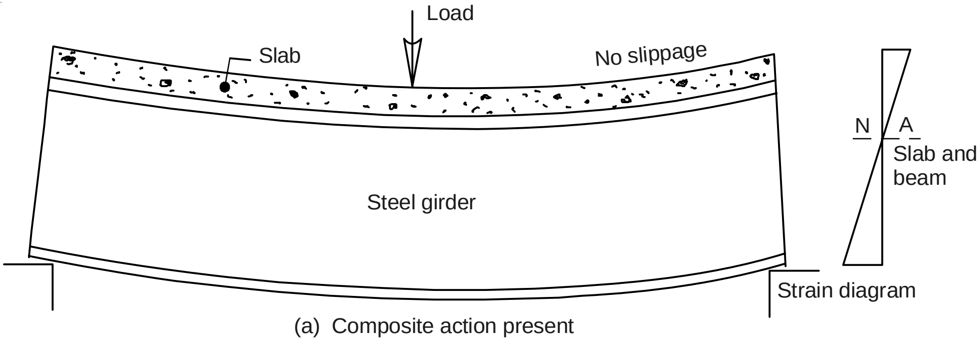
$$f = \frac{M}{Z} \tag{11.2}$$

where Z = section modulus [I/y].

The section modulus is dependent only on the geometry of the cross-section. By observation, we can see that the bigger the value of Z, the smaller the resulting stress. Therefore, it is in the best interest of the designer to increase the section modulus as much as possible. Composite sections provide substantial section modulus with minimum material and it is here that the principal advantage of composite action comes into play.

11.3 SHEAR CONNECTORS

The shear connectors are part and parcel of a composite deck system. The need for shear connectors can be understood by considering the interaction between the slab and the steel beam. If the slab simply rests on the steel beam, a phenomenon known as slippage occurs. As the loads are placed on the top of the slab, the top of the slab and the beam will be in compression while the bottom of the slab and the beam will be in tension. Both the slab and the steel beam behave independently deflecting like a beam. Since the bottom of the slab is in tension (tending to push outwards) and the top of the beam is in compression (tending to move inwards), the resulting effect is manifested by extension of the slab over the ends of the beam. This is shown in Fig. 11.1.



(a) Composite action present

Fig. 11.1 (Contd.)

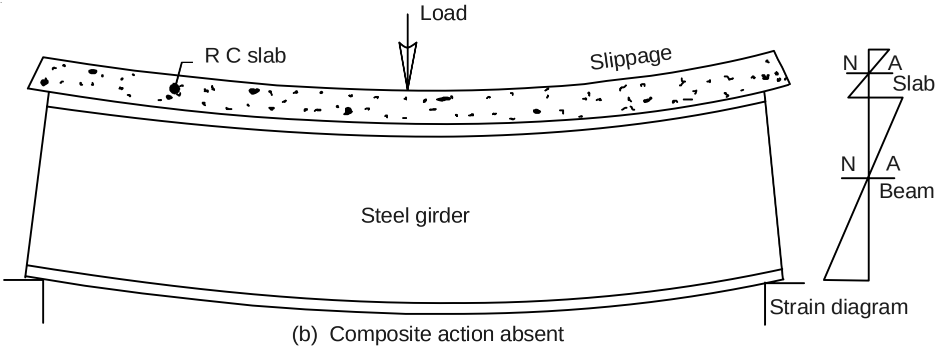


Fig. 11.1 Composite action.

It is possible to somehow connect the concrete slab and the steel beam such that they resist the loads like a common unit. Such a one-to-one unity between the two units can be achieved by providing **shear connectors** between the slab and the beam.

A shear connector is generally a metal element of particular shape, which extends vertically from the top flange of the supporting beam and gets embedded into the slab. Depending upon the magnitude of the shear force at the interface of the beam and the slab, a number of shear connectors can be placed along the length of the beam. With shear connectors in place, the slab and beam can now be analysed as a single unit. The composite section will now have a higher section modulus that allows the composite beam to resist higher loads. In a nutshell, the I-shaped beam gets replaced more or less by a T-shaped beam.

11.3.1 Design Requirements of Shear Connectors

The shear connectors have to be designed to facilitate conjoint action between the RC slab and the steel beam. Their basic function is

- ➢ To transfer the shear force at the interface of the slab and the beam without slip.
- ➢ To prevent separation of the slab from the steel beam in the perpendicular direction.

There are rigid and flexible shear connectors. The rigid types include channel angles, tee sections while the stud types of shear connectors come under the flexible type. The design criterion for the stud type of shear connector is explained here. The different types of shear connectors are sketched in Fig. 11.2.

For a welded stud connector of steel with minimum ultimate strength of 460 MPa, and yield strength of 350 MPa and elongation of 20%, the safe shear for each shear connector is given by

$$Q = 4.8hd\sqrt{f_{ck}} \tag{11.3}$$

where

Q = the safe shear resistance in newton of one shear connector
h = height of the stud in mm
d = diameter of the stud in mm
f_{ck} = characteristic compressive strength of concrete.

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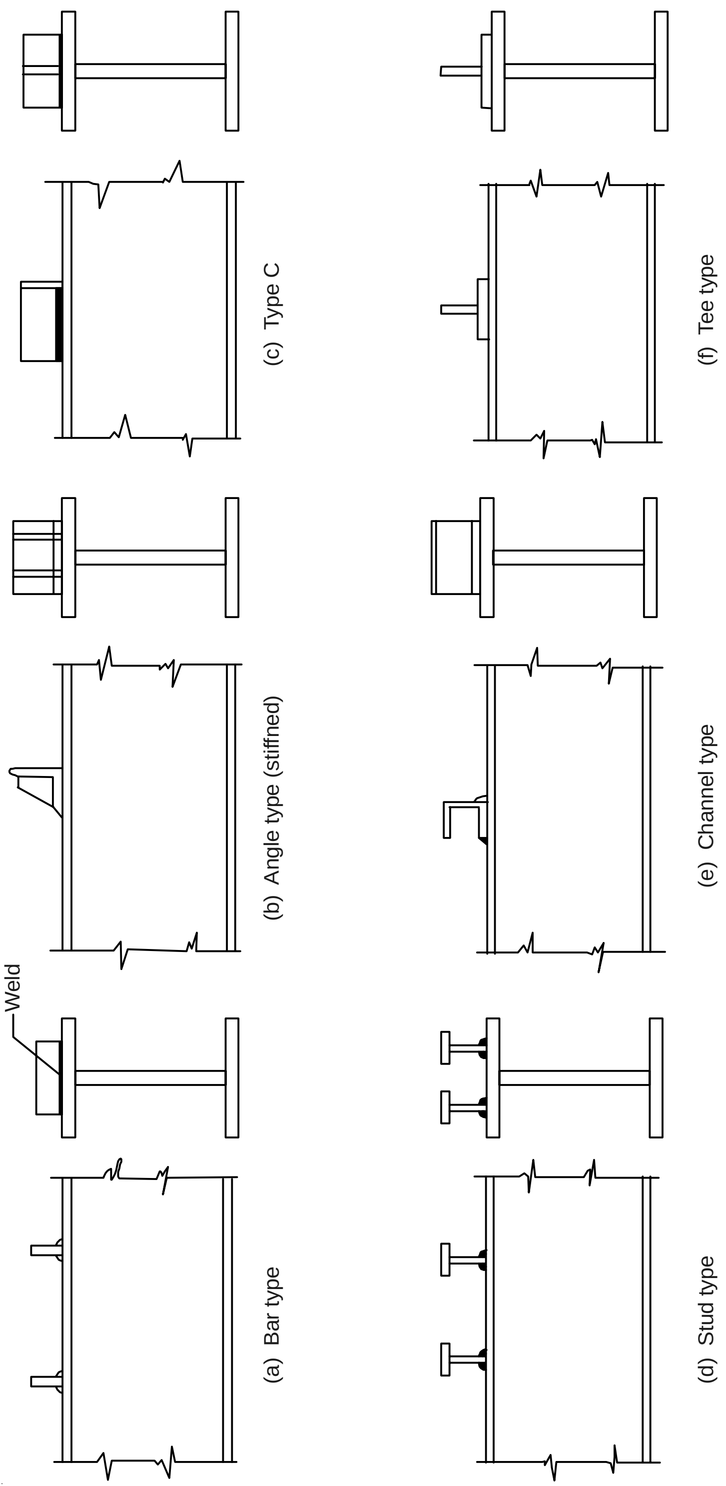

Fig. 11.2 Rigid (a, b, c,) and flexible (d, e, f,) types of shear connectors.

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The above equation is applicable for a ratio h/d less than 4.2. For a ratio h/d equal to or greater than 4.2, the safe shear resistance is given by

$$Q = 196d^2\sqrt{f_{ck}} \tag{11.4}$$

The spacing of the shear connectors can be determined from the formula,

$$p = \frac{NQ}{FS_h} \tag{11.5}$$

where

p = spacing of the connectors in mm
N = no. of connectors in a row
Q = safe shear resistance of a connector in newton
S_h = horizontal shear stress developed per unit length of the beam in N/m
F = factor of safety, generally taken as 2.

11.4 COMPOSITE OR TRANSFORMED SECTION

The composite slab-beam section is converted into a modified section where the concrete slab turns into an equivalent area of steel. This conversion is brought through the use of modular ratio m. The modular ratio m is given by

$$m = \frac{E_s}{E_c} \tag{11.6}$$

where

E_s = modulus of elasticity of steel in MPa
E_c = modulus of elasticity of concrete in MPa.

Figure 11.3 shows what a transformed section looks like. The design of a composite bridge deck can be understood by the example presented below.

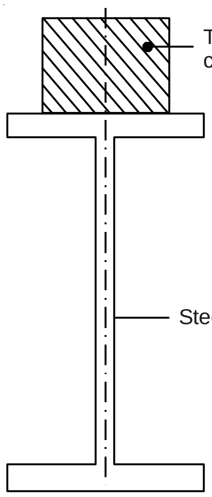

Fig. 11.3 Composite/Transformed section.

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EXAMPLE 11.1

Design a composite bridge deck consisting of an RCC slab on steel girders. The span of the bridge is 15 m. The other details are:

Road: Two lane Highway

Kerbs: 600 mm on either side

No. of steel girders: 4

Spacing of girders: 2.5 m c/c

Materials: M40 concrete, Fe 415 steel

Other details:

Bed level: 150 m
Bed width: 21 m
Stream bund top: 152.50 m
Road top level: 155.50 m
Hard rock level: 148.50 m
Wing walls: Return type.

Preliminary Dimensions

The cross-section of the deck with assumed dimensions is shown in Fig. 11.4

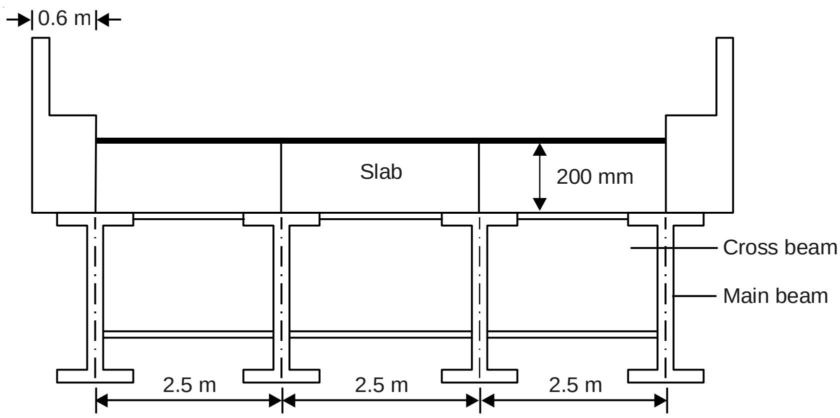

Fig. 11.4 Preliminary dimensions of the deck (Example 11.1).

Calculation of Dead Load and Live Load Bending Moments for the Slab

The slab is designed as a panel supported on all the four edges. The continuity of the slab over the supports in two directions is accounted by a continuity factor which is taken as 0.8. In this problem, the cross girders are assumed at 3 m c/c. Thus the interior panel of the deck slab need be treated as a two way slab. Pigeaud's method can be used for calculation of the dead load and live load bending moments.

The panel dimensions are 2.5 m × 3 m (Fig. 11.5).

Dead weight of the slab = 0.20 × 24 = 4.80 kN/m^2

Dead weight of the wearing coat = 0.08 × 22 = 1.76 kN/m

Total dead weight = 6.56 kN/m^2 ≈ 6.6 kN/m^2

For calculating the live load B.M., IRC-AA tracked vehicle is considered as it gives more severe effects. The lengthwise disposition of the wheel on the slab is not complete. However, proportionate loading has been considered.

The dispersed contact dimension of the wheel along the long span direction is

$$3.60 + 2 \times 0.08 = 3.76 \text{ m}$$

out of which only 3 m is available on the slab.

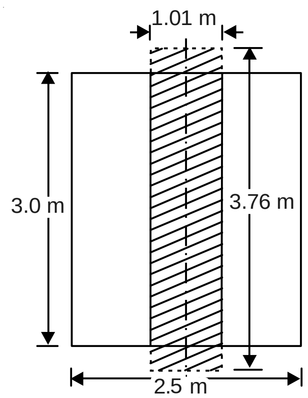

Fig. 11.5 Slab panel and Class AA wheel (Example 11.1).

Along the short span the dispersed dimension is (see Fig. 11.5)

$$0.85 + 2 \times 0.08 = 1.01 \text{ m}$$

The dimensionless ratios are

$$\frac{U}{B} = \frac{1.01}{2.5} = 0.40$$

$$\frac{V}{L} = \frac{3}{3} = 1.00$$

$$K = \frac{B}{L} = \frac{2.5}{3} = 0.833$$

From Pigeaud's curves, we get

$$m_1 = 0.08, \qquad m_2 = 0.0425$$

As the dispersion length (along long span) is not fully available on the panel, the load transmitted on account of the wheel, i.e. the magnitude of the wheel load gets reduced to a certain extent.

$$\text{Actual load to be considered} = \frac{350 \times 3}{3.76} = 279 \text{ kN}$$

Short span live load bending moment = 279[0.08 + (0.15 × 0.0425)] = 24.1 kN·m

Bending moment including impact factor (25%) and continuity factor,

$$= 1.25 \times 0.8 \times 24.1 = 24.1 \text{ kN·m}$$

Long span bending moment including continuity and impact factors,

$$= 1.25 \times 0.8 \times 279\ [0.0425 + (0.15 \times 0.08)] = 15.2 \text{ kN·m}$$

Dead load bending moment is calculated using Pigeaud's curves. Here, the entire dead load transmitted by the slab is symmetrical and is spread on the entire panel uniformly. So the dimensionless ratios are

$$\frac{U}{B} = 1.00,\ \frac{V}{L} = 1.00,\ K = \frac{B}{L} = 0.833$$

Total dead load (equivalent concentrated load at the centre of the slab)

$$= 6.6 \times 2.5 \times 3 = 49.5 \text{ kN}$$

Using Pigeaud curve for K = 0.8 and $1/K$ = 1.25, we get

$$m_1 = 0.045 \text{ and } m_2 = 0.028$$

Dead load bending moments considering the continuity factor:

$$\text{Along short span } = 0.8 \times 49.5[(0.045 + (0.15 \times 0.028)]$$
$$= 1.95 \text{ kN·m}$$
$$\text{Along long span } = 0.8 \times 49.5[(0.028 + (0.15 \times 0.045)]$$
$$= 1.38 \text{ kN·m}$$

Therefore the final design moments are

$$\text{Along short span} = 24.1 + 1.95 = 26.05 \text{ kN·m}$$
$$\text{Along long span} = 15.2 + 1.38 = 16.58 \text{ kN·m}$$

Design of the Slab

$$\text{Effective depth required} = \sqrt{\frac{26.05 \times 10^6}{2.31 \times 1000}} = 106.2 \text{ mm}$$

Providing 12 mm diameter bars, with a cover of 40 mm, we have

$$\text{Actual effective depth} = 200 - 40 - 6 = 154 \text{ mm}$$

$$\text{Area of steel} = \frac{26.5 \times 10^6}{200 \times 0.9 \times 154} = 939.75 \text{ mm}^2$$

Provide 12 mm diameter bars at 100 mm c/c.

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Effective depth along long span using 10 mm diameter rods

$$= 154 - 6 - 5 = 143 \text{ mm}$$

$$\text{Area of distribution steel} = \frac{16.58 \times 10^6}{200 \times 0.9 \times 143} = 644 \text{ mm}^2$$

Provide 10 mm diameter rods at 100 mm c/c.

Design of the Steel Girder

Calculation of dead load and live load bending moments

The grid pattern of the steel girders is shown in Fig. 11.6.

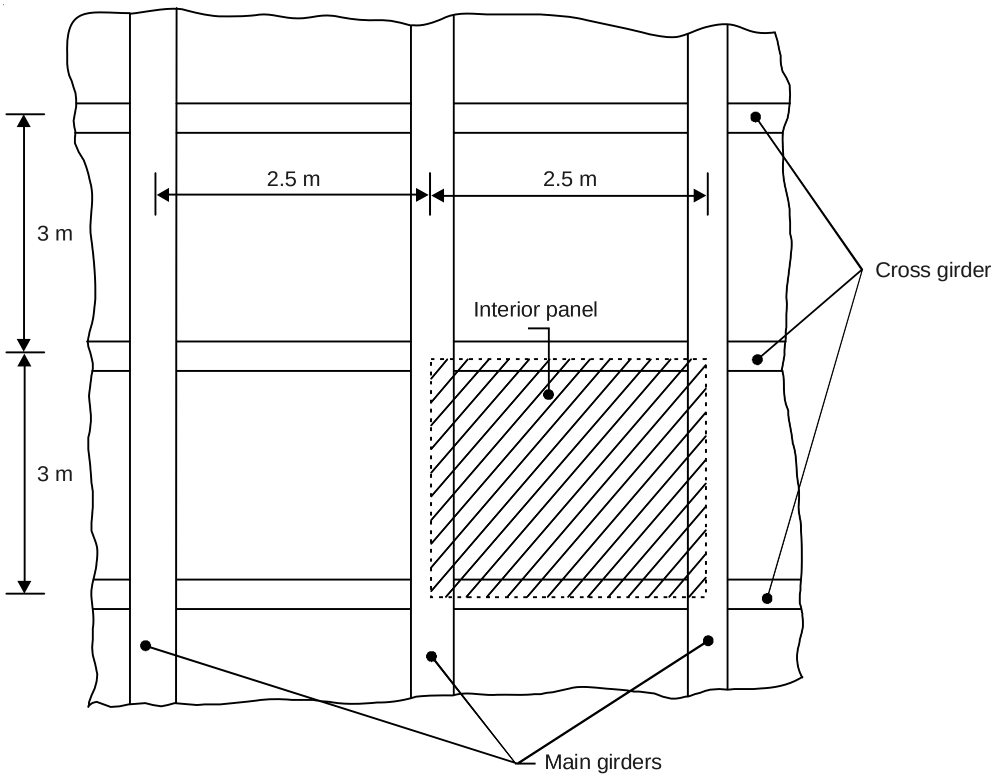

Fig. 11.6 Grid pattern of steel girders (Example 11.1).

The dead load on the girder owing to slab portion of 2.5 metre width (flange width is taken as centre to centre of girders).

$$= 6.6 \times 2.5 = 16.5 \text{ kN/m}$$

Self-weight of the steel girder is given by $(0.2L + 1)$ kN/m, i.e. $(0.2 \times 15 + 1) = 4$ kN/m so that the total load is 16.5 + 4 = 20.5 kN/m. The cross girders placed at regular intervals of 3 m shall impound some weight on the main girder in the form of concentrated load. This load (self-weight of cross girder is taken as 1 kN/m) is = 2.5 × 1 = 2.5 kN.

The dead load pattern on the steel girder is idealised in Fig. 11.7(a).

Maximum dead load bending moment which occurs at the centre of the beam is

$$= \frac{20.5 \times 15^2}{8} + (5 \times 7.5) - (2.5 \times 1.50) - (2.5 \times 4.50)$$

$$= 599 \text{ kN·m}$$

$$\text{Dead load shear force} = \frac{20.5 \times 15}{2} + \frac{4 \times 2.5}{2} = 158.75 \text{ kN}$$

Maximum live load bending moment is obtained when an IRC Class AA tracked wheel is placed at the centre of the beam. A single wheel is placed on the beam at the centre [Fig. 11.7(b)].

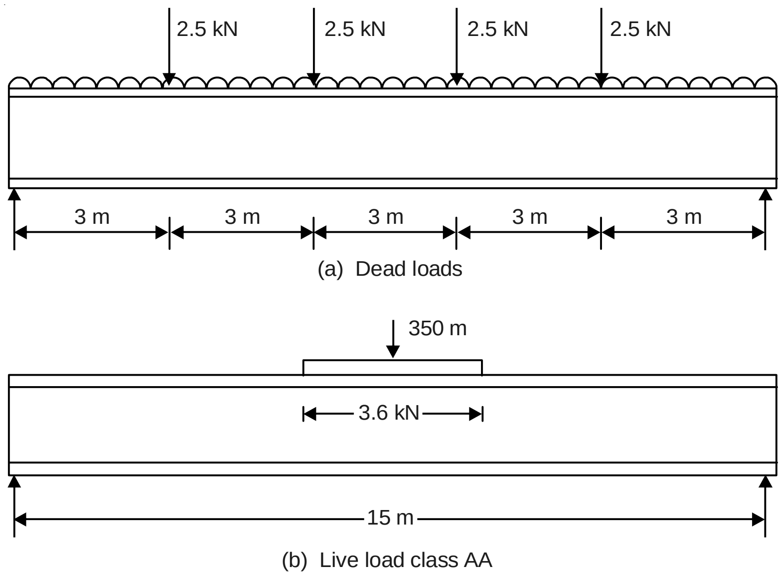

Fig. 11.7 Girder loading (Example 11.1).

$$\text{Maximum live load bending moment} = 175 \times 7.5 - \left(\frac{350}{3.6} \times \frac{1.8}{2}\right)$$

$$= 1225 \text{ kN·m}$$

Impact factor is 10%.

Actual live load bending moment = 1.1 × 1225 = 1347.5 kN·m

Design bending moment = 599 + 1347.5 ≈ 1946.5 ≈ 1947 kN·m

Design shear force = 158.75 + (1.1 × 175) ≈ 351.25 kN ≈ 352 kN

Dimensioning of the girder

The economical depth of the girder is given by

$$D = 5\sqrt[3]{\frac{M}{\sigma_{bc}}} = 5\sqrt[3]{\frac{1947 \times 10^6}{165}} = 1138.31 \text{ mm}$$

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A web depth of 1000 mm can be adopted. The thickness of the web may be taken as 10 mm. With these dimensions of the web, the shear stress in the web is

$$= \frac{352 \times 10^3}{1000 \times 10} = 35.2 \text{ MPa} < 100 \text{ MPa}$$

Flange plate

Approximate flange area required is given by

$$A_f = \frac{M}{\sigma_{bc} d} - \frac{A_w}{6}$$

$$= \frac{1947 \times 10^6}{165 \times 1000} - \frac{1000 \times 10}{6} = 10{,}133.33 \text{ mm}^2$$

Flange width may be taken as $L/40$ to $L/45$, i.e. 15000/40 to 15000/45 = 375 mm to 333 mm Adopt the flange width as 400 mm so that its thickness is 10,642.42/400 = 26.60 ≈ 30 mm The section so devised must be checked for maximum bending stress.

Moment of inertia of the section about the neutral axis is

$$I = \frac{10 \times 1000^3}{12} + 2\left(\frac{30^3 \times 400}{12} + 30 \times 400 \times 515^2\right)$$

$$= 7.2 \times 10^9 \text{ mm}^4$$

Bending tensile stress is

$$\frac{My}{I} = \frac{1947 \times 10^6 \times 530}{7.2 \times 10^9}$$

$$= 143.32 \text{ N/mm}^2 < 165 \text{ N/mm}^2$$

Check for shear

Permissible average shear stress is dependent on the web dimensions as the web is assumed to bear the shear. It depends on the ratio d/t which in this case is 1000/10 = 100.

Adopting a stiffener spacing of 1000 mm ($c = 1d$), from Table 6.6 of IS 800, average permissible shear stress is found to be 100 MPa.

But average shear stress developed is 37.4 N/mm^2, which is within the permissible limit.

Connection between Flange and Web

Shear force is critical at the junction of the web and flange, which is given by

$$\text{Average shear stress} = \frac{VAY}{I}$$

$$= \frac{352 \times 10^3 \times 400 \times 30 \times 515}{7.2 \times 10^9}$$

$$= 302.13 \text{ N/mm}$$

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A continuous weld on both sides may be provided. The strength of the weld is equated to the shear force to be resisted in order to arrive at weld thickness.
Strength of rivet is

$$= 2 \times 0.7 \times 102.5 \times s = 145s$$

or

$$145s = 302.13$$

Therefore,

$$s = 2.08 \text{ mm}$$

As 2.03 mm is not practicable, a minimum weld thickness of 5 mm is provided.

Design of Intermediate stiffeners

As the ratio d/t = 100 which is more than 85, the web needs to be stiffened with vertical intermediate stiffeners. The spacing of stiffeners is taken as 1000 mm (spacing can be from 0.33d to 1.5d). Intermediate stiffeners are supposed to provide moment of inertia as given by IS 800. Therefore,

$$I = \frac{1.5d^3t^3}{c^2}$$

$$= \frac{1.5 \times 1000^3 \times 10^3}{1000^2} = 15 \times 10^5 \text{ mm}^4$$

Assuming the thickness of the plate to be 10 mm, its outstanding length should not be greater than 16t, i.e. 16 × 10 = 160 mm, so a plate of cross-sectional dimension 10 mm × 100 mm can be selected. The moment of inertia (w.r.t. web central line) actually provided becomes

$$= \frac{10 \times 100^3}{3} = 33.3 \times 10^5 \text{ mm}^4$$

Connections of Vertical Stiffener to Web

The connection between the web and the stiffener has to sustain the shear force, which is given by

$$= \frac{125t^2}{h} = \frac{125 \times 10^2}{100} = 125 \text{ kN/m} = 125 \text{ N/mm}$$

Size of the weld

$$s = \frac{125}{0.7 \times 102.5} = 1.75 \text{ mm}$$

However, a minimum size of 5 mm for the weld may be adopted. The weld need not be continuous since the thickness of the weld is more than that actually required. Instead, the weld can be placed alternatively on either side. The length of the weld can be made equal to 10t, i.e. 10 × 10 = 100 mm and spacing may be provided at 100 mm c/c.

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End Bearing Stiffeners

An end bearing stiffener is designed as a column to take the reaction at the end of the girder.
Reaction at the end of the girder = 352 kN

The stiffener is designed as long column for which h/t, i.e. outstand length/thickness should not be greater than 12. If the outstand is assumed as 200 mm the required thickness would be t = 200/12 = 16.66 mm, say 18 mm

Bearing area to be provided = Reaction/Permissible bearing stress

$$= \frac{352 \times 10^3}{0.75 f_y} = \frac{352 \times 10^3}{0.75 \times 250} = 1877.33 \text{ mm}^2$$

Total bearing area provided = 2 × 200 × 18 = 7200 mm^2 > 1877.33 mm^2

It is assumed that along with the stiffener, a portion of the web will also take up the reaction. The length of the web which is assumed to take the share is taken as $20t_w$ = 20 × 10 = 200 mm.

Moment of inertia of the bearing area w.r.t. central web axis is

$$= \frac{18 \times 410^3}{12} + \frac{2 \times 200 \times 10^3}{12} = 86{,}184{,}583 \text{ mm}^4$$

Area = (400 × 10) + (2 × 200 × 18) = 11,200 mm^2

$$\text{Radius of gyration} = \sqrt{\frac{I}{A}} = \sqrt{\frac{86{,}184{,}583}{11{,}200}} = 93 \text{ mm}$$

Effective length of the stiffener (The stiffener is held in position but restrained against rotation)

= 0.7 × 1000 = 700 mm

Slenderness ratio = 700/93 = 7.5

Referring to Table 5.1 of IS 800, permissible stress σ_{ac} (MPa) in axial compression for steel of f_y = 250 MPa is 150 MPa.

$$\text{Therefore, the area actually needed} = \frac{352 \times 10^3}{150} = 2346.66 \text{ mm}^2 < 11{,}200 \text{ mm}^2$$

Connection between the Bearing Stiffener and the Web

The entire reaction is assumed to be taken up by the welds.
Here also, the welds can be cast intermittently on either side of the web.
The length available (both sides included) = 2(1000 – 40) = 1920 mm

$$\text{The welds have to sustain the shear force given by} = \frac{352 \times 10^3}{1920} = 183.33 \text{ N/mm}$$

$$\text{Size of the weld} = \frac{183.33}{0.7 \times 102.5} = 2.55 \text{ mm, say 3 mm}$$

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The length of the weld = 10t = 10 × 10 = 100 mm

Intermittent welds of 100 mm length and 3 mm size are provided on both sides of the web. Spacing may be provided at 100 mm c/c.

Design of the Composite Section (Refer Fig. 11.8)

The flange width of the composite section is taken as centre to centre distance of girders. Equivalent area of concrete

$$A_e = \frac{\text{Actual area of concrete slab}}{\text{Modular ratio}}$$

$$= \frac{2500 \times 250}{13} = 48.077 \text{ mm}^2$$

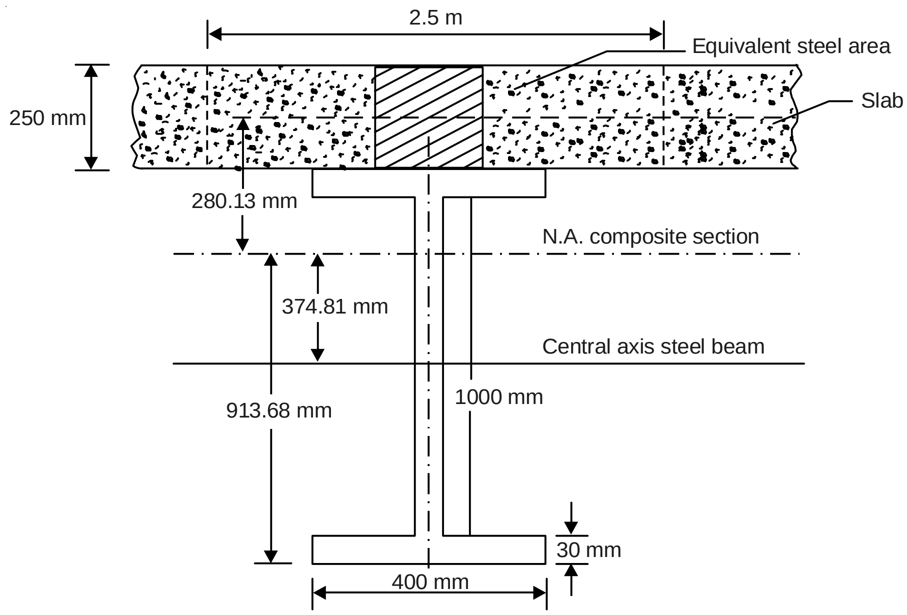

Fig. 11.8 Composite section (Example 11.1).

To find the neutral axis of the composite section, we have

$$Ay = \{[48{,}077 \times 1185] + [400 \times 30 \times 1045]\} + \{[1000 \times 10 \times 530] + [400 \times 30 \times 15]\}$$

$$= 74{,}991{,}245$$

$$A = 48{,}077 + (400 \times 30 \times 2) + (1000 \times 10) = 82{,}076 \text{ mm}^2$$

Therefore,

$$y = \frac{74{,}991{,}245}{82{,}076} = 913.68 \text{ mm}$$

Moment of inertia of the composite section w.r.t. neutral axis is

$$I_{\text{com}} = [48{,}077 \times 280.13^2] + \frac{400 \times 1060^3}{12} - \frac{390 \times 1000^3}{12} + 34{,}000 \times 374.87^2$$

$$= 1.575 \times 10^{10} \text{ mm}^4$$

Maximum shear force at the junction of the slab and the steel girder is

$$\tau = \frac{VAy}{I_{\text{com}}}$$

$$= \frac{352 \times 10^3 \times 48{,}077 \times 280.13}{1.575 \times 10^{10}} = 300.99 \text{ N/mm}$$

Total horizontal shear force on the width = 300.99 × 400 = 120,398 N

Shear Connectors

Using 20 mm shear connectors with height 4 × diameter = 4 × 20 = 80 mm, the capacity of the single connector is given by

$$Q = 4.8hd\sqrt{f_{ck}} = 4.8 \times 80 \times 20 \times \sqrt{40} = 48{,}573 \text{ N}$$

Therefore, the no. of studs required = 120,398/48,573 = 2.47 ≈ 3

Four studs are placed symmetrically w.r.t. centre line of the girder as shown in Fig. 11.9. The longitudinal spacing of the studs is given by

$$p = \frac{NQ}{\tau} = \frac{3 \times 48{,}573}{301} \approx 484 \approx 400 \text{ mm}$$

However, c/c spacing of 400 mm can be adopted.

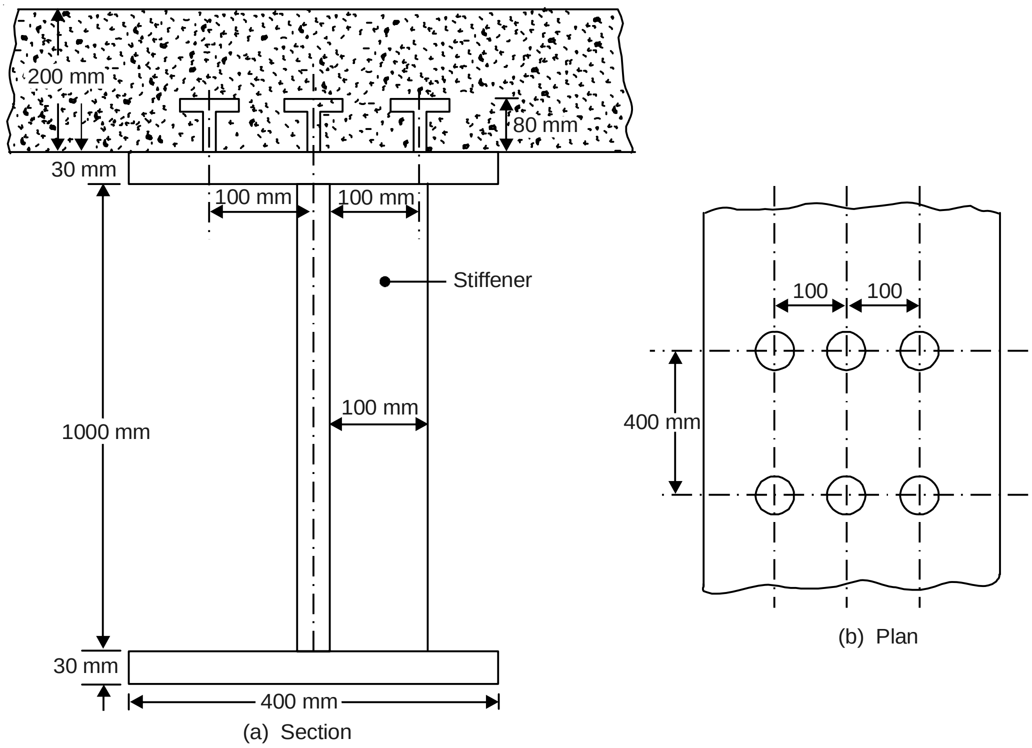

Fig. 11.9 Arrangement of shear connectors (Example 11.1).

Figures 11.10 and 11.11 show details of shear connectors and sectional views of the deck.

