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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 construction 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 much more than just pedestrian loading. To give the designers the ability to accurately model the live load on a structure, 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.

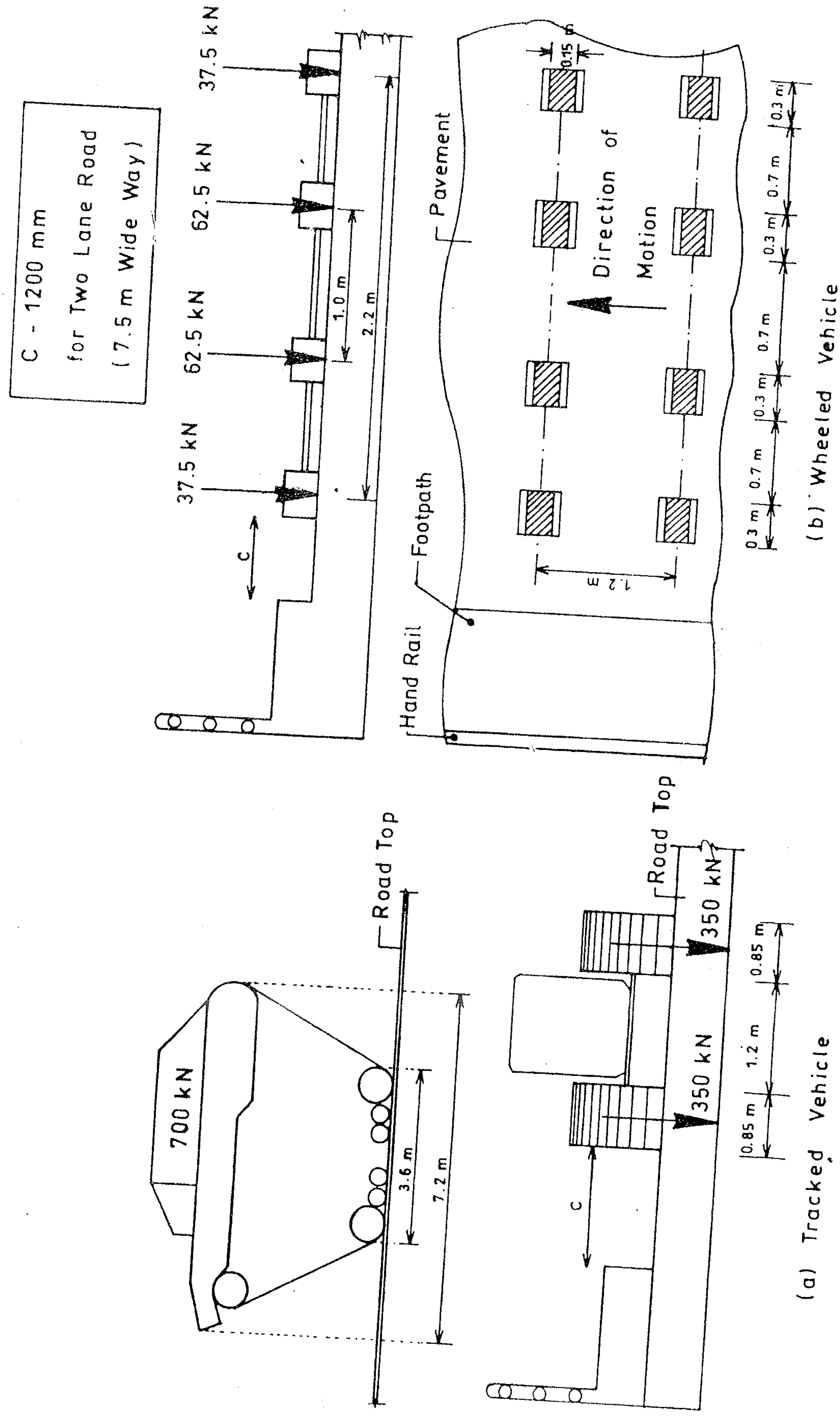
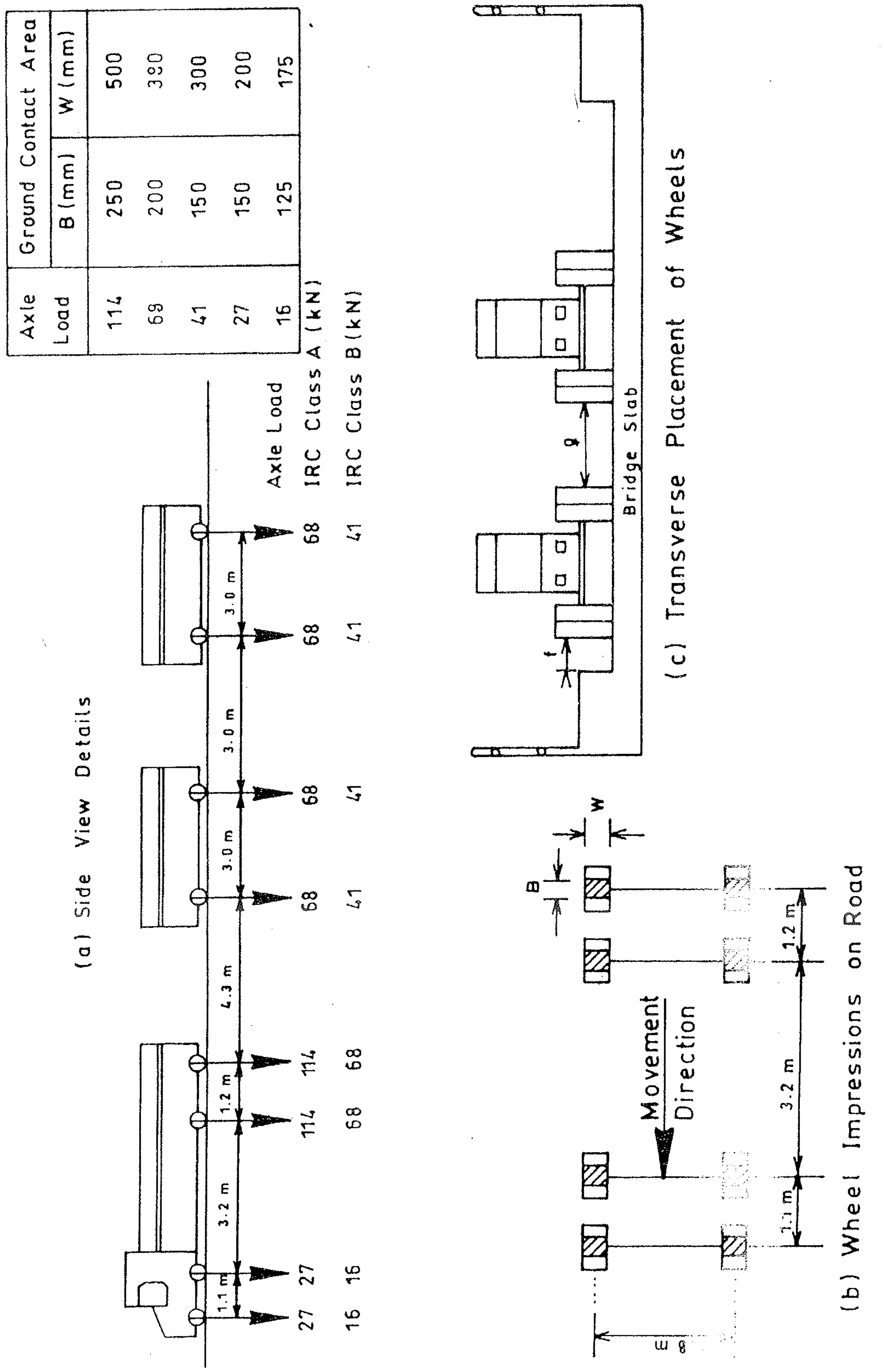


Fig. 4.1 IRC Class AA loadings.

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

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.

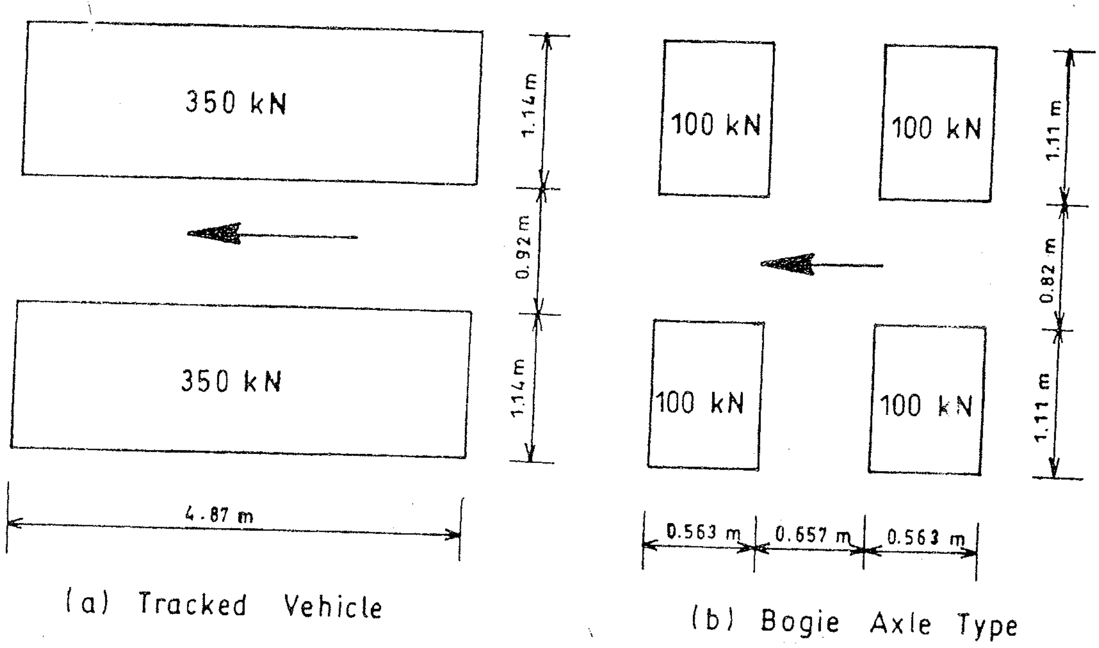


Fig. 4.3 Class 70R loadings.

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].

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 depending 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.

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

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.

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: M 25 concrete and Fe 415 steel.