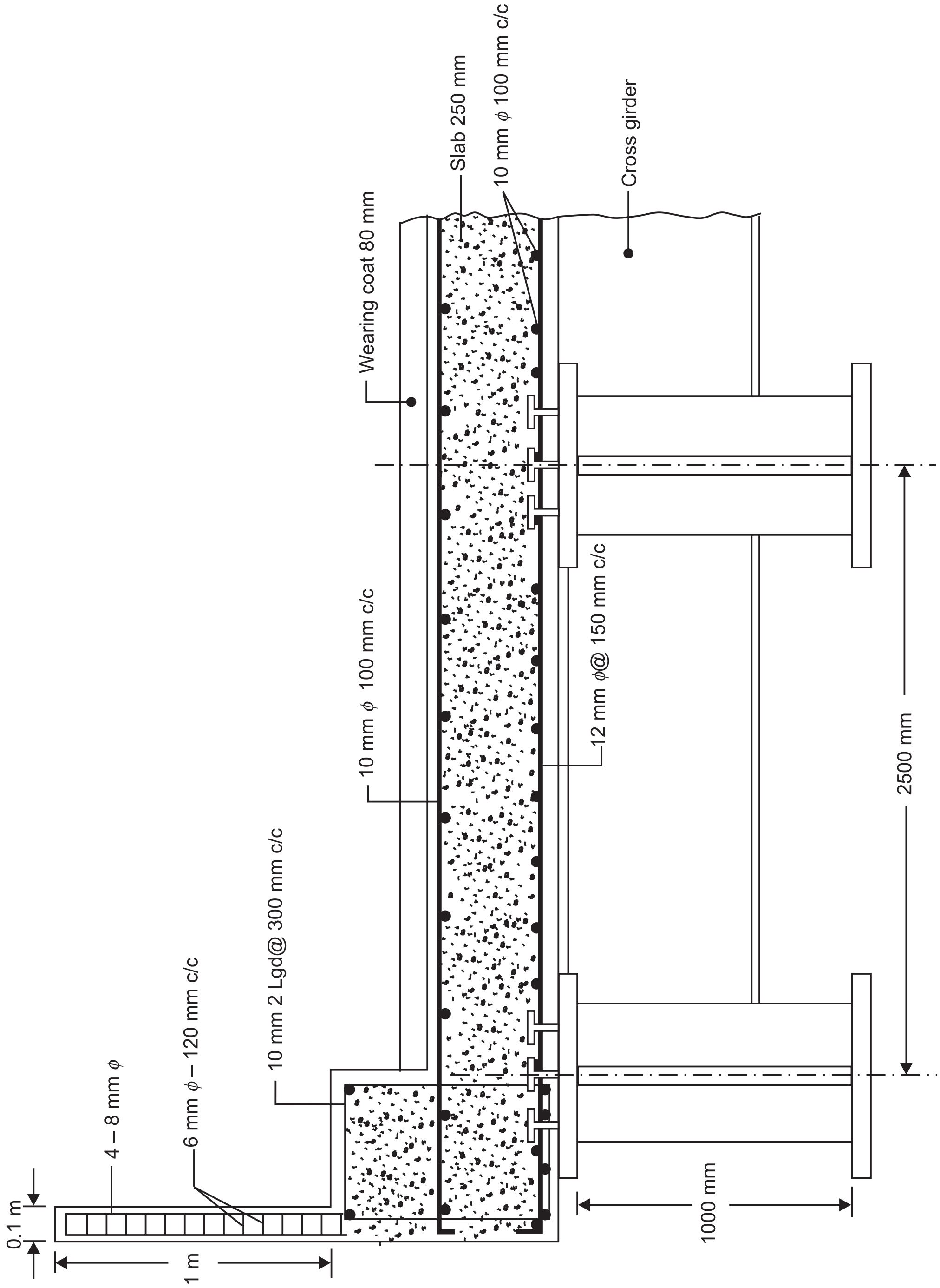

Fig. 11.10 Cross-section of the deck (Example 11.1).

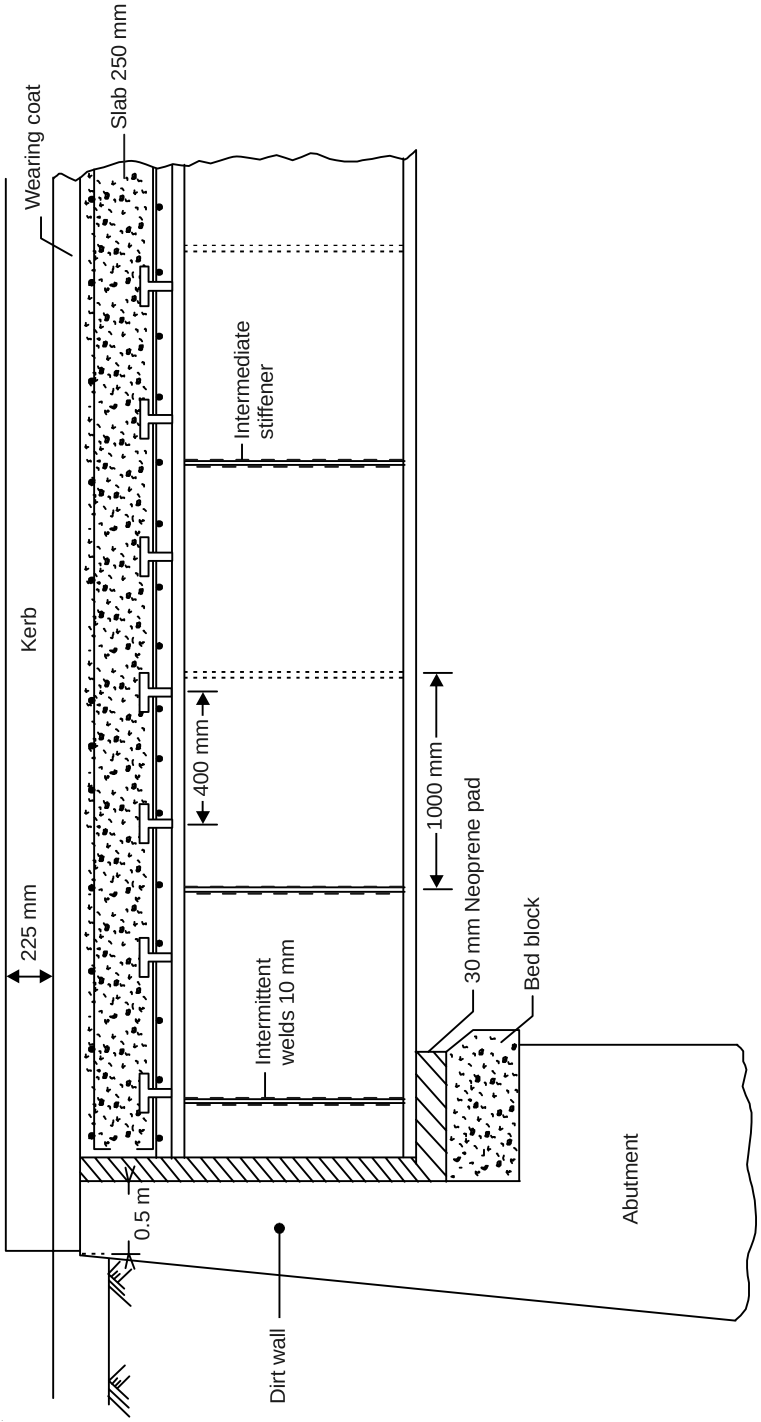

Fig. 11.11 Longitudinal section of the deck (Example 11.1).

Substructures

$$\text{Abutment top width} = \text{bearing + clearance + dirt wall}$$
$$= 0.50 + 0.10 + 0.50 = 1.1 \text{ m}$$

Abutment shall have vertical waterfront and back batter, such that its bottom width at a level of 150 m would be

$$= 0.4 \times (155.5 - 150) = 2.20 \text{ m}$$

Wing wall is of return type. The top width of the wing wall is 0.5 m. The wing wall is provided with a back batter of 1 in 6.

$$\text{Bottom width of the wing wall} = 0.5 + \frac{1}{6}(155.5 - 150) = 1.41 \text{ m, say } 1.4 \text{ m}$$

Plate 7 shows the detailed drawing of the bridge.

DESIGN PROBLEM

Design a composite bridge superstructure and substructure with the following data:

Span: 18 m
No. of lanes: Two
Live load: IRC Class AA
Material: M35 concrete and Fe 415 steel
Top level of the road embankment: 1000 m
Bed level of the stream: 992 m
HFL of the stream: 994.5 m
Top level of the stream bund: 995 m
Hard soil for foundation is available at: 900 m

Adopt the splayed type of wing walls. Draw the following views to suitable scale:

1. Half-sectional elevation and half-front elevation
2. Cross-section of the deck showing details of shear connectors.

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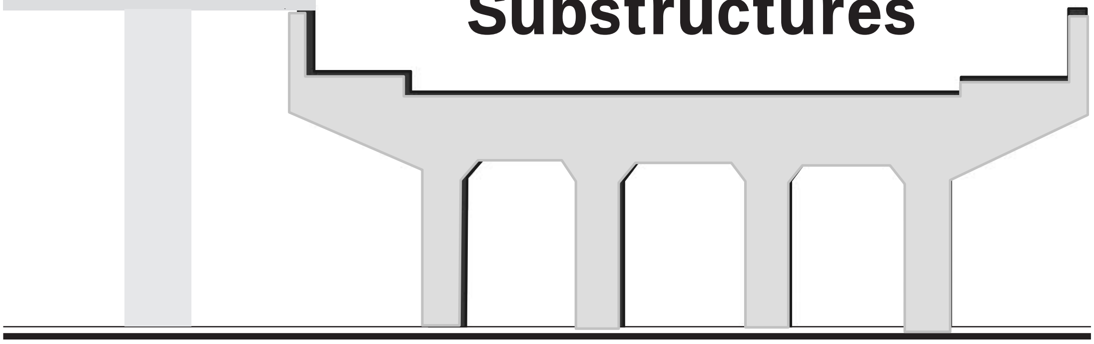
Chapter 12

Substructures



The substructures of a bridge are designed to support the superstructure of the bridge. This chapter deals with two major components of the substructure: Abutments and Piers.

12.1 ABUTMENTS

An abutment is a structure located at the end of a bridge. The basic functions of an abutment are:

- ➢ Supporting the bridge deck at the ends
- ➢ Retaining the approach road embankment
- ➢ Connecting the approach road to the bridge deck

There are different types of abutments. The selection of a particular form of abutment depends on the geometry of the site and size of the bridge. The simplest form of an abutment is a wall of considerable thickness provided with bridge seating arrangements at the top. A major difference between a conventional retaining wall and an abutment is that, an abutment is always associated with additional walls called *wing walls*. The major types of abutments currently in use are explained below.

Gravity abutment

A gravity abutment resists horizontal earth pressure from the rear, with its own dead weight. To be stable, this leads to massive-sized abutments. These abutments may be of mass concrete or stone masonry. A gravity abutment is composed of a back wall and splayed wing walls, which rest on foundation. The cross-section of this type of abutment is shown in Fig. 12.1.

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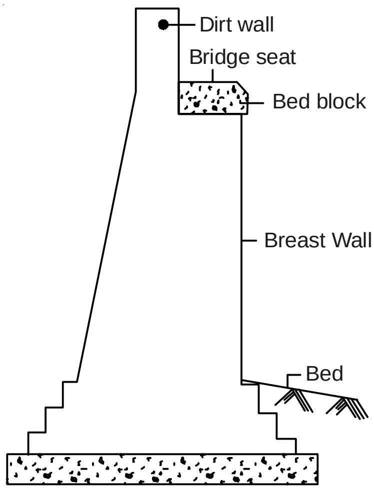

Fig. 12.1 Gravity abutment.

U-abutment

When the wing walls of a gravity abutment are placed at right angles to the back wall, the abutment is known as the U-abutment. The name 'U-abutment' is due to the shape of this abutment in plan. The wing walls are typically cast monolithically with the abutment back wall and cantilevered both vertically and horizontally. This is shown in Fig. 12.2.

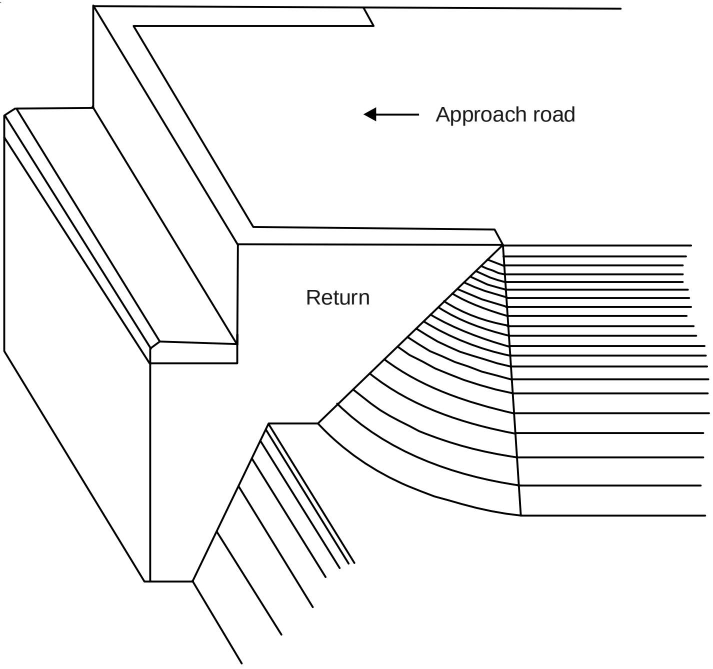

Fig. 12.2 U-abutment.

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Stub abutment

Stub abutments are relatively short abutments, which are placed on top of the embankment or slope. Sufficient rocky terrain must prevail at the site, so that the stub abutment can be supported on piles which extend through the embankment. This is portrayed in Fig. 12.3.

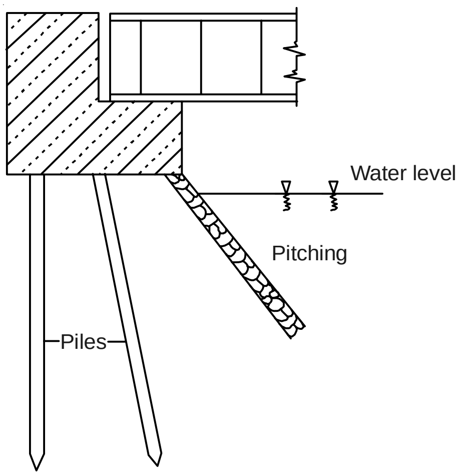

Fig. 12.3 Stub abutment.

Counterfort abutment

A counterfort abutment is very much similar to a counterfort retaining wall. In counterfort abutments, a thin wall called *counterfort* connects the breast wall to the footing. These counterforts are spaced at regular intervals so that the breast wall is designed as a supported slab rather than as a cantilever. Counterfort abutments are used when high abutments are required. This type of abutment is shown in Fig. 12.4.

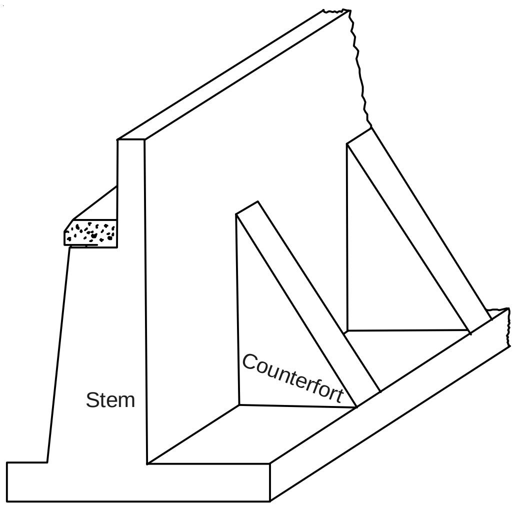

Fig. 12.4 Counterfort abutment.

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12.1.1 Stability Analysis of Abutments

The dimensions of the abutment such as top width, bottom width and front and back batters are fixed first. These dimensions depend on type of the abutment, height requirement, depth of foundation, etc. Table 12.1 shows the minimum top widths to be fixed for abutments based on their heights. The bottom width should be such as to create sufficient self-weight to enable stability. In any case, the bottom width should not be less than 0.4 times the height. The abutment so designed must be checked for stability. An abutment must be safe against:

- ➢ Overturning
- ➢ Sliding
- ➢ Eccentricity of the resultant with respect to centre of the base
- ➢ Maximum base pressure

Table 12.1 Minimum top width of piers and abutments (For slab and girder bridges)(IRC 40)

Span in meters	3 m	6 m	12 m	24 m	40 m	50 m *and above*
Top width of piers carrying simply supported spans (m)	0.50	1.00	1.2	1.6	2.0	2.2
Top width of abutments and of piers carrying continuous spans (m)	0.40	0.75	1.00	1.30	1.70	1.90

The stability against overturning is verified by calculating the factor of safety which is given by

$$F.S._{overturn} = (\Sigma \text{ Moments to resist overturning})/(\Sigma \text{ Overturning moments})$$

A generally accepted factor of safety against overturning is 1.5 or 2.0
The factor of safety against sliding is given by

$$F.S._{slide} = (\Sigma \text{ Resisting forces})/(\Sigma \text{ Sliding forces})$$

The factor of safety against sliding should not be less than 2.0.

The resultant of all the forces acting on the abutment must be such as to pass through a point located within the central portion of the base of the abutment (middle-third location). The eccentricity of the resultant corresponding to this position should be less than $b/6$, where b is the base width of the abutment. If the eccentricity is more than $b/6$, tension would develop at the bottom, which may consequentially lead to certain portion of the bottom loosing contact with the founding media. Finally, the maximum base pressure should not exceed the permissible stresses (bearing capacity) of the supporting media. The equation for maximum and minimum base pressures is given by

$$p = \frac{W}{b}\left[1 \pm \frac{6e}{b}\right] \tag{12.1}$$

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where

W = total vertical load on the abutment in kN

b = base width of the abutment in m

e = eccentricity of the resultant in m

12.2 PIERS

A pier is an intermediate supporting structure of a bridge. Piers are generally constructed using concrete although steel is also used. Piers are conventionally reinforced and help in:

- Sustaining dead load and live load
- Facilitating a long bridge to be converted into segments or bays
- Adding something to the appearance of the bridge as a whole

Presented below are some of the main types of piers constructed for bridges.

Solid piers

A solid pier can be made of concrete or stone masonry with cement mortar. It can also have facing of stone masonry with concrete hearting, as this enhances the aesthetic value of the bridge. Dimensioning of these types of piers are governed by experience and rules of thumb. The top width of the pier may be selected by using the following rules:

For spans between 5 to 10 m span/6

For spans between 10 to 20 m span/7

For spans more than 20 m .... span/8 to span/10

Table 12.1 shows the minimum top widths to be fixed for piers as stipulated by IRC 40. The piers are provided with cut and ease water for smooth passage of water. The detailed patterns of cut and ease water which can be provided for a bridge are highlighted in IRC 6. The length of the pier depends on the width of the superstructure to be carried. For stability, piers are provided with a batter of 1 in 12 to 1 in 24.

Other types

Other types of piers include: Cellular, Trestle, Hammerhead, and Framed. Each of these piers is of very high aesthetic value. Such piers are normally constructed for flyovers or bridges within urban areas. Figure 12.5 depicts these piers.

12.2.1 Loads on Piers

The following loads are considered while analysing the stability of a pier.

- Dead load from superstructure and self-weight
- Live load from traffic
- Impact effect
- Buoyancy effect

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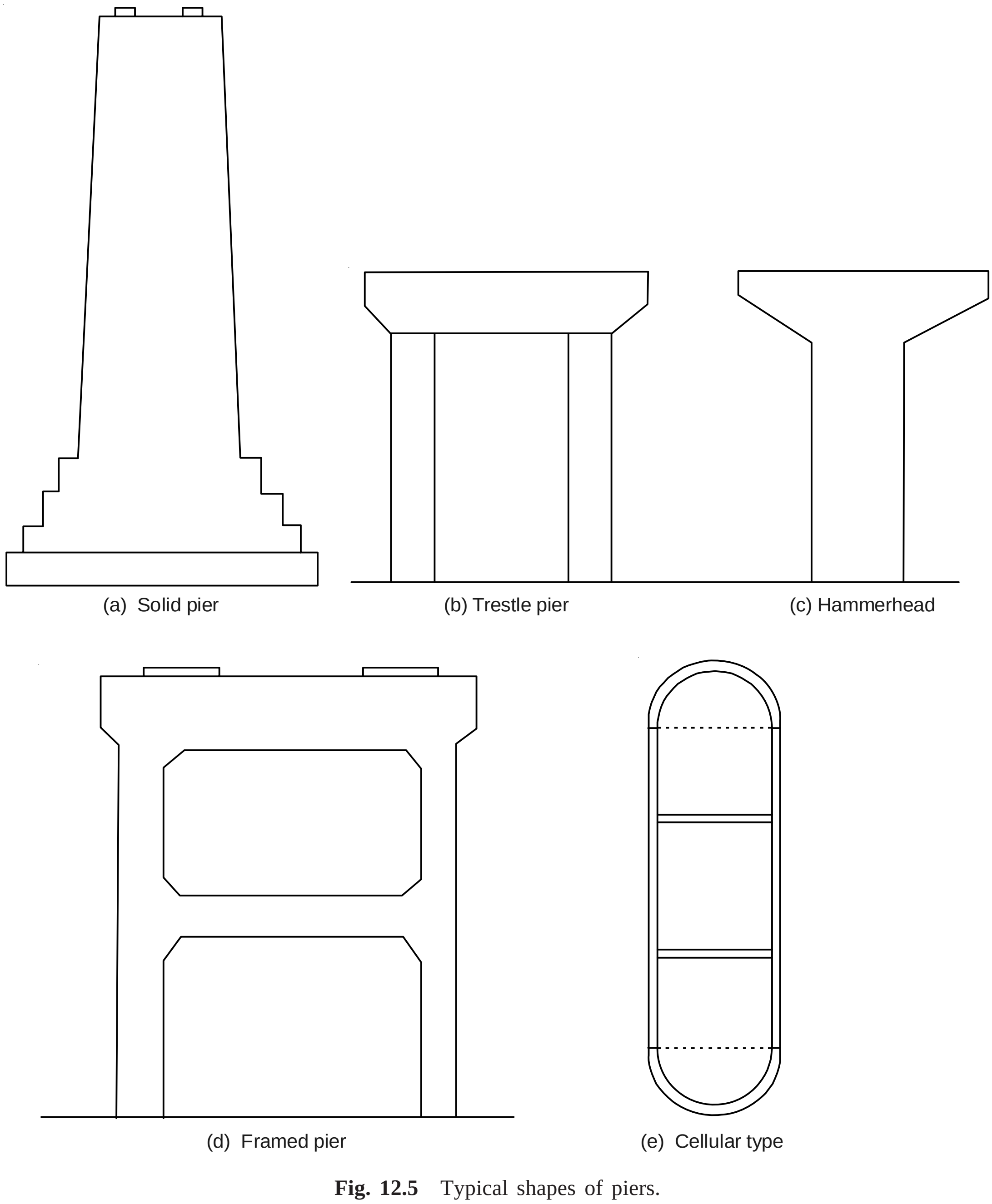

Fig. 12.5 Typical shapes of piers.

- Wind forces
- Wave forces
- Longitudinal force owing to braking of vehicles
- Seismic effects
- Forces owing to collision, for piers in navigable waterways

12.2.2 Analysis of Piers

After deciding the dimensions such as top width, side batter and base width, the pier should be analysed for adequacy of its dimensions. This can be done by checking the maximum and minimum base pressures developed by the pier. While calculating these base pressures, all the forces enumerated above are considered.

EXAMPLE 12.1

Verify the stability of the abutment shown in Fig. 12.6. The other salient details are given below:

Material of the abutment: Concrete
Density of the soil: 18 kN/m^3
Coefficient of friction: 0.6
Angle of repose of the soil: ϕ = 30°
Live load on the bridge: IRC Class AA (Tracked)

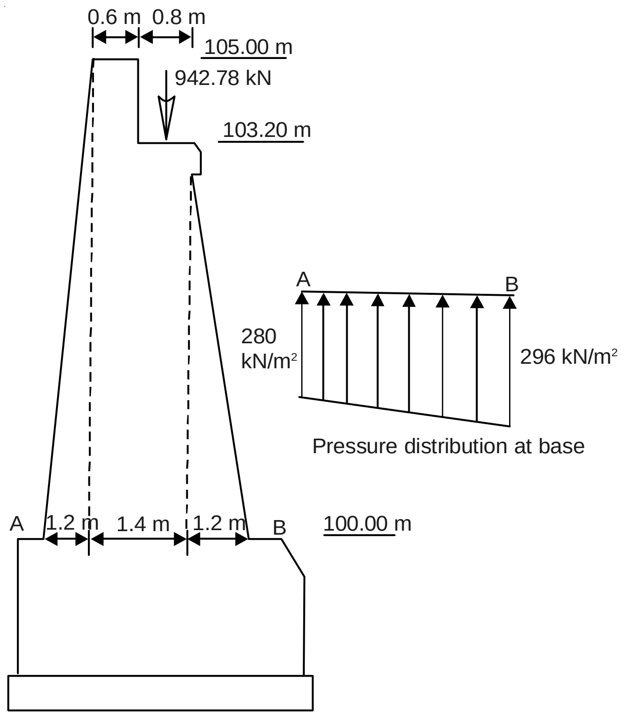

Fig. 12.6 Section of the abutment (Example 12.1).

Span of the bridge: 15 m

Angle of friction between the soil and concrete: $\delta = 18°$

The bridge deck consists of three longitudinal girders of 1.4 m depth with a deck slab of 200 mm depth.

Analysis

The stability of the abutment is verified at bed level.

Self-weight of the abutment

$= 24\,[(0.6 \times 5) + (0.8 \times 3.2) + (0.5 \times 1.2 \times 3.2) + (0.5 \times 1.2 \times 5)]$

$= 251.52$ kN

Dead load from superstructure

$= (3 \times 1.4 \times 0.3 \times 15 \times 24) + [(24 \times 0.2) + (22 \times 0.08)] \times 8.7 \times 15$

$= 453.60 + 856.08 = 1309.68$ kN

Dead load per abutment = 1309.68/2 = 654.84 kN

Dead load per metre run of abutment = 654.84/8.7 = 75.26 kN {where 8.7 m is the width of the deck}

Reaction due to live load

Live load reaction is maximum when the wheel is nearer to the support such that the tip of the wheel touches the support, letting the full portion of the wheel within the span.

Live load reaction = 700[15 – (3.6/2)]/15 = 616 kN

Total load

Total load = 251.52 + 75.26 + 616 = 942.78 kN

Earth pressure

Earth pressure is calculated using Coulomb's formula

$\phi = 30°$, $\tan\theta = 1.2/5$, therefore, $\theta = 13.50°$

Total earth pressure = $0.5 \times 18 \times 5^2 \times \cos 13.50°\ k_a$

where k_a is given by [active earth pressure coefficient]

$$k_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta\cos(\partial + \theta) + \sqrt{\dfrac{\sin(\partial + \phi)\sin\phi}{\cos(\partial + \phi)\cos\phi}}}$$

Upon substitution, $k_a = 0.853$

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Therefore,

$$\text{Earth pressure} = 0.5 \times 18 \times 5^2 \times \cos 13.50° \times 0.853 = 181.46 \text{ kN}$$

Horizontal component of earth pressure = 181.46 cos ($\delta + \theta$) = 181.46 cos (18 + 13.50)
= 154.72 kN

Vertical component of earth pressure = 181.46 sin 31.50 = 94.81 kN

$$\Sigma V = 94.81 + 942.78 = 1037.59 \text{ kN}$$

$$\Sigma H = 154.72 \text{ kN}$$

$$\text{Resultant} = \sqrt{(1037.59)^2 + (154.72)^2} = 1049.06 \text{ kN}$$

Check against overturning

The earth pressure is assumed to act at a height of $0.42h = 0.42 \times 5 = 2.1$ m

Moments having overturning effect = 2.1 × 154.72 = 324.91 kN.m

Restoring moments

= (0.6 × 5 × 2.3 × 24) + (0.5 × 1.2 × 3.2 × 24 × 0.8)
+ (0.8 × 3.2 × 24 × 1.6) + (0.5 × 1.2 × 5 × 24 × 3) + (942.78 × 1.6)
= 2025.20 kN·m

Factor of safety against overturning = 2025.20/324.91 = 6.23 > 2. Therefore, the abutment is safe against overturning.

Check against sliding

Factor of safety = (0.6 × 942.78)/154.72 = 3.65 > 2. Therefore, the abutment is safe against sliding.

Maximum and minimum base pressures

$$\text{Distance of the resultant from the toe} = \frac{2025.20 - 324.91}{1049.06} = 1.62 \text{ m}$$

Eccentricity of the resultant from the centre of the base

$$e = \frac{3.27}{2} - 1.62 = 0.015 < \frac{b}{6}$$

Therefore,

$$\text{Maximum pressure } p_{max} = \frac{942.78}{3.27}\left(1 + \frac{6 \times 0.015}{3.27}\right) = 296.24 \text{ kN/m}^2$$

$$\text{Minimum pressure } p_{min} = \frac{942.78}{3.27}\left(1 - \frac{6 \times 0.015}{3.27}\right) = 280.37 \text{ kN/m}^2$$

Stresses are within limits as the compressive stress for concrete is 2000 kN/m^2.

The abutment cross-section and the pressure distribution at the base are shown in Fig. 12.6.

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EXAMPLE 12.2

Verify the adequacy of the dimensions for the pier shown in Fig. 12.7. The following details are available:

Top width of the pier: 1.6 m

Height of the pier up to springing level: 10 m

c/c of bearings on either side: 1.00 m

Side batter: 1 in 12

High flood level: 1 m below the bearing level

Span of the bridge: 16 m

Loading on span: IRC Class AA

Road: Two-lane road with 1 m wide footpath on either side.

Superstructure: Consists of three longitudinal girders of 1.4 m depth with a deck slab of 200 mm depth. Rib width of girders = 300 mm

Material of the pier: Concrete M15

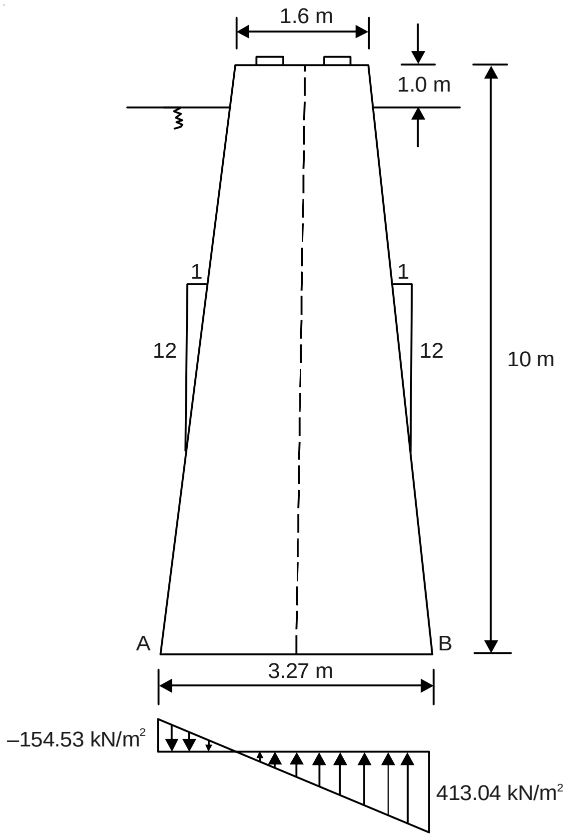

Fig. 12.7 Section of the pier and pressures at the base (Example 12.2).

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Analysis

Base width at bed level = 1.6 + (1/12) × (10 × 2) = 3.27 m

Pier length required = 7.5 + (2 × 1) = 9.50 m

Self-weight of the pier

Area at top = (9.5 × 1.6) + (2 × π × 0.8^2/2) = 17.21 m^2

Area at bottom = (3.27 × 9.5) + (2 × π × 1.6^2/2) = 39.11 m^2

Self-weight = (1/2) × (17.21 + 39.11) × 10 × 24 = 6578.40 kN

Moment of inertia with respect to *X-X* axis

$$= \frac{9.5 \times 3.27^3}{12} + \frac{2\pi \times 3.27^4}{128} = 33.29 \text{ m}^4$$

Dead load from the superstructure

This is due to longitudinal girders and deck slab of the superstructure.
Roughly, it is given by

= (3 × 1.4 × 16 × 0.3) 24 + (24 × 0.2 + 0.08 × 22) × (9.5 × 16)

= 1480.96 kN

Therefore,
Load per metre length of pier = 1480.96/9.5 = 155.89 kN

Design dead load = 6758.40 + 155.89 = 6914.29 kN

Stresses at bottom owing to dead load = 6914.29/35.11 = 176.79 kN/m^2

Stresses owing to buoyancy

Owing to buoyancy, the pier gets lifted, i.e. there is a relief in stress value. Therefore, stresses due to buoyancy are always negative.

Width of the pier at HFL = 1.6 + (2 × 0.1) = 1.8 m

Area of the pier at HFL = (1.8 × 9.5) + (π × 0.9^2/2) = 18.37 m^2

Submerged volume of the pier = [(18.37 + 39.11)/2] × 9 = 258.66 m^3

Reduction in weight of the pier owing to buoyancy = Weight of the displaced water
= 258.66 × 10 = 2586.6 kN

Stress at base = 2586.6/39.11 = – 66.13 N/mm^2

Stress owing to live load

Reaction owing to live load (Class AA) including impact = 1.1 × 700 = 770 kN

Maximum bending moment at base = 770 × 0.5 = 385 kN/m

Maximum stress at base = (770/39.11) + (385 × 3.27)/(33.29 × 2) = 38.59 kN/m^2

Minimum stress at base = (770/39.11) – (385 × 3.27)/(33.29 × 2) = 0.779 ≈ 0.8 kN/m^2

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Stresses owing to longitudinal force

Longitudinal force may be taken as 20% of IRC Class AA loading that is 0.2 × 700 = 140 kN

Moment owing to this force at base = 140 × 10 = 1400 kN·m

Stresses at base = ± (1400 × 3.27)/(33.29 × 2) = ± 68.75 kN/m^2

Stresses owing to water current

Velocity of water may be taken as 3 m/s

Water pressure = 5.2 kv^2 (k is a constant = 0.66 for semicircular cut and ease water).

= 5.2 × 0.66 × 3^2 = 30.88 kN/m^2

Area of the wetted surface of pier = 9(1.8 + 3.27)/2 = 22.81 m^2

Force owing to water current = 30.88 × 22.81 = 704.37 kN

For the worst effect, the current direction is taken as 20°

Force perpendicular to pier = 704.37 cos 20°

= 661.89 kN

Moment at the base owing to this force = 661.89 × (2/3) × 9 = 3971.34 kN·m

Stresses at the base owing to this force = ± [3971.34/33.29] × (3.27/2) = ±195.04 kN/m^2

Summation of all the stresses

Maximum stress = 176.79 + 38.59 – 66.13 + 68.75 + 195.04 = 413.04 kN/m^2

Minimum stress = 176.19 – 0.8 – 66.13 – 68.75 – 195.04 = –153.93 kN/m^2

The stresses developed at the base are within limits.

Negative pressure indicates development of tension at the bottom. This is undesirable. To abate this, the bottom and top widths of the pier may be slightly altered.

The pier cross-section and pressure distribution at the base are shown in Fig. 12.7

DESIGN PROBLEM

Verify the stability of the abutment of a bridge with the following details:

Top width: 1.5 m
Height: 4 m
Back batter: 1 in 6
Front face of the abutment is vertical
Material: Stone masonry
Unit weight of soil: 18 kN/m^3
Angle of repose: 30°
Superstructure: T-beam bridge of span 15 m
Loading: IRC Class AA

Assume suitable dimensions for the components of the superstructure.

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Chapter 13

Bridge Foundations

As the loads of the superstructure and substructures of the bridge get finally transferred to ground, the substructures need to be founded properly. The foundations of the bridge should have no differential settlements, because differential settlements generate moments in the superstructure which may be of very high magnitude. The depth of foundations should be placed below the depth of maximum scour, so that the foundations are not undermined.

13.1 TYPES OF FOUNDATIONS

There are two categories, i.e. shallow foundations and deep foundations. Whether deep or shallow, the foundation to a bridge is location-specific. If stable foundation stratums are available at lower depths, a shallow foundation is the right option (such as raft foundation, stepped foundation, etc.). If a good stratum is available at greater depths only, it is the deep foundation that would be required. Deep foundations are dealt with in this chapter. Deep foundations used for bridges are of two types:

1. Well foundations
2. Pile foundations

13.1.1 Well Foundations

Well foundations are predominantly used as bridge foundations. They are identified under three categories.

Box caission

This is a vessel, which is open at its top and closed at its bottom. The box could be of timber, steel or concrete. The box is constructed on shore and towed to the location where it is required. It is then sunk at the site from where it starts to function as a foundation. It is suitable for shallow depths.

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Open caisson

This is called well foundation in Indian context. It is essentially a cylindrical (sometimes square or rectangular) structure made of RCC or bricks. It has its top and bottom ends open. The central space is filled with sand. The top is then closed by a concrete cap. On this cap, either the abutment or the pier is constructed.

Pneumatic caisson

When it is required to go to very great depths for want of suitable founding strata, pneumatic caissons are used. Pneumatic caissons are wells with open bottoms and closed tops. In between the closed top and the open bottom, pressure is regulated so that labourers can work. These types are found to be exorbitantly expensive.