Design of the Slab

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.

$$\text{Overall depth of the slab} = 80 \times 5.5 = 440 \text{ mm}$$

Using 25 mm diameter bar and a clear cover of 30 mm, we have

$$\text{Effective depth of the slab} = 440 - 12.5 - 30 = 397.5 \text{ mm}$$

Effective span is the least of

(i) Clear span + Effective depth = 5.5 + 0.397 = 5.897 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

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

$$= 2.625 + 2.05 + 2.70 = 7.375 \text{ m} \approx 7.38 \text{ m}$$

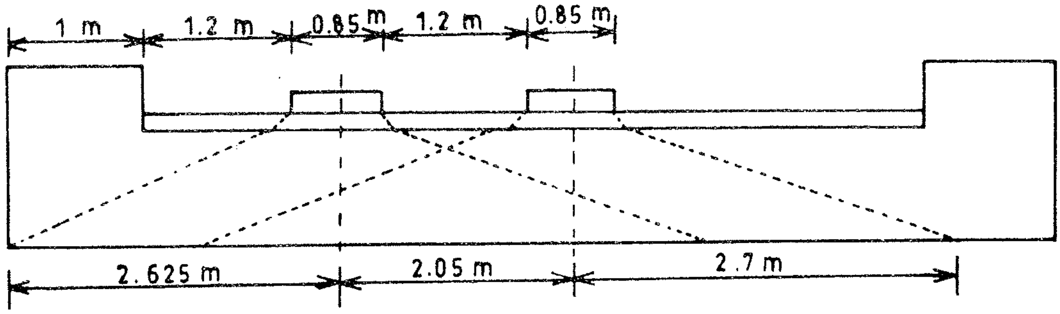


Fig. 7.7 Effective width of two wheels (For B.M.) (Example 7.2).

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).

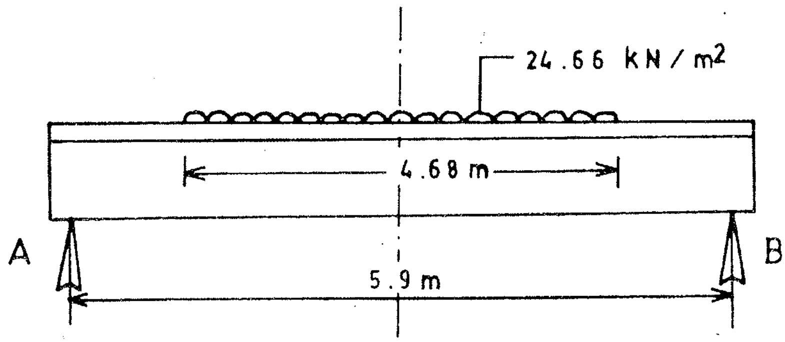


Fig. 7.8 Loading for B.M. (Example 7.2).

$$\text{Maximum live load B.M.} = \frac{24.66 \times 4.68}{2} \times \frac{5.9}{2} - 24.66 \times \frac{4.68}{2} \times \frac{4.68}{4} = 102.71 \text{ kN·m}$$

Design bending moment = dead load B.M. + live load B.M.

$$= 55.52 + 102.71 = 158.23 \text{ kN·m}$$

$$\text{Effective depth required} = \sqrt{\frac{158.23 \times 10^6}{1.1 \times 10^3}} = 379.27 \text{ mm}$$

Effective depth 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

$$= 397.5 - 12.5 - 5 = 380 \text{ mm}$$

$$\text{Area of distribution steel} = \frac{29.98 \times 10^6}{200 \times 0.9 \times 3.80} = 438.30 \text{ mm}^2$$

$$\text{Spacing of bars} = \frac{78.5}{438.30} \times 1000 = 179 \text{ mm say at 170 mm c/c.}$$

Check for shear stress

The loading should be arranged as shown in Fig. 7.9.

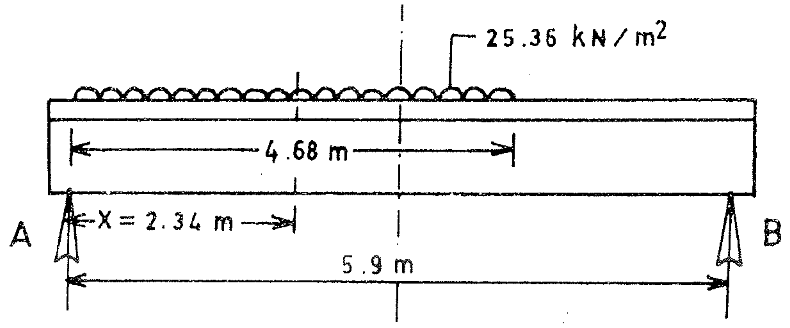


Fig. 7.9 Loading for S.F. (Example 7.2).

Distance of the centre of gravity of the concentrated load from the nearest support A

$$x = 4.68/2 = 2.34 \text{ m}$$

$$\text{Effective width of dispersion} = 2.88 \times 2.34 \left(1 - \frac{2.34}{5.9}\right) + 1.05 = 5.12 \text{ m}$$

Effective width for two wheels (Fig. 7.10) = 2.56 + 2.05 + 2.56 = 7.17 m

$$\text{Intensity of loading} = \frac{1.217 \times 700}{4.68 \times 7.17} = 25.36 \text{ kN/m}^2$$

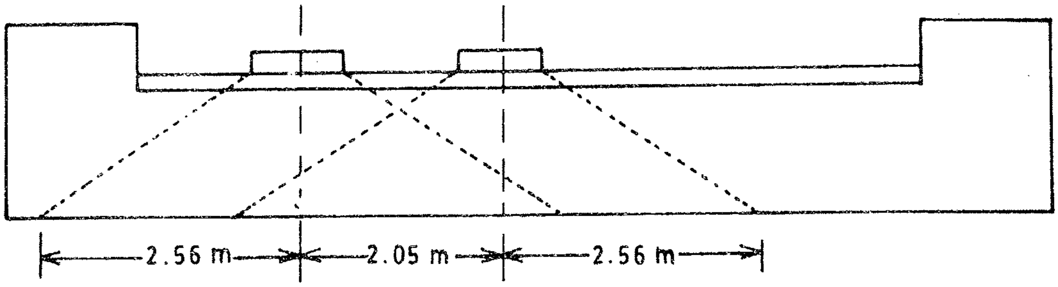


Fig. 7.10 Effective width for two wheels (For S.F.) (Example 7.2).

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

$$\tau_c = k_1 k_2 \tau_{co}$$

where

$$k_1 = (1.14 - 0.7 \times 0.397) = 0.722$$

$$k_2 = 1 \quad \text{and} \quad \tau_{co} = 0.4 \text{ N/mm}^2$$

Therefore,

$$\tau_c = 0.722 \times 1 \times 0.4 = 0.29 \text{ N/mm}^2$$

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:

$$\text{Dispersion length for one wheel} = 0.25 + 2(0.36 + 0.08)$$
$$= 1.13 \text{ m}$$

$$\text{Dispersion length for two wheels} = 1.2 + \frac{1.13}{2} + \frac{1.13}{2} = 2.33 \text{ m}$$

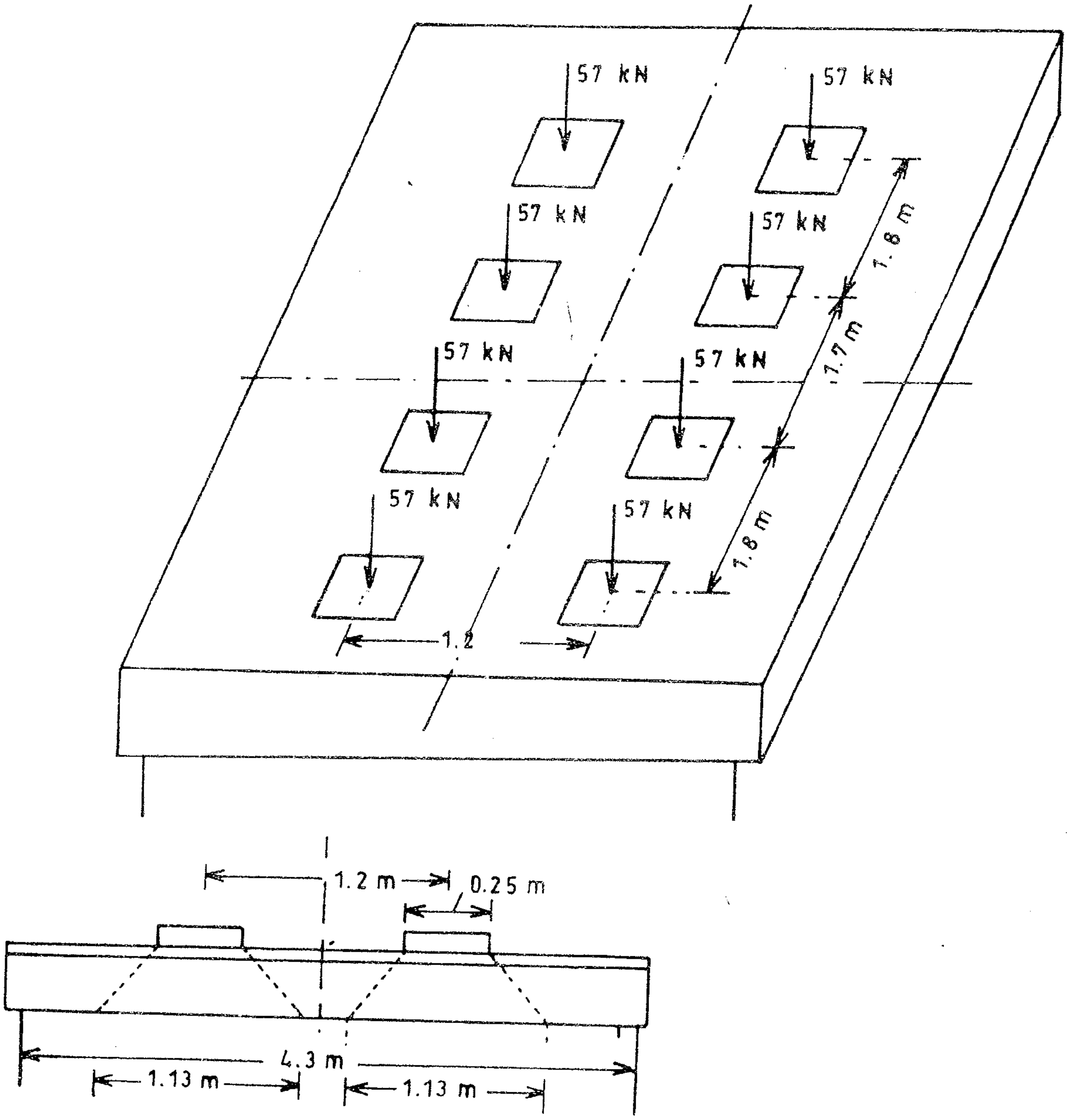


Fig. 7.11 Arrangement of IRC Class A and effective length of dispersion (Example 7.3).