13.1.2 Open Well Foundations

In India, well foundations are very popular. Wells can be of various shapes such as circular, rectangular, double-D, dumb-bell. These are shown in plan in Fig. 13.1. The selection of shape is left to the engineer-in-charge. However, it is governed by the following factors:

- ➢ Size and shape of the base of the pier or the abutment
- ➢ Difficulty in sinking (possibility of tilt)
- ➢ Stability requirement

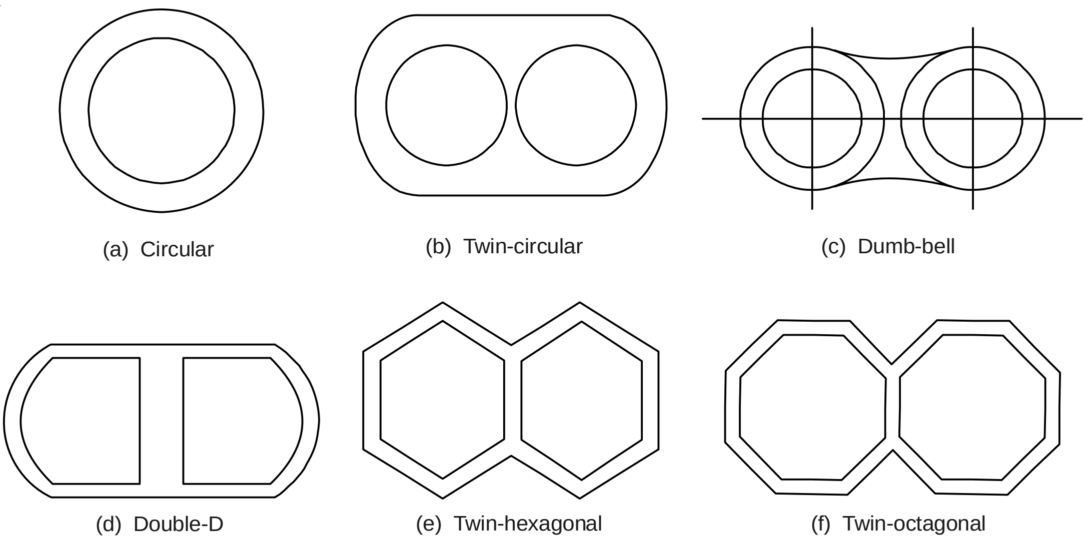

Fig. 13.1 Types of wells.

It has been found that a circular well is always advantageous because of the relatively low sinking effort involved.

13.1.3 Components of Well Foundations

The various components of a well foundation are shown in Fig. 13.2. The functions of the components illustrated in Fig. 13.2 are enumerated below:

Steining. It is generally made of concrete. It provides dead load during sinking. When placed on hard strata, the steining will take care of load transfer. It provides a firm grip below the scour level.

Well cap. It is a concrete slab covering the top of the well. Over this cap, an abutment or a pier is constructed.

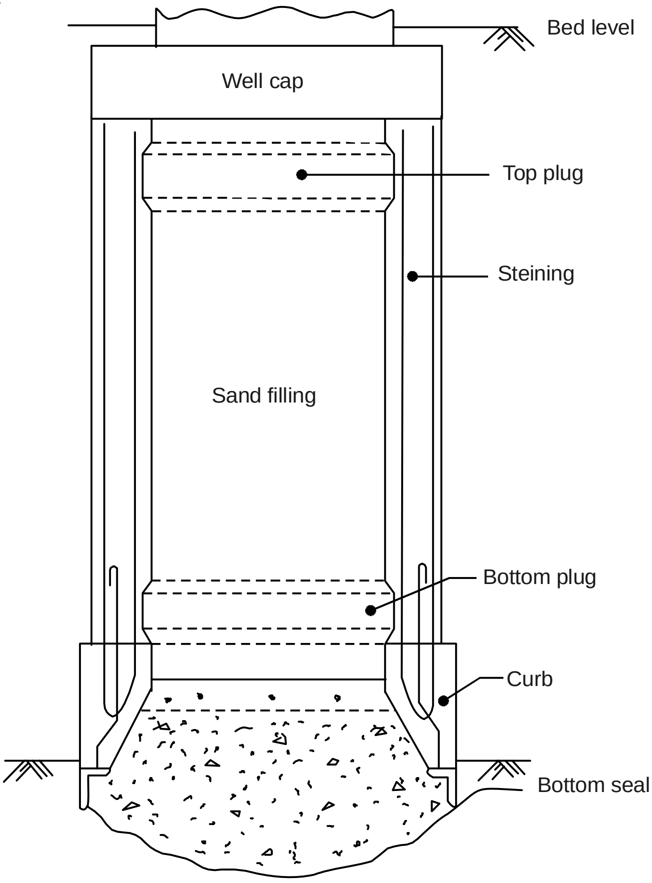

Fig. 13.2 Components of a well foundation.

Top plug. It is the covering provided over the sand filling. It also serves as a shuttering for laying the well cap.

Bottom plug. The bottom plug is a via media for transferring the load from steining to the soil.

Sand fill. It is used to fill the void created between the walls of the well. It increases the weight of the well, and thus adds to stability of the well.

Curb. This is the bottom most part of a well. It is shaped in the form of a wedge to facilitate easy sinking. It is provided with a mild steel angle all round.

13.2 DESIGN OF WELLS

Steining. The minimum thickness of steining shall not be less than 500 mm and satisfy the following relationship:

$$h = kd\sqrt{L} \tag{13.1}$$

where

h = minimum thickness of steining in m
d = external diameter of the circular well in m
L = depth of the well in m
k = constant
= 0.030 for sandy strata, 0.033 for clayey strata

The values of k for wells of different shapes are listed in IRC 78.

Reinforcement in well steining. Vertical reinforcement in the steining shall not be less than 0.12% of the gross-sectional area. This shall be equally distributed on both faces of steining. The vertical reinforcements are tied up with hoop steel not less than 0.04% of the volume per unit length of steining.

Cutting edge. The mild steel cutting edge shall be strong enough and not less than 40 kg/m, to facilitate sinking of the well through the types of strata expected.

Well curb. It is made of M20 concrete having minimum reinforcement of 72 kg/cu.m.

Bottom plug. Bottom plug shall be located at a depth of 300 mm from the top of the kerb. The thickness of the bottom plug is given by

$$t^2 = 1.18r^2 \frac{q}{f_c} \tag{13.2}$$

where

q = bearing pressure at the base in kN/m^2
r = radius of the well in m
f_c = flexural strength of concrete in kN/m^2.

Check for stresses

The stresses developed in the steining of the well can be verified at the location of maximum bending moment. The point where the bending moment is maximum is located below the scour level. It is given by

$$x = \sqrt{\frac{2FH}{\gamma_b(k_p - k_a)B}} \tag{13.3}$$

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where

F = factor of safety = 2.0
H = horizontal force at scour level in kN
γ_b = submerged unit weight of soil in kN/m^3
k_p and k_a = coefficients of passive and active earth pressure, respectively
B = outer diameter of the well in m

Maximum bending moment is

$$M_{max} = M_0 + (2/3)Hx \tag{13.4}$$

where M_0 = moment at the scour level in kN·m

Stress in the steining is then given by

$$\sigma_{max} = (V/A) + (M_{max}/Z) \tag{13.5}$$

This stress should be within the permissible limits.

13.3 PILE FOUNDATIONS

Piles are slender structural elements placed in the ground for:

- ➢ Receiving load from the superstructure and transferring it to ground by friction or bearing
- ➢ Increasing the bearing capacity of the soil

There may be wooden, steel or concrete piles depending on location and need. Nowadays, the concrete piles are used extensively. Concrete piles can be precast, cast in-situ or prestressed. Precast concrete piles are made in the workshop and transported to the site. Piles are cast in-situ, directly inside a hole covered with a casing. After concreting, the casing is removed.

13.3.1 Group of Piles

It is not possible to provide just one pile for the foundation of an abutment or a pier because the loads to be transferred become too large for a single pile. In bridge foundations, it is always a group of piles that is designed. The group is connected by a common cap. However, the load carrying capacity of the group is not the sum of carrying capacities of individual piles, instead it is slightly less. The spacing between the piles in a group is given by

$$S = \frac{mn\pi D}{2(m+n-2)} \tag{13.6}$$

where

m = no. of piles along columns
n = no. of piles along rows
D = diameter of the pile in m

The above relationship gives the minimum spacing for 100% efficiency.

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13.3.2 Design of Piles

Section. Precast concrete piles can be round, square or octagonal in section. The minimum size of the precast pile is 250 mm square, the maximum size is less than 600 mm. Based on the length of the piles, the following cross-sectional dimensions can be adopted:

Up to 10 m length: 250 mm (square)

10 m to 12 m: 300 mm (square)

12 to 15 m: 350 mm (square)

15 to 18 m: 400 mm (square)

More than 18 m: 450 mm (square)

Longitudinal reinforcement. The area of longitudinal reinforcement should not be less than the following stipulations (IRC 78):

(a) For piles with a length less than 30 times the least width = 1.25%

(b) For piles with a length 30 to 40 times the least width = 1.5%

(c) For piles with a length greater than 40 times the least width = 2%

Lateral reinforcement. These reinforcements are of great importance as they supplement the resisting driving stresses. These should be in the form of hoops, spirals or links. The minimum diameter should not be less than 6 mm. For a distance of about 3 times the least width or diameter from each end of the pile, the volume of lateral reinforcement shall not he less than 0.6% of the gross volume. In the body of the pile, the lateral reinforcement shall not be less than 0.2% of the gross volume.

EXAMPLE 13.1

A well foundation is to be designed for an abutment of 10 m × 5 m base dimensions. The well is founded on a sandy soil. The data available are as follows:

Height of bearing above the maximum scour level: 28 m

Permissible horizontal displacement of the bearing level: 50 mm

Height of the abutment: 6.0 m

Total vertical load including weight of the abutment and well (considering buoyancy effect): 20,000 kN

Total lateral load at the scour level = 400 kN

Submerged unit weight of soil: 9.5 kN/m^3

Design the well and verify the stresses in the steining.

Design

Considering the dimensions of the abutment base, the diameter of the well can be 10 m. Referring to Fig. 13.3, we have

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$$\text{Grip length} = \frac{\text{Height of the well above the scour level}}{3}$$

$$= \frac{22}{3} = 7.3 \approx 8.0 \text{ m}$$

Total height or depth of the well = 30 m

Thickness of steining

The minimum thickness is 500 mm. If the external diameter is 11 m (with minimum thickness of steining), the codal stipulation for thickness of the steining is

$$h = kd\sqrt{L} = 0.030 \times 11\sqrt{30} = 1.80 \text{ m}$$

A thickness of 1.80 m for the steining can be adopted.

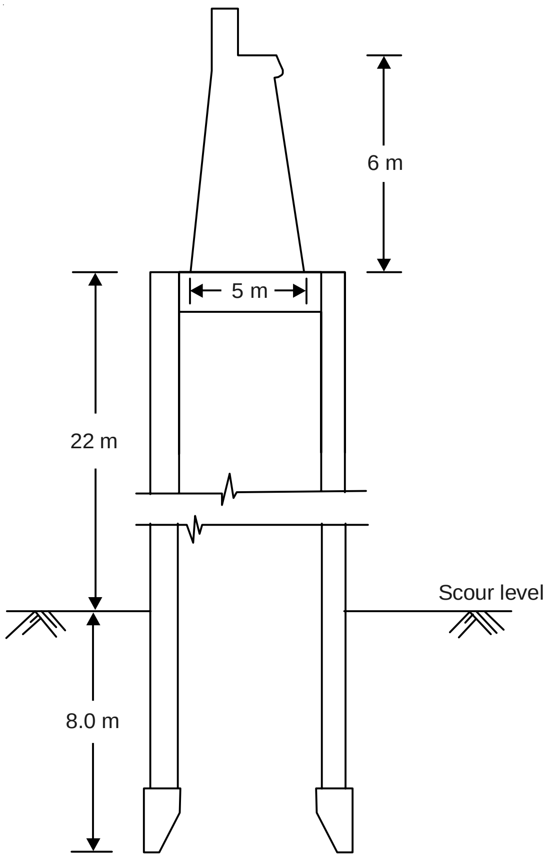

Fig. 13.3 Well foundation (Example 13.1).

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Reinforcement

Vertical reinforcement in the steining should not be less than 0.12% of the gross-sectional area. Providing 0.15%, we get

$$\text{Steel area} = \frac{0.15}{100} \times \frac{\pi}{4} \times (13.6^2 - 10^2)$$
$$= 0.10 \text{ m}^2 = 100{,}000 \text{ mm}^2$$

Since this is equally distributed on both faces of the steining, area of steel on one face is 50,000 mm^2.
No. of bars required (by providing 20 mm diameter rods) = 50,000/314.2 = 159

$$\text{Spacing with a clear cover of 50 mm} = \frac{\pi \times 13.5 \times 1000}{159} = 266.73 \text{ mm}$$

where 13.5 is the diameter of the hoop steel obtained by providing a cover of 50 mm on both sides (13.6 – 2 × 0.05 = 13.5)

A spacing of 250 mm could be adopted.

Hoop reinforcement should not be less than 0.04% of the volume/unit length of the steining. Providing 0.05%, we get

Volume of the steining/unit length = 66.72 × 1 = 66.72 m^3

Volume of steel = (0.05 × 66.72)/100 = 0.033 m^3/m

Using 12 mm diameter hoops, length of hoop = π × 13.5 = 42.41 m

Steel to be provided per unit height = 0.033 × 7850 = 259.05 kg

Weight of each hoop = (42.41 × π × 0.012^2 × 7850)/4 = 37.65 kg

No. of hoops = 259.05/37.65 = 6.88 = 9 hoops

Adopt 3 hoops each face at 300 mm c/c.

Bottom plug

The thickness of the bottom plug is

$$t^2 = 1.18\, r^2 \frac{q}{f_c}$$
$$= \frac{1.18 \times 5^2 \times 20{,}000}{66.72 \times 7000}$$
$$= 1.263 \text{ m, say } 1.5 \text{ m}$$

Check for section adopted

It can be shown that zero shear force (Max. B.M.) will be at a distance x from the scour level, that is

$$x = \sqrt{\frac{2FH}{\gamma_b (k_p - k_a)\beta}}$$

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Assuming $\phi = 30°$, $\delta = \phi/2 = 30/2 = 15°$, we get

$$k_a = \left[\frac{\cos\phi}{\sqrt{(\cos\delta + \sqrt{\sin(\theta+\delta)\sin\phi}}}\right]^2$$

$$= \left[\frac{\cos 30}{\sqrt{(\cos 15)} + \sqrt{\sin 45 \sin 30}}\right]^2$$

and

$$k_p = \left[\frac{\cos 30}{\sqrt{(\cos 15)} - \sqrt{\sin 45 \sin 30}}\right]^2$$

By calculations, $k_a = 0.30$ and $k_p = 5$. Therefore

$$x^2 = \frac{2 \times 2 \times 400}{9.5\,(5 - 0.3)13.6}$$

or

$$x^2 = 2.63$$

or

$$x = 1.62 \approx 1.60 \text{ m}$$

Maximum B.M. is $M_{max} = M_0 + (2/3)Hx$

M_0 is the bending moment at the scour level. Bending moment will exist at this level because of horizontal displacement.

$$\text{Tilt at the scour level} = (50/36) \times 8 = 11.11 = 12 \text{ mm}$$

$$\text{Moment owing to tilt} = 20{,}000 \times 0.012 = 240 \text{ kN·m}$$

$$M_{max} = 240 + (2/3) \times 400 \times 1.6 = 666.66 \text{ kN·m}$$

$$A = \pi/4\,[13.6^2 - 10^2] = 66.72 \text{ m}^2$$

$$\text{Moment of inertia} = \pi/64\,[13.6^4 - 10^4] = 1188.41 \text{ m}^4$$

Maximum stress in the steining is

$$\sigma_{max} = \frac{20{,}000}{66.72} + \left(\frac{666.66}{1188.41} \times \frac{13.6}{2}\right)$$

$$= 303.57 \text{ kN/m}^2$$

This stress is safe as it is within limits.

The details of the longitudinal-section and the cross-section are given in Plate 8.

EXAMPLE 13.2

The foundation for substructures of a bridge consists of 16 piles to carry a total load of 8000 kN. The piles are spaced at 1.5 m c/c. They are driven through soft ground to a hard stratum available at a depth of 12 m. Design the pile foundation using M20 concrete and Fe 415 steel.

Design

Permissible stresses are

$$\sigma_{cc} = 5 \text{ N/mm}^2$$

$$\sigma_{st} = 230 \text{ N/mm}^2$$

$$\sigma_{bc} = 7 \text{ N/mm}^2$$

Allowing for an extra length of 0.5 m, the length of the pile is 12.50 m.
Square piles of 300 mm side length can be selected. Here,

L/B = 12.5/0.3 = 41.66 > 12, therefore, the pile is designed as a long column.

Reduction coefficient for reduction of stress values is given by

$$c_r = (1.25 - L/48B) = (1.25 - 41.66/48) = 0.38 \approx 0.40$$

Safe stresses are

$$\sigma_{cc} = 0.4 \times 5 = 2 \text{ N/mm}^2$$

$$\sigma_{st} = 0.4 \times 230 = 92 \text{ N/mm}^2$$

Load carrying capacity of the pile is

$$P = (\sigma_{cc} A_{cc} + \sigma_{st} A_{st})$$

Load on each pile is = 8000/16 = 500 kN

Therefore,

$$500 \times 10^3 = 2(300 \times 300) + 92(A_{st})$$

or

$$A_{st} = 3478.26 \text{ mm}^2$$

Since the length of the pile is more than 40 times the lateral dimension, the minimum area of steel is 2% of the gross area, i.e. [(2/100) × 300 × 300] = 1800 mm^2

Therefore, the area of steel is 3478.26 mm^2.

8 bars of 25 mm diameter are provided (area of steel = 3926.98 mm^2) with a cover of 40 mm all round.

Lateral reinforcement

The lateral reinforcement should be 0.2% of the gross volume. Using 6 mm diameter ties, we have

$$\text{Volume of one tie} = [\pi \times 6^2/4]\,[4(300 - 80)] = 24{,}881.41 \text{ mm}^3$$

If p is the pitch of the ties in mm, then

$$\text{Volume of pile/Pitch length} = (300 \times 300)p = 90{,}000p$$

or

$$\frac{0.2}{100} \times 90{,}000p = 24{,}881.41$$

or

$$p = 138 \text{ mm, say } 130 \text{ mm}$$

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Lateral reinforcement near the pile head

This reinforcement is provided in order to take care of the driving stresses.
This reinforcement is provided at 0.6% of the gross volume for a length of $3 \times b = 900$ mm

Using 6 mm helical ties, volume of spiral/mm length of pile = $\dfrac{0.6}{100} \times 300 \times 300 \times 1 = 540 \text{ mm}^3$

If p is the pitch of the spirals, then

$$p = \frac{\text{circumference of the spiral} \times \text{area of cross-section}}{\text{volume}}$$

$$= \frac{\pi \times 180 \times 28.3}{540}$$

$$= 29.63 \text{ mm, say } 30 \text{ mm}$$

These spirals are provided in addition to normal ties.

Lateral reinforcenrent near the pile end

Volume of tie is 0.6% of the gross volume. Using 8 mm dia ties, we have

Volume of each tie = $\dfrac{\pi \times 8^2}{4}[4(300 - 80)] = 44{,}233.62 \text{ mm}^3$

Therefore,

$$\frac{0.6}{100} \times 90{,}000\, p = 44{,}233.62$$

or

$$p = 81.91 \text{ mm, say } 80 \text{ mm c/c}$$

In addition, 2 pairs of 12 mm diameter spacer forks with 6 mm dia links may be provided at 1000 mm c/c.

The longitudinal section and the cross-section of the pile are shown in Plate 8.

DESIGN PROBLEMS

1. Design a well foundation for a bridge using the following particulars:

   Diameter of the well: 3 m

   Depth of the well: 15 m

   Type of soil: Stiff clay

   Materials: M25 concrete, Fe 415 steel.

2. An abutment of a major bridge is to be founded on 15 piles driven through soft clay for a depth of 15 m. The total load expected is around 12,000 kN. Design the pile. Use M25 concrete and Fe 415 steel.

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Chapter 14

Bearings and Expansion Joints

14.1 BEARINGS

Bearings are mechanical arrangements provided in the superstructure to transmit the load to the substructure. They can be thought of as the interface or via media between the superstructure and the substructure. The main functions of bearings are:

- ➢ To transmit vertical loads to the substructure, i.e. to the pier or abutment
- ➢ To facilitate movement caused by thermal changes (expansion and contraction)
- ➢ To provide rotational movement of the girders

14.1.1 Forces on Bearings

Bearings are intended to transmit the forces and sustain the translational and rotational movements of the bridge structure. Four major forces are considered.

- ➢ Reactive forces
- ➢ Longitudinal forces
- ➢ Uplift forces
- ➢ Transverse forces

The reactions generated at the end of structural members create reactive forces on the bearings. The longitudinal and transverse forces may be caused due to earthquakes, thermal expansion and contraction, braking effect, etc.

14.1.2 Types of Bearings

Depending on functional behaviour, there can be two types of bearings, i.e. fixed bearings and expansion bearings. Fixed bearings allow only rotation while expansion bearings allow both rotation and translation. The causes for movement in bearings may be due to creep in concrete,

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shrinkage, settlement, uplift forces and thermal forces. The sudden application of brakes may also initiate some movement at the bearing level. Generally, for each span, one end of the bridge is provided with fixed bearings and the other end with the expansion type. Thus the free end allows for movement and the fixed end holds the bridge. If both ends are provided with fixed bearings, internal stresses are sure to develop in bridge components. Some of the commonly used bridge bearings are explained in the succeeding paragraphs.

Rocker bearings

Rocker bearings are suitable only for straight steel bridges. The type of bearing consists of a pin to accommodate large live load deflections as well as large vertical loads. The rocker bearing is used for span lengths of 15 m and more. It is made of steel and connected to the substructure through a steel plate (bedplate). It is bolted or welded to the primary bridge structure component. To prevent the rocker in a bearing from making translational movements, *pintles* or *shoes* are used which resist transverse forces. A shoe is a trapezoidal extrusion which extends upwards from the base plate. A groove (recess) is made to hold the rocker pin which in turn is covered by the top shoe. Figure 14.1 shows parts of a rocker bearing. The rocker pin is designed for the maximum longitudinal force. The IRC 83 stipulates the following specifications for rocker pins.

- ➢ The diameter *d* of the rocker pin shall not be less than 16 mm.
- ➢ The pin shall be fitted to a depth of 0.5*d* in the groove.
- ➢ The minimum clearance above the top surface of the rocker pin shall be 2.5 mm.

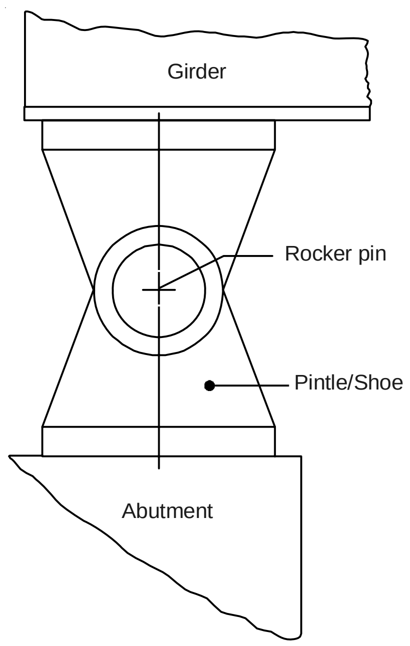

Fig. 14.1 Rocker bearing.

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Rocker and roller bearings

In this type of bearing, translational movement is facilitated by a group or nest of rollers while rotational movement is achieved by the rocker pin. Roller bearings are intended for spans of moderate length. A general drawback of this type is its susceptibility to collect dust. This may lead to freezing of the bearing in which case the rollers cease to roll. A rocker and roller bearing is shown in Fig. 14.2. The codal stipulations for such bearings are:

- ➢ The minimum diameter of the roller shall be 75 mm.
- ➢ The ratio of the length of the roller to its diameter shall normally be not more than 6 (but not more than 10 in any case).
- ➢ The gap between the rollers shall not be less than 50 mm in the case of multiple rollers.

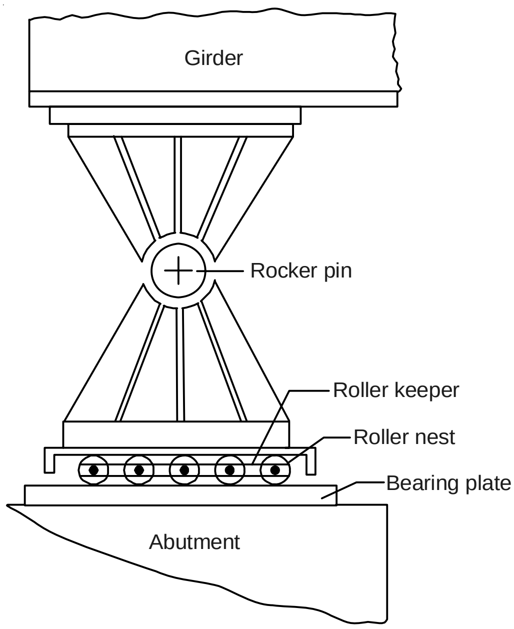

Fig. 14.2 Rocker and roller bearing.

The allowable working loads per unit length of cylindrical rollers (on flat surface) shall be taken as:

For mild steel:

(i) Single and double rollers: 8*D* N/mm of length.
(ii) Three or more rollers: 5*D* N/mm of length.

For high tensile steel:

(i) Single and double rollers: 10*D* N/mm of length.
(ii) Three or more rollers: 7*D* N/mm of length.

where *D* is the diameter of the roller in mm

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Sliding plate bearings

A sliding bearing, as its name implies, utilizes one plate sliding against the other to accommodate translational movement. As this bearing does not allow rotational movement, it is recommended to be used for span lengths of less than 15 m. Sliding bearings can be fabricated out of steel, although other materials such as teflon and bronze are also used. This type of bearing may only be used for straight steel bridges. A sliding bearing becomes unstable when subjected to lateral forces.

Elastomeric bearings

Elastomer is a polymeric substance obtained after vulcanization of rubber. Polychloroprene may also be used as a synthetic elastomer. An elastomeric bearing can consist of an unreinforced elastomeric pad or a reinforced elastomeric bearing may also be fabricated by binding together alternate layers of rubber and steel plates. The speciality of this bearing is that it takes direct compressive load, shearing force and moment by undergoing appropriate deformation. The movements of elastomeric bearings are shown in Fig. 14.3. These types of bearings are relatively a new invention. The advantages of elastomeric bearings are:

- They have no moving part, therefore, they require no maintenance.
- The height of the bearing being less, it calls for a lower headroom, thus, effecting reductions in the cost of approaches.
- In the event of a crack or split in the bearing, it can be easily accessed and replaced with a new one.

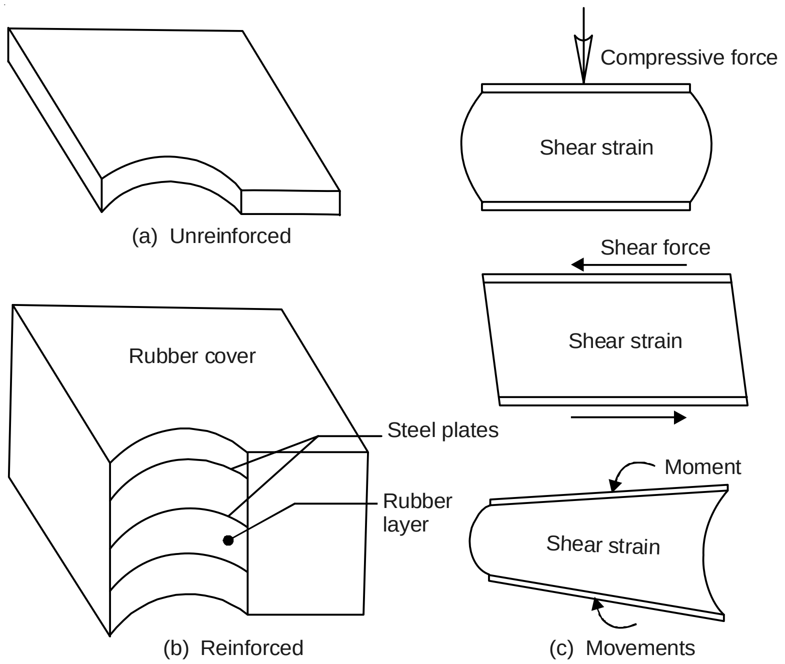

Fig. 14.3 Elastomeric bearings.

An elastomeric bearing should satisfy the following conditions as per IRC 83 (Part II) code.

1. Hardness should be 60 ± 5 degrees on IRHD Scale (International Rubber Hardness Scale).
2. Minimum tensile strength should be 17 MPa.
3. Minimum elongation at break shall be 400%.
4. Shear modulus of the elastomer shall be in the range from 0.8 MPa to 1.20 MPa.
5. Adhesion strength of the elastomer to steel plates shall not be less than 7 kN/m.

14.2 DESIGN OF UNREINFORCED ELASTOMERIC BEARINGS

1. Plan dimensions. The preferred dimensions of elastomeric bearings are given in Table 14.1 below. However, interpolation of plan dimensions can be made if the situation warrants.

Table 14.1 Standard plan dimensions of elastomeric bearings (IRC: 83-1987, Part II)

Size (*Index no.*)	*Width* (a) (mm)	*Length* (b) (mm)
1	160	250
2	160	320
3	200	320
4	200	400
5	250	400
6	250	500
7	320	500
8	320	630
9	400	630
10	400	800

2. The vertical (axial) stiffness of the elastomer is represented by its *shape factor*. The shape factor S of the elastomer is given by the ratio

$$\text{(Loaded surface area)/(Surface area free to bulge)} = \frac{ab}{2t(a+b)} \tag{14.1}$$

where a and b are plan dimensions of the pad, and t is the thickness of the pad.

3. Thickness. The thickness of a bearing is governed by its shear movement. If u is the translational shear deformation (Fig. 14.4), then

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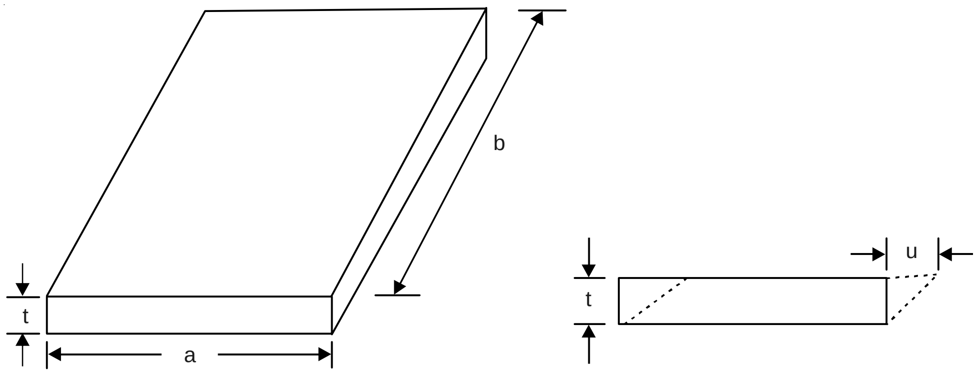

Fig. 14.4 Bearing dimensions and deformation.

$$u = t \tan\phi \tag{14.2}$$

$$\tan\phi = \frac{H_c + H_s}{GA} \tag{14.3}$$

where

G = modulus of rigidity in N/mm²

H_c = sustained horizontal load in newton

H_s = sustained dynamic horizontal load in newton

The value of u should be less than $0.7t$, such that $t > 1.43u$.

4. Average compressive stress. This is given by

$$\sigma_m = \frac{P}{A_e} \tag{14.4}$$

where

P = total vertical load in newton

A_e = effective plan area excluding shear deformation in mm²

$$= \frac{a-u}{b} \tag{14.5}$$

The average stress so calculated should be less than *2GS.*

5. To prevent slip. The slip of a bearing is due to high horizontal force and low vertical force. To avoid slip, the following conditions need to be met with

(a) $\sigma_m = \dfrac{P_c}{A_e} > \left(1 + \dfrac{a}{b}\right)$ MPa (14.6)

(b) $(H_c + H_s) < f(P_c + P_s)$ (14.7)

where

P_c and P_s = sustained and dynamic vertical load, respectively in newton

H_c and H_s = sustained and dynamic horizontal load, respectively in newton

f = coefficient of friction (average value = 0.3).

6. In order that a bearing does not overturn or topple, the thickness of the bearing is restricted to less than $a/5$.

14.3 BASIS FOR SELECTION OF BEARINGS

The successful behaviour of a bridge structure depends to a large extent upon the functioning of its bearings as anticipated in the design. The designer should have a clear understanding of the nature of forces developed in the structure. Accordingly, only the bearings which can successfully perform the considered functions, should be selected. When selecting a bearing, the factors listed below should be carefully considered:

1. High vertical load taking capability
2. Movement capability to cope with horizontal movements
3. Rotational capability
4. Capability to resist external horizontal forces like wind forces and centrifugal forces
5. Good seismic resistance, i.e. capability to dissipate energy at high displacement levels
6. Overall cost (i.e. initial cost, maintenance cost, etc.) should be low
7. Aesthetic considerations—low height bearings will add pleasing looks to a bridge than high bearings
8. Environmental conditions like physical environment should be considered for proper functioning of bearings during their lifespan. Roller bearings and sliding bearings face problems in dusty and desert conditions. Steel bearings should be avoided in the vicinity of water.

EXAMPLE 14.1

Design a mild steel rocker bearing for transmitting the superstructure reactive load of 1200 kN.

Allowable pressure on bearing block: 3.8 MPa

Permissible bending stress: 0.66 f_y = 165 MPa

Permissible bearing stress: 100 MPa

Permissible shear stress: 100 MPa

Design

Area of the bedplate

Area = load/permissible bearing stress

$= 1200 \times 10^3/3.8 = 315{,}789.47 \text{ mm}^2 \approx 320{,}000 \text{ mm}^2$

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A bedplate of size 400 mm × 800 mm can be provided.

Diameter of the rocker

The load taken by the rocker will be almost in the form of line load. It can be taken as 4D N/mm, where D is the diameter of the rocker in mm. Thus, we have

$$(4D \times 800) = 1200 \times 10^3$$

or

$$D = 375 \text{ mm, say } 400 \text{ mm}$$

Therefore,

$$\text{Radius of the rocker} = 200 \text{ mm}$$

Rocker pin

With reference to Fig. 14.5, the rocker pin is supported by the legs of the rocker bearing. It will experience bending in between the two legs.

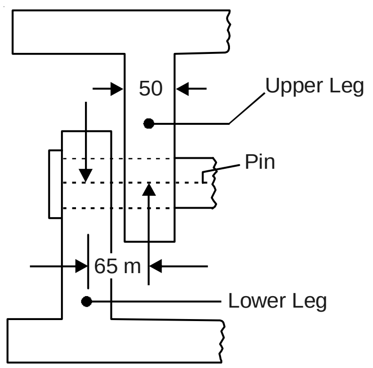

Fig. 14.5 Reaction transfer to pin (Example 14.1).

Load on one leg = (1/2) × 1200 × 10^3 = 600 × 10^3 N

Distance between the upper and lower legs = 25 + 25 + 15 = 65 mm

Bending moment = 600 × 10^3 × 65 = 39 × 10^6 N·mm

Section modulus, $Z = M/f_b$ = 39 × 10^6/165 = 236,363.6 mm^3

Therefore,

$$d^3 = 236{,}363.6 \times (32/\pi)$$

or

$$d = 134.02 \text{ mm, say } 140 \text{ mm}$$

Check for bearing stress

$$\text{Bearing area of the pin} = \text{thickness of the leg} \times \text{diameter}$$
$$= 50 \times 140 = 7000 \text{ mm}^2$$

Bearing stress = load/bearing area

$$= \frac{600 \times 10^3}{7000} = 85.71 \text{ N/mm}^2 < 100 \text{ N/mm}^2$$

Check for shear stress

Area of cross-section of the pin = $(\pi/4) \times 140^2$ = 15,393.80 mm^2

Shear stress = 600 × 10^3/15,393.80 – 38.97 N/mm^2 < 100 MPa

The details of the rocker bearing are displayed in Fig. 14.6.

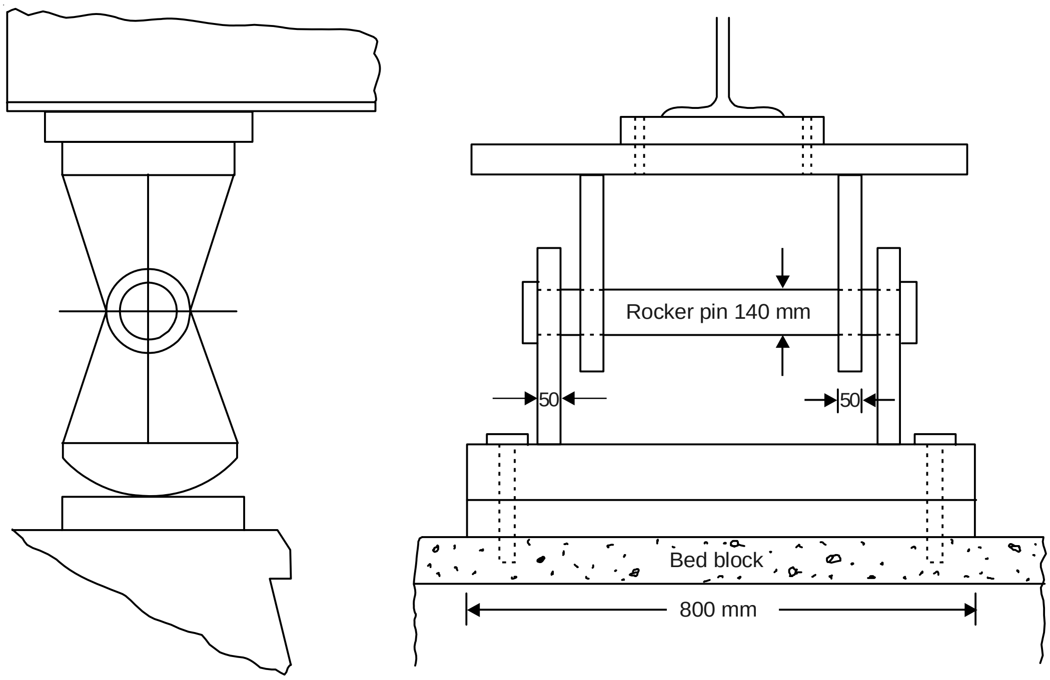

Fig. 14.6 Details of the rocker bearing (Example 14.1).

EXAMPLE 14.2

A reaction of 2500 kN is expected at the supports of a 20 m spanned T-beam bridge. Design a rocker and roller bearing. The other details are:

Allowable pressure on rollers: 5 N/mm diameter/mm length
Bearing pressure on rocker pin: 30 N/mm^2
Allowable pressure on bearing plate: 2000 N/mm^2
Allowable pressure on concrete bed block: 3.8 N/mm^2

Design

Rocker pin

As the minimum diameter stipulated is 75 mm, try a 120 mm pin.

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Bearing area = 120 L
Thus,

$$120 \times L \times 30 = 2500 \times 10^3$$

or

$$L = 694 \text{ mm, say 700 mm long pin}$$

Rollers

100 mm diameter rollers can be tried.
If L is the total length of the rollers, load taken by rollers = 5 × L × diameter
Thus,

$$5 \times L \times 100 = 2500 \times 10^3$$

or

$$L = 5000 \text{ mm}$$

Provide 6 rollers of 900 mm length each (Total length provided = 5400 mm).

Total width of the roller nest with a gap of 50 mm between rollers is

$$= 6 \times 100 + 5 \times 50 = 850 \text{ mm}$$

Allowance should be made for the movement of the rollers on either side, normally taken as 0.8 mm/m of span = 0.8 × 16 = 12.8 ≈ 15 mm which is for one side only. Therefore,

For either side = 15 × 2 = 30 mm

Width of the bearing plate = 850 + 30 = 880 mm, say 900 mm

Area of the bed block required = $(2500 \times 10^3)/3.8 = 658 \times 10^3$ mm^2

Adopt a bed block of 1.2 m × 1.2 m. This bearing is sketched in Fig. 14.7.

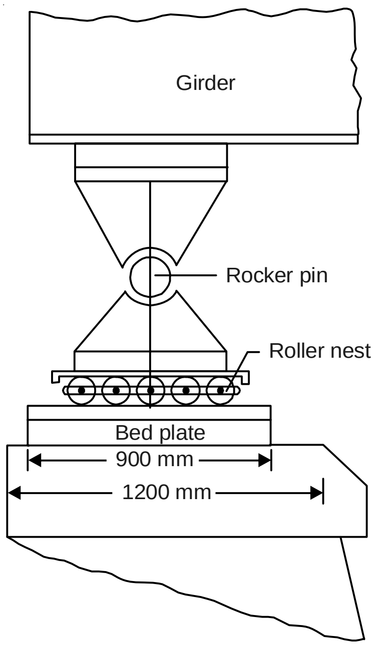

Fig. 14.7 Rocker roller bearing (Example 14.2).

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EXAMPLE 14.3

Design an elastomeric unreinforced neoprene pad bearing to suit the following data:

Vertical load (sustained): 200 kN
Vertical load (dynamic): 40 kN
Horizontal force: 60 kN
Modulus of rigidity of elastomer: 1 N/mm^2
Friction coefficient: 0.3

Design

Total vertical load = 200 + 40 = 240 kN

Horizontal force = 60 kN

Select the preferred dimensions from Table 14.1.

$$a = 250 \text{ mm}, \quad b = 500 \text{ mm}$$

Thickness (t) should be less than $a/5$, select a thickness of 30 mm

Area $A = 250 \times 500 = 125{,}000 \text{ mm}^2$

$\tan\phi = H/GA = (60 \times 10^3)/(1 \times 125{,}000) = 0.48$

$u = t \tan\phi = 30 \tan\phi = 30 \times 0.48 = 14.40$ mm

But, $t > 1.43u > 1.43 \times 14.40 = 20.59 < 30$ mm, therefore the design is safe.

Axial stress

$$\text{Shape factor } (S) = \frac{ab}{2t(a+b)} = \frac{250 \times 500}{2 \times 30(250+500)} = 2.77$$

$\sigma_m = P/A' < 2GS$

$A' = (a - u)b = (250 - 14.40)500 = 117{,}800 \text{ mm}^2$

$P = (240 \times 10^3)$ N

Therefore,

$$\sigma_m = \frac{240 \times 10^3}{117{,}800} = 2.03 < 2 \times 1 \times 2.77 = 5.54. \text{ Therefore, the design is safe.}$$

$$\sigma'_m = \frac{P_c}{A'} > \left(1 + \frac{a}{b}\right)$$

$$= \frac{200 \times 10^3}{117{,}800} = 1.697 > \left(1 + \frac{250}{500} = 1.5\right). \text{ Therefore, the design is safe.}$$

14.4 EXPANSION JOINTS

Expansion joints are designed to take up longitudinal and transverse movements of a bridge caused by thermal expansion, contraction and certain loading conditions. The type of joint selected for a deck is generally dependent on the type and magnitude of movement that a joint has to accommodate. An expansion joint in a bridge has to satisfy the following requirements:

- It should allow the movement of the bridge so that the stresses caused by temperature, shrinkage and loading are relieved.
- It should not allow percolation of water.
- It should be durable and structurally strong.
- It should be accessible for easy inspection and maintenance.
- It should be strong enough to withstand knocking of wheels of heavy vehicles.

Deck joints are of two types: open and closed. An open joint is nothing more than an opening between the concrete deck and an adjacent structural element (deck/deck, deck/abutment, deck/approach slab). In a closed joint, the gap between the adjacent elements of the deck is covered by a sealant. A mechanical system is provided to take up the movement of the bridge. Open joints are prone to leakage and deterioration and can handle small longitudinal movements only. Owing to these reasons, open joints are not being implemented in new constructions and therefore they are found in old bridges only.

14.4.1 Closed Joints

Closed or filled joints are widely adopted for bridges. This type of joint consists of a sealant, which is either inserted or hot poured into the joint. These types of joints are also suitable for rehabilitation work, where upgradation of existing joints (damaged) is required. Different types of closed joints are available; they are briefly discussed below.

Compression seal joint

This joint is made by squeezing a sealant material into an open joint. An adhesive lubricant is also provided along with the sealant. The common material used is the extruded neoprene. This material takes up the movement of the bridge by getting itself compressed. For added durability, compression seals are combined with steel angles at the deck slab edge to form an armoured joint. This type of joint is used for decks, which are expected to sustain movements ranging from 12 mm to 60 mm. However, a common problem with this type of joints is the loosening of bond between the seal and the concrete surface. This loosening in turn gives rise to loss of compression. Occasionally, this may lead to the seal popping out of the joint. This type of joint is shown below in Fig. 14.8.

Strip seal joint

This type of joint consists of an elastomeric material, which is placed between the dual rails that are anchored to the face of the joint opening. The most commonly used material is neoprene rubber. Here, the material is mechanically fitted into the steel rail assemblies. These joints can accommodate a larger movement than that by compressive seals (up to 100 mm).

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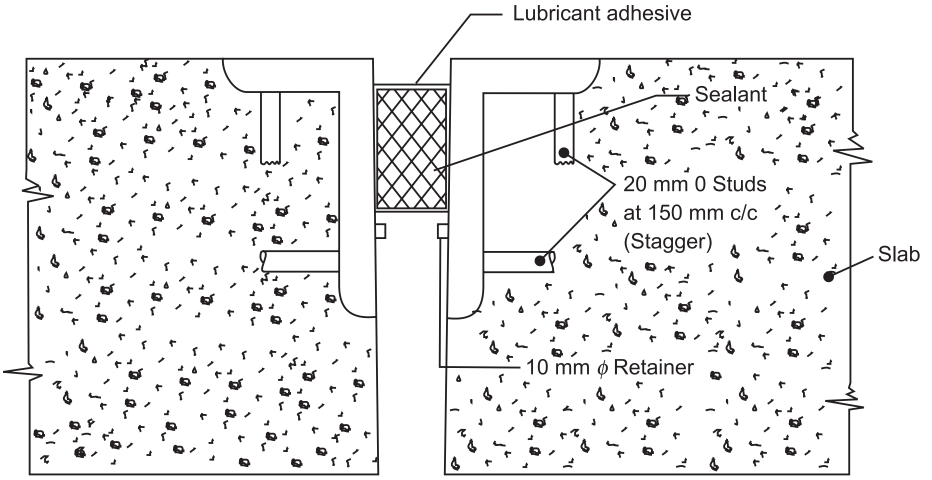

Fig. 14.8 Compression seal joint.

Modular joints

A modular joint uses multiple strip seals to accommodate very large deck movements. The seals are fitted between rolled beams, which run along the length of the joint. This type of joint can accommodate movements ranging from 900 mm to 1200 mm. It is used for skewed and curved decks.

DESIGN PROBLEMS

1. Design a steel rocker roller bearing to transmit a load of 2000 kN. The span of the girder is 50 m. The a1lowable pressure on rollers is 4 N/mm diameter/mm length. The bearing pressure on the rocker pin is 30 N/mm^2. The allowable pressure on the steel bearing plate is 2000 N/mm^2. The allowable pressure on the bed block of the abutment is 1000 N/mm^2.
2. Design an elastomeric unreinforced neoprene pad bearing to be placed beneath a girder of a bridge to suit the data given below:

   Span of the girder: 12 m

   Dimensions: 300 mm × 1400 mm

   No. of girders: 3

   Slab thickness: 300 mm

   No. of lanes: 2

   Live load: IRC Class AA

   Horizontal force: 100 kN

   Shear modulus of elastomer: 1 N/mm^2

   Coefficient of friction: 0.35

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Chapter 15 Prestressed Concrete Bridge Decks

15.1 INTRODUCTION

It is a well-known fact that the concrete is weak in tension and strong in compression. However, prestressing can be used to ensure that concrete remains within its tensile and compression capacity under the heavy loads. Prestressing of a bridge normally involves application of an external force to the concrete by the use of wires, strands or bars, and this can greatly increase the strength of concrete. Prestressing of concrete in bridges has resulted in longer and slender spans, improved aesthetics and increased economy in construction. With prestressing, the slab bridges can have the spans in the range of 10–20 m. While slab-beam (T-beam) can have the span range of 20–40 m. There are many vivid examples around the world of bridges with prestressed concrete decks such as the ones shown in Figs. 15.1 and 15.2.

Fig. 15.1 A single prestressed box girder bridge.

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Fig. 15.2 Fifth street bridge over great Miami-River.

Prestressed concrete bridges include a wide variety of different forms, from cast in situ to precast, from beams to box girders, and from simply supported to cable stayed. Their functions range from the carrying of pedestrians to road or rail traffic and they make up a significant proportion of the bridge stock in existence today.

The design of prestressed concrete bridges both influences and is dependent on the construction process envisaged. The construction sequence and practical considerations in positioning the tendons influence the prestress layout much more than the desire to achieve a concordant profile. The deck section and concrete shape is often dictated by the placement of the prestress tendons and their anchorages, while the need for rapid construction or difficulties with access may dictate the type of structure and construction methodology adopted.

15.2 PRINCIPLES OF PRESTRESSING

The prestressing of the concrete happens due to force transfer between the prestressed tendon and the concrete. Tendons are pulled and stretched and then firmly anchored against the concrete at the end of the section. The tension in the tendon is balanced by the compression in the concrete. In this way, external compression force is applied to the concrete and is used to counter the tensile stresses generated under the bending moments and the shear forces present due to heavy loading.

The tendons are placed either within the concrete member as internal tendons, or alongside the concrete as external tendons, and can be unbounded or bounded to the concrete. They can be pre-tensioned or post-tensioned. However, their effect on the concrete, and the basic principles of design are the same in all cases.

To apply the prestressing force, such as the one depicted in Fig. 15.3, jacks can be placed at either end of the beam and made a permanent part of the structure. Such a configuration, however, would be extremely susceptible to movement at the supports which

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could lead to reduction in prestress force. A more popular solution is to utilize a steel cable or tendon, which is embedded within the concrete beam and anchored at the ends of the member. When the prestressig force is applied to the tendon prior to the concrete being poured, the beam is said to be *pre-tensioned*. When the force is applied after the concrete has hardened the beam is called *post-tensioned*.

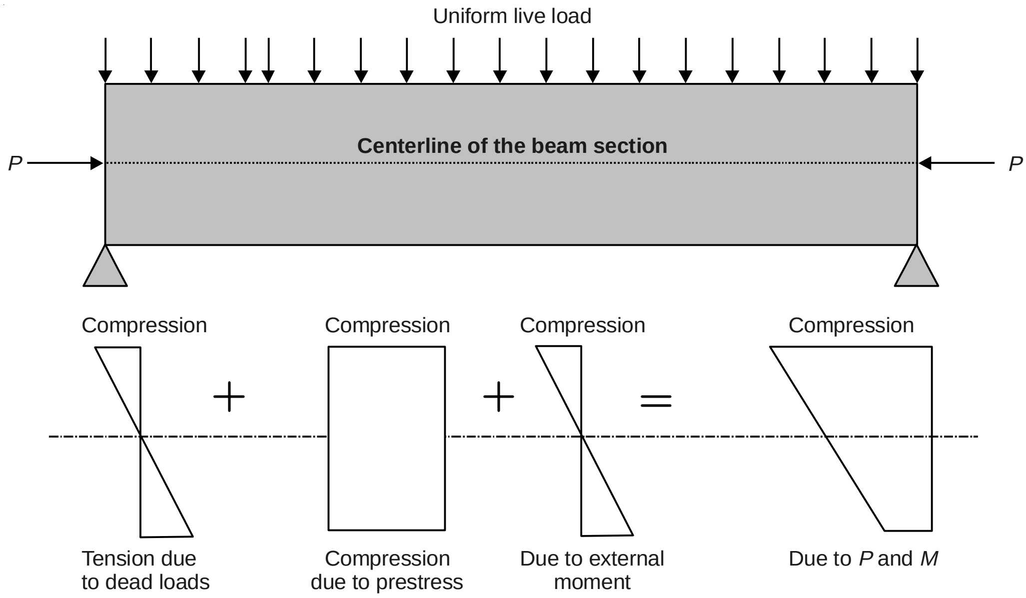

Fig. 15.3 Stress distribution at mid span of a simple rectangular beam prestressed concentrically.

Stresses at the bottom of the beam as shown in Fig. 15.3, are caused by dead loads and live loads, can only be eliminated if the compressive stress induced by the prestressing force *P*, is equal to the magnitude of the tensile stress induced by the applied loads. Like conventional reinforced concrete, prestressed concrete must also account for the effects of shrinkage and creep. If the beam is to eliminate all tensile stresses, then the prestressing force must be greater than the tensile stresses alone in order to account for the additional deformational loads. One drawback to this process is that the top fibre of the beam must resist both the compressive forces from the applied load and the prestressing force.

15.3 PRE-TENSIONING

Pre-tensioning is used to describe a method of prestressing in which the tendons are tensioned before the concrete is placed, and the prestress is transferred to the concrete when suitable cube strength is reached. The pre-tensioning is a four-stage process:

Stage 1: Tendons and reinforcement are positioned in the beam mould.

Stage 2: Tendons are stressed to about 70–80% of their ultimate strength.

Stage 3: Concrete is cast into the beam mould and allowed to cure to the required initial strength.

Stage 4: When the concrete has cured, the prestressing force is released and the tendons anchor themselves in the concrete.

Pre-tensioned beams are sometimes fabricated in casting yards where the tendons are anchored between large abutments known as pre-tensioning beds, which can be as much as 150 m apart. An alternative method is to embed the anchors within the beam itself.

One major advantage of pre-tensioning approach is that it lends itself well to the mass production of beams. As prestressed bridges became more popular, the standard beam sizes accepted, fabricators found themselves producing prestressed pre-tensioned concrete beams much in the same fashion steel fabricators had been churning out wide flanged beams.

One method of enhancing the performance of pre-tensioned beams is to deflect the prestressing tendons upward and downward along the length of the beam. This process is known as *draping* or *barping*. Figure 15.4 shows the process of pre-tensioning.

Fig. 15.4 Pre-tensioning of tendons.

15.4 POST-TENSIONING

In a post-tensioned beam, the steel tendons are stressed after the concrete has had time to harden. The tendons are incorporated into the concrete beam either in bonded fashion or unbonded way. Bonded tendons are placed within preformed voids in the concrete member. These voids could be formed by metal ducts or plastic tubes. After the concrete has hardened and the post-tensioning stress is applied, the space between the hole and the tendon is filled with grout so that the tendon and the tube assembly becomes *bonded* to the surrounding beam.

Unbonded tendons are simply greased and wrapped in paper. After the post-tensioning force is applied, they are left as is or *unbonded* to the surrounding concrete. With respect to bridges the bonded tendons are more popular. However, both the methods require the incorporation of an anchorage at the ends of post-tensioned member.

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The post-tensioning is also a four-stage process:

Stage 1: Cable ducts and reinforcement are positioned in the beam mould. The ducts are usually raised towards the neutral axis at the ends to reduce the eccentricity of the prestressing force.

Stage 2: Concrete is cast into the beam mould and allowed to cure to the required initial strength.

Stage 3: Tendons are threaded through the cable ducts and tensioned to about 70–80% of their ultimate strength.

Stage 4: Wedges are inserted into the end anchorages and the tensioning force on the tendons is released. Grout is then pumped into the ducts to protect the tendons.

For post-tensioning, the tendon is pulled and stretched using a hydraulic jack and the resulting force is transferred directly on to the hardened concrete through the tendon anchor. The tendon consists of bars, single strands or multi-strands that can be arranged with a varying vertical and horizontal profile along the bridge deck, which allows for the most efficient arrangements of prestressing to be adopted. Post-tensioning can be used on many different types of structures, including precast beams, in situ or precast box girders and cable stayed structures.

Internal tendons are placed inside a duct cast into the concrete and installed either before or after the concrete is placed, while external tendons are installed in ducts placed outside the concrete section after the concreting has been completed. The force is generated in a post-tensioned tendon by the use of a stressing jack. The jack pulls the strands and transfers the force from the tendon on to the anchor plate and into concrete. Reinforcements in the surrounding concrete resist the local tensile stresses around the anchor and assists in transferring the force into the deck. Figure 15.5 shows the post-tensioning bars and Fig. 15.6 depicts the process of prestressing.

Fig. 15.5 Post-tensioning bars.

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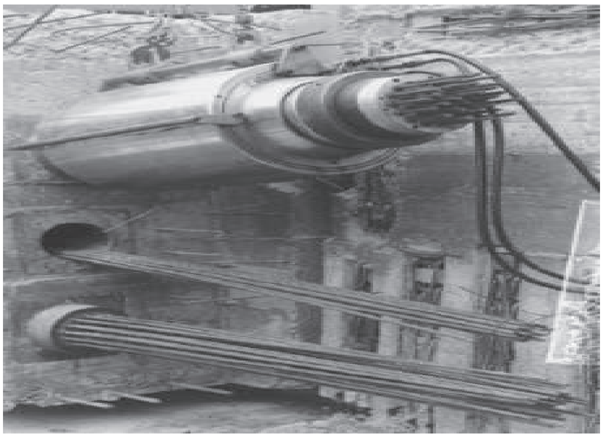

Fig. 15.6 Post-tensioning process.

While pre-tensioning has the advantage of mass production, post-tensioning is advantageous when very large elements are being fabricated which cannot be transported to the project site. The ability to post-tension precast or cast-in-situ members is another advantage, which provides the designer with added flexibility. Where pre-tensioned beams are typically produced in standard sizes and geometries, post-tensioned girders can be customized to fit site-specific conditions.

15.5 STRANDS, TENDONS AND BARS

The most common form of prestressing involves 7-wire strand, which is made up of individual cold-drawn wires with six outer wires twisted around an inner core wire. The strand is stress relived and is usually of a low relaxation grade. Usually strands could be galvanized for a greater protection against corrosion. For post-tensioning, 13 mm or 15 mm diameter, 7-wire strand is used, either singly for pre-tensioning or in bundles to form multi-strand tendons. The most common post-tensioned tendon sizes utilize 7, 12, 19 or 27 strands to suit the standard anchor blocks available, although systems are available for tendons incorporating up to 55 strands when necessary. At the ends of tendons the strands are anchored either by splaying out the wires and encasing them in the concrete as a dead-end anchorage, or by passing them through an anchor arrangement and fixing them into an anchor block for live-end anchorage.

Prestressing bars are available in different diameters from 15 mm to 75 mm and are used in post-tensioned construction. They typically have a minimum ultimate characteristic tensile strength between 1000 MPa and 1100 MPa, although a higher strength steel grade is available. Jacking forces range from 135 kN to over 3000 kN. Bars are formed in straight lengths by the hot rolling of steel rods and are either smooth or deformed on the surface. The bars are generally placed into ducts cast into the concrete between two anchor blocks located on the concrete surface. Bars are stressed by pulling from one end, using a stressing jack placed against the anchorage arrangement, and then held in place by a nut assembly. Figure 15.7 shows a picture of prestressing strand.

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Fig. 15.7 Prestressing strand.

15.6 ANCHORAGES

At each end of a tendon the force is transferred into the concrete by an anchorage system. For pre-tensioned strands the anchorage is by bond and friction of the bare strand cast into the concrete, while for post-tensioned tendons anchorage is achieved by using anchor blocks or an encased dead end anchor. For a post-tensioning, the bar is held in place by the threaded nut and the force from the bar is transferred through the threads and the nut and on to the anchor block that is cast into the concrete. The hole on the near side of the anchor block assists with the grouting of the duct and the anchorage arrangement after stressing has been completed.

Stressing of multi-strand tendons is undertaken using jacks placed over the anchorage and tendon. The jack grips each strand and pulls the tendon until the required force is generated. Wedges are then pushed into place around the strand and seated into the anchor block, so that on the release of the force by the jack the wedges grip the strand and transfer the force on to the anchorage and into the concrete. The strands and wedges are seated into holes formed in the anchor block, which rests against the bearing plate and trumpet, cast into the concrete. A typical anchorage block (end block) is shown in Fig. 15.8.

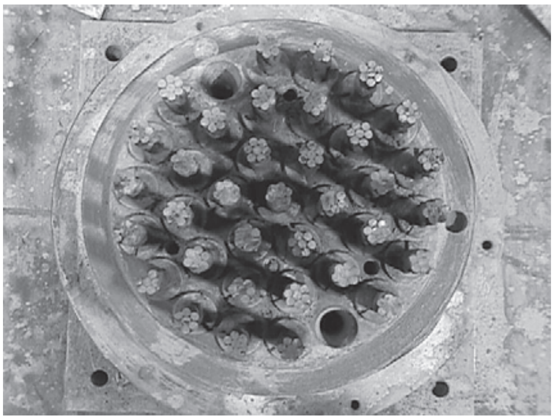

Fig. 15.8 Anchorage block.

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15.7 END BLOCK

The end zone (or end block) of a post-tensioned member is a flared region which is subjected to high stress from the bearing plate next to the anchorage block. It needs special design of transverse reinforcement. The design considerations are bursting force and bearing stress.

The stress field in the end zone of a post-tensioned member is complicated. The compressive stress trajectories are not parallel at the ends. The trajectories diverge from the anchorage block till they become parallel. Based on Saint Venant's principle, it is assumed that the trajectories become parallel after a length equal to the larger transverse dimension of the end zone. Figure 15.9 shows the external forces and the trajectories of tensile and compressive stresses in the end zone.

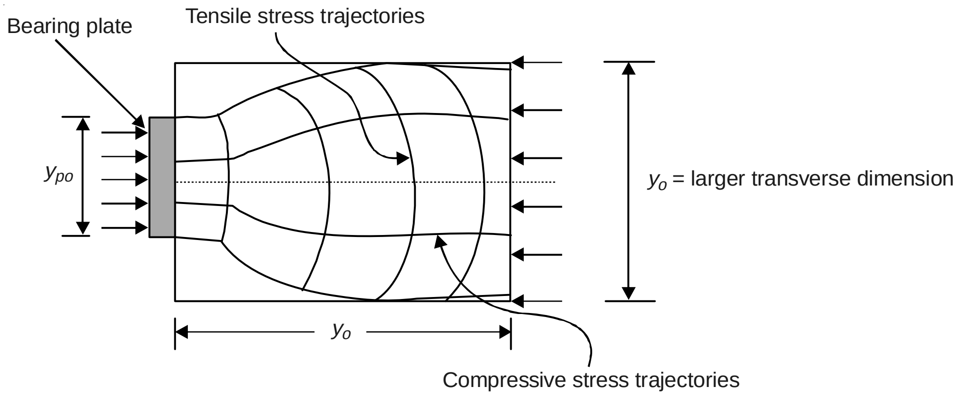

Fig. 15.9 Stress trajectories at the end zone of a post-tensioned beam.

The local zone is the region behind the bearing plate and is subjected to high bearing stress and internal stresses. The behaviour of the local zone is influenced by the anchorage device and the additional confining spiral reinforcement. The general zone is the end zone region which is subjected to spalling of concrete. The stress is compressive for a distance $0.1y_o$ from the end. Beyond that it is tensile. The tensile stress increases and then drops down to zero within a distance y_o from the end.

The transverse tensile stress is known as *splitting tensile stress*. The resultant of the tensile stress in a transverse direction is known as the *bursting force* (F_{bst}).

Transverse reinforcement is provided in each principle direction based on the value of F_{bst}. This reinforcement is called end zone reinforcement or anchorage zone reinforcement or bursting links. The reinforcement is distributed within a length from $0.1y_o$ to y_o from an end of the member. The amount of end zone reinforcement in each direction (A_{st}) can be calculated from the following equation:

$$A_{st} = \frac{F_{bst}}{f_s} \tag{15.1}$$

The stress in the transverse reinforcement (f_s) is limited to $0.87f_y$. When the cover is less than 50 mm, f_s is limited to a value corresponding to a strain of 0.001. The end zone reinforcement is provided in several forms, some of which are proprietary of the construction firms. The forms are closed stirrups, mats or links with loops. The local zone is further strengthened by confining the concrete with spiral reinforcement. The performance of the reinforcement is determined by testing end block specimens. The end zone may be made of high strength concrete. The use of dispersed steel fibres in the concrete (fibre reinforced concrete) reduces the cracking due to the bursting force. Proper compaction of concrete is required at the end zone. Any honeycomb of the concrete leads to settlement of the anchorage device. If the concrete in the end zone is different from the rest of the member, then the end zone is cast separately.

15.8 STEPS FOR DESIGNING A POST-TENSIONED PRESTRESSED CONCRETE DECK SLAB

Step 1: Assume overall thickness of the slab to be 50 mm/meter span of the deck.

Step 2: Find out the dead load moment and shear force.

Step 3: Find the maximum live load bending moment and shear force for a particular IRC loading under consideration.

Step 4: Check whether actual section modulus provided for the slab is more than the minimum section modulus of the slab section. The minimum section modulus is given by:

$$Z_{\min} = \left(\frac{M_q + (1-\eta)M_g}{f_{br}} \right) \tag{15.2}$$

where M_q is dead load bending moment, M_g is live load bending moment, η is loss ratio, $f_{br} = (\eta f_{ct} - f_{tw})$, f_{ct}: permissible stress in concrete at transfer to be obtained from IRC :18–2000, f_{tw}: permissible tensile stress under working loads.

Step 5: Find the minimum prestressing force required. The minimum prestressing force is given by the relation:

$$P = \left(\frac{A(f_{\inf} Z_b + f_{\sup} Z_t}{Z_b + Z_t} \right) \tag{15.3}$$

where A is cross-sectional area of slab per one-metre width, $f_{\inf}$ and $f_{\sup}$ are the prestress in the concrete developed at the top and bottom fibres respectively. Z_b and Z_t are the section modulus of the bottom and top fibres.

Step 6: Calculate the prestress that can be sustained by each cable by assuming the permissible level of presterss. Find out the spacing of cables.

Step 7: Find the eccentricity of the cables using the formula:

$$e = \left(\frac{Z_t Z_b (f_{\inf} - f_{\sup})}{A(f_{\sup} Z_t + f_{\inf} Z_b)} \right) \tag{15.4}$$

Step 8: Check for stresses at service loads.

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Step 9: Check for the ultimate shear resistance of support section of the slab. If ultimate shear force is less than 50% ultimate shear strength, no shear reinforcement is required.

Step 10: Design supplementary reinforcement.

Step 11: Design the end block reinforcement.

15.9 DESIGN EXAMPLE: POST-TENSIONED PRESTRESSED CONCRETE DECK SLAB

Data for the Design

- Clear span: 9 m.
- Clear width of roadway: 7.5 m.
- Foot paths: 600 mm wide on either side.
- Thickness of wearing coat: 100 mm at the centre of the road.
- Live load: IRC class AA tracked vehicle.
- Type of construction/structure: class-I.
- Materials: M-50 grade concrete and 7 mm dia high strength strands with ultimate tensile strength at 1500 MPa. The cable consists of 12 strands anchored at the end with a suitable diameter anchor block.
- Compressive strength of concrete at transfer (f_{ci}) = 40 MPa.

Permissible stresses

The permissible compressive stresses in the concrete at transfer and at working loads as recommended in IRC-18 (2000) are as follows:

- $f_{ct} < 0.5\, f_{ci} = 0.5 \times 40 = 20$ MPa
- Loss ratio = η = 0.8
- Permissible compressive stress in concrete under service loads (f_{cw}) = 0.33 f_{ck} (f_{ck} is characteristic strength of concrete, in this example, it is 50 MPa), $f_{cw} = 0.33 \times 50 = 16.5$ MPa
- Allowable tensile stress in concrete at initial transfer of prestress (f_{tt}) = 0
- Allowable tensile stress in concrete under service loads (f_{cw}) = 0

Dead load bending moment and shear force

Since the slab is prestressed, the thickness may be reduced and could be taken at 50 mm per metre span of the slab.

Overall thickness of the slab: 9 × 50 = 450 mm.
Width of the bearing: 400 mm.
Therefore, the effective span = 9 + 0.4 = 9.4 m.

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Dead weight of the slab = (0.45 × 24) = 10.8 kN/m^2
Dead weight of wearing coat: (0.1 × 22) = 2.2 kN/m^2
Total dead load = 10.8 + 2.2 = 13 kN/m^2
Dead load bending moment (M_g) = (13 × 9.4^2)/8 = 143.6 kN·m.
Dead load shear force = (13 × 9.4)/2 = 61.1 kN.

Live load bending moment

The maximum live load bending moment is generated in the slab when the IRC class AA tracked wheel is centrally placed on the slab.

Impact factor is taken as 10% as the span is more than 9 m.
Effective length of the load = 3.6 + 2(0.45 + 0.1) = 4.7 m.
The arrangement of the wheel on the span of the slab is shown in Fig. 15.10.

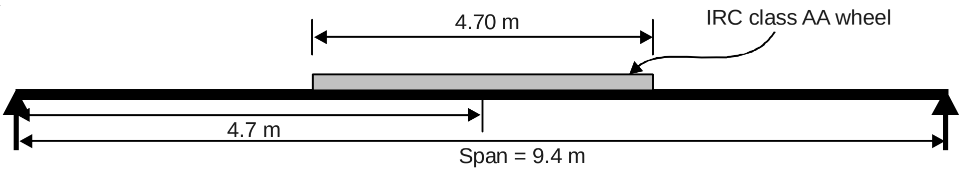

Fig. 15.10 Placement of IRC class AA load for maximum bending moment.

Effective width of load dispersion perpendicular to span is given by (refer Chapter 7):

$$b_{ef} = \alpha x\left(1 - \frac{x}{l}\right) + b_1 \qquad (15.5)$$

Width of the slab = 7.5 + 2 × 0.6 = 8.7 m
Therefore, $k = B/l$ = 8.7/9.4 = 0.92, from IRC–21–2000, α = 2.36. b_1 = (0.85 + 2 × 0.1) = 1.05 m.

Substituting the values,

$$b_{ef} = 2.36 \times 4.7 \times \left(1 - \frac{4.7}{9.4}\right) + 1.05 = 6.596 \approx 6.6 \text{ m}$$

Figure 15.11 shows the placement of IRC class AA wheel as per the spacing requirements stipulated by IRC: 6–2000.

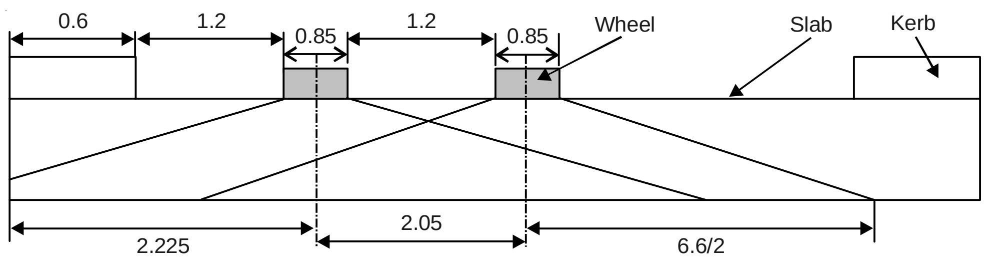

Fig. 15.11 Effective width of dispersion for IRC class AA tracked wheel.

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Referring to Fig. 15.11, the width of dispersion for two tracks is given by

$$B = 2.225 + 2.05 + 3.3 = 7.575 \text{ m}$$

Therefore, the intensity of live load $= \dfrac{1.10\times 700}{4.7\times 7.575} = 21.63 \text{ kN/m}^2$

Maximum bending moment due to live load

$$= 21.63 \times 4.7 \times 0.5 \times 4.7 - 21.63 \times 4.7 \times 0.5 \times 4.7 \times 0.25$$

$$M_q = 179.17 \text{ kN·m}$$

Maximum live load shear

The placement of wheel for obtaining maximum shear force is as shown in Fig. 15.12.

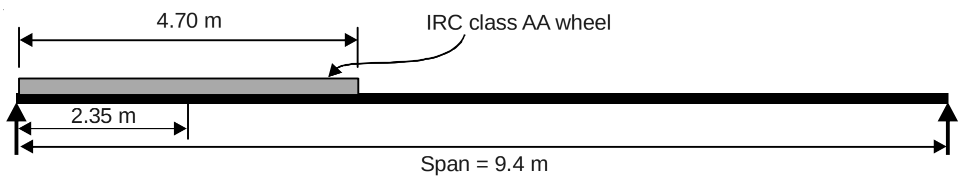

Fig. 15.12 Placement of IRC class AA load for maximum shear force.

Effective width of dispersion of a wheel:

$$b_{ef} = 2.36\times 2.35\times\left(1-\frac{2.35}{9.4}\right)+1.05 = 5.21 \text{ m}$$

Net effective width of dispersion: B = 2.225 + 2.05 + 5.21/2 = 6.9 m

Intensity of live load to be considered for computing shear force $= \dfrac{1.10\times 700}{4.7\times 6.9} = 23.74 \text{ kN/m}^2$

Maximum live load shear force = (23.74 × 4.70 × (9.4 – 2.35))/9.4 = 83.68 kN

Dead load shear force = 61.1 kN

Total Design Shear = 83.68 + 61.1 = 144.8 kN.

Verification of section modulus provided

As slab provides a symmetrical section, section modulus

$$Z_t = Z_b = Z = \left(\frac{1000\times 450^2}{6}\right) = 33.75 \times 10^6 \text{ mm}^3$$

The permissible stress in concrete at transfer (f_{ct}) is obtained from IRC 18–2000.

$$f_{ct} = 20 \text{ MPa},\ f_{cw} = 16.5 \text{ MPa},\ f_{tw} = 0,\ \eta = \text{loss ratio} = 0.8,$$

$$f_{br} = (\eta f_{ct} - f_{tw}) = (0.8 \times 20 - 0) = 16 \text{ MPa}$$

The minimum section modulus to be provided is given by

$$Z_{\min} = \left(\frac{M_q + (1-\eta)M_g}{f_{br}}\right) \tag{15.6}$$

$$= \left(\frac{179.17 + (1-0.8)143.6}{16}\right) \times 10^6 = 12.99 \times 10^6 < 33.75 \times 10^6 \text{ mm}^3$$

Therefore, the section provided is sufficient to resist the service loads.