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).

$$= 1.00 + 1.80 + 1.70 + 1.80 + \frac{3.63}{2} = 8.115 \text{ m}$$

$$\text{Intensity of distributed load} = \frac{1.44 \times 4 \times 114}{8.115 \times 2.33} = 34.72 \text{ kN/m}^2$$

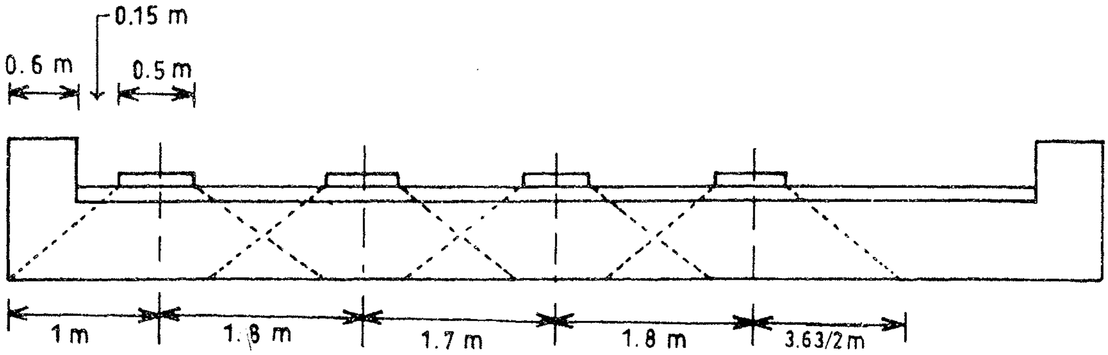


Fig. 7.12 Effective width for four IRC Class A wheels (Example 7.3).

Maximum live load bending moment

$$= \frac{34.72 \times 2.33 \times 4.3}{4} - \frac{34.72 \times 1.165 \times 1.165}{2} = 63.40 \text{ kN·m}$$

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).

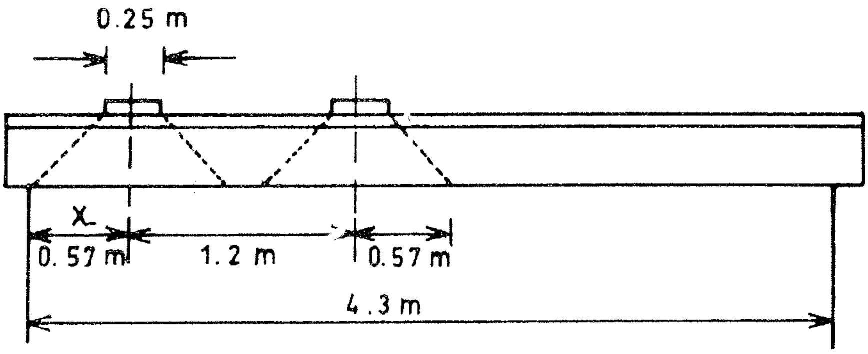


Fig. 7.13 Class A loads for maximum B.M. (Example 7.3).

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

$$= \frac{1.44 \times 4 \times 114}{7.463 \times 2.33} = 37.76 \text{ kN/m}^2$$

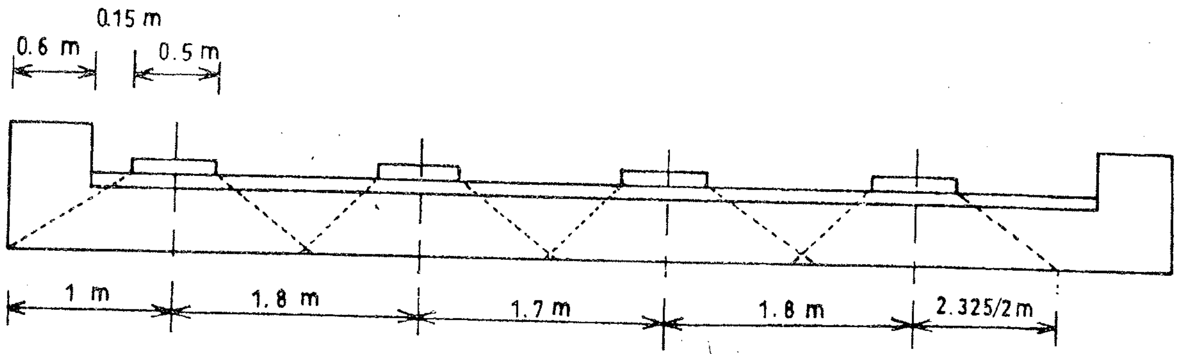


Fig. 7.14 Effective width for four IRC Class A wheels (For S.F.) (Example 7.3).

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: M25 concrete, Fe 415 steel
   Abutment: Standard sections
   Wing wall: Return 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: M30 concrete, Fe 415 grade steel

Overall depth of the slab = 4 × 90 = 360 mm

Using 20 mm diameter bars with a clear cover of 30 mm, we have

Effective depth of the slab d = 360 − 30 − 10 = 320 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 + 0.32 = 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 = 0.36 × 24 = 8.64 kN/m²

Dead load of the wearing coat = 0.08 × 22 = 1.76 kN/m²

10.40 kN/m²

$$\text{Dead load bending moment} = \frac{Wl^2}{8}$$

$$= \frac{10.4 \times 4.3^2}{8} = 24 \text{ kN·m}$$

$$\text{Dead load shear force} = \frac{10.4 \times 4.3}{2} = 22.4 \text{ kN}$$

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.

$$\text{Proportional load to be considered} = \frac{4.3 \times 700}{4.5} \text{ (see Fig. 7.3)} = 684 \text{ kN}$$

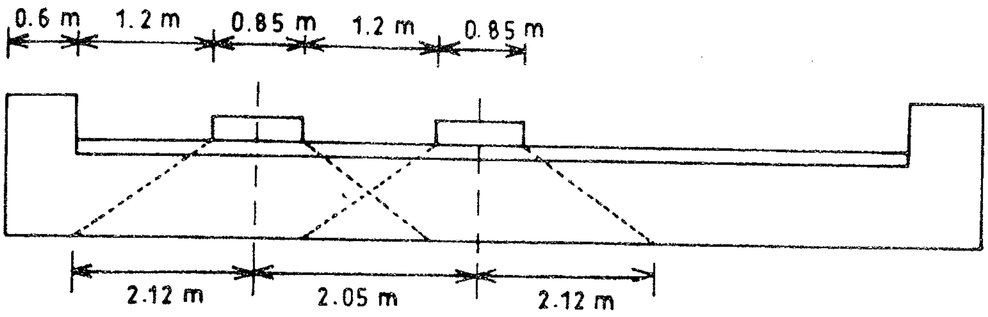


Fig. 7.4 Net effective width for two wheels (Example 7.1).

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 is

$$= \frac{31.56 \times 4.3^2}{8}$$

$$= 72.94 \text{ kN·m}$$

Design bending moment = Dead load B.M. + Live load B.M.

= 24 + 72.94 = 96.94 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

= 22.4 + 67.85 = 90.25 kN

Slab

$$\text{Effective depth required} = \sqrt{\frac{96.94 \times 10^6}{1.1 \times 1000}} = 296.86 \text{ mm}$$

Effective depth provided = 320 mm

$$\text{Area of longitudinal reinforcement} = \frac{96.94 \times 10^6}{200 \times 0.9 \times 320} = 1683 \text{ mm}^2$$

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.

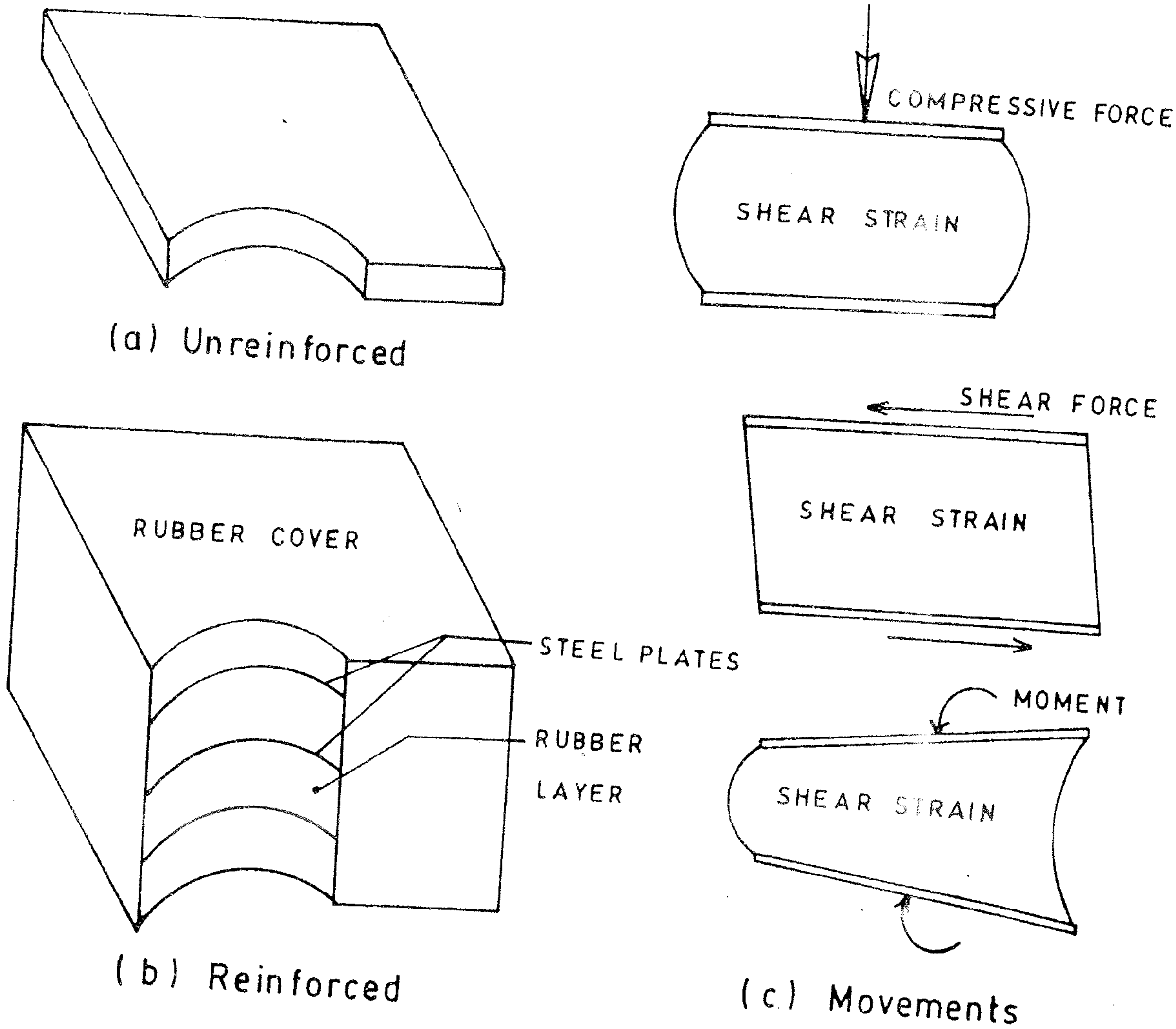


Fig. 14.3 Elastomeric bearings.

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)

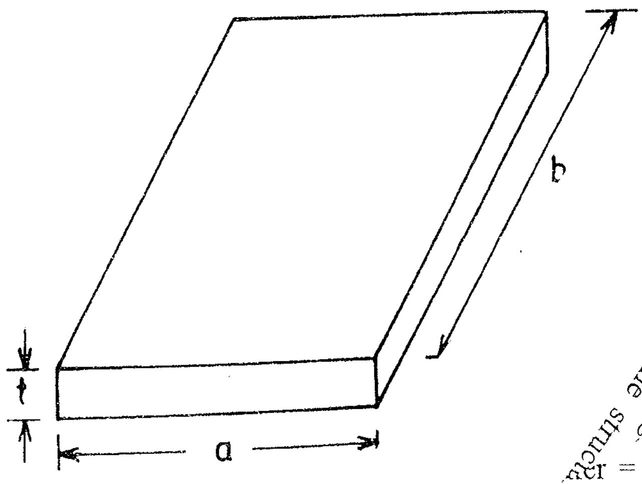
Size (*Index no.*)	*Width* (a) (mm)	*Length* (b) (mm)
1	160	250
2	160	320
3	200	320
4	200	400
5	250	400
6	250	500
7	320	500
8	320	630
9	400	630
10	400	800

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 thickr

3. Thickness. The thickness of a bearing is governed by its shear ... ~al shear deformation (Fig. 14.4), then



...er = 200 mm

Fig. 1/

$$u = t \tan \phi \quad (14.2)$$

$$\tan \phi = \frac{H_c + H_s}{GA} \quad (14.3)$$

where

G = modulus of rigidity in N/mm^2

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

$$\sigma_m = \frac{P}{A_e} \quad (14.4)$$

where

P = total vertical load in newton

A_e = effective plan area excluding shear deformation in mm^2

$$= \frac{a - u}{b} \quad (14.5)$$

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

(ii) Steel bridges

Impact Factor for Different Spans

Span length (m)	2	6	10	20	40	60	80	100
Lane – I	1.64	1.51	1.30	1.18	1.10	1.07	1.05	1.04
Lane – II	1.32	1.20	1.15	1.09	1.05	1.03	1.02	1.02

10. Highway Loadings of Belgium

The impact factor is given by the relation

$$\phi = 1 + \frac{0.377^{v}}{\sqrt{L/\alpha}}\sqrt{1 + \frac{2Q}{P}}$$

where v = speed in km/hour (always greater than 60)

L = span length in metres

$\alpha = (L/f_s)$

f_s = static deflection in metres due to dead weight

Q = moving loads on the bridge deck in tones

P = dead weight of the bridge in tones

11. Highway Loadings of Italy

The impact factor ϕ expressed as a function of span length L is given by the Formula,

$$\phi = 1 + \frac{(100 - L)^2}{100(250 - L)}$$

This relation is applicable for spans up to 100 m.
For spans exceeding 100 m, the value of ϕ is unity.

12. Highway Loadings of Netherlands

The impact coefficient 'S' for bridges carrying normal traffic given by the formula,

$$S = 1 + \frac{40}{(100 + L)}$$

where L is the span in metres.

Width of dispersion parallel to span = [0.25 + 2(0.5 + 0.08)] =1.41 m

The 114 kN wheels are placed symmetrically on the span as shown in Fig. 3.18.

M_{max} = [(27.36 × 0.5 ×.2.61) × 3.2] – [(27.36 × 0.5 × 2.61) × 0.25 × 2.61]
= 91.00 kN·m

The value of bending moment is significantly lower than the value of 113 kN.m obtained for IRC Class AA loading.

3.8 DESIGN OF REINFORCED CONCRETE SLAB SUPPORTED ON ALL SIDES (TEE BEAM AND SLAB DECK) FOR IRC CLASS A LOADS

The slab panel of a reinforced concrete Tee beam and slab deck is 2.5 m wide between main girders and 4 m between cross girders. Design the slab for IRC Class A loading. Adopt M-20 Grade concrete and Fe-415 grade HYSD bars.

1. Data

Two way slab panel 2.5 m wide by 4 m supported on all the four sides and continuous over main and cross girders.

Loading; IRC Class A train

Materials: M-20 Grade concrete and Fe-415 Grade HYSD bars

Thickness of slab = 200 mm

Wearing coat = 80 mm

2. Permissible Stresses

σ_{cb} = 6.67 N/mm^2 $\quad$ m = 10 $\quad$ Q = 0.762

σ_{st} = 200 N/mm^2 $\quad$ j = 0.91

3. Bending Moments

The arrangement of wheel loads is as shown in Fig. 3.19.

$W_1 = W_2$ = 57 kN, $\quad$ L = 4 m, $\quad$ B = 2.5 m

B.M due to W_1

u = [0.5 + (2 × 0.08)] = 0.66 m

v = [0.25 + (2 × 0.08] = 0.41 m

(u/B) = (0.66/2.5) = 0.264

(v/L) = (0.41/4) = 0.102

$K = (B/L)$ = (2.5/4) = 0.625

Using Pigeaud's curve (Refer Fig. 3.5) for $K = 0.6$, the moment coefficients are read out as

$$m_1 = 0.188 \text{ and } m_2 = 0.148$$

Short span moment $= M_B = W(m_1 + 0.15\, m_2)$

$= 57\,[0.188 + (0.15 \times 0.148)] = 11.99$ kN·m

Long span moment $= M_L = W(m_2 + 0.15\, m_1)$

$= 57\,[0.148 + (0.15 \times 0.188)] = 10.04$ kN·m

B.M due to load W_2 (Unsymmetrical load)

An imaginary load equal to W_2 is placed symmetrically as shown in Fig. 3.20.

$W_2 = 57$ kN

$$\text{Intensity of load} = \left[\frac{57}{(0.41 \times 0.66)}\right] = 210.6\,\text{kN/m}^2$$

$(u/B) = (0.66/2.5) = 0.264$

$(v/L) = (2.81/4) = 0.702$

$K = (B/L) = (2.5/4) = 0.625$

Using Pigeaud's curve (Fig. 3.5) for $K = 0.6$, read out the values of moment coefficients $m_1 = 0.12$ and $m_2 = 0.038$. The bending moments in the short and long span are computed as

$M_B = W(m_1 + 0.15\, m_2)$

$= [210.6 \times 2.81 \times 0.66]\,[0.12 + (0.15 \times 0.038)]$

$= 49.10$ kN.m

$M_L = W(m_2 + 0.15\, m_1)$

$= [210.6 \times 2.81 \times 0.66]\,[0.038 + (0.15 \times 0.12)]$

$= 21.87$ kN·m

Subtracting the moments due to the load as shown in Fig. 3.21,

$(u/B) = (0.66/2.5) = 0.264$

$(v/L) = (1.99/4) = 0.498$

$K = (B/L) = (2.5/4) = 0.625$

Using Pigeaud's curve (Fig. 3.5) for $K = 0.6$, read out the moment coefficients $m_1 = 0.142$ and $m_2 = 0.049$

∴ $M_B = W(m_1 + 0.15\, m_2)$

$= [210.6 \times 1.99 \times 0.66]\,[0.142 + (0.15 \times 0.049)]$

$= 41.30$ kN·m

$M_L = W(m_2 + 0.15\, m_1)$

$= [210.6 \times 1.99 \times 0.66]\,[0.049 + (0.15 \times 0.142)]$

$= 19.44$ kN·m

Moments due to W_2 are computed as

$M_B = 0.5[49.10 - 41.30] = 3.90$ kN·m

$M_L = 0.5[21.87 - 19.44] = 1.22$ kN·m

Applying continuity and impact factors, the total live load moments are given by

$M_B = (1.25 \times 0.8)[11.99 + 3.90] = 15.89$ kN·m

$M_L = (1.25 \times 0.8)[10.04 + 1.22] = 11.26$ kN·m

Dead weight of slab = $(0.2 \times 24) = 4.8$ kN/m^2

Dead weight of wearing coat = $(0.08 \times 22) = 1.76$ kN/m^2

Total dead load = 6.56 kN/m^2

Referring to Pigeaud's curve (Fig. 3.10)

$(u/B) = 1$, $(v/L) = 1$ and $K = (B/L) = 0.625$

$m_1 = 0.049$ and $m_2 = 0.015$

Moments due to dead load are computed as

$M_B = 65.60[0.049 + (0.15 \times 0.015)] = 3.36$ kN·m

$M_L = 65.60[0.015 + (0.15 \times 0.049)] = 1.468$ kN·m

Taking continuity into effect

$M_B = (0.8 \times 3.36) = 2.688$ kN·m

$M_L = (0.8 \times 1.468) = 1.174$ kN·m

Total design moments are given by

$M_B = (15.89 + 2.688) = 18.578$ kN·m

$M_L = (11.26 + 1.174) = 12.434$ kN·m

4. Design of Slab

The effective depth required is computed as

$$d = \sqrt{\frac{M}{Qb}} = \sqrt{\frac{18.578 \times 10^6}{0.762 \times 10^3}} = 156\,\text{mm}$$

Adopt effective depth = 160 mm and overall depth = 200 mm

Tension reinforcement is computed as

$$A_{st}(\text{for } M_B) = \left[\frac{(18.578 \times 10^6)}{(200 \times 0.91 \times 160)}\right] = 638\,\text{mm}^2$$

Use 12 mm diameter bars at 150 mm centres
Using 10 mm bars in the long span direction
Effective depth = [160 – (6 + 5)] = 149 mm

$$A_{st}(\text{for } M_L) = \left[\frac{(12.434 \times 10^6)}{(200 \times 0.91 \times 149)}\right] = 459 \text{ mm}^2$$

Use 10 mm diameter bars at 150 mm centres.