Prestressing force required

The required prestressing force is given by

$$P = \left(\frac{A(f_{\inf} Z_b + f_{\sup} Z_t)}{Z_b + Z_t}\right)$$

$$f_{\sup} = \left(f_{tt} - \frac{M_g}{Z_t}\right) = 0 - \frac{143.6 \times 10^6}{33.75 \times 10^6} = -4.25 \text{ MPa}$$

$$f_{\inf} = \left(\frac{f_{tw}}{\eta} + \frac{(M_q + M_g)}{\eta Z_b}\right) = 0 + \frac{(179.17 + 143.6)10^6}{0.8 \times 33.75 \times 10^6} = 11.95 \text{ MPa}$$

$$\therefore \quad P = \left(\frac{1000 \times 450 \times (11.95 \times 33.75 \times 10^6 - 4.25 \times 33.75 \times 10^6)}{2 \times 33.75 \times 10^6}\right) = 1732.5 \times 10^3 \text{ N} = 1732.5 \text{ kN}$$

Using cables containing 12 strands of 7 mm diameter stressed to 1200 MPa,

Force in each cable = $(12 \times \pi \times 7^2 \times 1200)/(4 \times 1000)$ = 554 kN

$$\therefore \text{ Spacing of cables} = \frac{(1000 \times 554)}{1732.5} = 319.76 \text{ mm} \approx 320 \text{ mm c/c}$$

Eccentricity of cables

The eccentricity of the cable is measured at the centre of span. It is given by the relation:

$$e = \left(\frac{Z_t Z_b (f_{\inf} - f_{\sup})}{A(f_{\sup} Z_t + f_{\inf} Z_b)}\right) \tag{15.7}$$

$$e = \left(\frac{33.75^2 \times 10^{12} (11.95 + 4.25)}{1000 \times 450 \times 33.75 \times 10^6 (-4.25 + 11.95)}\right) = 157.8 \approx 158 \text{ mm}$$

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The eccentricity is to be made available at the centre of the span only. To achieve this, the cables should be given parabolic profile.

Check for stresses

(a) Check for failure by yielding of steel

As per IRC 18–2000, for under-reinforced section:

$$M_u = 0.9 d_p A_p f_p \tag{15.8}$$

where, A_p = The area of high tensile steel = (12 × 38.5 × 1000)/320 = 1443.75 mm^2

f_p = The ultimate tensile strength of steel without definite yield point or stress at 4% elongation whichever is higher (taken as 1500 MPa).

d_p = The depth of the slab from the maximum compression edge to the CG of the steel tendons (225 + 158 = 383 mm).

$\therefore M_u = 0.9 \times 383 \times 1443.75 \times 1500 = 746 \times 10^6$ N·mm

(b) Failure by crushing of concrete

As per IRC 18–2000, for a rectangular section, the ultimate moment required for crushing of concrete is given by

$$M_u = 0.176 b d_b^2 f_{ck} = 0.176 \times 1000 \times 383^2 \times 50 = 1290 \times 10^6 \text{ N·mm}$$

The actual ultimate moment of resistance to be chosen is smaller of the two values calculated in above steps.

$$\therefore \quad M_u = 746 \times 10^6 \text{ N·mm}$$

According to IRC 18–2000,
Required ultimate capacity of the section = $1.5M_g + 2.5M_q$

$$= 1.5 \times 143.6 + 2.5 \times 179.17 = 663.32 \text{ kN·m or } 663.32 \times 10^6 \text{ N·mm}$$

Therefore, the ultimate moment capacity of the section (746×10^6 N·mm) is greater than the required ultimate moment.

(c) Check for ultimate shear strength

Ultimate shear force = 1.5 × dead load shear + 2.5 × live load shear

$$V_u = 1.5 \times 61.1 + 2.5 \times 83.68 = 300.85 \text{ kN}$$

As per IRC 18–2000, the ultimate shear resistance of support section uncracked in flexure is given by

$$V_{co} = 0.67 bd \sqrt{f_t^2 + 0.8 f_{cp} f_t} \tag{15.9}$$

b = width of slab = 1000 m

d = overall depth of slab = 450 mm

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f_t = principal tensile stress given by $0.24\sqrt{f_{ck}} = 0.24\sqrt{50}$ = 1.69 MPa

f_{cp} = compressive prestress at centroidal axis = $\eta(P/A) = \dfrac{0.8 \times 1732.5 \times 10^3}{1000 \times 450} = 3.1$ MPa

$$\therefore \quad V_{co} = 0.67 \times 1000 \times 450\sqrt{1.69^2 + 0.8 \times 3.1 \times 1.69} = 800384.55 \text{ N} = 800.4 \text{ kN}$$

Since the ultimate shear force (V_u) is less than 50% of the ultimate shear resistance V_{co}, no shear reinforcement is required.

Design of supplementary reinforcement

Supplementary reinforcement may be provided at 0.18% of the gross sectional area.

$$A_s = (0.18 \times 1000 \times 450)/100 = 810 \text{ mm}^2$$

10 mm dia HYSD bars may be provided at 200 mm c/c both at top and bottom faces of the slab in both the directions.

Design of end block

The concentric cables each carrying a force of 554 kN are placed at 320 mm c/c create a bursting force at the support section. Owing to this force, bursting tension will be developed across the section of the slab near the support. Suitable tension reinforcement is to be provided to counteract this effect. The bursting tension force may be computed using the Table 15.1 as recommended in IRC 18–2000.

Table 15.1 Design bursting tensile force in end blocks

Y_{po}/Y_o	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_k	0.23	0.2	0.17	0.14	0.11

Here, $2Y_{po}$ = side of loaded area (320 mm), P_k = Tendon force (554 kN)

Y_o = Side of end block (150 mm), F_{bst} = bursting tensile force

$\therefore$ Y_{po}/Y_o = 150/320 = 0.468 = 0.5 for this ratio, F_{bst}/P_k = 0.17

$\therefore$ F_{bst} = 0.17 × 554 = 94.18 kN

Using 10 mm dia Fe-250 grade steel bars as end block reinforcement, the area of steel required would be = $(94.18 \times 10^3)/(0.87\, f_y) = (94.18 \times 10^3)/(0.87 \times 250) = 433 \text{ mm}^2$

10 mm dia bars at 100 mm centre in the vertical and horizontal directions have been provided at distances of 100 mm and 200 mm respectively.

The longitudinal and cross section of the deck slab is shown in Figs. 15.13 and 15.14.

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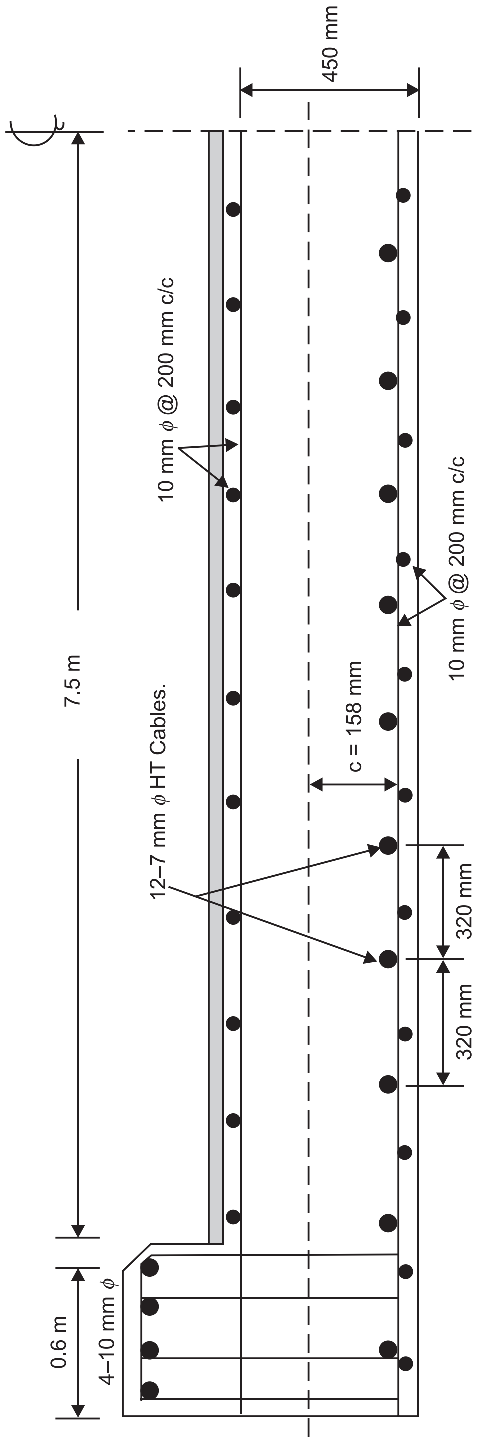

Fig. 15.13 Cross-section of slab at centre of span.

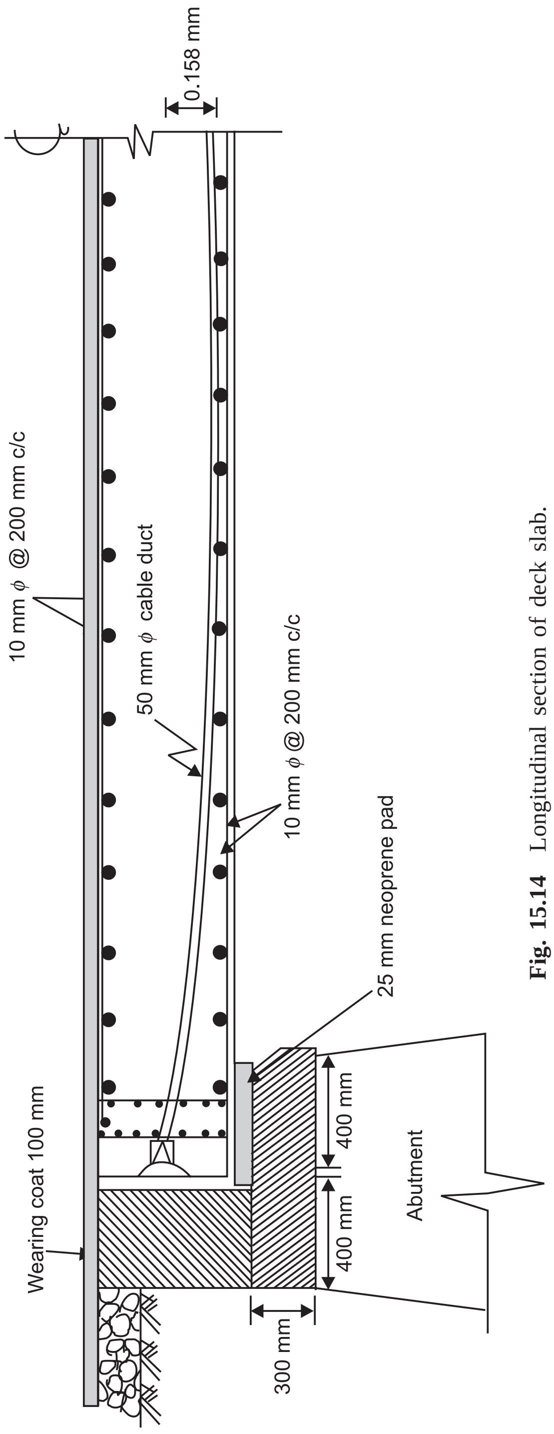

Fig. 15.14 Longitudinal section of deck slab.

15.10 DESIGN EXAMPLE: POST-TENSIONED PRESTRESSED CONCRETE T-BEAM BRIDGE DECK

Data for the Design

- Effective span: 25 m.
- Clear width of roadway: 7.5 m.
- Foot paths: 1 m wide on either side.
- Thickness of wearing coat: 100 mm at the centre of the road.
- Spacing of cross girders: 5 m c/c.
- Live load: IRC class AA tracked vehicle.
- Type of construction: class-I.
- Materials: M-40 grade concrete for deck slab and M-50 grade concrete for girders, 7 mm dia high strength strands with ultimate tensile strength at 1500 MPa. The cable consists of 12 strands anchored at the end with a suitable diameter anchor block.
- Compressive strength of concrete at transfer (f_{ci}) = 40 MPa.

Permissible stresses and design constants

The permissible compressive stresses in the concrete at transfer and at working loads as recommended in IRC–18 (2000) are as follows:

- $f_{ct} < 0.5\, f_{ci} = 0.5 \times 40 = 20$ MPa
- Loss ratio = η = 0.8
- Permissible compressive stress in concrete under service loads (f_{cw}) = 0.33 f_{ck} (f_{ck} is characteristic strength of concrete, in this example, it is 50 MPa), f_{cw} = 0.33 × 50 = 16.5 MPa
- Allowable tensile stress in concrete at initial transfer of prestress (f_{tt}) = 0
- Allowable tensile stress in concrete under service loads (f_{cw}) = 0

The design constants are computed as follows:

For M40 concrete and Fe 415 steel, as per IRC 21–2000

$$n = \frac{10 \times 200}{10 \times 200 + 13.33} = 0.4$$

$$j = 1 - \frac{n}{3} = 1 - \frac{0.4}{3} = 0.866$$

$$Q = \frac{1}{2} \times 13.33 \times 0.4 \times 0.866 = 2.3$$

The preliminary dimensions of the different components of the deck are as shown in Fig. 15.15.

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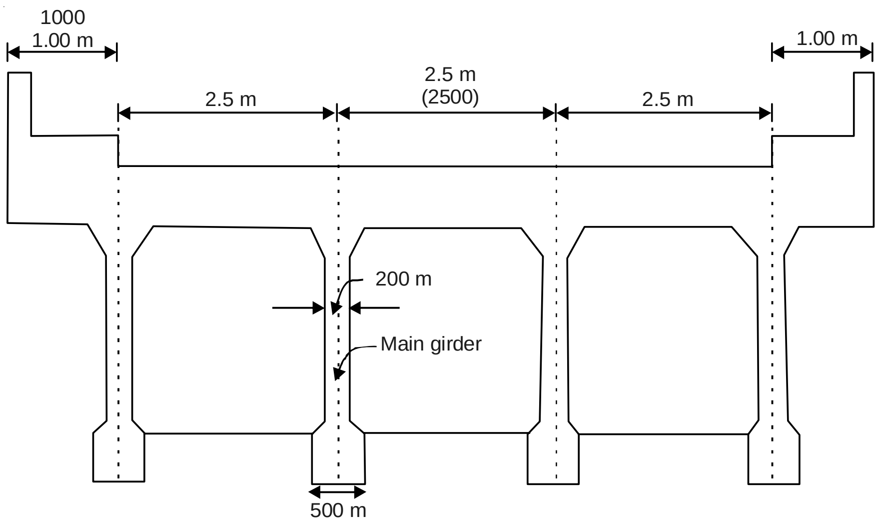

Fig. 15.15 Cross-section of bridge deck.

Design of interior slab panel

(a) Dead load bending moment and shear force

Dead weight of the slab = (1 × 1 × 0.25 × 24) = 6.00 kN/m^2

Wearing coat = (0.1 × 22) = 2.2 kN/m^2

Total design load = 8.2 kN/m^2

Total dead load on the panel = 5 × 2.5 × 8.2 = 102.5 kN

As the panel is loaded with uniformly distributed dead laod, the ratios $U/B = 1$, $V/L = 1$, $k = B/L = 2.5/5 = 0.5$ and $1/k = 2.0$. Referring to the Pigeaud's curve (Fig. 15.16), we find: $m_1 = 0.047$ and $m_2 = 0.01$. Using the moment coefficients, the dead load bending moments in short and long span directions are arrived at

$$M_{BD} = 102.5(0.047 + 0.15 \times 0.01) = 4.97 \text{ kN·m}$$

$$M_{LD} = 102.5(0.01 + 0.15 \times 0.047) = 1.74 \text{ kN·m}$$

$$\text{Dead load shear force} = (0.5 \times 8.2 \times 2.3) = 9.43 \text{ kN}$$

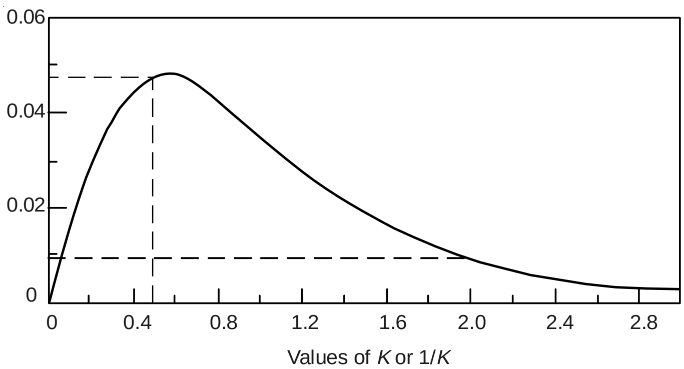

Fig. 15.16 Moment coefficients for slabs completely loaded with uniformly distributed load. m_1 and m_2 correspond to k and $1/k$ respectively.

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(b) Live load bending moment and shear force

In order to generate maximum live load bending moment, the IRC Class AA tracked wheel (single) is placed on the panel of the slab as shown in the Fig. 15.17.

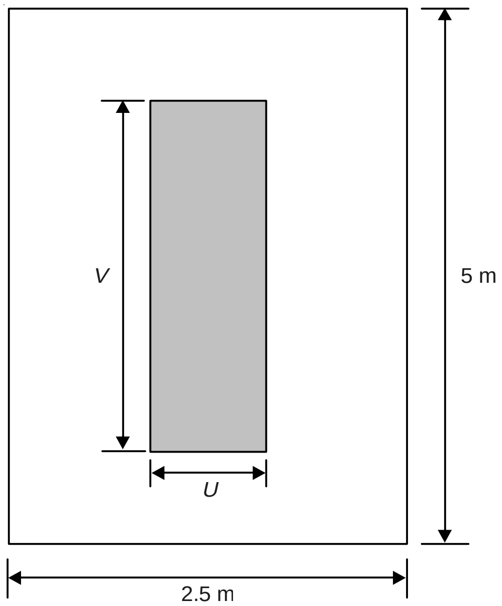

Fig. 15.17 Placement of IRC Class AA tracked wheel on the panel.

The dispersion length of the wheel = U = (0.85 + 2 × 0.1) = 1.05 m
The dispersion width of the wheel = V = (3.60 + 2 × 0.1) = 3.8 m
The ratios U/B = 1.05/2.5 = 0.42, V/L = 3.8/5.0 = 0.76 and $K = B/L$ = 2.5/5.0 = 0.5

Referring to Pigeaud's curve corresponding to $K = 0.5$, the moment coefficients for short and long span of the slab are: $m_1 = 0.095$ and $m_2 = 0.025$

Short span and long span live load bending moments are:

$$M_{BL} = 350(0.095 + 0.15 \times 0.025) = 34.6 \text{ kN·m}$$

$$M_{LL} = 350(0.025 + 0.15 \times 0.095) = 13.73 \text{ kN·m}$$

As the slab is continuous, design live load bending moment may be taken as 80% of the actual. Considering impact factor to be 25%.

$$M_{BL} = 1.25 \times 0.80 \times 34.6 = 34.6 \text{ kN·m}$$

$$M_{LL} = 1.25 \times 0.80 \times 13.73 = 13.73 \text{ kN·m}$$

(c) Live load shear force

Live load shear force can be calculated by approximate method. In order to obtain maximum shear, the wheel is placed such that the dispersion is contained within the interior panel of the slab.

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Span ward dispersion length of the wheel load = 0.85 + 2(0.1 + 0.25) = 1.55 m

The arrangement of the wheel is as shown in Fig. 15.18.

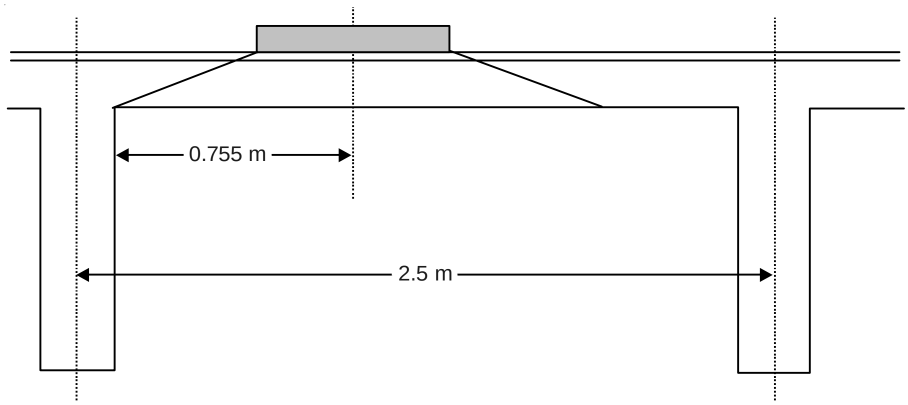

Fig. 15.18 Placement of wheel on the panel for maximum shear force.

Effective width of slab = $\alpha x\left(1-\frac{x}{l}\right)+b_1$

Clear length of the panel = (5 – 0.2) = 4.8 m, $\therefore$ B/L = 4.8/2.3 = 2.08.

From IRC 21–2000, for B/L = 2.08, α is 2.6 for continuous slab.

Effective width of the slab = $2.6\times 0.775\times\left(1-\frac{0.775}{2.3}\right)+(3.6+2\times 0.1)=5.136\text{ m}$

Live load per metre width of the slab = 350/5.136 = 68.15 kN = 68 kN
Shear force per metre width of the slab = 68(2.3 – 0.775)/2.3 = 45.1 kN
Shear force considering impact = 1.25 × 45 = 56.25 kN.

Design of the slab

Total M_B = 34.6 + 4.97 = 39.57 kN·m
M_L = 13.83 + 1.74 = 15.6 kN·m

Effective depth of the slab = d = $\sqrt{\frac{M}{QB}}=\sqrt{\frac{39.57\times 10^6}{2.3\times 1000}}=131.16\text{ mm}$

Adopting effective depth as 200 mm, the area of steel is:

$$A_{st}=\frac{39.57\times 10^6}{200\times 0.86\times 200}=1150\text{ mm}^2$$

Using 14 mm bars at 130 mm c/c (area of steel actually provided A_{st} = 1184 mm^2)
Effective depth available along long span using 10 mm bars = 200 – 7 – 5 = 188 mm

Area of steel in transverse direction, $A_{sd} = \dfrac{15.6 \times 10^6}{200 \times 0.86 \times 188} = 482.43\ \text{mm}^2$

The bars are to be placed at 160 mm c/c.

(d) Check for shear stresses

The design shear force = dead load shear + live load shear = 9.43 + 56.25 = 65.68 kN

Nominal shear stress $= \dfrac{V}{bd} = \dfrac{65.68 \times 10^3}{1000 \times 210} = 0.312\ \text{N/mm}^2$

For percentage of steel $\dfrac{100 \times 1184}{1000 \times 210} = 0.56$ and for M40 concrete, permissible shear stress is 0.32 N/mm^2. Considering multiplication factor of 1.10, the actual permissible shear stress is found to be 1.10 × 0.32 = 0.352. Therefore, the shear stress in the slab is within permissible limits.

Design of longitudinal girder

First, it is required to find Courbon's reaction factor, for this IRC class AA loads are arranged for maximum eccentricity as shown in Fig. 15.19.

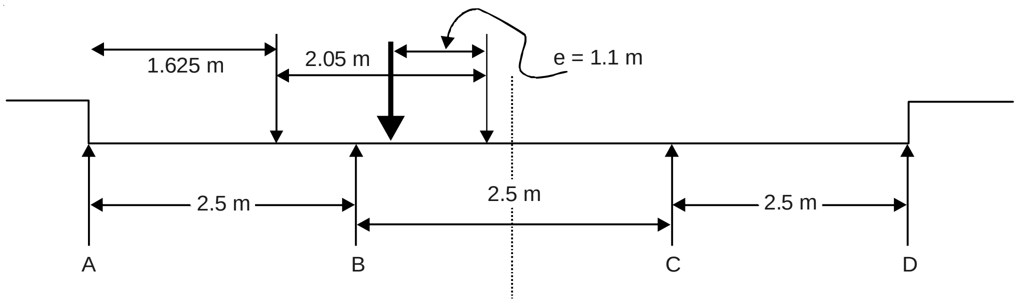

Fig. 15.19 Transverse arrangement of IRC class AA tracked vehicle.

Reaction factor for exterior girder (A or D)

$$R_A = \frac{2W_1}{4}\left(1 + \frac{4I \times 3.75 \times 1.1}{(2I \times 3.75^2) + (2I \times 1.25^2)}\right) = 0.764W_1$$

Reaction factor for interior girder (B or C)

$$R_B = \frac{2W_1}{4}\left(1 + \frac{4I \times 1.25 \times 1.1}{(2I \times 3.75^2) + (2I \times 1.25^2)}\right) = 0.588W_1$$

Here, W = 700 kN, $W_1 = W/2$, ∴ $R_A = 0.764 \times W/2 = 0.382\ W$, $R_B = 0.588 \times W/2 = 0.294W$

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Dead load from the deck of the bridge

The cantilever portion of the deck is shown in Fig. 15.20.

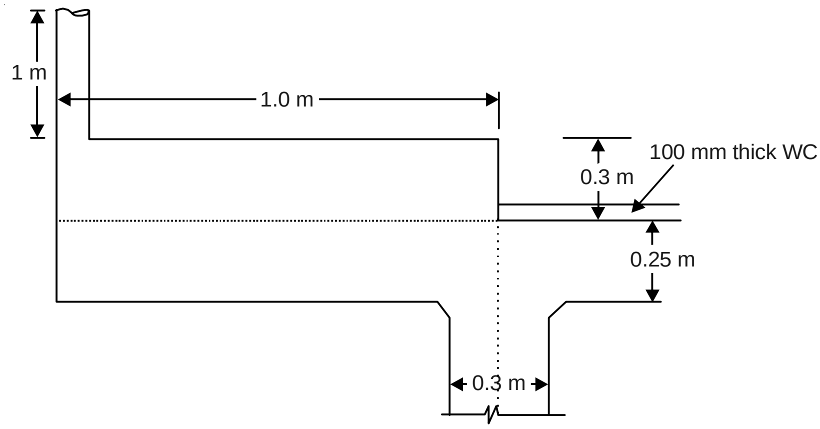

Fig. 15.20 Cantilever portion of the deck.

Loading from parapet railing: 0.8 kN/m

Loading from footpath: (0.3 × 24 × 1) = 7.2 kN/m

Loading from deck slab = (0.25 × 1 × 24) = 6.0 kN/m

Total dead load from one portion of the cantilever = 14 kN/m

Total dead load from the deck = [Dead load of two cantilever portions] + [Dead load of slab]

$$= 2 \times 14 + 8.2 \times 7.5 = 89.5 \text{ kN}$$

This dead load is assumed to be equally shared by all the four girders. Thus, load per girder is 89.5/4 = 22.37 kN.

Dead load of the main girder

Assuming a depth of 60 mm per metre span of the girder, the overall depth of the main girder is 60 × 25 = 1500 mm. The section of the girder is assumed as shown in Fig. 15.21. The bottom flange is wide enough to accommodate cables.

The self-weight/metre run of the girder (by considering the section to be plain)

$$= 0.5 \times 0.45 \times 24 + 1.0 \times 0.2 \times 24 = 10.2 \text{ kN/m}$$

Weight of the cross girder (assuming the depth to be 1 m) = 1 × 0.2 × 24 = 4.8 kN/m

Dead load bending moment and shear of the main girder

With the dead loads arrived in the previous step, the dead load bending moment and shear force is calculated:

Reaction of cross girder on main girder = 4.8 × 2.5 = 12 kN

Reaction from the deck slab = 22.37 kN/m

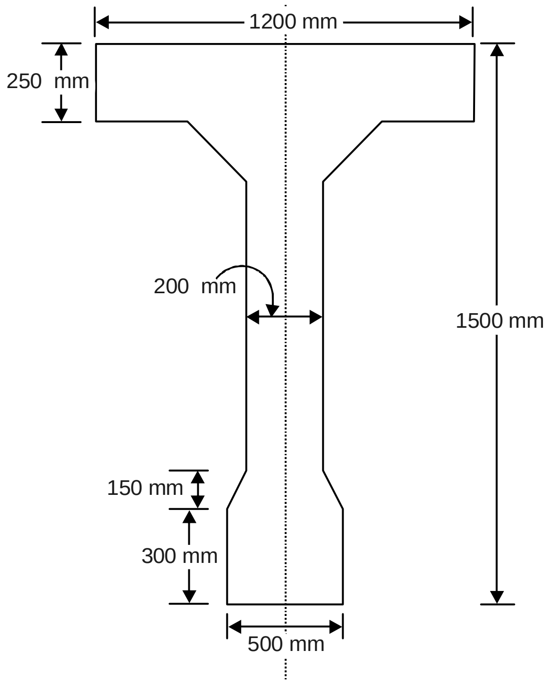

Fig. 15.21 Cross-section of the girder.

Total dead load on the girder including self-weight = 22.37 + 10.2 = 32.6 kN/m
The loading is sketched in Fig. 15.22.

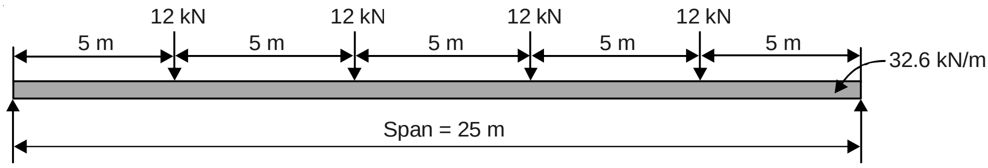

Fig. 15.22 Arrangement of dead loads on the main girder.

The maximum dead load shear force and bending moment is computed using the Fig. 15.20.

Maximum shear force = reaction at the support = 0.5 × (12 × 4 + 36.2 × 25) = 475.5 kN

$$\text{Maximum bending moment} = \frac{32.6 \times 25^2}{12} + 24 \times 12.5 - 12 \times 7.5 - 12 \times 2.5 = 3008.125 \text{ kN-m}$$

Live load bending moment

Live load bending moment in the girder is calculated using the Fig. 15.23.

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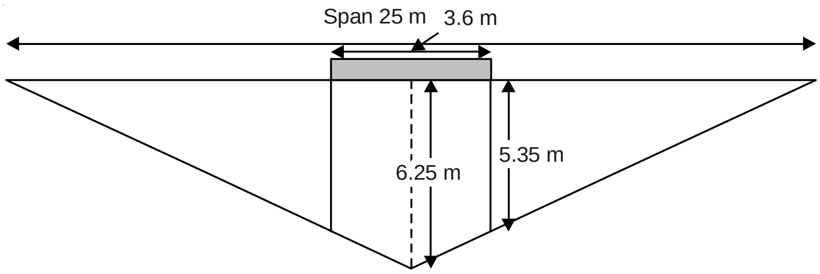

Fig. 15.23 Influence diagram for maximum bending moment.

Bending moment at the center of the girder = 0.5 × (6.25 + 5.35) × 700 = 4060 kN-m

Bending moment for the outer girder considering impact factor and reaction factor

= 1.1 × 0.382 × 4060 = 1706.01 kN-m

Bending moment for the inner girder is = 1.1 × 0.294 × 4060 = 1313 kN-m.

Live load shear force in girder

The disposition of the wheels for computing shear force is shown in Fig. 15.24.

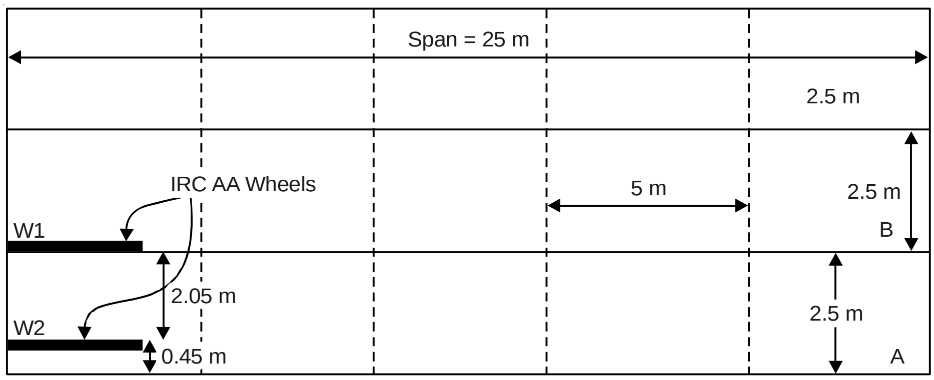

Fig. 15.24 Disposition of wheels for finding shear force in longitudinal girders.

Reaction of W2 on girder B = (350 × 0.45)/2.5 = 63 kN

Reaction of W2 on girder A = (350 × 2.05)/2.5 = 287 kN

Total load on girder *B* = (350 + 63) = 413 kN

Using the magnitude of reaction arrived in above steps; maximum reaction (shear force) in the girders is calculated.

Maximum reaction in girder B = (413 × 23.2)/25 = 383.26 kN

Maximum reaction in girder A = (287 × 23.2)/25 = 266.4 kN

The design live load shear force for inner girder considering impact factor is

= 383.3 × 1.1 = 421.6 kN

For outer girder = 1.1 × 266.4 = 293 kN

A summary of design bending moments and shear forces are given in Table 15.2.

Table 15.2 Summary of design moments and shear forces in beams

Bending Moment	Owing to dead load	Owing to live load	Design value
Outer Girder	3008 kN-m	1706 kN-m	4714 kN-m
Inner Girder	3008 kN-m	1313 kN-m	4321 kN-m
Shear force	**Owing to dead load**	**Owing to live load**	**Design value**
Outer Girder	475.5 kN	293 kN	769 kN
Inner Girder	475.5 kN	421.6 kN	897 kN

Sectional properties of the girder

The cross section of the beam is simplified to ease out the calculations pertaining to the sectional properties. Thus, the top flange is 1200 mm × 250 mm, web = 200 mm × 800 mm and the bottom flange = 500 mm × 450 mm

Area of cross section = A = 1200 × 250 + 200 × 800 + 500 × 450 = 69.75 × 10^4 mm^2

Distance of the centroid axis from top = y_t

$$= (1200 \times 250 \times 125 + 200 \times 800 \times 650 + 500 \times 450 \times 1275)/(69.75 \times 10^4)$$

$$= 614.15 \text{ mm}$$

Distance of the centroid axis from the bottom = y_b = 1500 – 615.15 = 884.85 mm

Moment of inertia of the section about centroid axis is

$$I = \frac{1200 \times 250^3}{12} + 1200 \times 250 \times (614.15 - 125)^2 \text{ (MI of top flange)}$$

$$+ \frac{200 \times 800^3}{12} + 200 \times 800 \times (650 - 614.15)^2 \text{ (MI of web)}$$

$$+ \frac{500 \times 450^3}{12} + 500 \times 450 \times (884.85 - 850)^2 = 14.37 \times 10^{10} \text{ mm}^4$$

Section modulus of bottom section = Z_b = (I/y_b) = (14.37 × 10^{10})/884.85 = 1.624 × 10^8 mm^3

Section modulus of top section = Z_t = (I/y_t) = (14.37 × 10^{10})/614.15 = 2.33 × 10^8 mm^3

Check for the adequacy of the section adopted

Making use of the section properties, the adequacy of the section is verified. Various design parameters to be used are listed.

f_{ck} = 50 MPa, η = 0.85, f_{ci} = 40 Mpa, f_{ct} = 20 MPa, f_{cw} = 16.5 MPa,

$f_{br} = (\eta f_{ct} - f_{tw})$ = (0.85 × 20 – 0) = 17 MPa, $f_{tr} = (f_{cw} - \eta f_{tt})$ = 16.5 MPa, M_q = 1706 kN-m,

M_g = 3008 kN-m, $M_d = M_q + M_g$ = 4714 kN-m

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$f_{inf} = f_{tw}/\eta + M_d/(\eta \times Z_b) = 0 + (4714 \times 10^6)/(0.85 \times 1.624 \times 10^8) = 34.14$ MPa

$f_{sup} = f_{tt} - M_g/Z_t) = 0 - (3008 \times 10^6)/(2.33 \times 10^8) = -6.45$ MPa

Required section modulus for the bottom section of the beam

$$Z_{min} = \left(\frac{M_q + (1-\eta)M_g}{f_{br}}\right) = \frac{(1706 + (1-0.85) \times 3008) \times 10^6}{17}$$

$$= 1.268 \times 10^8 < 1.624 \times 10^8 \text{ mm}^3$$

Hence, section provided is just adequate.

Prestressing force

With a maximum cover of 150 mm, the eccentricity provided for the prestressing force is (884.5 – 150) = 734.5 mm. Prestressing force is computed using the formula

$$P = \frac{(A \cdot f_{inf} Z_b)}{(Z_b + A.e)} = \frac{(69.75 \times 10^4 \times 34.14 \times 1.624 \times 10^8)}{(1.624 \times 10^8 + 69 \times 10^4 \times 734.5)} = 4.725 \times 10^6 \text{ N} = 4725 \text{ kN}$$

Using 7-strands of 15.2 mm diameter in a cable, force in each cable = (7 × 181.45 × 1500) = 1905 kN

Therefore number of cables = 4725/1905 = 2.48 ≈ 3

Area provided by three cables = 3 × 7 × 181.45 = 3810.45 mm^2

The arrangement of cables at the central section of the girder is shown in Fig. 15.25.