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The spacing of the shear connector is computed by the relation,

$$p = \left(\frac{\Sigma Q}{V_L}\right) \quad \text{or} \quad \left(\frac{\Sigma Q_u}{V_{Lu}}\right)$$

where p = Spacing of the shear connector (mm)
Q = Safe shear resistance of one connector (kN)
Q_u = Ultimate shear resistance of one connector (kN)
V_L = Longitudinal working shear per unit length
V_{Lu} = Ultimate longitudinal shear per unit length.

The longitudinal shear (working or ultimate) is computed using the equations,

$$V_L = \left(\frac{V A_C \bar{Y}}{I}\right) \quad \text{and} \quad V_{Lu} = \left(\frac{V_u A_C \bar{Y}}{I}\right)$$

where V = Vertical shear due to dead load placed after composite section is effective and working live load with impact
V_u = Vertical shear due to ultimate loads computed with load factors of 1.5 for dead load and 2.5 for live load.
A_C = Transformed compressive area of concrete above the neutral axis of the composite section
$\bar{Y}$ = Distance from neutral axis to the centroid of the area A_C.
I = Second moment of area of the whole transformed composite section.

The design of a composite bridge deck using steel plate girders with cast *in-situ* reinforced concrete slab using stud connectors is presented in the following example.

9.3 DESIGN EXAMPLE

Design a composite bridge deck with reinforced concrete slab and steel plate girders to cover a span of 18 m.
Clear width of road way = 7.5 m
Footpath: 1 m on either side
Spacing of main girders = 2 m
Materials: concrete M-20 grade and Fe-415 grade tor steel, rolled steel sections with yield stress of 236 N/mm^2.
Design the reinforced concrete deck slab and the steel plate girders with shear connectors.

Draw the following views to a suitable scale:

(a) The cross-section of the deck slab continuous over steel girders and cross-section of the steel girders.

(b) Longitudinal elevation of the steel girders showing the details of the shear connectors.

1. Cross-section the Deck

The cross-sectional details of the deck slab assumed are as shown in Fig. 9.2

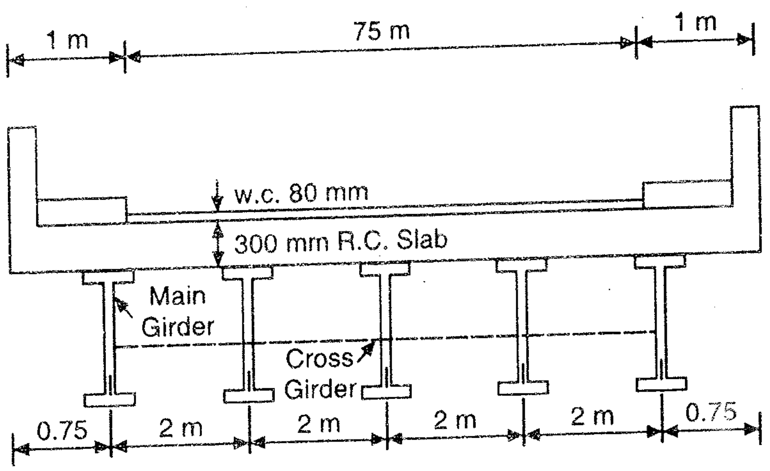


Fig. 9.2 Cross-section of Deck Slab.

2. Design of Deck Slab

Panel dimensions = 2 m by 4.5 m

Dead weight of slab = (0.3×24) = 7.20 kN/m^2

Dead weight of W.C. = (0.08×22) = 1.76

Total load = 9.00 kN/m^2

3. Live Load B.M.

Live load is I.R.C. Class AA tracked vehicle

Referring to Fig. 9.3

$u = (0.85 + 2 \times 0.08) = 1.01$ m

$v = (3.6 + 2 \times 0.08) = 3.76$ m

$(u/B) = (1.01/2.0) = 0.50$

$(v/L) = (3.76/4.50) = 0.83$

$K = (B/L) = (2.0/4.5) = 0.45$

From Pigeaud's curves (Refer Fig. 3.3)

for $K = 0.5$ read out the values of

$$m_1 = 0.085 \text{ and } m_2 = 0.017$$

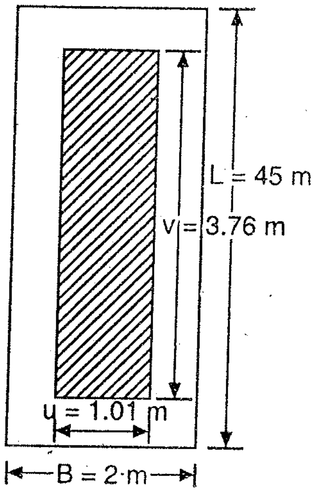


Fig. 9.3 I.R.C. Class A Wheel Load.

Short span moment $M_B = W(m_1 + 0.15\ m_2)$

$= 350\ (0.085 + 0.15 \times 0.017) = 31$ kN·m

B.M. including impact and continuity factor

$= (1.25 \times 0.8 \times 31) = 31$ kN·m

Long span moment $M_L = W(m_2 + 0.015\ m_1)$

$= 350\ (0.017 + 0.015 \times 0.085) = 10.5$ kN·m

B.M. including impact and continuity factor

$= (1.25 \times 0.8 \times 10.5) = 10.5$ kN·m

4. Dead Load B.M.

Dead load of deck slab = 9 kN/m^2

Total dead load/panel = $(9 \times 2 \times 4.5) = 81$ kN

$(u/B) = 1$ and $(v/L) = 1$

$K = (B/L) = (2/4.5) = 0.445\ (1/K) = 2.25$

Using Pigeaud's curves (Refer Fig. 3.9), read out

$m_1 = 0.047$ and $m_2 = 0.006$

$M_B = 81\ (0.047 + 0.15 \times 0.006) = 3.94$ kN·m

Taking continuity into account

$M_B = (0.08 \times 3.94) = 3.15$ kN·m

$M_L = 81\ (0.006 + 0.15 \times 0.047) = 1.05$ kN·m

Taking continuity into account

$M_L = (0.8 \times 1.05) = 0.84$ kN·m

The design bending moments are

$$M_B = (31 + 3.15) = 34.15 \text{ kN·m}$$
$$M_L = (10.5 + 0.84) = 11.34 \text{ kN·m}$$

5. Design of Section

For M-20 grade concrete and Fe-415 grade tor steel,

$$Q = 0.762, \quad j = 0.91, \quad \sigma_{st} = 200 \text{ N/mm}^2$$

$$d = \sqrt{(M / Q \cdot b)} = \sqrt{(34.15 \times 10^6) / 0.762 \times 10^3} = 212 \text{ mm}$$

Overall depth = 300 mm
Effective depth = 260 mm

$$A_{st} = [(34.15 \times 10^6)/(200 \times 0.91 \times 260)]$$
$$\text{(Short span)} = 722 \text{ mm}^2$$

Provide 12 mm $\bar{\phi}$ bars at 140 mm centres.

Effective length for long span using 10 mm $\bar{\phi}$ bars.

$$= (260 - 6 - 5) = 249 \text{ mm.}$$
$$A_{st} = [(11.34 \times 10^6)/(200 \times 0.91 \times 249)]$$
$$\text{(Long span)} = 250 \text{ mm}^2$$

Provide 10 mm $\bar{\phi}$ bars at 150 mm centres.

6. Design of steel plate girder

Spacings of main girders	= 2m
Spacings of cross girders	= 4.5 m
Dead load on girder = (9 × 2)	= 18 kN/m
Self-weight of main girder (0.2L + 1) kN/m	= 4 kN/m
Total load = W	= 22 kN/m

Self-weight of cross girders (assumed as 1 kN/m)

$$= (2 \times 1) = 2 \text{ kN.}$$

(a) Dead Load Moments
Referring to Fig. 9.4

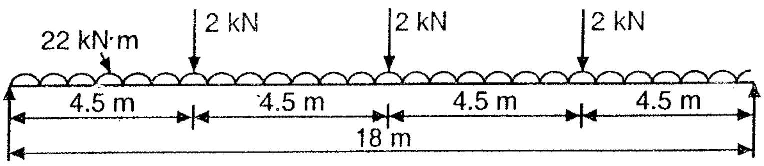


Fig. 9.4 Dead Loads on Plate Girder.

The maximum dead load moment is computed as

$$M_{max} = [(22 \times 18^2)/8 + (2 \times 18)/4 + (2 \times 4.5)] = 909 \text{ kN·m}$$

(b) Live Load Moments

Referring to Fig. 9.5

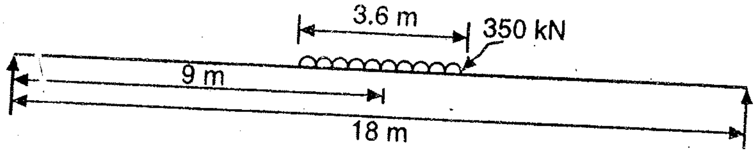


Fig. 9.5 Live Load on Plate Girder.

The maximum live load moment is computed as

$$M_{max} = [(350 \times 9)/2 - (350 \times 0.9)/2] = 1418 \text{ kN·m}$$

Impact factor = 10 percent

Live load B.M = (1418 × 1.1)	= 1560 kN·m
Dead load B.M	= 909
Design B.M	= 2470 kN·m

(c) Shear Forces

Dead load shear = [(22 × 18)/2 + 2 + 2/2] = 201 kN

Live load shear with impact factor

= 1.1 [(350 × 16.2 × 18] = 347 kN

Total design shear = V = (201 + 347) = 548 kN.

(d) Proportioning of Trial Section of Web Plate

Approximate depth of girder = 1/8 to 1/10 span = 18/10 = 1.8 m

Economical depth

$$= 5^3\sqrt{M/\sigma_b} = 5^3\sqrt{2470 \times 10^6/165} = 1230 \text{ mm}$$

Web depth based on shear considerations assuming 10 mm thick plate is,

$$d = V/(\tau \times 0) = [(548 \times 10^3)/(85 \times 10)] = 644.7 \text{ mm}$$

Try web 1000 mm by 10 mm.

(e) Flange Plates

Approximate flange area required

$A_f = [(M/\sigma_b \cdot d) - (A_w/6)] = [(2470 \times 10^6)/(165 \times 1000) - (10 \times 1000)/6]$
$= 13303 \text{ mm}^2$

Flange width $B = L/40$ to $L/45$ = 450 mm to 400 mm

Thickness of plate = (13303/500) = 26.6 mm

Adopt flange plates 500 mm by 30 mm.

The section selected is shown in Fig. 9.6

(f) Check for Maximum Stresses

$$I = [(10 \times 1000^3)/12 + 2\ (30 \times 500)\ 515^2]$$
$$= 879 \times 10^7\ \text{mm}^4$$

Bending tensile stress $= \sigma_b = (My/I)$

$$= (2470 \times 10^6 \times 530)/(879 \times 10^7)$$
$$= 149\ \text{N/mm}^2 < 165\ \text{N/mm}^2$$

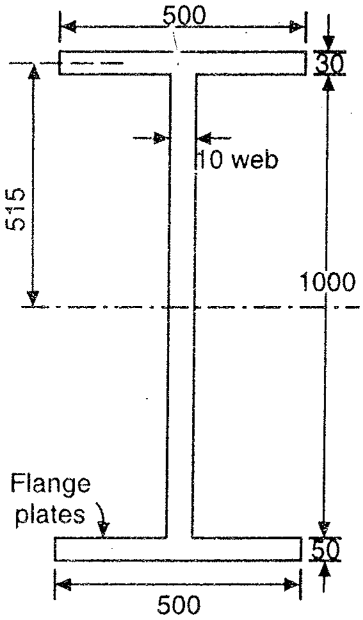


Fig. 9.6 Cross-section of Plate Girder.

Average shear stress $= (548 \times 10^3)/(1000 \times 10) = 55\ \text{N/mm}^2$

Permissible average shear stress depends upon the ratio of $(d/t) = (1000/10) = 100$

Using stiffener spacing $c = 1000\ \text{mm} = d$

From Table 8.2, allowable average shear stress is 87 N/mm^2

Hence the average shear stress is within safe permissible limits.

(g) Connection between Flange and Web

Maximum shear force at the junction of web and flange is given by,

$$\tau = (Va\bar{y}/I)$$

where $V = 548 \times 10^3$ N

$a = (500 \times 30) = 15{,}000\ \text{mm}^2$

$I = 879 \times 10^7\ \text{mm}^4$

$\bar{y} = 515$ mm

$\tau = [(548 \times 10^3 \times 15 \times 10^3 \times 515)/(879 \times 10^7)] = 483$ N/mm 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

$= 1200 \times 10^3/3.8 = 315{,}789.47 \text{ mm}^2 \approx 320{,}000 \text{ mm}^2$

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,

$$\text{Radius of the rocker} = 200 \text{ mm}$$

Permissible bearing stress

$= \sigma_p = 189$ N/mm^2 (IRC: 24)

$\therefore$ Bearing area required $= [(548 \times 10^3)/189] = 2900$ mm^2

If two plates are used

Total area provided $= (2 \times 180 \times 15)$

$= 5400$ mm^2 $>$ 2900 mm^2

The length of web plate which acts along with stiffener plates in bearing the reaction $= 20t = (20 \times 10) = 200$ mm

$$I = [(15 \times 370^3)/12 + (2 \times 200 \times 10^3)/12]$$
$$= 6334 \times 10^4 \text{ mm}^4$$

Area $\quad A = [(360 \times 15) + (400 \times 10)] = 9400$ mm^2

λ = Slenderness Ratio $= (L/r)$

$$r = \sqrt{I/A} = \sqrt{(6334\pi\, 10^4)/9400} = 82 \text{ mm}$$

Effective Length of Stiffener $= (0.7 \times 1000) = 700$ mm

$$\lambda = (700/82) = 8.53$$

From Table 8.3 (IRC: 24),

Permissible stress σ_{ac} in axial compression is obtained as 138 N/mm^2

$\therefore$ Area required $= [(548 \times 10^3)/138] = 3971$ mm^2 9400 mm^2

(k) Connection between Bearing Stiffener and Web

length available for welding using alternate intermittent welds

$= 2(1000 - 40) = 1920$ mm

Required strength of weld $= [(548 \times 10^3)/1920] = 286$ N/mm

Size of weld $= [286/(0.7 \times 102.5)] = 3.98$ mm

Use 5 mm fillet weld

Length of weld $\not< 10\,t \not< (10 \times 10) = 100$ mm

Use 100 mm long 5 mm welds alternately.

(l) Properties of the Composite Section[6]

Referring to Fig. 9.7

$A_{ce} = [(2000 \times 300)/13] = 46154$ mm^2

Modular ratio $= m = 13$

The centroid of the composite section is determined by first moment of the areas about the axis XX.

$$A\,\bar{y} = [(46154 \times 1210) + (500 \times 30 \times 1045) + (1000 \times 10 \times 530) + (500 \times 30 \times 15)]$$
$$= 77046340$$
$$A = [46154 + (500 \times 30) + (1000 \times 10) + (500 \times 30)]$$
$$= 86154 \text{ mm}^2$$

Assuming continuous weld on either side, strength of weld of size 's' is
$= (2 \times 0.7 \times s \times 102.5) = 145\,s$

$\therefore \quad 145\,s = 483,\ s = 3.33$ mm

Use 5 mm fillet weld, continuous on either side,

(h) Intermediate Stiffeners

Since $(d/t) = (1000/10) = 100 < 85$

Vertical Stiffeners are required.

Spacing of Stiffeners $= 0.33\,d$ to $1.5\,d$

$= (0.33 \times 1000)$ to (1.5×1000)

$= 333$ mm to 1500 mm

Adopt 1000 mm spacing. Hence $c = 1000$ mm

The intermediate stiffeners are designed to have a minimum moment of inertia of,

$$I = [(1.5\,d^3t^3)/c^2] = [(1.5 \times 1000^3 \times 10^3)/1000^2] = 15 \times 10^5 \text{ mm}^4$$

Using 10 mm thick plate

Maximum width of plate not to exceed 12t for flats

Use a plate 10 mm × 80 mm $h = 80$ mm

$$I = [(10 \times 80^3)/3] = 17 \times 10^5 \text{ mm}^4 > 15 \times 10^5 \text{ mm}^4$$

(i) Connection of Vertical Stiffener to Web

Shear on welds connecting stiffener to web

$$= [(125 \times t^2)/h] \text{ kN/m}$$

where t = web thickness (mm)

h = outstand of stiffener (mm)

Shear on welds $= [(125 \times 10^2)/80] = 156.25$ kN/m $= 156.25$ N/mm

Size of welds $= [156.25/(0.7 \times 102.5)] = 2.17$ mm

Use 5 mm minimum size intermittent welds.

Effective length of weld $\not< 10\,t \not< (10 \times 10) = 100$ mm

Use 100 mm long, 5 mm fillet welds alternately on either side.

(j) End Bearing Stiffener

Maximum shear force = 548 kN

The end bearing stiffener is designed as a column

$$(h/t) \not> 12$$

where h = outstand

t = thickness

If $h = 180$ mm, $\quad t = (180/12) = 15$ mm

Use 180 mm by 15 mm size plate

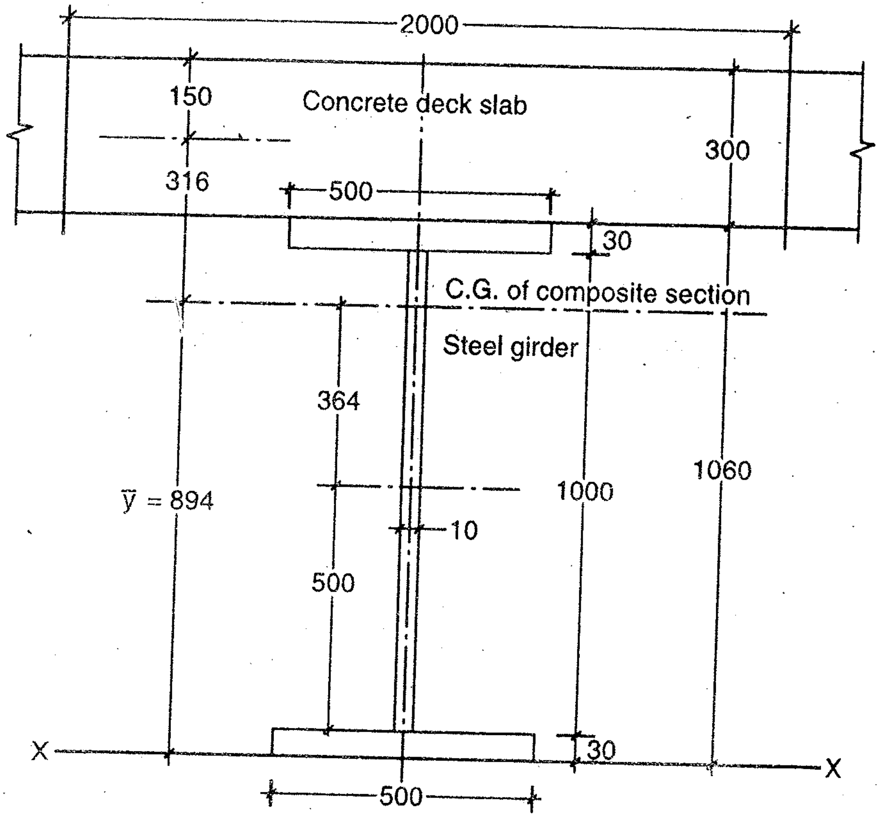


Fig. 9.7 Properties of Composite Section.

$\therefore \quad \bar{y} = (77046340/86154) = 894 \text{ mm}$

$I_{\text{comp}} = [(46154 \times 316^2) + (500 \times 1060^3)/12 - (400 \times 1000^3)/12 + (40000 \times 364^2)]$

$= 1.957 \times 10^{10} \text{ mm}^4$

Maximum shear force at junction of slab and girder is given by

$$\tau = (V a \bar{y} / I)$$

where

$V = 548$ kN

$a = 46154 \text{ mm}^2$

$I = 1.957 \times 10^{10} \text{ mm}^4$

$\bar{y} = 316$ mm

$\therefore \quad \tau = [(548 \times 10^3 \times 46154 \times 3160)/(1.957 \times 10^{10})]$