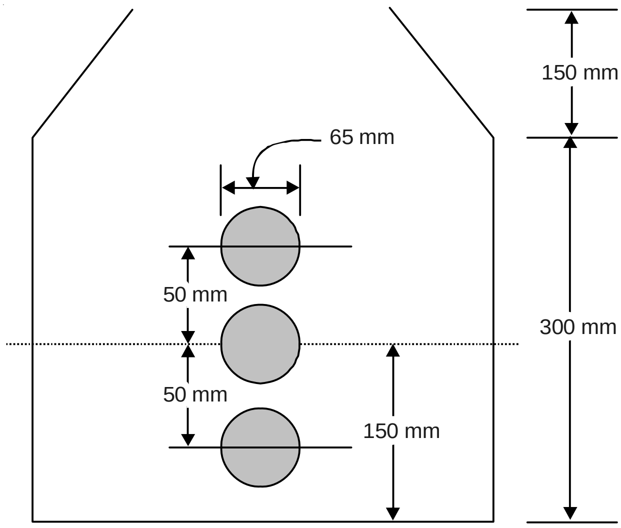

Fig. 15.25 Placement of cables at centre span section.

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Permissible tendon zone at support section

To avoid stress concentration at the supports, the cables are to be placed in such a way to satisfy the eccentricity requirements.

$$e \leq \frac{Z_b f_{cr}}{P} - \frac{Z_b}{A} = \frac{1.624 \times 10^8 \times 20}{4725 \times 10^3} - \frac{1.624 \times 10^8}{69.75 \times 10^4} = 687.4 \text{ mm}$$

$$e \geq \frac{Z_b f_{tw}}{\eta P} - \frac{Z_b}{A} = 0 - \frac{1.624 \times 10^8}{69.75 \times 10^4} = -232.83 \text{ mm}$$

The cables are arranged on a parabolic profile providing an eccentricity of 150 mm towards the top flange of the beam at support section. The arrangement of cables is shown in Fig. 15.26.

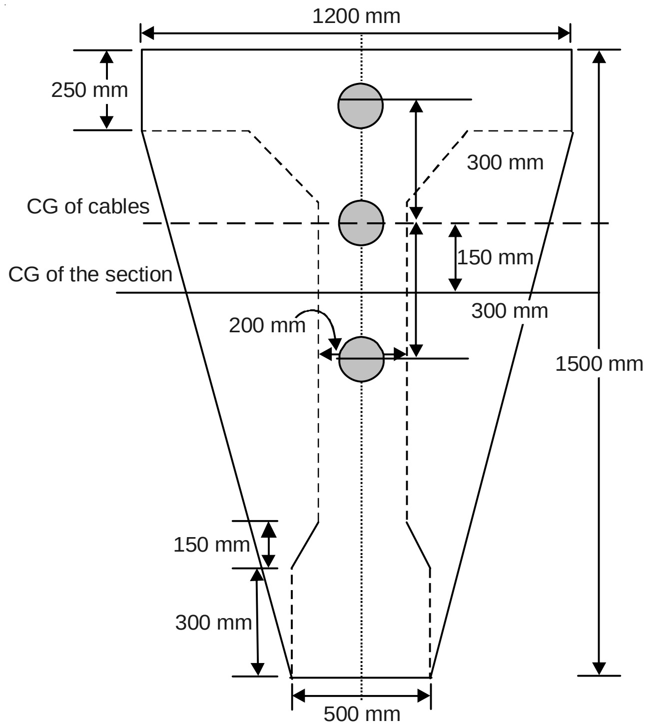

Fig. 15.26 Arrangement of cables at support section.

Check for stresses

The stress levels in the section of the beam located at centre of the span are checked. We have

$P/A = (4725 \times 10^3)/(69.75 \times 10^4) = 6.77$ MPa

$(P.e)/Z_t = (4725 \times 10^3 \times 734.5)/(2.33 \times 10^8) = 14.88$ MPa

$(P.e)/Z_b = (4725 \times 10^3 \times 734.5)/(1.624 \times 10^8) = 21.37$ MPa

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$(M_g/Z_t) = (3008 \times 10^6)/(2.33 \times 10^8) = 12.90$ MPa

$(M_g/Z_b) = (3008 \times 10^6)/(1.624 \times 10^8) = 18.52$ MPa

$(M_q/Z_t) = (1706 \times 10^6)/(2.33 \times 10^8) = 7.32$ MPa

$(M_q/Z_b) = (1706 \times 10^6)/(1.624 \times 10^8) = 10.50$ MPa

Stresses at transfer of prestress

In top fibres $\sigma_t = (P/A) - (P_e/Z_t) + (M_g/Z_t) = 6.77 - 14.88 + 12.90 = 4.79$ MPa

In bottom fibres $\sigma_b = (P/A) + (P_e/Z_b - (M_g/Z_b) = 6.77 + 21.37 - 18.52 = 9.62$ MPa

Stresses at working stage

In top fibres $\sigma_t = (\eta P/A) - (\eta P_e/Z_t) + (M_g/Z_t) + (M_q/Z_t)$

$= 0.85 \times 6.77 - 0.85 \times 14.88 + 12.9 + 7.32 = 13.32$ MPa

In bottom fibres $\sigma_t = (\eta P/A) + (\eta P_e/Z_b) - (M_g/Z_b) - (M_q/Z_b)$

$= 0.85 \times 6.77 + 0.85 \times 21.37 - 18.52 - 10.50 = -5.1$ MPa

It is observed that the stresses at bottom as well as top layers of the girders under the two conditions considered are well within the limits.

Check for ultimate flexural strength of the beam

The ultimate moment to be considered (IRC: 18–2000)

$$M_u = 1.5M_g + 2.5M_q$$
$$= 1.5 \times 3008 + 2.5 \times 1706 = 8777 \text{ kN-m}$$

Under ultimate load conditions, the failure may either occur by yielding of the steel (under-reinforced) or by the direct crushing of the concrete (over reinforced). Ultimate moment of resistance of the section, under these two alternative conditions of failure is calculated by the following formulae and the smaller of the two values shall be taken as the ultimate moment of resistance of the section for design.

(i) Failure by yielding of steel

$$M_u = 0.9 d_b A_s f_p$$

where, A_s=the area of high tensile steel.

d_b = the depth of the beam from the maximum compression edge to the centre of gravity of the steel tendons.

f_p = the ultimate tensile strength for steel without definite yield point or yield stress at 4% elongation whichever is higher for steel with a definite yield point.

$= 0.9 \times 1350 \times 3810 \times 1860 = 8.61 \times 10^9$ N-mm

(ii) Failure by crushing of concrete

For T-beam section, the ultimate moment is given by

$$M_u = 0.176\, b d_b^2 f_{ck} + \frac{2}{3} \times 0.8(B_f - b)\left(d_b - \frac{t}{2}\right) t f_{ck}$$

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where, b = the width of the web, B_f0 = the width of the flange and t = thickness of flange.

$$M_u = 0.176 \times 200 \times 1350^2 + \frac{2}{3} \times 0.8 \times (1200 - 200) \times \left(1350 - \frac{250}{2}\right) \times 250 \times 50$$

$$= 8.23 \times 10^9 \text{ N-mm} = 8230 \text{ kN-m}$$

As the required ultimate moment 8230 kN-m < 8777 kN-m the section is safe.

Check for ultimate shear strength of the beam

The ultimate shear force to be considered = $1.5S_g + 2.5S_q$ where, S_g is dead load and S_q is live load shear force.

$$V_u = 1.5 \times 475.5 + 2.5 \times 421.6 = 1767.25 \text{ kN}$$

According to IRC 18–2000, the ultimate shear strength of the section uncracked in flexure, V_{co} corresponds to the occurrence of a maximum principal tensile stress, at the centroidal axis of the section, of $f_t = 0.24\, f_{ck}$. In the calculation of V_{co}, the value of prestress at the centroidal axis has to be taken as $0.8\, f_{cp}$. The value of V_{co} is given by

$$V_{co} = 0.67bd\sqrt{f_t^2 + 0.8\, f_{cp}\, f_t} + \eta P \sin\theta \tag{15.10}$$

where b = width of rib, d = overall depth of the member, f_t maximum principal tensile stress $= 0.24\sqrt{f_{ck}}$ = 1.7 MPa, f_{cp} = compressive stress at centroidal axis due to prestress taken as positive = (hP/A) = 5.75 MPa

Eccentricity of cables at the centre span = 734.5 mm

Eccentricity of cables at support = 150 mm

Net eccentricity = 734.5 – 150 = 584.5 mm

Slope of the cable = $\theta = 4.e/L$ = (4 × 584.5)/(25 × 1000) = 0.093

$$V_{co} = 0.67 \times 200 \times 1500 \times \sqrt{1.7^2 + 0.8 \times 5.75 \times 1.7} + (0.85 \times 4725 \times 10^3 \times 0.093)$$

$$= 1031306.6 \text{ N} = 1032 \text{ kN}$$

Ultimate shear resistance considered = 1767.25 kN ≈ 1767 kN

Ultimate shear capacity of the section = 1032 kN

Balance shear = 1767 – 1032 = 735 kN

Shear reinforcement is to be designed to resist the balance shear

Using 10 mm diameter stirrups the spacing is given by

$$S = \frac{0.87 \times f_y \times A_{sv} \times d}{V} = \frac{0.87 \times 415 \times 2 \times 79 \times 1450}{735 \times 10^3}$$

$$= 112.53 \text{ mm}$$

Provide 10 mm diameter stirrups at 110 mm c/c near support and at a spacing of 220 mm c/c near the centre of the span.

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Design of supplementary reinforcement

Longitudinal supplementary reinforcement at 0.15% of gross cross-sectional area is provided to limit the shrinkage cracks.

$$\text{Area of steel} = A_s = (0.0015 \times 69.75 \times 10^4) = 1046.25 \text{ mm}^2$$

14 mm diameter bars (8 numbers) are placed in the compression flange of the beam as shown in Fig. 15.27.

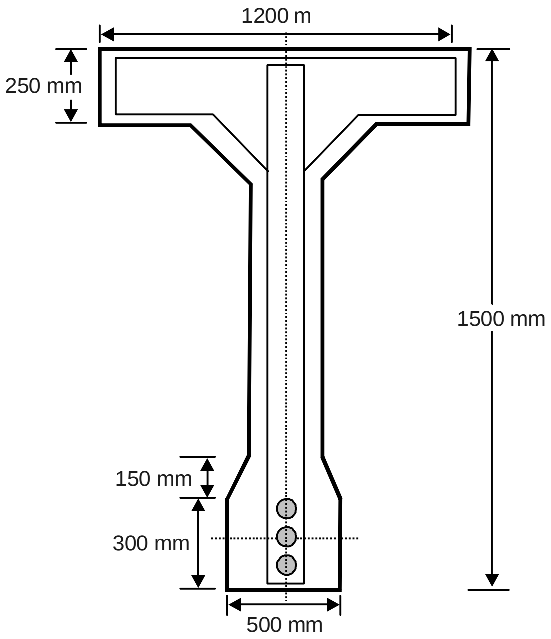

Fig. 15.27 Reinforcement at centre of span section.

Design of end block

End block is designed to distribute the concentrated prestressing force at the anchorage. It shall have sufficient area to accommodate anchorages at the jacking end and shall preferably be as wide as the narrowest flange of the beam. Length of end block in no case be less than 600 mm nor less than its width. Generally, end blocks are provided at supports for a length of 1.5 m. The bursting forces generated during the post-tensioning should be assessed on the basis of the ultimate strength. The bursting force, F_{bst} existing in an individual square end block located by symmetrically placed square anchorage or bearing plate, may be derived from Table 15.1.

P_k = force in each cable = 1905 kN, $2y_{po}$ = 225 mm, and $2y_o$ = 900 mm
The ratio (y_{po}/y_o) = 0.25, the bursting force = 0.23 × 1905 = 438.15 kN
Area of steel required to resist this tension is = $(438.15 \times 10^3)/(0.87 \times 415) = 1214 \text{ mm}^2$
Provide 12 mm bars at 100 mm c/c in the horizontal and transverse direction as shown in Fig. 15.28.

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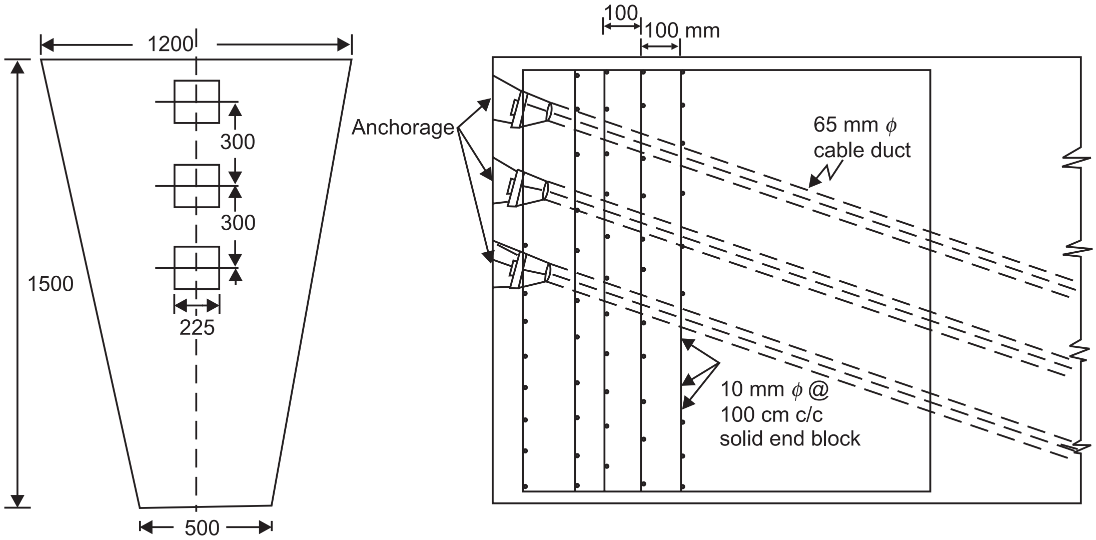

Fig. 15.28 Equivalent prisms and anchorage zone reinforcement.

DESIGN PROBLEMS

1. Design a prestressed concrete slab for the following data:

   Span (clear): 4.5 m
   Live load: IRC class 70R
   Road: National Highway
   Foot Path: 1 m on either side.
   Materials: M40 concrete and high strength steel.
   The compressive stress permissible in concrete during transfer: 16 MPa.

2. Design a post-tensioned prestressed concrete T-beam and slab bridge deck to suit the following data:

   Effective span: 28 m
   Road width: 7.5 m with kerbs 600 mm wide on either side.
   Loading: IRC class AA tracked vehicle.
   Materials: M50 concrete and high tensile steel confirming to IS:6006 and supplementary reinforcement comprising Fe-415 grade HYSD bars. Permissible stresses are as per IRC 18–2000. Loss ratio = 0.80.

Chapter 16

Artificial Intelligence in Bridge Engineering

16.1 INTRODUCTION

Artificial intelligence (AI) is a field of science and engineering concerned with the computational understanding of what is commonly called intelligent behaviour, and with the creation of artifacts (software, robots etc…) that exhibit such behaviour.

The field of artificial intelligence (AI) consists of long-standing intellectual and technological efforts addressing several interrelated scientific and practical aims, important ones are:

- Constructing intelligent machines, whether or not these operate in the same way as people do;
- Formalizing knowledge and mechanizing reasoning, both commonsense and refined expertise, in all areas of human endeavour;
- Using computational models to understand the psychology and behaviour of people, animals, and artificial agents;
- Making the working with computers as easy and helpful as working with skilled, cooperative, and possibly expert people.

AI has been around since the 1950s, and since then, it has fuelled many visions, dreams, and hopes. The details of realizing the promises included: a machine that will assist in its own programming (that is, a machine that learns), a machine that will be creative by evolving its programmes randomly through mutations (that is, achieving creativity by genetic programming), and the improvement in the hardware. In this Chapter, AI is seen simply as a collection of mainly symbolic computational techniques developed in AI research such as: heuristic search, constraint propagation, inductive learning that will match the functionality desired by the engineering task at hand.

AI applied to bridge engineering has developed into a large collection of studies specializing in solving various engineering subtasks. In a few cases, these studies were conducted closely with practitioners and in most others without such involvement. Even when

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the tasks addressed were significant, such as the conceptual design of bridges, the knowledge utilized was rather limited.

Bridge engineering is characterized by the following:

- For some tasks, there are no experts or there is significant historical knowledge that is available and therefore, rule based expert systems could not be coded.
- Some of the bridge engineering tasks are coupled with their solutions reqire diverse and intricate knowledge.
- Bridge engineering evolves continually by the introduction of new loading requirements, new materials, new structural forms, new quality construction methods, and other technologies such as smart or intelligent structures.

In order to delineate the role that AI techniques can play in bridge engineering, it is required to glance at engineering tasks related to bridges so that it is possible to identify the area that have the greatest benefit from AI techniques. The main tasks associated with bridge engineering are:

1. *Decision to commission:* The decision to commission or replace an old bridge draws upon many economical, political, and cultural issues. These issues determine much of the overall context in which a new bridge project is situated. By understanding them one can explain, for example, which kind of bridge suits a particular area.

2. *Design:* The design of a bridge is very complex. It starts with reconnaissance to find potential locations for the bridge together with some rough designs that need to pass some public approval. The actual design follows through a conceptual design stage in which aesthetic and economical issues play a major role, and follows through a detailed design stage. Designers need to exercise their judgment regarding many issues such as aesthetics, cost, the determination of loads and failure modes, the selection of appropriate models and analysis techniques, the interpretation of analysis results, constructability, and maintainability.

3. *Construction:* This stage involves a separate design in which the detailed design is recreated by the construction company. Included in this design are the selection and planning of the erection method together with the strength calculation of the bridge. Construction involves many planning activities at various levels and finally, it involves the actual erection of the bridge.

4. *Operation:* This is basically the function for which the bridge is built. It may involve determining the flow patterns of vehicles at different times, monitoring the weight of trucks or the transportation of hazardous materials.

5. *Maintenance:* This involves routine and on-demand inspection of the condition of the bridge that may be followed by decisions to perform rehabilitation procedures, limit the bridge loads, or even close the bridge. This activity involves considerable judgment of many issues and their potential effects on the state of the bridge and its remaining service life. Maintenance decisions of a particular bridge are dependent on the overall status of the bridge.

6. *Replacement:* The decision to replace an existing bridge is a part of maintenance activity. It initiates the loop back to the commissioning of the bridge. The replacement decisions

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must be carefully planned and even has to be designed in the early stages and should satisfy the environmental concerns.

The tasks listed above are mutually interactive. Isolating a task from the rest may result in misinformed solutions that are inferior to those that consider the information generated by other tasks. Figure 16.1 maps the information flow in bridge engineering at a certain level of abstraction. It is here that AI comes into picture. It is logical to expect practical benefit from AI techniques that are based on an overall life-cycle view of bridge engineering, which includes the interactions between different tasks.

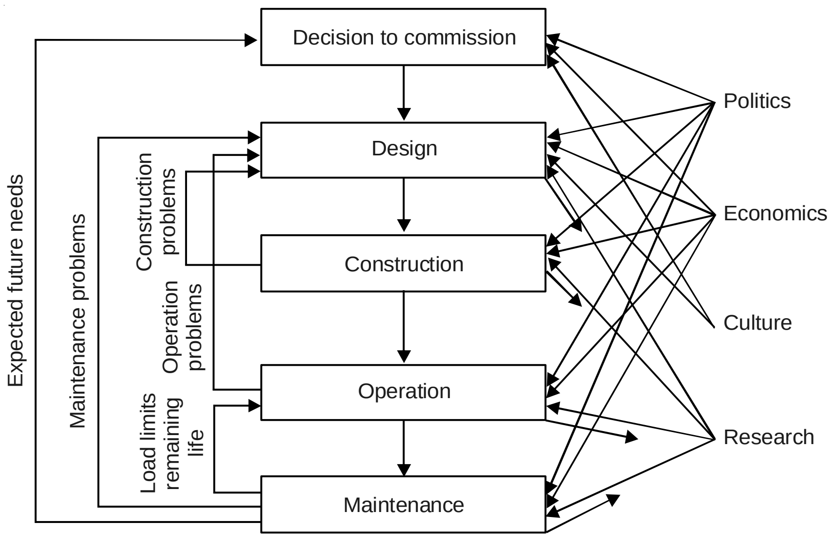

Fig. 16.1 The life cycle of information flow of a bridge (Reproduced from Yoram Reich).

16.2 AI RESEARCH IN BRIDGE ENGINEERING

Most of the reported works on AI techniques for bridge engineering has focused mainly on design (preliminary) and maintenance because of their relative importance in bridge engineering. Design has large impact on the success of any bridge and it is also considered to be the creative or artistic of all the tasks. This isolation permitted the use of AI techniques for attempting to solve this isolated task. However, the design applications have always drawn information generated by the other tasks. The second issue–maintenance has been pursued because it constitutes the most acute problem in bridge engineering.

16.2.1 Decision to Commission

There seems to be no work on the applications of AI techniques to this task. Nevertheless, there are numerous studies dealing with risk evaluation of large construction projects. The work cited refers to encoding of interdisciplinary knowledge on risk management for a large construction project into a system and using it over a long period of time by industry.

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16.2.2 Design

There have been several applications of AI techniques in bridge design. Starting from the conceptual design to the aesthetics of bridges many issues have been addressed.

In the conceptual design, the precise location of the bridge at the site is determined together with is form and major components. Some of the published research works covering this task are highlighted below:

- ➢ A commercial rule based expert system that takes care of preliminary design of bridges was implemented.
- ➢ Dynamic constraint satisfaction algorithm that can handle continuous variables that support early stages of preliminary design of bridges has been implemented.
- ➢ Using the knowledge of algorithmic procedure prevalent in Japan, minimum cost selection of superstructure and substructure of bridge was done.
- ➢ An expert system was developed to address precise site selection through the integration with a geographic information system.
- ➢ An expert system for the conceptual design of bridges was developed. The knowledge for the system was garnered from several experts.
- ➢ Preliminary design of bridges was modelled as a mapping from a large number of input parameters to a large number of outputs by a artificial neural network.
- ➢ An expert system for the preliminary design of cable-stayed bridges. This experimental system showed that learning from existing designs can result in knowledge that can be used to synthesize bridges of a similar type. The system was implemented using two machine-learning tools integrated with a detailed numerical analysis programme.
- ➢ An expert system was developed for selection of bridge superstructure.
- ➢ An expert system integrated with analysis tools was developed to design several types of trussed bridges. The expert rules were constructed by learning from simulations.
- ➢ A system was developed for designing highway bridges including their drawing production.
- ➢ Expert system was developed for the design of bridge abutments and piers.
- ➢ A rule and frame based shell was integrated with a finite element analysis programme for analyzing alternative designs. It also included visual scanned images for assisting its use in practice and education.
- ➢ A system was developed to design bridge foundation using commercial expert system tool.

16.2.3 Aesthetics

Bridge aesthetics is also an important issue because huge bridge structures can easily dominate the landscape. Several projects have addressed this issue.

- ➢ A learning algorithm was developed to learn the proportions between various bridge components as well as to encode overall aesthetics of the bridge using a case-based approach.

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- Based on the rules extracted from the experts, a system that can quantitatively evaluate the aesthetics of the bridges has been developed.
- A CAD tool that integrates knowledge and image processing capabilities has been developed. This package was used to assist the production of aesthetic bridge forms.

16.2.4 Analysis

The analysis of bridge is basically a quantitative procedure which is based on sound technical principles. However, the practical execution of the analysis involves expertise and heuristics. Several AI based applications have addressed this issue.

- A system that integrates heuristics extracted from many sources was developed to determine the dynamic response of highway bridges.
- A generic tool for building design critics was developed and used to implement a bridge design critic and a constructibility critic for reinforced concrete bridges.
- Several systems were developed for assisting designers in designing against fatigue.

16.2.5 Loads

Determination of loads is also a very important step in bridge design. Following applications have been developed.

- Seismic loading estimator was developed using object oriented programming language. Objects in this package associate with rules and procedures that encode the knowledge.
- Application of evolutionary algorithm based data mining approach to determine gross vehicle weights and vehicle velocities from bridge measurement data.

16.2.6 Planning

This stage involves determining the optimal construction schedules. Following are the reported applications of AI.

- Expert system that can interpret drawings and determines the construction work needed and schedule it.
- A system with augmented object oriented features developed with a panel of experts and potential users to eliminate the effects of errors in bridge fabrication.

16.2.7 Erection

This stage involves planning the erection process including construction of false work and making sure that the bridge sustains the loads during the erection. Following applications of AI have been reported:

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- ➢ A commercial system called welding advisor was developed by consulting firm and has been used on site for selection of appropriate welding procedure and diagnosing field welding defects. The system uses production rules implemented using commercial expert system and spreadsheet tools.
- ➢ Logic programming concept was used to develop a package that can select erection method for a particular bridge.

16.2.8 Monitoring

This task involves the continuous checking of the structure conditions. Following are the development of applications of AI in this area.

- ➢ A blackboard system was developed to monitor the bridge in-situ. This system contained knowledge about interpreting measured signals from radar sensors. A preliminary experiment with data collected suggested that the approach could be useful for the analysis of bridge deck deterioration.
- ➢ A system that can provide early warning against collapse. This system compares the dynamic response of the bridge due to normal loading to a predetermined bridge signature.
- ➢ A neural network model for in-situ dynamic analysis of bridges.
- ➢ An expert system was developed for monitoring of a cable-stayed bridge. The system can also detect faults in its instrumentation.
- ➢ A real time neural network system for monitoring single spanned bridges and reducing the mid span deflection.

Added to this, significant work has been done on AI applications to bridge maintenance including support for inspection, evaluation, retrofits, and scheduling activities.

16.3 AI AND RELATED TECHNIQUES

There are other computational techniques that need to be integrated together with AI. It is desirable that all these new tools need to be integrated together with existing tools used presently by practioners. The emerged tool should be able to address information modelling, information analysis and information communication. The information modelling facility should be used to model information originating from various data acquisition tools, such as sensors. The major technologies for supporting this facility are:

- Databases for storing product models and other information related to bridges.
- User interfaces.
- Multimedia technologies.

Some AI techniques that can play a supportive role will include:

- Knowledge representation techniques.

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- Knowledge acquisition tools.
- Machine learning techniques.

The information analysis facility is to facilitate the user to extract pieces of knowledge from the enormous amount of data. The AI related technologies that play a major role in this context are:

- Machine learning concepts and tools.
- Natural language processing for extracting information from textual information.

The information communication facilities rely on having compressed information into usable, comprehensible pieces of knowledge that can be transferred. These pieces can be in the form of sets of heuristic rules, relationships between parameters of design and bridge performance. Networking and distributed database technology can provide the means for people to communicate and share the information stored.

16.4 NEURAL NETWORKS FOR PREDICTION OF SCOUR DEPTH AROUND BRIDGE PIERS

The depth of scour is an important parameter for determining the minimum depth of foundations as it reduces the lateral capacity of the foundation. It is for this reason that extensive experimental investigation has been conducted in an attempt to understand the complex process of scour and to determine a method of predicting scour depth for various pier situations. To date, no generic formula has been developed that can be applied to all pier cases to determine the extent of scour that will develop.

The mechanism of flow around a pier structure is so complicated, that it is difficult to establish a general empirical model to provide accurate estimation for scour. Interestingly, each of the proposed empirical formula yields good results for a particular data set, an alternative approach, artificial neural network (ANN) has been extensively used to estimate the equilibrium and time dependent scour depth with numerous reliable database. Numerous ANN models, multi-layer perceptron using back propagation algorithm (MLP/BP) and radial basis function using orthogonal least-squares algorithm (RBF/OLS), Bayesian Neural Network (BNN) and Single Artificial Neural Network (SANN) were used.

The equilibrium scour depth was modelled as a function of five variables; flow depth, mean velocity, critical flow velocity, mean grain diameter and pier diameter. The time variation of scour depth was also modelled in terms of equilibrium scour depth, equilibrium scour time, scour time, mean flow velocity and critical flow velocity. In all the published works, the training and testing data were selected from the experiments and from valuable references.

Scour is a local phenomenon that takes place in the vicinity or around a structure (piers, piles, abutments etc.) in flowing water, due to modification of flow pattern, results in increase of local shear stress. This, in effect, dislodges the material on stream bed, results in local scour.

River flow past a pier or an abutment causes three-dimensional flow separations—a system of vortex pairs developed in separated flow. Between them, the primary vortex is more dominating, wraps round the pier in the form of horse shoe-vortex. The magnitude and strength of this vortex depends on the geometry of pier and the magnitude of approaching velocity.

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The estimation of correct depth of scour below the stream bed is very important, because it determines the depth of foundation. The phenomenon of bridge pier scour is of paramount concern to hydraulics engineering profession, because without this detailed knowledge, bridge failures can occur. As per National Bridge Register (NBR) of America, out of 577,000 bridges, more than 26,000 of them have been found to be scour-critical, due to erroneous prediction of scour depth during engineering design. In this context, Indo-Gangitic belt of the Indian sub-continent is interwoven with mighty rivers like Indus, Ganga, Brahamaputra and their innumerable tributaries. The alluvial is so deep that in some cases even up to a depth of 100 metres no rock strata is found. Moreover the river beds are highly errodable. In order to protect the bridge piers against scouring the foundations have to be taken very deep. In the bridge across river Ganga at Varanasi, the maximum depth of scour estimated was around 60 m.

Artificial Neural Network (ANN) models are attractive in the domain of estimation of local scour around bridge piers. This is due to their adoptive nature where learning by examples replaces or making functions in search of solutions. This architecture renders computational models more attractive in domains of very little or incomplete understanding of the problem to be solved but where broad training data base is accessible. From the literatures it appears that ANNs provide higher level of accuracy in solving a particular problem in comparison with experimental and theoretical results. ANN may therefore be a viable alternative in the prediction of local scour depth around bridge piers, provided reliable data base is available.

16.4.1 Factors Influencing the Local Scour Depth

Approach velocity

- ➢ Under clear water condition, the local scour depth increases with velocity (Ettema 1980) till it reaches a maximum value at critical velocity.
- ➢ When the approach velocity exceeds the threshold value, the problem becomes a live-bed problem and scour is about 10% less than the threshold depth (Laursen 1963).
- ➢ Meliville (1988) has shown that as the velocity exceeds the threshold value, the local scour depth first decreases and then increases.

Flow depth

- ➢ At shallow depths the local scour around piers increases with flow depth, but as the depth increases, the scour depth becomes independent of flow depth (Laursen 1966, Breuser et al. 1977, Ettema 1980).
- ➢ According to Malville (1988), the local scour depth is independent of depth of flow, so long as the two vortex rollers do not interfere with each other.
- ➢ Since the vortex strength increases with depth of flow may result in more scour depth.

Sediment size

- ➢ The maximum value of the clear water local scour depth in non-ripple forming sediments is unaffected by sediment size as long as the ratio of the obstruction to the median size of the sediments (D/d50) is greater than or equal to 50 (Ettema 1988).

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- ➢ However, d50 is limited to single size particle, but as d50/h increases the vortex strength also increases.

Pier shape

- ➢ Blunter the pier greater is the scour, when it faces the flow.
- ➢ Very little significance is attached to the shape on the down stream of the pier.

Pier size

- ➢ The relation between pier size and equilibrium scour depth are quite diverging. Keeping all the factors constant, the scour depth varies as Dβ, β varies from 0.5 to 1.

Angle of attack

- ➢ As the angle of attack increases so also the scour depth for all shapes expect cylindrical pier.

Constriction ratio

- ➢ The ratio of width of the flume to the size of the (B/D) influences the equilibrium scour depth. Shen et al. (1969) suggested that, for clear water experiments, the flume width should be at least eight times the diameter or the size of the pier. The same ratio for live-bed scour should be at least ten times the pier size (Chiew 1984).

Sediment grade

- ➢ Both the sediment grade and the equilibrium scour depth decreases with standard deviation of the particle size distribution (Ettema 1980; & Garde, 1995). This is due to the formation of armour layer at the base of the scour hole.

16.4.2 Estimation of Scour Depth

Equilibrium scour depth is the maximum scour depth that occurs under given conditions. Equilibrium scour depth around circular pier in a steady flow over a bed of uniform spherical and cohesionless sediment depends on: flow and sediment characteristics and pier geometry.

$$d_s = f(\rho, \gamma, U, y, \rho, d_{50}, \sigma_g, g, D, Sh, Al) \tag{16.1}$$

where

- ρ = fluid density,
  = Kinematic viscosity,
- U = mean flow velocity,
- Y = depth of flow,
  = sediment density,
- g = acceleration due gravity,
- D = pier width normal to the flow,
- Sh and Al = parameters describing the shape and alignment of the pier.

By dimensional analysis, the above equation transforms to

$$\frac{d_s}{D} = f\left(\frac{UD}{\gamma}, \frac{U^2}{gd_{50}}, \frac{y}{D}, \frac{\rho_s}{\rho}, \frac{d_{50}}{D}, \sigma_g, Sh, Al\right) \tag{16.2}$$

Such a flow phenomenon can be represented as

$$\frac{d_s}{d} = K_f, K_y, K_d, K_0, K_s, K_\alpha \tag{16.3}$$

where

- K_f = flow parameter,
- K_y = flow depth,
- k_d = sediment size,
- K_0 = sediment gradation,
- K_s and k_α = pier shape and alignment.

From the above equation it can be inferred that the scour around pier is a complex flow phenomenon and not amenable for a generic solution. It is for this reason that extensive experimental investigations have been conducted to understand the complex process of scour and to predict the depth of scour under different situations. Numerous empirical formulae have been derived to determine the extent of scour that will develop. In the following a summary of these formulae is presented.

Model studies were carried out on piers in connection with the Hardinge Bridge Works over river Ganga in India. They found that scour depth could be expressed as,

$$\frac{H_s}{D} = 2.32\left(\frac{q_1^{2/3}}{D}\right)^{0.78} \tag{16.4}$$

where q_1 is the discharge intensity in $m^3/s/m$.

A major disadvantage of this relation is the combination of undisturbed water depth and scour depth. The formula was later modified by using regime depth relations by researchers. Based on the analysis of scour data on 17 bridges in the Indo-Gangetic plains following formula was proposed.

$$D_{se} = 2H_m \tag{16.5}$$

H_m, known as Lacey's regime depth is given as,

$$H_m = 0.47\left(\frac{Q}{f}\right)^{1/3} \tag{16.6}$$

where,

D_{se} = scour depth measured below the water surface = H_s

H_m = Lacey's mean depth, equal to cross-sectional area divided by surface width

F = Lacey's silt factor = $1.76\sqrt{d_m}$

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d_m = mean size of the sediment in mm
Q = design flood discharge in m^3/s
h_s = scour depth measured below the ambient bed level and
h = depth of flow.

This is popularly known as 'Lacey-Inglis' method of estimating scour. This method is purely empirical in nature and gives combined scour caused due to flow modification by introduction of a pier, flow constriction due to guide bunds, and flow concentration due to non-uniform distribution of flow.

Laursen and Toch (1956) from the model studies for live-bed load scour at square nosed pier aligned with the flow, proposed a design curve (Reported by Neill, 1964) as:

$$\frac{h_s}{D} = 1.5\left(\frac{h}{D}\right)^{0.3} \quad (16.7)$$

This method does not take account of bed-material size and possible over-rates the influence of depth of flow. Laursen (1963) also suggested multiplying factors for various nose shapes.

Grade (1995) proposed a generalized relation for scour depth prediction as:

$$\frac{h_s}{h} = \left(\frac{4\eta_1\eta_2\eta_3}{\alpha}\right)F_r^{\,n} \quad (16.8)$$

in which F_r = Froude number = $\dfrac{U}{\sqrt{gh}}$

η_1, η_2, and η_3 are the functions of the particle drag coefficient C_D, Froude number F_r and foundation shape, respectively, and η is a function of C_D. α is the opening ratio, $\dfrac{B-D}{B}$, where B is the width of the channel.

This method was proposed for local scour at bridges crossing and applied to live bed conditions.

Laursen (1963) analyzed existing laboratory data and some field data and worked out the following formula for maximum local scour depth at a circular pier at the initiation condition of sediment motion.

$$h_{sm} = 1.05\, K_\theta K_s D^{3/4} \quad (16.9)$$

K_θ and K_s are factors accounting for angle of attack and pier shape respectively. He also proposed empirical values of K_θ and K_s for different conditions. The formula gives roughly the same answer as Laursen and Toch's (1956) equation above when the depth of flow is unity but the formula is not dimensionally homogenous.

Shen et al. (1969) proposed a clear water scour formula in terms of pier Reynolds number R_D on the basis of the vortex strength model, as

$$h_s = 0.000223(R_D)^{0.619} \quad (16.10)$$

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where

R_D = VD/v, and,
V = average velocity
D = diameter of pier
v = kinematic viscosity of water.

The formula is also referred to as Shen I formula in order to distinguish it from Shen II a (clear water scour), and Shen II b (live-bed scour) formulae.

Shen II a: $$\frac{h_s}{h} = 2F_r^{0.43}\left(\frac{D}{h}\right)^{0.645} \tag{16.11}$$

Shen II b: $$\frac{h_s}{h} = 3.4F_r^{2/3}\left(\frac{D}{h}\right)^{2/3} \tag{16.12}$$

It may be added that Shen et al's relations are on the conservative side and the plots show many scatters of data.