$= 408$ N/mm

Total shear force at junction = (408 × 500) = 204000 N

Using 20 mm diameter mild steel studs, capacity of one shear connector is given by

$$Q = 196\, d^2 \sqrt{f_{ck}}$$

where $\quad H = 5d - (5 \times 20) = 100$ mm

$d = 20$ mm

$f_{ck} = 20$ N/mm^2

$Q = 196 \times 20^2 \sqrt{20} = 350615$ N

Number of studs required in one row = (204000/350615) < 1

Provide a minimum of 2 mild steel studs in a row

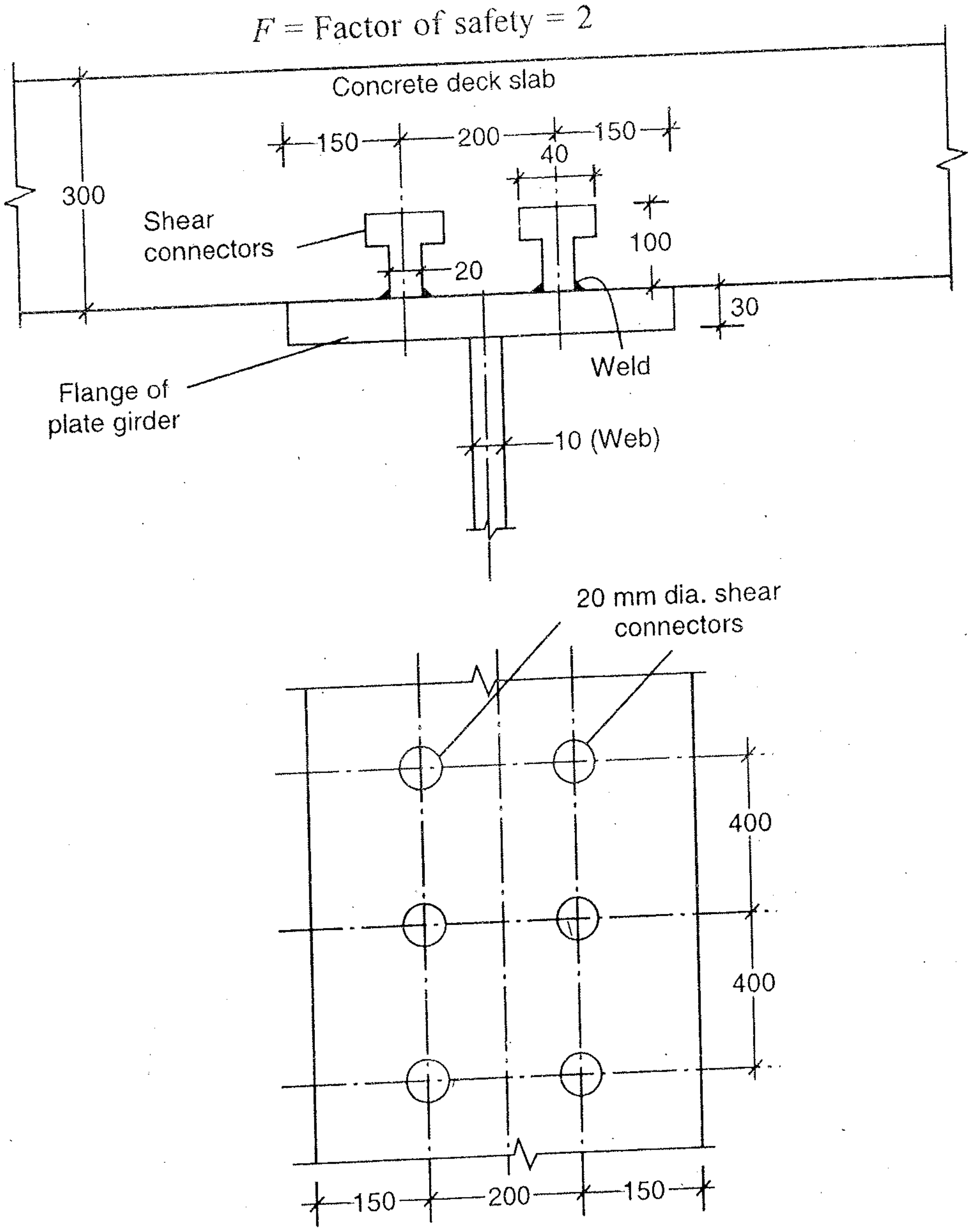
(m) Pitch of shear connectors = $p = [(NQ/(F\tau)]$

where $\quad N$ = Number of shear connectors in a row

Q = Capacity of one shear connector

τ = Horizontal shear per unit length

F = Factor of safety = 2



9.8 Details of Shear Connectors.

10 Prestressed Concrete Bridges

10.1 GENERAL ASPECTS

Prestressed Concrete is ideally suited for the construction of medium and long span bridges. Ever since the development of Prestressed Concrete by Freyssinet[1] in the early nineteen thirties, the material has found extensive application in the construction of long span bridges[2] gradually replacing steel which needs costly maintenance due to the inherent disadvantages of corrosion under aggressive atmospheric conditions.

Solid slabs are used for the span range of 10 to 20 m while Tee beam slab decks are suitable for spans in the range of 20 to 40 m. Single or multicell box girders are preferred for larger spans of the order of 30 to 70 m. Prestressed Concrete[3] is ideally suited for long span continuous bridges in which precast box girders of variable depth are used for span exceeding 50 m. Prestressed Concrete, has been widely used throughout the world for simply supported, continuous, balanced cantilever, suspension, hammer head and bridle chord type bridges in the span range from 20 to 500 m.

10.2 ADVANTAGES OF PRESTRESSED CONCRETE BRIDGES

Prestressed Concrete made up of high strength concrete[4] and high tensile steel[5] has distinct advantages when used for bridge construction. The salient benefits resulting from the use of prestressed concrete in bridges are out lined as follows:

1. The use of high strength concrete and high tensile steel results in slender sections which are aesthetically superior coupled with overall economy.

2. Prestressed concrete bridges can be designed as class 1 type structures without any tensile stresses under service loads resulting in a crack free structure.
3. In comparison with steel bridges, prestressed concrete bridges require very little maintenance.
4. Prestressed concrete is ideally suited for composite bridge construction in which precast prestressed girders support the cast *in-situ* slab deck. This type of construction is very popular since it involves minimum disruption of traffic.
5. Post tensioned prestressed concrete finds extensive applications in long span continuous girder bridges of variable cross section resulting in sleek structures and with considerable savings in the overall cost of construction.
6. In recent years, partially prestressed concrete[6] (type 3 structure) is preferred for bridge construction with considerable savings in the quantity of costly high tensile steel used in the girder.

10.3 PRE-TENSIONED PRESTRESSED CONCRETE BRIDGES

Pretensioned prestressed concrete bridge decks generally comprise precast pretensioned units used in conjunction with cast *in-situ* concrete resulting in composite bridge decks ideally suited for small and medium spans in the range of 20 to 30 m. In general, pretensioned girders are provided with straight tendons. The use of seven wire strands have been found to be advantageous in comparison with plain or indented wires. Deflected strands are employed in larger girders in U.S.A.

In U.K., the precast prestressed I and inverted T-beams have been standardized by the Cement and Concrete Association for the use in the construction of bridge decks of spans varying from 7 to 36 m. Standard I and T units are widely employed in high way bridge beams in U.S.A. Recently in U.K., Y-beams have been developed to replace the M-beams introduced in 1960. The design and development of the Y-beams which are superior to M-beams are ideally suited for medium spans of 15 to 30 m.

The typical cross section of the standard inverted Y-beams[7] developed by the research group in U.K. is shown in Fig 10.1 and the section properties of the Y-beam are compiled in Table 10.1. The salient features of composite bridge decks with precast pretensioned standard beams are shown in Fig. 10.2.

Table 10.1 Section Properties of Standard Y-Beams (U.K)

Section	Depth	Area	Height of Centroid above Soffit Y_b	Section Modulus Top fibre Z_t	Section Modulus Bottom fibre Z_b	Approximate Self Weight
	(mm)	(mm^2)	(mm)	($mm^3 \times 10^6$)	($mm^3 \times 10^6$)	kN/m
Y-1	700	309202	255.24	24.85	43.40	7.42
Y-2	800	339882	298.68	35.02	58.78	8.14
Y-3	900	373444	347.12	47.88	76.27	8.95
Y-4	1000	409890	399.71	63.53	95.41	9.82
Y-5	1100	44920	455.72	82.06	116.02	10.78
Y-6	1200	491433	514.50	103.58	138.00	11.78
Y-7	1300	536530	575.54	128.15	161.31	12.86
Y-8	1400	584708	638.54	155.98	186.01	14.02

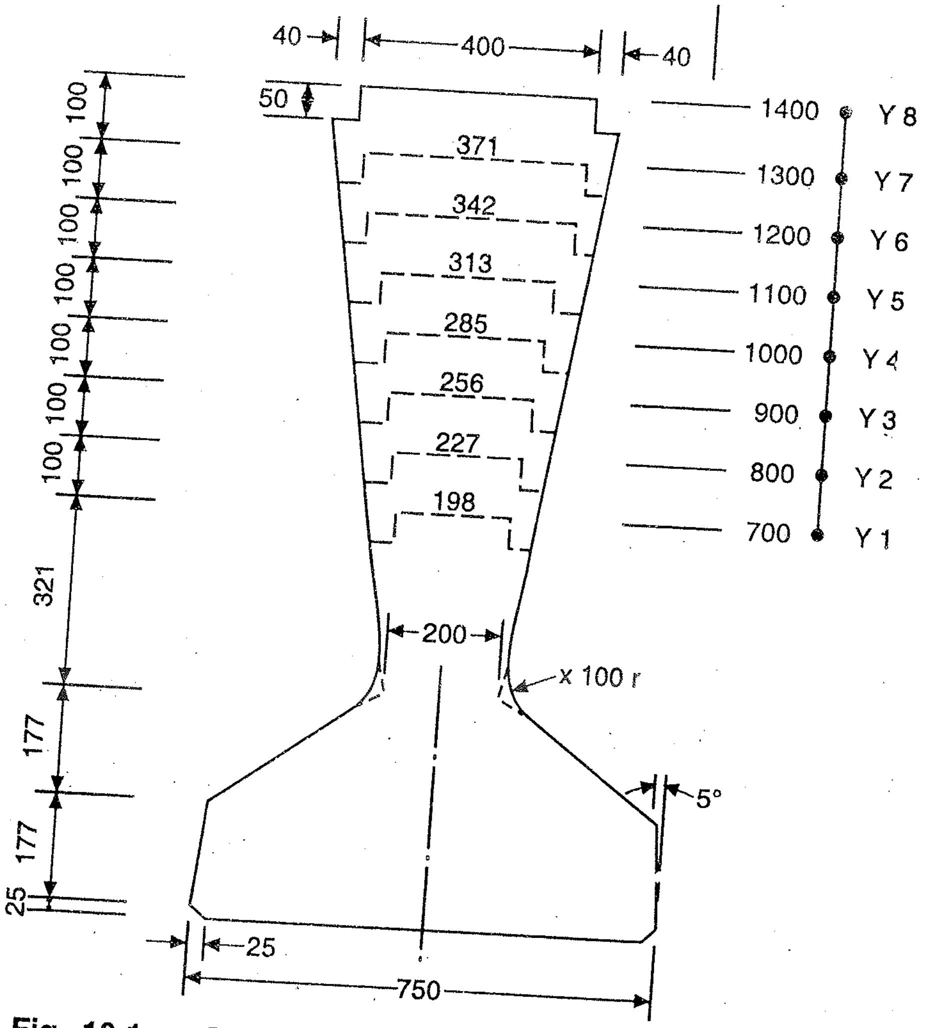


Fig. 10.1 Cross Section of Standard Y-Beams (U.K.).