Following equations for local scour at piers were proposed:

For clear water scour:

$$\frac{h_s}{D} = 2.42\left(\frac{2U}{U_i} - 1\right)\left(\frac{U^2}{gD}\right)^{1/3} \tag{16.13}$$

For live bed scour:

$$\frac{h_{sm}}{D} = 2.42\left(\frac{U_i^2}{gD}\right)^{1/3} \tag{16.14}$$

where U_I = mean flow velocity at the initiation condition of sediment.

Coleman (1971) analysed the data of Shen et al. (1969) and the results of his own experiments on circular piers under conditions of continuous sediment transport and proposed the relation

$$\frac{h_{sm}}{D} = 1.49\left(\frac{U_i^2}{gD}\right)^{1/10} \tag{16.15}$$

The relation with minor changes, approximates to $H_{sm} = 1.4D$. Design relation for circular piers was proposed based on the notion that for live-bed scour and depth of flow greater than twice the size of the pier, the maximum local scour depth is a function of pier shape and size only, as

$$h_{sm} = 1.4D \tag{16.16}$$

The highly simple relation was obtained from laboratory and field data.

$$\frac{h_s}{D} = f\left(\frac{U}{U_i}\right) 2.0 \tanh\left(\frac{h}{D}\right) K_\theta K_s \tag{16.17}$$

in which,

$$f\left(\frac{U}{U_i}\right) = 0 \qquad \text{for } \frac{U}{U_i} < 0.5$$

$$= 2\frac{U}{U_i} - 1 \qquad \text{for } 0.5 \le \frac{U}{U_i} \le 1.0$$

$$= 1.0 \qquad \text{for } \frac{U}{U_i} > 1.0$$

The value of K_s was recommended as

K_s = 1.0, for circular rounded shape, K_s = 0.75, for streamlined shape, and K_s = 1.30, for rectangular shape. They also proposed the use of the Laursen charts for the angle of attack.

Based on the vortex strength approach, a scour depth relation for the clear water case is given by:

$$\frac{h_{sm}}{D} = (\alpha_1 Fr_d - \alpha_2) \tanh\left(\alpha_3 \frac{h}{D}\right) \tag{16.18}$$

in which
$$Fr_d = \frac{U}{\sqrt{sgd}}$$

$$\alpha_1, \alpha_2, \alpha_3 = f_n\left(d\frac{\sqrt{sgd}}{v}\right)$$

s = specific gravity of the submerged particles and
f_n = functional relation.

Jain (1981) analysed the available scour data and proposed the enveloping equation for the maximum clear water scour as:

$$\frac{h_{sm}}{D} = 1.84\left(\frac{h}{D}\right)^{0.3} (Fr_i)^{0.25} \tag{16.19}$$

where, $Fr_i = \dfrac{U_i}{\sqrt{gh}} \Rightarrow$ Froude number at the sediment initiation motion.

Structurally, the relation (4.19) is similar to Laursen and Toch (1956)'s equation with the effect of sediment size included.

On the basis of the vortex strength approach, following scour depth relation, was proposed.

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$$h_{sm} = 538(C_o)^{1.28} \quad (16.20)$$

in which, $C_o = U_o r_o$

where, $U_o = 0.092 D^{-0.5} U^{0.83}$ and $r_o = 0.1D$

Besides being dimensionally non-homogenous, the relation (4.20) is valid for fine sands and for clear water conditions only.

Kothyari et al. (1992) approximated shear stress at the bottom of scour hole by assuming that shear stress under the vortex is a function of the cross-sectional area of the vortex and that the vortex expanded to fit the scour hole and they gave the following procedure for finding the scour depth as a function of time:

- ➢ Compute τ_p, t probabilistic shear stress, and $d_s = 0$ with

$$\tau_p, t = 4.0\ \tau\mu \left[\frac{A_0}{A_1}\right] = \rho\mu^2 \quad (16.21)$$

- ➢ Calculate t_o, the time required to remove the particle, with $t_0 = \dfrac{C_1 d}{p_0 u^* j}$;
- ➢ Increment d_s by one grain diameter over time t_o;
- ➢ Increment the time by t_o, recomputed τ_p, t for the incremented d_s and repeat the procedure;
- ➢ Scour ends when $\tau_p, t \le \tau_c$

Garde et al. (1989) proposed the following relations for clear water and live-bed scour depths respectively

Clear water:

$$\frac{h_s}{D} = 0.66\left(\frac{D}{d}\right)^{0.75}\left(\frac{h}{d}\right)^{0.16}\left(\frac{U^2 - U_c{}^2}{sgd}\right)^{0.4}\alpha^{-0.3} \quad (16.22)$$

Live-bed scour:

$$\frac{h_s}{D} = 0.88\left(\frac{D}{d}\right)^{0.67}\left(\frac{h}{D}\right)^{0.4}\alpha^{-0.3} \quad (16.23)$$

In which U_c is the approach flow velocity at the initiation of scour and α is the opening ratio, $\left(\dfrac{B-D}{B}\right)$.

Relationship for estimation of design scour around circular bridge piers was proposed by Kothyari et al (1992) as:

$$\frac{h_{sm}}{d_m} = 1.22\left[\frac{b}{d_m}\right]^{0.67}\left(\frac{D}{d_m}\right)^{0.4}\left(\frac{B-b}{B}\right)^{0.3} \quad (16.24)$$

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where

h_{sm} = maximum scour depth below the general bed level of the river
b = pier diameter
B = centre to centre spacing of the piers
D = flow depth at HFL
U = flow velocity at HFL

Many relationships are available for determination of the design scour depth around circular piers in straight flows under live-bed scour conditions (Garde and Kothyari, 1989).

The process of local scour at bridge piers is also time dependant. Peak flood flow may lost only a number of hours or a few days in reality. According to Melville (1999), the relation between the depths of local scour at a bridge pier (d_s) at particular time t, in a flow can be written as:

$$d_s = f(\rho, \mu, U, y, g, d_{50,} U_c, t, t_e) \tag{16.25}$$

where t_e time for equilibrium depth of local scour to develop. Based on above equation Melville and Chiew (1999) presented the following formulale:

$$\frac{d_s}{d_{se}} = \varepsilon\alpha\rho\{-0.03\}\left[\frac{U_c}{U}\ \ln\left(\frac{t}{t_c}\right)\right]^{1.6} \tag{16.26}$$

From the above equation, the relationship between d_s and its dependent parameters can be written as:

$$d_s = f(d_{se}, U, U_c, t, t_c) \tag{16.27}$$

Further the non-dimensional form is:

$$\frac{d_s}{d_{se}} = f\left(\frac{U}{U_c}, \frac{t}{t_e}\right) \tag{16.28}$$

Ettema (1980) developed a model that was applicable only to the upstream of the scour hole. He found that the width of the erosion area at the bottom of the scour hole varied according to D/d, where D is the pier diameter and d the depth of water and equation for the time of scour was:

$$\frac{\partial\left(\frac{ds}{D}\right)}{\sigma t} = \frac{K_1 d}{K_2 K_4 D^3}\frac{\sqrt{1+\cot^2\phi}}{\sqrt{\cot\phi}} I_p \tag{16.29}$$

where k_1 is a volume constant, k_2 and k_4 are area constants and ϕ is the sediment submerged angle of repose.

Two different mathematical functions to fit local scour that is time dependent have been suggested:

$$d_s(t) = a\left[1 - \frac{1}{(1+abt)}\right] + c\left[1 - \frac{1}{(1+cdt)}\right] \tag{16.30}$$

$$d_s(t) = a\left[1 - e\alpha p(-bf)\right] + c\left[1 - e\alpha p(-dt)\right] \tag{16.31}$$

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where *a*, *b* and *c* are curve fitting coefficients and *t* is the time elapsed. The duration of time must be at least that used in the experiments in order to produce accurate results.

16.4.3 Artificial Neural Network (ANN)

Artificial Neural Network (ANN) is used in hydrology and water resources system modelling. An ANN mimics the biological neural network in two ways:

- ➢ Acquiring of knowledge via learning process, and
- ➢ Accumulation of knowledge through connections known as synaptic weights.

ANNs are especially useful as pattern-recognition tools for generalization of input and output mapping.

An ANN consists of interconnected neurons receiving one or more inputs and mapping an output. An ANN is trained to learn the relationship between the input and the output. To achieve training, a number of sets of inputs and outputs are fed to the ANN. A training algorithm achieves the training through optimal weights. Back Propagation (BP) and Radial Basis Function (RBF) are the most commonly used network-training algorithms in hydrologic and water resources engineering are briefly discussed here.

Back Propagation (BP)

Back propagation is widely used algorithm for training ANNs. This architecture is made up of interconnected nodes in at least three layers:

- ➢ The input layer accepts the input map.
- ➢ The output layer produces the result.
- ➢ The hidden layers sequentially transform the input into the output.

The transfer function used in BP network is sigmoid function. During training BP networks process the pattern in a two-step procedure. In the first or forward phase of learning an input pattern is fed to the network, with initially assigned weights to provide the output. At the output layer the error is estimated from the corresponding output pattern derived in training set. In the second or backward phase, the error from the output is propagated back through the network to adjust the interconnection weights between layers. The process is repeated until the output map is acceptable.

Radial Basis Function Networks (RBFN)

Radial Basis Neural Networks are feed-forward neural networks with only one hidden layer and *radial basis function* (RBF) as activation functions. The hidden layer performs nonlinear transformation with no adjustable parameters. The RBF networks are trained with modified form of gradient descent training.

The primary difference between the RBF and the BF is in the nature of nonlinearities associated with the hidden node. The nonlinearity in back propagation is implemented by a fixed function such as sigmoid. While RBF base its nonlinearities on the data in the training set (ASCE, 2000a).

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16.4.4 Application of ANNs in Prediction of Scour around Bridges

The prediction of depth of scour is an important parameter to decide the depth of foundation. It is for this reason that extensive experimental investigation are undertaken to understand the complex process of scour. Till date, no generic formula has been developed that can be applied to all pier environment to determine the extent of scour. As explained above, numerous empirical formulae have been developed to predict the scour, each varies significantly, highlighting the fact that there is lack of knowledge in estimating scour depth in obtaining a universal solution. This gap may be bridged through Artificial Neural Network Models.

ANN has been widely applied in various areas of hydrology and water resource engineering. ANN was used to predict the flow conditions when interfacial mixing in stratified estuaries commences. The neural network results were compared to the semi-theoretical solution based on a combination of results from flow theory, turbulent flow theory and interfacial friction experiments. Although neither of the two solutions was perfect in every respect, they were sufficiently close to one another. Engineers can now compute the critical velocity at which interfacial mixing commences at a particular location in a stratified estuary or fjord.

Different networks were developed to predict the scour depth based on the input parameters of wave height, wave period, water depth, pile diameter, Reynold's number, maximum wave particle velocity, maximum shear velocity, Shield's parameter and Keulegan-Carpenter number. The neural network was able to provide a better alternative to the statistical curve fitting with a weight matrix developed to predict non-dimensional scour depth from the input of wave height, wave period, water depth and pile diameter.

Researchers have investigated the performance of neural networks as potential models capable of forecasting daily stream flows. An appropriate model was identified and a comparison approach was used to evaluate it against a conceptual model presently in use. It was found that the neural networks outperform the deterministic model for up to 5-day-ahead forecasts. It was also found that the results obtained with the neural network were far superior to the ones obtained with the classic model.

In another attempt, an ANN model was used to estimate the natural sediment discharge in rivers in terms of sediment concentration. Several trials were done to design a suitable architecture of the network. The model was trained with measured field data of variables selected on the basis of fluid and sediment dynamics. Model verification was done with a large number of data from several rivers. The results indicated that a neural network approach estimates sediment concentration well compared to conventional methods.

It has been demonstrated that the feasibility of training an ANN for accurately predicting transient water levels in a complex multilayered ground-water system under variable state, pumping, and climate conditions. The ANN was trained to predict transient water levels in response to changing pumping and climate conditions. The trained ANN was validated with ten sequential seven-day periods and the results compared against both measured and numerically simulated ground-water levels. The results indicate that the ANN technology has the potential to serve as a powerful prediction and management tool for many types of ground-water problems.

ANN models are attractive in the area of estimation of local scour around bridge piers. This is because of their adaptive nature where learning by example replaces programming or

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making functions in solving problems. This feature renders computational models very appealing in domains, where one has little or incomplete understanding of the problem to be solved but where training data examples are available. It has been proved that ANN provided a higher level of accuracy in solving a particular problem when compared to experimental and theoretical results. ANN may therefore be a viable alternative in the estimation of local scour depth around bridge piers, provided a reliable database is available.

QUESTIONS

1. What is AI? List the useful applications of AI in bridge engineering.
2. What is ANN? How are they useful in bridge engineering?
3. Sketch the life cycle of information flow in bridge engineering.
4. List the various empirical formulae for scour depth around bridge piers.
5. Explain how the scour depth can be predicted using ANN.

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Appendix

Computer Programs for Structural Designs

Program 1. Program to design pipe culverts

This program is to be typed in Turbo-C or Turbo-C++ editor. The functions listed are to be typed and stored in separate files and included in the main program.

```
#include<conio.h>
#include<stdio.h>
#include<math.h>
#include“pipel.c”
#include“pipeal.c”
#include“pipebl.c”
#include“pipecl.c”
#include“pipe4.c”
void main()
{float A,Q,v,D,H,ad,a,tebs,ls,Ds,ss;
float sp,sf,wl,p;
int ltyp,np,iccs,isp,dl,ds;
double Cs,Ce,d;
clrscr();
printf(“Enter the value of discharge(in metric cube per second):”);
scanf(“%f ”,&Q);
printf(“Enter the value of velocity of water through pipes(in metres per second):”);
scanf(“%f”,&v);
A=Q/v;
Printf(“A=%f/n”,A);
Printf(“\nSelect the standard pipe internal diameter:\n”);
Printf(“\nDiameters available are given below:(in metres)”);
Printf(“\n________________________________________”);
Printf(“\n0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 ”);
Printf(“\n ________________________________________”);
do{
printf(“\nEnter the internal diameter selected:”);
scanf(“%f”,&d);
```


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```
if((d!=0.5)&&(d!=0.6)&&(d!=0.7)&&(d!=0.8)&&(d!=0.9)&&(d!=1.0)&&(d!=l.1)&&(d!=1.2))
{printf("\nSelected diameter is not standard one");
}}while(d!=0.5&&d!=0.6&&d!=0.7&&d!=0.8&&d!=0.9&&d!=1.0&&d!=l.l&&d!=l.2);
np=pipeal(A,d);
printf("\n Number of pipes:",np);
do
{
printf("\nEnter the type of loading for the culvert.");
printf("\nEnter 1 for Class AA tracked and 2 for Class A.");
    scanf("%d",&ltyp);
    if(ltyp!=l&&ltyp!=2)
    {printf("\nSorry, it is absurd.");}}while(ltyp!=l&&ltyp!=2);
    if(ltyp= =1) p=700;
    if(ltyp= =2) p=228;
printf("\nEnter the height of the embankment in metres.");
scanf("%f",&H);
Cs=pipel(H,d);
Printf("\nThe value of Influence Coefficient for Embankment height\n",H);
Printf("and diameter",d,"is:",Cs,"\n");
Printf("\nEnter the value of Coefficient of Earth pressure from the code.");
Scanf("%f",&Ce);
if(d= =0.5) {D=0.65;ls=2.78;ss=6.82;tebs=35.93;}
if(d= =0.6) {D=0.77;ls=3.18;ss=9.01;tebs=43.11;}
if(d= =0.7) {D=0.87;ls=3.18;ss=12.27;tebs=50.30;}
if(d= =0.8) {D=0.99;ls=4.66;ss=15.04;tebs=57.48;}
if(d= =0.9) {D=l.l0;ls=4.66;ss=20.30;tebs=64.67;}
if(d= =1.0) {D=l.23;ls=4.66;ss=23.52;tebs=71.85;}
if(d= =1.1) {D=l.33;ls=4.66;ss=29.99;tebs=79.00:}
if(d= =1.2) {D=l.44;ls=5.55;ss=35.57;tebs=86.22;}
printf("Outter Diameter\n",D);
printf("\n Enter the density of soil(in kN/m*m*m):");
scanf("%f",&wl);
sf=pipe4(Ce,wl,D,Cs,p,tebs);
printf("Factor of safety is\n",sf);
    if(sf<=1.0)
    printf("\nEarthen bedding is provided.");
    else
        if(sf>1.0&&sf<=2.7)
    printf("\nFirst class bedding is provided.\n");
        Ds=7850;
ad=(d+D)*0.5;printf("\nAverage Diameter\n",ad);
ds=pipeb();
iccs=pipebl(ss,ad,Ds,ds);
printf("\n C/C spacing of",ds,"mm spirals",iccs,"mm.\n");
dl=pipec();
isp=pipecl(ls,ad,Ds,dl);
printf("\n C/C spacing of",dl,"mm longitudinal steel",isp,"mm.");
    getch();
}
```

File 1. Pipe1.c

This function returns the interpolated value of coefficient Cs, depending on the value of the diameter and height of the embankment.

```
#include<stdio.h>
float Cs;
float pipel(float H,double d)
{
if(((H>=0.1)&&(d= =0.5))&&((H<=0.2)&&(d= =0.5))) Cs=0.246 – (H – 0.1)*0.18;
if(((H>=0.2)&&(d= =0.5))&&((H<=0.3)&&(d= =0.5))) Cs=0.228 – (H – 0.2)*0.3;
if(((H>=0.3)&&(d= =0.5))&&((H<=0.4)&&(d= =0.5))) Cs=0.198 – (H – 0.3)*0.29;
if(((H>=0.4)&&(d= =0.5))&&((H<=0.6)&&(d= =0.5))) Cs=0.169 – (H – 0.4)*0.26;
if(((H>=0.6)&&(d= =0.5))&&((H<=0.8)&&(d= =0.5))) Cs=0.117 – (H – 0.6)*0.17;
if(((H>=0.8)&&(d= =0.5))&&((H<=1.0)&&(d= =0.5))) Cs=0.083 – (H – 0.8)*0.115;
if(((H>=1.0)&&(d= =0.5))&&((H<=2.0)&&(d= =0.5))) Cs=0.060 – (H – 1.0)*0.0215;
if(((H>=2.0)&&(d= =0.5))&&((H<=3.3)&&(d= =0.5))) Cs=0.017 – (H – 2.0)*0.009;
if(((H>=3.0)&&(d= =0.5))&&((H<=4.0)&&(d= =0.5))) Cs=0.008 – (H – 3.0)*0.003;
if((H>4.0)&&(d= =0.5))                           Cs=0.005;
if(((H>=0.1)&&(d= =0.6))&&((H<=0.2)&&(d= =0.6))) Cs=0.247 – (H – 0.1)*0.13;
if(((H>=0.2)&&(d= =0.6))&&((H<=0.3)&&(d= =0.6))) Cs=0.234 – (H – 0.2)*0.24;
if(((H>=0.3)&&(d= =0.6))&&((H<=0.4)&&(d= =0.6))) Cs=0.210 – (H – 0.3)*0.28;
if(((H>=0.4)&&(d= =0.6))&&((H<=0.6)&&(d= =0.6))) Cs=0.182 – (H – 0.4)*0.51;
if(((H>=0.6)&&(d= =0.6))&&((H<=0.8)&&(d= =0.6))) Cs=0.131 – (H – 0.6)*0.185;
if(((H>=0.8)&&(d= =0.6))&&((H<=1.0)&&(d= =0.6))) Cs=0.094 – (H – 0.8)*0.13;
if(((H>=1.0)&&(d= =0.6))&&((H<=2.0)&&(d= =0.6))) Cs=0.068 – (H – 1.0)*0.046;
if(((H>=2.0)&&(d= =0.6))&&((H<=3.0)&&(d= =0.6))) Cs=0.022 – (H – 2.0)*0.012;
if(((H>=3.0)&&(d= =0.6))&&((H<=4.0)&&(d = =0.6))) Cs=0.010 – (H – 3.0)*0.004;
if((H>4.0)&&(d= =0.6))                           Cs=0.006;
if(((H>=0.1)&&(d= =0.7))&&((H<=0.2)&&(d= =0.7))) Cs=0.247 – (H – 0.1)*0.11;
if(((H>=0.2)&&(d= =0.7))&&((H<=0.3)&&(d= =0.7))) Cs=0.236 – (H – 0.2)*0.21;
if(((H>=0.3)&&(d= =0.7))&&((H<=0.4)&&(d= =0.7))) Cs=0.215 – (H – 0.3)* 0.29;
if(((H>=0.4)&&(d= =0.7))&&((H<=0.6)&&(d= =0.7))) Cs=0.186 – (H – 0.4)*0.23;
if(((H>=0.6)&&(d= =0.7))&&((H<=0.8)&&(d= =0.7))) Cs=0.140 – (H – 0.6)*0.19;
if(((H>=0.8)&&(d= =0.7))&&((H<=1.0)&&(d= =0.7))) Cs=0.102 – (H – 0.8)*0.135;
if(((H>=1.0)&&(d= =0.7))&&((H<=2.0)&&(d= =0.7))) Cs=0.075 – (H – 1.0)*0.051;
if(((H>=2.0)&&(d= =0.7))&&((H<=3.0)&&(d= =0.7))) Cs=0.024 – (H – 2.0)*0.014;
if(((H>=3.0)&&(d= =0.7))&&((H<=4.0)&&(d= =0.7))) Cs=0.010 – (H – 3.0)*0.004;
if((H>4.0)&&(d= =0.7))                           Cs=0.006;
if(((H>=0.1)&&(d= =0.8))&&((H<=0.2)&&(d= =0.8))) Cs=0.249 – (H – 0.1)*0.09;
if(((H>=0.2)&&(d= =0.8))&&((H<=0.3)&&(d= =0.8))) Cs=0.240 – (H – 0.2)*0.20;
if(((H>=0.3)&&(d= =0.8))&&((H<=0.4)&&(d= =0.8))) Cs=0.220 – (H – 0.3)*0.24;
if(((H>=0.4)&&(d= =0.8))&&((H<=0.6)&&(d= =0.8))) Cs=0.196 – (H – 0.4)*0.235;
if(((H>=0.6)&&(d= =0.8))&&((H<=0.8)&&(d= =0.8))) Cs=0.149 – (H – 0.6)*0.195;
if(((H>=0.8)&&(d= =0.8))&&((H<=1.0)&&(d= =0.8))) Cs=0.110 – (H – 0.8)*0.135;
if(((H>=1.0)&&(d= =0.8))&&((H<=2.0)&&(d= =0.8))) Cs=0.083 – (H – 1.0)*0.056;
if(((H>=2.0)&&(d= =0.8))&&((H<=3.0)&&(d= =0.8))) Cs=0.027 – (H – 2.0)*0.014;
```


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```
if(((H>=3.0)&&(d= =0.8))&&((H<=4.0)&&(d= =0.8))) Cs=0.013 – (H – 3.0)*0.006;
if((H>4.0)&&(d= =0.8))                           Cs=0.007;
if(((H>=0.1)&&(d= =0.9))&&((H<=0.2)&&(d= =0.9))) Cs=0.249 – (H – 0.1)*0.08;
if(((H>=0.2)&&(d= =0.9))&&((H<=0.3)&&(d= =0.9))) Cs=0.241 – (H – 0.2)*0.16;
if(((H>=0.3)&&(d= =0.9))&&((H<=0.4)&&(d= =0.9))) Cs=0.225 – (H – 0.3)*0.23;
if(((H>=0.4)&&(d= =0.9))&&((H<=0.6)&&(d= =0.9))) Cs=0.202 – (H – 0.4)*0.23;
if(((H>=0.6)&&(d= =0.9))&&((H<=0.8)&&(d= =0.9))) Cs=0.156 – (H – 0.6)*0.195;
if(((H>=0.8)&&(d= =0.9))&&((H<=1.0)&&(d= =0.9))) Cs=0.117 – (H – 0.8)*0.14;
if(((H>=1.0)&&(d= =0.9))&&((H<=2.0)&&(d= =0.9))) Cs=0.089 – (H – 1.0)*0.06;
if(((H>=2.0)&&(d= =0.9))&&((H<=3.0)&&(d= =0.9))) Cs=0.029 – (H – 2.0)*0.015;
if(((H>=3.0)&&(d= =0.9))&&((H<=4.0)&&(d= =0.9))) Cs=0.014 – (H – 3.0)*0.006;
if((H>4.0)&&(d= =0.9))                           Cs=0.008;
if(((H>=0.1)&&(d= =1.0))&&((H<=0.2)&&(d= =1.0))) Cs=0.249 – (H –0.1)*0.07;
if(((H>=0.2)&&(d= =1.0))&&((H<=0.3)&&(d= =1.0))) Cs=0.242 – (H – 0.2)*0.14;
if(((H>=0.3)&&(d= =1.0))&&((H<=0.4)&&(d= =1.0))) Cs=0.228 – (H – 0.3)*0.23;
if(((H>=0.4)&&(d= =1.0))&&((H<=0.6)&&(d= =1.0))) Cs=0.205 – (H – 0.4)*0.215;
if(((H>=0.6)&&(d= =1.0))&&((H<=0.8)&&(d= =1.0))) Cs=0.162 – (H – 0.6)*0.195;
if(((H>=0.8)&&(d= =1.0))&&((H<=1.0)&&(d= =1.0))) Cs=0.123 – (H – 0.8)*0.14;
if(((H>=1.0)&&(d= =1.0))&&((H<=2.0)&&(d= =1.0))) Cs=0.095 – (H – 1.0)*0.063;
if(((H>=2.0)&&(d= =1.0))&&((H<=3.0)&&(d= =1.0))) Cs=0.032 – (H – 2.0)*0.017;
if(((H>=3.0)&&(d= =1.0))&&((H<=4.0)&&(d= =1.0))) Cs=0.015 – (H – 3.0)*0.005;
if((H>4.0)&&(d= =1.0))                           Cs=0.010;
if(((H>=0.1)&&(d= =1.1))&&((H<=0.2)&&(d= =1.1))) Cs=0.249 – (H – 0.1)*0.07;
if(((H>=0.2)&&(d= =1.1))&&((H<=0.3)&&(d= =1.1))) Cs=0.242 – (H – 0.2)*0.14;
if(((H>=0.3)&&(d= =1.1))&&((H<=0.4)&&(d= =1.1))) Cs=0.228 – (H – 0.3)*0.23;
if(((H>=0.4)&&(d= =1.1))&&((H<=0.6)&&(d= =1.1))) Cs=0.205 – (H – 0.4)*0.215;
if(((H>=0.6)&&(d= =1.1))&&((H<=0.8)&&(d= =1.1))) Cs=0.162 – (H – 0.6)*0.195;
if(((H>=0.8)&&(d= =1.1))&&((H<=1.0)&&(d= =1.1))) Cs=0.123 – (H – 0.8)*0.14;
if(((H>=1.0)&&(d= =1.1))&&((H<=2.0)&&(d= =1.1))) Cs=0.095 – (H – 1.0)*0.063;
if(((H>=2.0)&&(d= =1.1))&&((H<=3.0)&&(d= =1.1))) Cs=0.032 – (H – 2.0)*0.017;
if(((H>=3.0)&&(d= =1.1))&&((H<=4.0)&&(d= =1.1))) Cs=0.015 – (H – 3.0)*0.005;
if((H>4.0)&&(d= =1.1))                           Cs=0.010:
if(((H>=0.1)&&(d= =1.2))&&((H<=0.2)&&(d= =1.2))) Cs=0.249 – (H – 0.1)*0.07;
if(((H>=0.2)&&(d= =1.2))&&((H<=0.3)&&(d= =1.2))) Cs=0.242 – (H – 0.2)*0.12;
if(((H>=0.3)&&(d= =1.2))&&((H<=0.4)&&(d= =1.2))) Cs=0.230 – (H – 0.3)*0.21;
if(((H>=0.4)&&(d= =1.2))&&((H<=0.6)&&(d= =1.2))) Cs=0.209 – (H – 0.4)*0.19;
if(((H>=0.6)&&(d= =1.2))&&((H<=0.8)&&(d= =1.2))) Cs=0.171 – (H – 0.6)*0.2;
if(((H>=0.8)&&(d= =1.2))&&((H<=1.0)&&(d= =1.2))) Cs=0.131 – (H – 0.8)*0.135;
if(((H>=1.0)&&(d= =1.2))&&((H<=2.0)&&(d= =1.2))) Cs=0.l04 – (H – 1.0)*0.068;
if(((H>=2.0)&&(d= =1.2))&&((H<=3.0)&&(d= =1.2))) Cs=0.036 – (H – 2.0)*0.016;
if(((H>=3.0)&&(d= =1.2))&&((H<=4.0)&&(d= =1.2))) Cs=0.020 – (H – 3.0)*0.009;
if((H>4.0)&&(d= =1.2))                           Cs=0.011;
return(Cs);}
```


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File 2. Pipe4.c

This function calculates the safety factor.

```
float sf;
float pipe4(float Ce,float w1,float D,float Cs,float p,float tebs)
{float w,wl,f;
w=Ce*wl*D*D,
wl=6*Cs*p;
f=(tebs/1.5) – (wl/1.5);
sf=f/w;
sf=abs(sf);
return(sf);}
```

File 3. Pipea1.c

This function returns the number of pipes required for the pipe culvert.

```
#include<stdio.h>
int np;
int pipeal(float A,float d)
{ float a;
a=3.142*d*d*0.25;
np=A/a;
np=int(np)+1;
return(np);}
```

File 4. PipeB.c

This function evaluates the spacing of spiral reinforcement.

```
#include<stdio.h>
int ds;
int pipeb()
{
do
{printf("Enter the diameter of mild steel to be used as spiral reinforcement.");
scanf("%f",&ds);
if(ds< 8||ds>12)
{printf("range is 8 mm to 12 mm.";}}while(ds<8||ds>12);
return(ds); }
```

File 5. Pipeb1.c

This function evaluates the spacing of spiral reinforcement.

```
#include<stdio.h>
#include"pipeb.cpp"
int iccs;
float pipebl(float ss,float ad,float Ds,float ds)
{
float ccs,ns,wel;
wel=3.142*(ds/1000)*(ds/1000)*0.25*Ds;
ns=ss/(3.142*ad*wel);
```


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```
ccs=1000/ns;
iccs=int(ccs);
return(iccs);}
```

File 6. Pipec.c

This function evaluates the diameter of longitudinal steel

```
#include<stdio.h>
int dl;
int pipec()
{
do
{printf("Enter the diameter of mild steel to be used as longitudinal reinforcement.\n");
    scanf("%d",&dl);
    if(dl< 6||dl>8)
    {printf("range is 6 mm to 8 mm.");}}while(dl<6||dl>8);
returm(d1);}
```

File 7. Pipec1.c

This function calculates the spacing of longitudinal reinforcement.

```
#include<stdio.h>
#include"pipec.cpp"
int isp;
float pipecl(float ls,float ad,float Ds,float dl)
{float we2,n,sp;
we2=3.142*(dl/l000)*(dl/l000)*0.25*Ds;
n=ls/(3.142*ad*we2);
    sp=1000/n;
    isp=int(sp);
return(isp);}
```

Program 2. Program to design deck slab culverts

Type this program and save this as the main program segment with file name Slab.cpp or stab.c. Supplementary files can then be opened with the names mentioned herein.

```
#include<stdio.h>
#include<conio.h>
#include<math.h>
#include<dos.h>
#include<process.h>
#include"livmoaa.c"
#include"livshaa.c"
#include"livmoaw.c"
#include"design3.c"
#include"livshaw.c"
#include"areal.c"
#include"area3.c"
#include"shear.c"
float l,td,b,fp;
```


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```
void main ()
    {float clsp,de,cc,be,11,12,dw,dc,dwc,wd,md,vd;
    double imf,c,dr,Ast,sm,Md,ded,sd,tovc,M,V;
    int ltyp,dm,dd,ds,grc,nl:
    float Astd,toc,tov,y,ml,vl;
clrscr();
do{
printf("\nEnter clear span(Support to support distance in metres):");
scanf("%f",&clsp);
if(clsp>8.0||clsp<1.0)
{printf("Sorry this span is not economical.\n");

printf("Try again\n");
getch();}}while(clsp>8.0||clsp<1.0);
printf("\nEnter the grade of concrete:(M20-M35)");
scanf("%d",&grc);
td=80.0*clsp;
printf("\nTotal depth of the slab=%f",td,"mm\n");
do{
do{
printf("\nEnter the diameter of steel for main reinforcement in mm:");
scanf("%d",&dm);
if(dm<20||dm>36)
{printf("diameter range is 20 mm to 36 mm\n");
    getch();}} while(dm<20||dm>36);
    cc=30.0;
de=td-(dm/2+cc);
de=de/1000;
printf("\nEffective depth of slab=%f",de);
gctch();
if(clsp<3.0)
be=0.3;
if(clsp>=3.0&&clsp<=8.0)
be=0.4;
11=clsp+de;
12=clsp+be;

1=(11<12)?11:12;
printf("\nEffective span of the slab:%f",1);
getch();
dw=0.08;
dc=24.0;
dwc=22.0;
wd=((td*dc)/l000.0)+(dw*dwc);
printf("\ndead load :%f".wd);
md=wd*1*1/8;
printf("\nDead load bending moment:%f",md,"kN·m");
vd=wd*1/2;
printf("\nDead load shear force:%f",vd,"kN");
getch();
c=7.5;
```

```
do {
printf("\nEnter the width of the footpath(in metres):");
scanf("%f",&fp);
if(fp<0.6||fp>l.5)
{printf("\nFootpath width range is 0.6 m to1.5 m");}}while(fp<0.6||fp>1.5);
b=c+(2.0*fp);
do{
printf("\nEnter the type of IRC loading you prefer to use:\n");
printf("\nEnter 1 for IRC-AA,2 for IRC-A:");
scanf("%d",&ltyp);

if(ltyp!=1 && ltyp!=2)
{printf("\nSorry,it is absurd.\n");}}while(ltyp!=1&&ltyp!=2);
if(ltyp= =1)
{ml=livmoaa(1,td,b,fp); printf("\nLive load bending moment is,%f",ml,"kN·m");
vl=livshaa(i,td,b,fp);
printf("\nLive load shear force is,%f",vl,"kN");}
else
   if(ltyp= =2)
{ml=livmoaw(l,td,b,fp);
printf("\nLive load bending moment is:%f",ml,"kN·m");
vl=livshaw(l,td,b,fp);
printf("\nLive load shear force:%f",vl,"kN");}
j=design(grc);
Q=design2();
getch();
M=ml+md;
printf("\nDesign bending moment is,%f",M,"kN·m");
V=vl+vd;
printf("\nTotal design shear is,%f",V,"kN");
dr= sqrt((M*1000000.0)/(Q*1000.0)); dr=dr/1000;
printf("\nDepth required is,%f",dr); getch();
if(dr>de)
{printf("\nRedesign the section by increasing the total depth of the slab by 50 mm.");getch();
td=td+50;}}while(dr>de);

if(dr<de)
printf("\nDepth of the section is,%f",de);
{Ast=asteel(de,M,j);
sm=spmsteel(dm);
printf("\nArea of main steel reinforcement is,%f",Ast);
printf("\nSpacing per metre is,%f",sm,"mm");}
do
{printf("\nEnter the diameter of distribution steel");
scanf("%d",&dd);
if(dd<8.0||dd>20.0)
printf("\nDiameter range is 8 mm to 20 mm."); getch();
}while(dd<8||dd>20);
Astd=adsteel(ml,md,de,j,dm,dd);
sd=sdsteel(dd);
printf("\nArea of distribution steel=%f", Astd);
```


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```
printf("\nSpacing of distribution steel per metre is,%f",sd,"mm");
toc=design3(grc);
tovc=shear(de,Ast,toc);
tov=(V*1000)/(de*1000*1000);
if(tov<tovc)
{printf("\nShear stress is within the permissible limits.");}
getch();}
```

The following files are to be included.

File 1. Livmoaa.c

This file contains a function which calculates the live load bending moment for Class AA and returns the same to the main function.

```
#include<stdio.h>
float livmoaa(float 1,float td,float b,float fp)
{float le,be,k,wlm,imf,ml;
float bw,tbm,wim,r,x;
r=b/l;

if(r<=0.1)                    k=0.40;
if((r>=0.1) && (r<=0.2)) k=(r–0.1)*4.0+0.4;
if((r>=0.2) && (r<=0.3)) k=(r–0.2)*3.6+0.8;
if((r>=0.3) && (r<=0.4)) k=(r–0.3)*3.2+1.16;
if((r>=0.4) && (r<=0.6)) k=(r–0.4)*2.4+1.48;
if((r>=0.6) && (r<=0.7)) k=(r–0.6)*1.6+1.96;
if((r>=0.7) && (r<=1.1)) k=(r–0.7)*1.2+2.12;
if((r>=1.1) && (r<=1.2)) k=(r–1.1)*0.4+2.60;
if((r>=1.2) && (r<=1.4)) k=(r–1.2)*0.8+2.64;
if((r>=1.4) && (r<=1.9)) k=(r–1.4)*0.4+2.80;
if(r>=l.9)                    k=3.0;
if(1<=5.0) imf=0.25;
else
imf=25–(3.75*(1–5)); imf/=100;

td=td/1000;
le=3.6+2*(td+0.08);
if(le>l)l=le;
bw=(0.85+2.0*0.08);
x=1/2;
be=(k*x*(1–x/l)+bw;
tbm=fp+1.2+0.425+2.05+be/2;
if(tbm>b)
tbm=b;
wim=700*(1+imf);
wlm=wim/(le*tbm);
ml=((wlm*le)/2*x)–(((wlm*le)/2)*le/4);
return (ml);}
```


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File 2. 1ivshaw.c

This file contains a function which calculates the live load shear for Class A load and returns the same to the main function.

```
#include<stdio.h>
float livshaw(float 1,float td, float b,float fp)
{float le,tle,k,wls,bw,tbs,wis,r,x,imf,vl,be;
le=0.25+2*(td/l000+0.08);
tle=le/2+1.2;
x=le/2;
bw=0.5+2*0.08;
r=b/l;
if(r<=0.l)    k=0.40;
if((r>=0.1)&&(r<=0.2)) k=(r–0.1)*4.0+0.4;
if((r>=0.2)&&(r<=0.3)) k=(r–0.2)*3.6+0.8;
if((r>=0.3)&&(r<=0.4)) k=(r–0.3)*3.2+1.16;
if((r>=0.4)&&(r<=0.6)) k=(r–0.4)*2.4+1.48;
if((r>=0.6)&&(r<=0.7)) k=(r–0.6)*1.6+1.96;
if((r>=0.7)&&(r<=1.1)) k=(r–0.7)*1.2+2.12;
if((r>=1.1)&&(r<=1.2)) k=(r–1.1)*0.4+2.60;
if((r>=1.2)&&(r<=1.4)) k=(r–1.2)*0.8+2.64;
if((r>=1.4)&&(1<=1.9)) k=(r–1.4)*0.4+2.80;
if(r>=1.9) k=3.0;
be=k*x*(l–x/l)+bw;
tbs=fp+0.15+0.25+1.8+1.7+1.8+be/2;
if(tbs>b)
tbs=b;
imf=4.5/(6.0+1);
wis=4*114*(l+imf):
wls=wis/(tle*tbs);
vl=(wls*tle*(l–tle/2))/l;
return (vl);
}
```

File 3. 1ivmoaw.c

This file contains a function which calculates the live load bending moment for IRC Class A loading and returns the same to the main function.

```
#include<stdio.h>
float livmoaw(float 1,float td,float b,float fp)
{
float imf,le,bw,r,x,be,tbm,wim;
float wlm,k,tle,re,ml;
imf=(4.5/(6+1));
le=0.25+2*(td/1000+0.08);
tle=le+l.2;
bw=(0.5+2*0.08);
r=b/l;
```

```
if((r>=0.1) && (r<=0.2)) k=(r–0.1)*4+0.4;
if((r>=0.2) && (r<=0.3)) k=(r–0.2)*3.6+0.8;
if((r>=0.3) && (r<=0.4)) k=(r–0.3)*3.2+1.16;
if((r>=0.4) && (r<=0.6)) k=(r–0.4)*2.4+1.48;
if((r>=0.6) && (r<=0.7)) k=(r–0.6)*1.6+1.96;
if((r>=0.7) && (r<=1.1)) k=(r–0.7)*1.2+2.12;
if((r>=1.1) && (r<=1.2)) k=(r–1.1)*0.4+2.60;
if((r>=1.2) && (r<=1.4)) k=(r–1.2)*0.8+2.64;
if((r>=1.4) && (r<=1.9)) k=(r–1.4)*0.4+2.8;
if(r>=1.9)               k=3.0;
x=((l*0.5) – 0.6);
be=k*x*(l–x/l)+bw;
tbm=fp+0.l5+0.25+1.8+1.7+1.8+be/2;
wim=4*114*(l+imf);
wlm=wim/(tle*tbm);
re=wlm*tle/2;
ml=(re*l/2) – (wlm*tle*0.5*tle*0.25);
return(m1);)
```

File 4. Design 3.cpp

This function returns the basic shear values listed in IRC-21 for different grades of concrete.

```
#include<stdio.h>
#include "design2.c"
float toc;
float design3(int grc)
{
        if(grc= =20)
            toc=0.34;
    else
        if(grc= =25)
            toc=0.40;
    else
        if(grc= =30) toc=0.45;
    else
        if(grc= =35)
            toc=0.50;
            return(toc);}
```

File 5. area3.c

This file contains a function which returns the spacing distance of distribution steel to the main function.

```
#include<stdio.h>
#include"area2.c"
float sdsteel(int dd)
{float sd,astd;
astd=(3.14*dd*dd)/4;
```


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```
sd=(astd/Astd)*1000.0;
sd=int(sd);
if(sd>200.0)
sd=200.0;
return(sd);}
```

File 6. shear.c

This file contains a function which returns the permissible shear stress value to the main function.

```
#include<stdio.h>
#include<process.h>
float tovc;
float shear(float de, float Ast, float toc)
{
float p,k1,k2,tovc;
p=(l00*Ast)/(de*1000000);
kl=1.14–(0.7*de):
if(k1<0.5)
k1=0.5;
K2=(0.5+0.25*p);
if(k2<1)
k2=1;
tovc=k1*k2*toc;return(tovc);}
```

File 7. 1ivshaa.c

This file contains a function which calculates the live load shear force for Class AA loading and returns the same to the main function.

```
#include<stdio.h>
float livshaa(float 1,float td,float b,float fp)
{float wlv,wiv,tbs,be,x,le,k,r,bw,imf,vl;
r=2;
if(r>=0.1)                     k=0.40;
if((r>= 0.1) &&(r<=0.2)) k=(r–0.1)*4.0+0.4;
if((r>= 0.2) &&(r<=0.3)) k=(r–0.2)*3.6+0.8;
if((r>= 0.3) &&(r<=0.4)) k=(r–0.3)*3.2+1.16;
if((r>= 0.4) &&(r<=0.6)) k=(r–0.4)*2.4+1.48;
if((r>= 0.6) &&(r<=0.7)) k=(r–0.6)*1.6+1.96;
if((r>= 0.7) &&(r<=1.1)) k=(r–0.7)*1.2+2.12;
if((r>= 1.1) &&(r<=1.2)) k=(r–l.l)*0.4+2.60;
if((r>= 1.2) &&(r<=1.4)) k=(r–1.2)*0.8+2.64;
if((r>= 1.4) &&(r<=1.9)) k=(r–1.4)*0.4+2.80;
if(r>1.9)                      k=3.0;
if(l<=5.0) imf=0.25;
else
imf=25–(3.75*(1–5));
imf/=100;
```


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```
td=td/1000;
le=3.6+2*(td+0.08);
if(le>l)le=l;
bw=0.85+(2*0.08);
x=le/2.0;
be=(k*x*(l–x/l))+bw;
tbs=fp+l.2+0.425+2.05+be/2;
if(tbs>b)
tbs=b;
wiv=700*(l+imf);
wlv=wiv/(le*tbs);
vl=(wlv*le*(l–x))/l;
return (vl);}
```

File 8. area1.c

This file contains a function which determines the spacing of main steel and returns the same to the main function.

```
#include<stdio.h>
#include "area.c"
float spmsteel(int dm)
{
   float ast,sm;
ast=3.14*dm*dm*0.25;
sm=(ast/AST)*1000;
sm=int(sm);
if(sm>l50.0)
sm=150.0;
return(sm);}
```

File 9. area.c

This file contains a function which calculates the area of main reinforcement and returns the same to the main function.

```
#include<stdio.h>
float ast;
float asteel(float de,float m,float j)
{ float sigst=200.00;
ast=(m*l000)/(sigst*de*j);
return(ast);}
```

Program 3. Program to design box culverts

```
#include<stdio.h>
#include<dos.h>
#include<math.h>
#include<conio.h>
#include<process.h>
#include<string.h>
```

```
#include<boxlive.c>
#include"boxcasel.c"
#include"boxcase2.c"
#include"boxcase3.c"
#include"design4.c"
#include"areaia.c"
#include"arealb.c"
#include"area3a.c"
#include"1.c"
#include"bo2.c"
#include"1b.c"
#include"lc.c"
#include"el.c"
#include"e2.c"
#include"e3.c"

void main(){
clrscr();

float l,h,d,dl,ll,tw,dc=25000,sw,pv,lts,wsw,reb,ka,lpuws,clsp,Dvw2,lpt, lpb;
double fi,R11,R12,he,hew,uws,em2[4],em3[4],fem[4],d[4]:
float Kts,Kbs,Kvw,ha l,hd l,fbmpt,nbmtl,fbmpb,nbmbl,nbml,ssbmv2;
float nbmt2,nbmb2,ha2,nbm2,nbmt3,nbmb3,ha3,nbm3,hd2,hd3,M,M1,de,dr;
int grc,dm,dd;
float b=l000,sm,sd,M2,Ms,Aas,sas,Astd,Mbl,Mb,Astb,Astlb,M2b,Msb,Aasb,sasb;
float smb,vet,veb,spe,m,sigst,ce,cc=30;
do{
printf("Enter the span of the box culvert (in metres):");
scanf("%f",&clsp);
if(clsp>4.0)
{printf("\nSpan is not economical.");
printf("Please try again.\n");getch();}} while(clsp>4.0);
do{
printf("Enter the height of the vent of the box culvert (in metres):"); scanf("%f",&h);
if(h>4.0)
{printf("\nHeight is not economical.");
printf("P1ease try again.\n");getch();}} while(h>4.0);
do{
printf("\nEnter the angle of repose in radians:");scanf("%f",&fi);
if(fi<0.523||fi>0.785)
printf("\nAngle of repose between 0.523 to 0.785 is desirable.");
getch();}while(fi<0.523||fi>0.785);
ka=(l-sin(fi))/(l+sin(fi));
printf("\nCoefficient of active earth pressure:%f",ka);
printf("\nEnter the grade of concrete used M(20–35):");
scanf("%d",&grc);
printf("bEnter the unit weight of soil(newton per metric cube):");
scanf("%f",&uws);
printf("\nEnter the level of road(in metres).");
scanf("%f",&R11);
```

```
printf("\nEnter the level of the top of the slab (in metres)"),
scanf("%f",&R12);
d=clsp/l0,
l=clsp+d,
h=h+d,
do
{
printf("Dimensions(centre to centre):lXh\n");
he=R11–R12;
{if(hs=0.0)
hew=0.0;
else
    if(he>0.0)
hew=he*1*1*uws;}
sw=d*1*1*dc;
dl=sw+hew;
printf("\nThe live load on the box culvert is calculated for IRC AA tracked vehicle.");
ll=boxlive(l,d);
printf("\n Live load due to IRC AA tracked vehicle:%f",ll);
its=d1+ll;
printf("\nTotal load on top slab %f",lts);
wsw=h*d*dc;
printf("\nWeight of each side wall:%f",wsw);
reb=((lts*1)+(2*wsw))/1;
fbmpt=lts*l*1/8;
fbmpb=reb*l*l/8;
printf("\nUpward soil reaction at base:%f",reb);
lpt=(11+hew)*ka;
lpuws=ka*uws*h;
lpb=1pt+lpuws;
vet=0.5*lts1;
veb=0.5*reb*l;
printf("\nFor the horizontal top slab vertical reactions at ends due to u.d.l:%f",vet);
printf("\nFor the horizontal bottom slab vertical reactions at ends due to u.d.l:%f",veb);
Kts=0.5;
Kbs=0.5;
Kvw=l.0;
df[0]=Kts/(Kts+Kvw);
df[l]=Kvw/(Kts+Kvw);
df[2]=Kvw/(Kbs+Kvwj;
df[3]=Kbs/(Kbs+Kvw);
printf("CASE 1\n");
printf("_________  \n");
casel(lpt,lpb,reb,l,lts,df,eml);
{for(int j=0;j<4;j++)
printf("%f\n", eml[j],"N.m");
nbmtl=fbmpt–eml[2];
nbmbl=fbmpb–eml[0];
H1=forces1(h);
hal=(eml[2]–eml[0])/h+Hl;
hdl=forcesla(h,hal);
```


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```
ssbmvl=forces2(h);
nbml=((eml[0]+eml[2])*0.5–ssbmvl;)
case2(lpt,lpb,reb,l,h,lts,df,em2);
printf("CASE 2\n");
printf("_____ \n");
{for(int j=0;j<4;j++)
printf("em2[j]= %f",em2[j],"N.m");
nbmt2=fbmpt–em2[2];
nbmb2=fbmpb–em2[0];
H2=forces3(h);
ba2=(em2[2]–em2[0])/h+H2;
hd2=forces3a(h,ha2);
ssbmv2=forces4(h);
nbm2=(em2[0]+em2[2])*0.5–ssbmv2;}
printf("CASE 3\n");
printf("———————— \n");

case3(hew,uws,reb,l,h,lts,df,em3);
{for(int j=0;j<4;j++)
printf("em3[j]=%f",em3[j],"N.m");
nbmt3=fbmpt–em3[2];
nbmb3=fbmpb–em3[0];
H3=forces5(h);
ha3=(em3[0]–em3[2])/h+H3;
hd3=forces4a(h,ha3);
ssbmv3=forces4b(h);
nbm3=(em3[0]+em3[2])*0.5+ssbmv3;}

printf("\n Section at centre is designed for maximum B.M. induced at centre\n");
    Ml=(nbmt1>nbmt2)?nbmtl:nbmt2;
    M=(M1>nbmt3)?Ml:nbmt3:
printf("\nDesign bending moment at centre is, %f",M);
do
{
printf("\nEnter the diameter of steel of main reinforcement :");
scanf("%d",&dm);
if(dm<20||dm>40)
printf("\nDiameter range is between 20 mm to 40 mm.");
} while(dm<20||dm>40);
float cc=30;
de=(d*1000)–(dm/2+cc);
j=design(grc);
Q=design2();
    dr=sqrt((M*1000)/(Q*b));
printf("\n Depth required is:%f",dr);
if(dr>de)
{printf("\nRedesign the section by increasing the depth by 0.03 m.");
    d=d+0.03;)
    l=clsp+d;
    h=h+0.03;}while(dr>de);
    if(dr<de)
```


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```
printf("\n Depth of the section is:%f",de);
  Ast=asteell (de,M j);
  sm= spmsteel(dm);
  printf("\n Area of main steel reinforcement is:%f",Ast);
  printf("\n Spacing per metre is:%f",sm,"mm\n");
  printf("\n Bend:%f", Ast*0.5)
printf("up near the supports at a distance of:%f", 1/5,"from edges.");
  do {
  printf("\n Enter the diameter of distribution steel.");
  scanf("%d",&dd);
  if(dd<8.0/ldd>20.0)
  printf("\nDiameter range is between 8 mm to 20 mm.");
  }while(dd<8.0 dd>20.0);
  Astd=0.2*d*10000;
printf("%f",Astd);
  sd=sdsteel1(dd,Astd);
printf("\nProvide %d mm dia bars at %f mm on each face",dd,sd*2);
printf("Section at ends\n");
printf("***************************\n");
  M2=(eml[2]>em2[2])?eml[2]:em2[2];
  Ms=(M2>em3[2])?M2:em3[2];
printf("Design bending moment is:%f",Ms);
  Astl=astee12(de,Ms,j);
  printf("\n Area of steel reinforcement is:%f",Astl);
  printf("\n Area available from bars bent at mid-section is:%f",Ast*0.5);
  Aas=Astl–0.5*Ast;
  printf("\nArea required,%f",Aas);
  sas=sasteel(dd,Aas);
printf("\nProvide %d mm diameter bars at %f mm c/c", dd,sas);
printf("################################\n");
printf("DESIGN OF BOTTOM SLAB\n");
printf("___________________________ \n");
printf("\n Section at centre is designed for maximum B.M. induced at centre");
  Mbl=(nbmbl>nbmb2)?nbmb1:nbmb2;
  Mb=(Mb1>nbmb3)?M1:nbmb3;
printf("\nDesign bending moment at centre is:%f",Mb);
Astb=asteel2b(de,Mb,j);
smb=spmsteel1 (dm);
printf("\n Area of main steel reinforcement is:%f",Astb);
printf("\n Spacing per metre is,%f mm c/c",smb);
printf("\n Bend:%f up at the outer face near the supports", Astb*0.5):
printf("\nProvide %d mm diameter bars at %f mm c/c on each face", dd,sd*2);
printf("\nSection at ends");
printf("*********************\n");
  M2b=(eml [0]>em2[0])?eml [0]:em2[0];
  Msb=(M2b>em3[0])?M2:em3[0];
printf("Design bending moment is:%f",Msb);
  Astlb=asteel3(de,Msb,j);
printf("\n Area of steel reinforcement is:%f",Astl b);
printf("\n Area available from bars bent at mid-section is:%f",Astb*0.5);
Aasb=Astlb–0.5*eAstb;
```


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```
printf("\nArea required:%f",Aasb);
sasb=sasteel2(dd,Aasb);
printf("\nProvide %d mm bars at %f mm c/c", dd,sasb);
printf("############################\n");
printf("DES1GN OF SIDE WALLS\n");
printf("___________________ \n");
float el=(nbml>nbm2)?nbml:nbm2;
float e2=(el>nbm3)?el:nbm3;
float e3=(e2>eml[0])?e2:em1[0];
float e4=(e3>em2[0])?e3:em2[0];
float me=(e4>em3[0])?e4:em3[0];
float e=(me)/veb;
ce=cc+(dm*0.5);
   if(e/d <0.25)
   spe=spel (e,veb,dm,b,d,m,ce,sigcbc,sigco);
   else
      if(0.25<e/d<1.5)
   spe=spe2(me,veb,dm,b,d,grc,ce);
   else
      if(e/d>1.5)
      spe=spe3(e,veb,dm,b,d,m,ce,sigcbc,sigst);
   printf("Provide %d mm bars at %f mm c/c", dm,spe);
   printf("Provide distribution steel of %d mm at %f mm c/c",dd,sas);
   getch();}
```

The following functions are to be created in separate tiles and included in the main program. The function of individual modules is explained here.

File 1. 1b.c

This function calculates the spacing of additional steel for the top slab.

```
float sasteel2(int dd,float Aasb)
{float sasb,asasb;
asasb=3.14*dd*dd/4;
sasb=(asasb/Aash)*1000;
sasb=int(sasb);
if(sasb>200)
sasb=200;
return(sasb);}
```

File 2. 1.c

This function calculates the spacing of additional steel for the bottom slab.

```
float sasteel(int dd,float Aas)
{float sas,asas;
asas=3.14*dd*dd/4;
sas=(asas/Aas)* 1000;
sas=int(sas);
if(sas>200)
sas=200;
retum(sas);}
```


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File 3. 1c.c

This function calculates the area of steel and section of the bottom slab.

```
#inciude<stdio.h>
float Astlb;
float asteel3(float de,float Msb,float j)
{float sigst=200.0;
Astl b=(Msb*l000)/(sigst*de*j);
   return(Ast1b);}
```

File 4. 3.c

This function calculates the axial thrust for the top slab (loading case :3)

```
#include<stdio.h>
#include<conio.h>
#include”box3.c”
float H3;
float forced(float h)
{
H3=(em3[0]–em3[2]–1pt3*h*h*0.5+(0.5*(1pb3–1pt3)*h*h)/3)/h;
   return H3;}
```

File 5. bo1.c

This function calculates the area of steel for the bottom slab.

```
#include<stdio.h>
float Astb;
float asteel2b(float de,float Mb,float j)
{float sigst=200.0;
Astb=(Mb* l000)/(sigst*de*j);
   return(Astb);}
```

File 6. bo2.c

This function calculates the spacing of steel for the bottom slab.

```
#include<stdio.h>
#include“bol.c”
float spmsteel1(int dm)
{float astb,smb;
astb=3.14*dm*dm*0.25;
smb=(astb/Astb)*1000;
smb=int(smb);
if(smb>l50.0) smb=150.0;
return(smb);}
```


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File 7. box1ive.c

This function returns the live load imposed on the box culvert owing to IRC Class AA live load.

```
#include<stdio.h>
float boxlive(float 1,float d)
{float b,r,k,bw,dw=0.08,x,e,te,1e,1l,imf=0.25;
int n;
b=7.5;
r= b/l;
if(r<=0.1)    k=0.40;
if((r>=0.1)&&(r<=0.2))  k=(r–0.1)*4.0+0.4;
if((r>=0.2)&&(r<=0.3))  k=(r–0.2)*3.6+0.8;
if((r>=0.3)&&(r<=0.4))  k=(r–0.3)*2.8+1.16;
if((r>=0.4)&&(r<=0.5))  k=(r–0.4)*2.4+1.44;
if((r>=0.5)&&(r<=0.6))  k=(r–0.5)*1.6+1.16;
if((r>=0.6)&&(r<=0.8))  k=(r–0.6)*1.2+1.84;
if((r>=0.8)&&(r<=1.0))  k=(r–0.8)*0.8+2.08;
if((r>=1.0)&&(r<=1.1))  k=(r–1.0)*0.8+2.24;
if((r>=1.1)&&(r<=1.2))  k=(r–1.1)*0.8+2.28;
if((r>=1.2)&&(r<=1.3))  k=(r–1.2)*0.4+2.36;
if((r>=1.3)&&(r<=1.4))  k=(r–1.3)*0.8+2.40;
if((r>=1.4)&&(r<=1.5))  k=(r–1.4)*0.0+2.48;
if((r>=1.5)&&(r<=1.80))  k=(r–1.5)*0.8+2.48;
if(r>=1.8)         k=2.60;
x=0.5*1;
bw=0.85+2*(dw);
e=k*x*(l–x/l)+bw;
te=1.625+2.050+e/2;
if(te>b)
te=b;
le=3.6+(2*(d+dw));
{if(le<l)
ll=700000*(l+imf)/(le*te);
else
   if(le>l)
ll=(700000*1*(l+imf))/(le*l*te);
return(ll);}}
```

File 8. boxcasel.c

This function returns the simply supported bending moment in vertical wall for Case 1.

```
#include“forcel a.c”
float ssbmvl;
float forces2(float 1)
{
ssbmvl=lpt1*1*1/8+(1pb1–1ptl)*1*1/16;
return(ssbmv1);}
```


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file boxcase2.c

```
#include“force3a.c”
float sshmv2;
float forces4(float h)
{
ssbmv2=(lpb2*h*h/8)+(lpt2–lpb2)*h*h/16;
return(ssbmv2);}
```

File 9. boxcase3.c

This function returns the simply supported bending moment in vertical wall for Case 3.

```
#include”force4a.c”
float sshmv3;
float forces4b(float h)
{
ssbmv3=–(lpt3*h*h/8)+(lpb3–lpt3)*h*h/l6;
return(ssbmv3);}
```

File 10. design.c

This function returns the design constant j to the main function.

```
#include<stdio.h>
float n,j,Q,sigcbc;
float design(int grc)
{
const float m=10.0,sigst=200.0;
printf(“\n Grade of steel used in the Design: Fe415\n”);
{
if(grc==20) sigcbc=6.7;
if(grc==25) sigcbc=8.3;
if(grc==30) sigcbc=10.0;
if(grc==35) sigcbc=11.5;
n=(m*sigcbc)/((m*sigcbc)+sigst);
j=l-n/3;return(j);}}
```

File 11. design2.c

This funchon returns the design constant “Q” based on the type of the concrete grade.

```
#include<stdio.h>
#include”design.c”
float design2(){
Q=0.5*n*j*sigcbc; return(Q);
```


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File 12. design3.c

This function rerturns the permissible shear stress depending on the grade of concrete used.

```
#include<stdio.h>
#include “design2.c”
float toc;
float design3(int grc)
{
if(grc= =20)
     toc=0.34;
else
  if(grc= =25)
     toc=0.40;
    else
     if(grc= =30)
     toc=0.45;
    else
     if(grc= =35)
       toc=0.50;
return(toc);}
```

File 13. design4.c

This function returns the permissible axial (direct) compressive stress value.

```
#include<stdio.h>
#include “design3.c”
float sigco;
float design4(int grc)
{
if(grc= =20)
     sigco=5.0;
else
  if(grc= = 25)
     sigco=6.2,
  else
   if(grc= = 30)
     sigco=7.5,
    else
     if(grc= =35)
       sigco=8.5;
return(sigco);}
```


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File 14. areaia.c

This function returns the spacing of main steel.

```
#include<stdio.h>
#include"areaa.c"
float spmsteel(int dm)
{float ast,sm;
ast=3.14*dm*dm*0.25;
sm=(ast/Ast)*1000;
sm=int(sm);
if(sm>l50.0) sm=150.0;
return(sm);}
```

File 15. area3a.c

This function returns the spacing of distribution steel.

```
#include<stdio.h>
float sdsteell(int dd,float Astd)
{float sd,astd;
astd=(3.14*dd*dd)/4;
sd=(astd/Astd)*1000.0;
sd=int(sd);
if(sd>200.0)
sd=200.0;
return(sd);}
```

File 16. area.c

This function returns the area of steel for end section of the top slab.

```
#include<stdio.h>
float Ast;
float asteell(float de,float M,float j)
{float sigst=200.0;
Ast=(M*1000)/(sigst*de*j);
return(Ast);}
```

File 17. box1.c

This function returns the final end moments for Case 1. The moment distribution method used is coded here.

```
#include<stdio.h>
#include<conio.h>
#include<math.h>
float lpt1,lpbl;
double em1[4];
void casel(float lpt,float lpb,float reb,float 1, float lts,double* df,double* eml)
{
```


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```
double fem[4],bal[4],sum1,sum2,co[2],bald[4],x;
lptl=lpt;
lpbl=lpb;
fem[0]=reb*1*1/12;
fem[1]=–(lpt1*1*)/12–0.5*(1pb1–1pt1)*1)*1/10;
fem[2]=(lpt1*1*1)/12+(0.5*(1pb1–1pt1)*1*1)/15;
fem[3]=–(lts*1*1)/12;
suml=fem[0]+fem[l];
suml= –sum1;
bal[0]=sum1 *df[0];
bal[l]=sum1 *df[l];
sum2=fem[2]+fem[3];
sum2= –sum2;
ba1[2]=sum2*df[2];
ba1[3]=sum2*df[3];
co[0]=0.5*ba1[2];
co[l]=0.5*bal[l];
int i=0;
while(i<l0)
{
    bald[0]=df[0]*(–co[0]);
    bald[l]=df[l]*(–co[0]);
    bald[2]=df[2]*(–co[1]);
    bald[3]=df[3]*(–co[l]);
    bal[0]=bal[0]+bald[0];
    bal[l]=bal[l]+bald[l]+co[0];
    bal[2]=bal[2]+bald[2]+co[l];
    bal[3]=bal[3]+bald[3];
    co[0]=0.5*bald[2];
    co[1]=05*ha1d[l]; i++;}
    for(i=0;i<4;i++)
    *em1++=fem[i]+bal[i];}
```

File 18. box2.c

This function returns the final end moments for Case 2.

```
#include<stdio.h>
#include<conio.h>
#include<math.h>
float lpt2,lpb2;
double em2[4];
void case2(float lpt,float lpb,float reb,float 1,float h, float lts,double* df,double* em2)
{
double fem[4],bal[4],sum1,sum2,co[2],bald[4];
lpt2=lpt;
lpb2=lpb–9800*h;
fem[0]=reb*1*1/12;
fern[1]= –(lpb2*1*1)/12–(0.5*(lpt2–lpb2)*1)*1/15;
fem[2]=(lpb2*1*1)/12+(0.5*(lpt2–lpb2)*1*1)/l0;
fem[3]=–(lts*l*l)/12;
```

```
sum1=fem[0]+fem[1];
sum1= –sum1;
bal[0]=suml*df[0];
bal[1]=suml*df[1];
sum2=fem[2]+fem[3];
sum2= –sum2;
ba1[2]=sum2*df[2];
ba1[3]=sum2*df[3];
co[0]=0.5*ba1[2];
co[l]=0.5*bal[l];
int i=0;
while(i<10)
{
    bald[0]=df[0]*(–co[0]);
    bald[1]=df[l]*(–co[0]);
    bald[2]=df[2]*(–co[1]);
    bald[3]=df[3]*(–co[1]);
    bal[0]=bal[0]+bald[0];
    bal[1]=bal[1]+bald[1]+co[0];
    bal[2]=bal[2]+bald[2]+co[1];
    bal[3]=bal[3]+bald[3];
    co[0]=0.5*bald[2];
    co[1]=0.5*bald[1]; i++;}
    for( i=0;i<4;i++)
    *em2++=fem[i]+bal[i];}
```

File 19. box3.c

This function returns the final end moments for Case 3.

```
#include<stdio.h>
#includ<conio.h>
#include<math.h>
float lpt3,lpb3;
double em3[4];
void case3(double hew,double uws,float reb,float 1,float h, float lts,double* df,double* em3)
{
double fem[4],bal[4],suml,sum2,co[2],bald[4];
lpt3=hew/3;
lpb3=9800*h+lpt3–(uws*h/3);
fem[0]=reb*1*1/12;
fem[1]=–(lpt3*1*1)/12+(0.5*(lpb3–lpt3)*1)*1/10;
fem[2]=(lpt3*1*1)/12–(0.5*(lpb3–lpt3)*1*1)/15;
fem[3]=–(lts*1*1)/12
suml=fem[0]+fem[1];
suml=–suml;
bal[0]=suml*df[0];
bal[1]=suml*df[1];
sum2=fem[2]+fem[3];
sum2=–sum2;
bal[2]=sum2*df[2];
```

```
bal[3]=sum2*df[3];
co[0]=0.5*bal[2];
co[1]=0.5*bal[1];
int i=0;
while(i<10)
{
bald[0]=df[0]*(–co[0]);
bald[1]=df[1]*(–co[0]);
bald[2]=df[2]*(–co[1]);
bald[3]=df[3]*(–co[1]);
bal[0]=bal[0]+bald[0];
bal[1]=bal[1]+bald[1]+co[0];
bal[2]=bal[2]+bald[2]+co[1];
bal[3]=bal[3]+bald[3];
co[0]=0.5*bald[2];
co[1]=0.5*bald[1];i++;}
for(i=0;i<4;i++)
*em3++=fem[i]+bal[i];}
```

File 20. force1a.c

This function returns the thrust in bottom slab for Case 1.

```
#include<stdio.h>
#include<conio.h>
#include"forcel.c"
float hd1;
float forces la( float h ,float ha1)
{
hd1=((lpt1+lpb1)*0.5*h)–ha1;
return(hd1);}
```

File 21. force1.c

This function returns the thrust in the top slab for Case 1.

```
#include<stdio.h>
#include<conio.h>
#include"bon1.c"
float H1;
float forces1(float 1)
{ H1=(+lptl*1*1*0.5+(0.5*(lpb1–lpt1)*1*1)/3)/1;
return(H1);}
```

File 22. force3.c

This function returns the thrust in the top slab for Case 2.

```
#include<stdio.h>
#include<conio.h>
#include"box2.c"
float H2;
```

```
float forces3(float h)
{ H2=(+1pb2*h*h*0.5+((lpt2–lpb2)*h*h)/3)/h;
return(H2);}
```

File 23. force3a.c

This function returns the thrust in the bottom slab for Case 2.

```
#include<stdio.h>
#include<conio.h>
#include“force3.c”
float hd2;
float forces3a( float h ,float ha2)
{
hd2=(lpt2+lpb2)*0.5*h–ha2;
return(hd2);}
```

File 24. e1.c

This function returns the spacing of steel in vertical wall in case of less eccentricity.

```
#include<stdio.h>
float spe1(float e,float veb,int dm,float b,float d,float m,float ce,float sigcbc,float sigco)
{float Ae,Ie,sigcc,sigc,spea1,Ase,f,spe;
do {
printf(“Enter spacing of main steel in mm”);
scanf(“%f”,&speal);
if(spea1>300.00)
speal=300.0;
Asc=1000/speal*(3.14*dm*dm)*0.25;
Ae=b*d+((1.5*m–1)*Asc);
Ie=(b*d*d*d/2)+(1.5*m–1)*Asc*(d*0.5–ce)*(d*0.5–ce);
sigcc=veb/Ae;
sigc=veb*e/Ie*d*0.5;
f=(sigcc/sigco)+(sigc/sigcbc);} while(f>1);
spe=spea1;
return(spe);}
```

File 25. e2.c

This function returns the spacing of steel in vertical wall in case of less eccentricity.

```
#include<stdio.h>
float spe2(float me,float veb,int dm,float b,float d,int grc,float ce)
{float spea2,Asc,spe,fck,Mu,Vu,p,r,rl,r2,r3,
Mu=l.5*me,
Vu=l.5*veb;
fck=grc;
r1=(Mu*1000)/(fck*b*d*d*1000000);
r2=Vu/fck*b*d*1000);
r3=ce/d*1000);
```


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```
printf("Referring to the interaction curves (SP-16) for Fe 415 for ,%f,%f,%f\n",rl,r2,r3);
printf("r=");
scanf("%f",&r);
p=r*fck;
Asc=p*b*d*1000/100;
if(Asc<(0.8*h*d*l0))
Asc=0.8*b*d*l0;
spea2=(3.l4*dm*dm*0.25/Asc)* 1000;
spe=int(spea2);
return(spe);}
```

File 26. e3.c

This function returns the spacing of steel in vertical wall in case of high eccentricity.

```
#include<stdio.h>
#include<math.h>
float spe3(float e,float veb,int dm,float b,float d,float m,float ce,float sigcbc,float sigst)
{float spea3,Asc,spe,n,c,t,nl,n2,X,Y,Y1;
do{
printf("Enter spacing of main steel in mm\n");
scanf("%f",spea3);
if(spea3>300.00)
spea3=300.0;
Asc=l000/spea3*(3.14* dm*dm)*0.25;
X=((m+(1.5*m)–l)*Asc)/b;
Yl=sqrt(m+(l.5*m)–l)*Asc–(2*b*((m*(d–ce)+ce*(l.5*m)–l)));
Y=Yl/b;
nl=abs(–X+Y);
n2=abs(–X–Y);
n=(n1>n2)?n1:n2;
c=veb*e/(((0.5*b*n)*(d–ce–n/3))+1.5*m–1)*Asc*(n–ce)/n);
t=m*c*(d–ce–n);}while(c>sigcbc&&t>sigst);
spe=spea3;
return(spe);}
```

File 27. area1b.c

This function returns the additional steel for the bottom slab.

```
#include<stdio.h>
float Astl;
float asteel2(float de,float Ms,float j)
{float sigst=200.0;
Astl=(Ms*l000)/(sigst*de*j);
return(Ast1);}
```


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File 28. force4a.c

This function returns the axial thrust for the bottom slab for Case 3.

```
#include<stdio.h>
#include“3.c”
float hd3;
float forces4a(float h,float ha3)
{hd3=0.5*(lpb3–lpt3)*h–(lpt3*h)–ha3;
return(hd3);}
```


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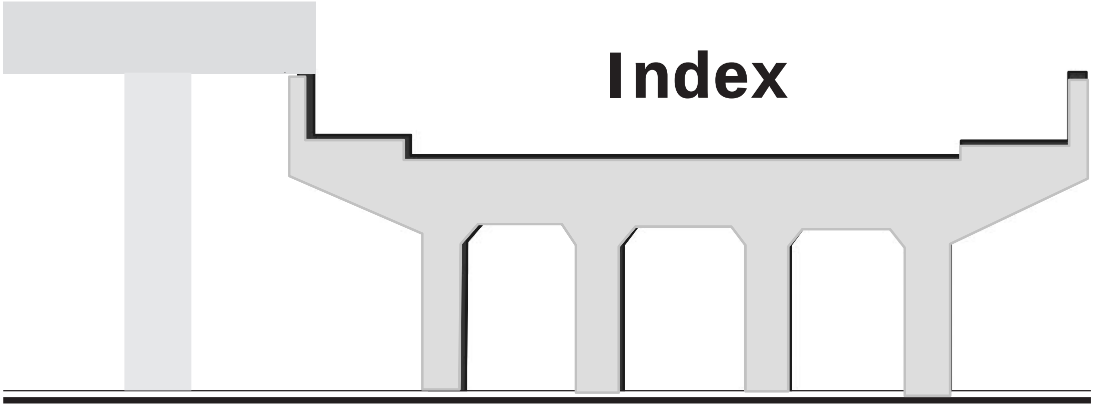

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THE AUTHORS

T.R. Jagadeesh, Ph.D., is Principal, H.M.S. Institute of Technology, Tumkur. His areas of interest include hydraulic design of bridges, fluid dynamics, rainwater harvesting and computer applications in hydraulic engineering. He has authored several books and presented numerous papers in national and international conferences.

M.A. Jayaram, Ph.D., is Professor, and Director of the Department of Master of Computer Applications, Siddaganga Institute of Technology, Tumkur. His areas of interest include structural design of bridges, applications of soft computing in civil engineering, data mining and evolutionary computing algorithms.

Dr. Jayaram has published/presented more than 50 papers in national and international journals and conferences. He has authored several books, including *Mechanics of Materials with Programs in C*, published by PHI Learning, New Delhi.

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