10.4 POST-TENSIONED PRESTRESSED CONCRETE BRIDGE DECKS

Post tensioned bridge decks are generally adopted for longer spans exceeding 20 m. Bridge decks with precast post tensioned girders of either Tee type or box type in conjunction on with a cast *in-situ* slab is commonly adopted for spans exceeding 30 m. Post tensioning facilitates the use of curved cables which improve the shear resistance of the girders.

Post tensioning is ideally suited for prestressing long span girders at the site of construction without the need for costly factory type installations like pretensioning beds. Segmental construction[8] is ideally suited for post tensioning work. In this method a number of segments can be combined by prestressing,

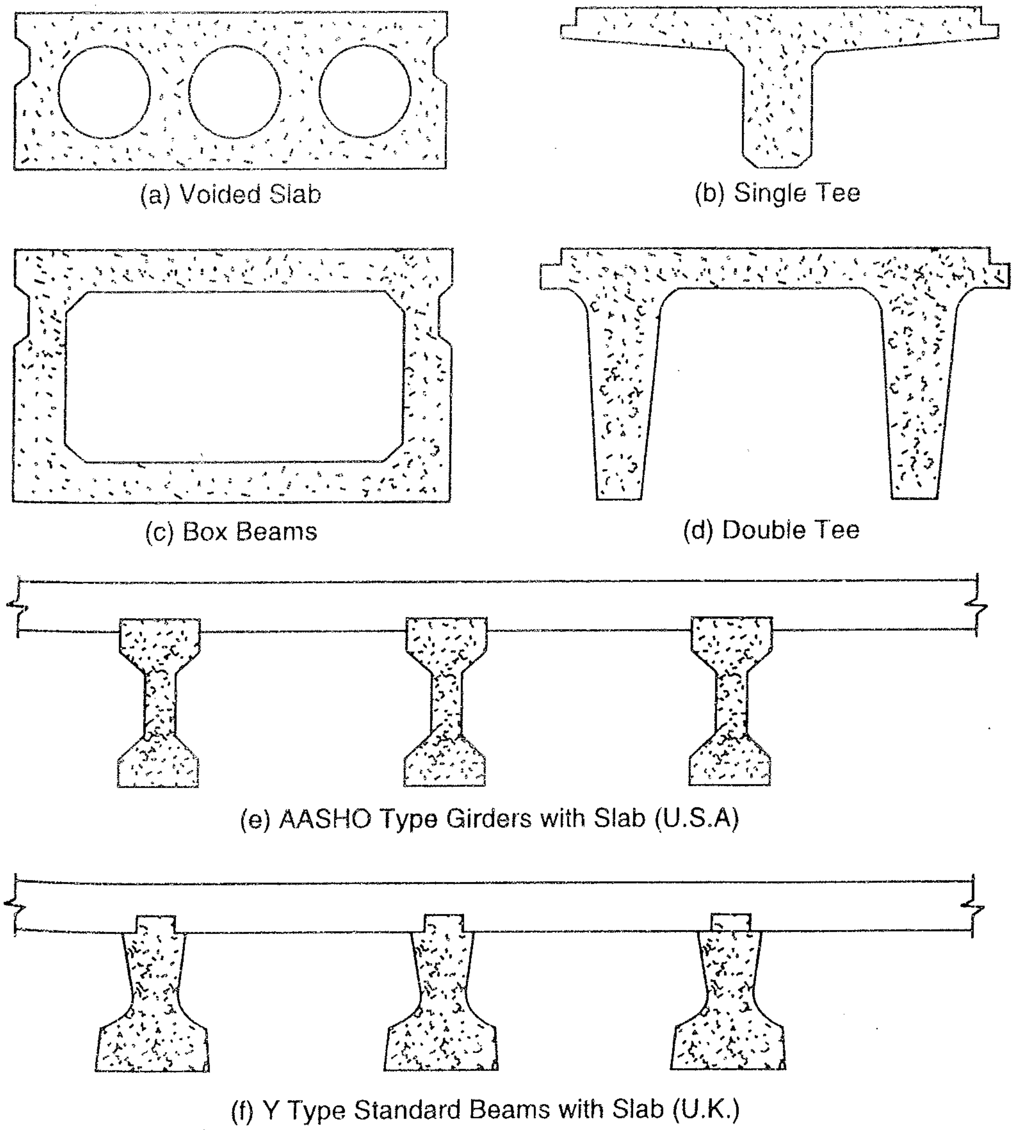
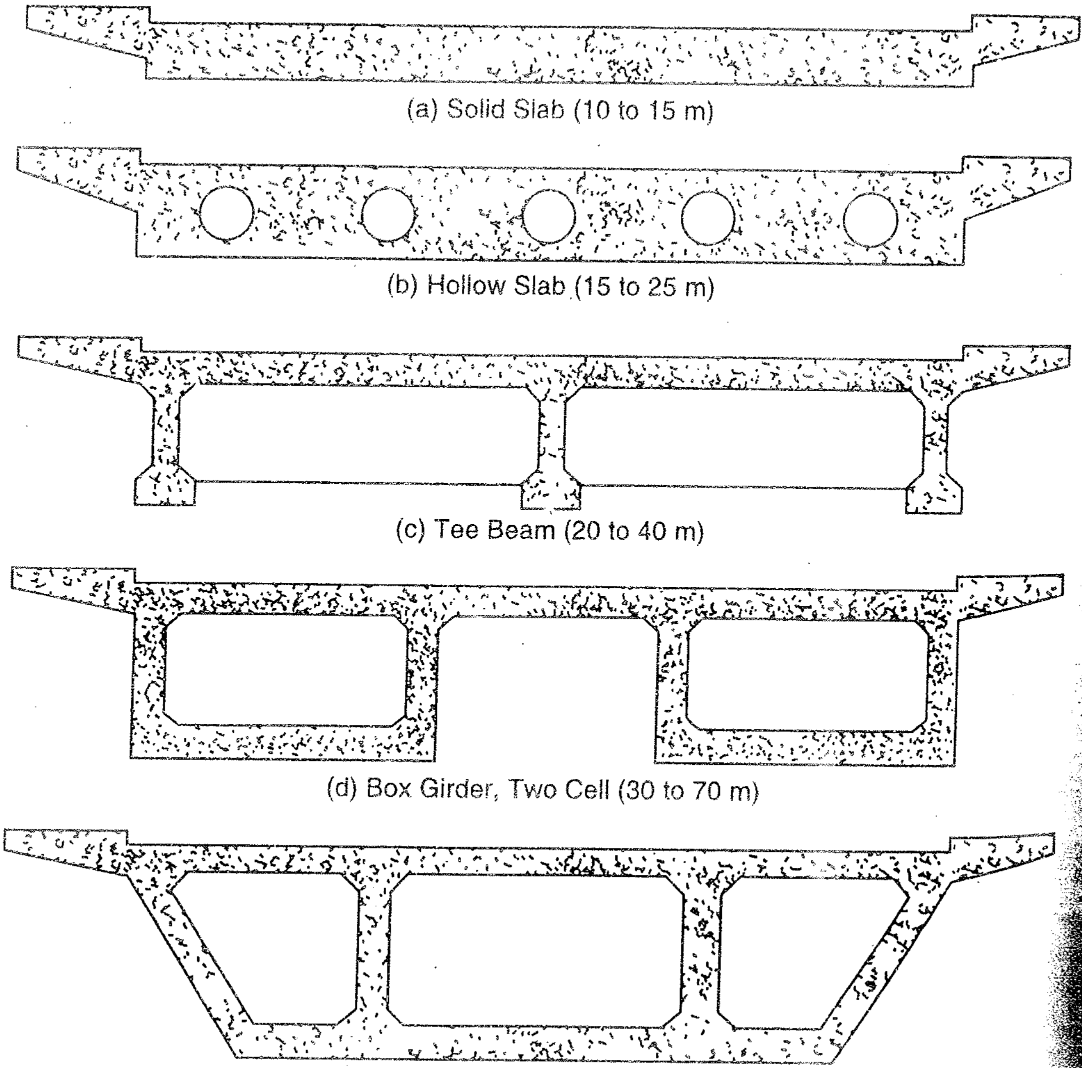


Fig. 10.2 Typical Cross Sections of Pretensioned Prestressed Concrete Bridge Decks.

resulting in an integrated structure. In India a large number of long span bridges have been constructed using the cantilever method of construction.[9] Some of the notable examples being the Barak bridge at Silchar built in 1960 with a main span of 130 m and the Lubha bridge in Assam with a span of 130 m between the bearings. Long span continuous prestressed concrete bridges are invariably built up of multicelled box girder segments of variable depth using the post tensioning system. Typical cross sections of post tensioned prestressed concrete bridge cocks are shown in Fig. 10.3. The salient features of cantilever construction method using cast *in-situ* segments and precast concrete elements are shown in Figs. 10.4(a) and (b).



(a) Solid Slab (10 to 15 m)

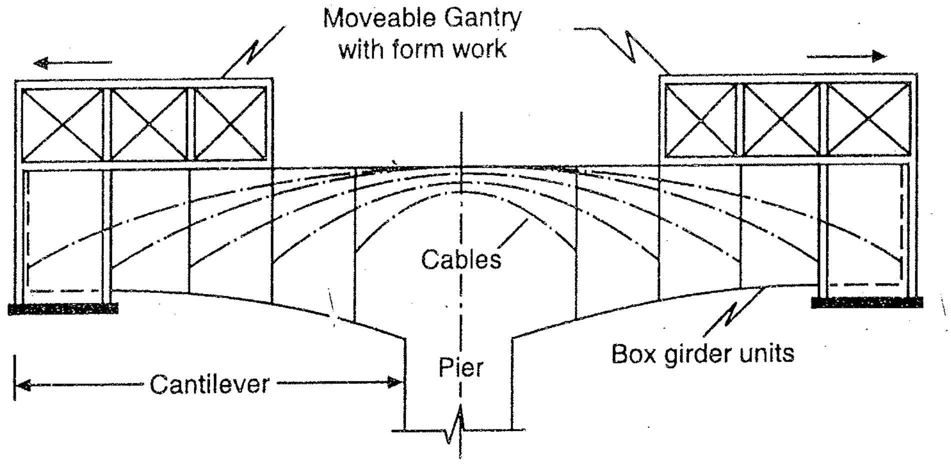
(b) Hollow Slab (15 to 25 m)

(c) Tee Beam (20 to 40 m)

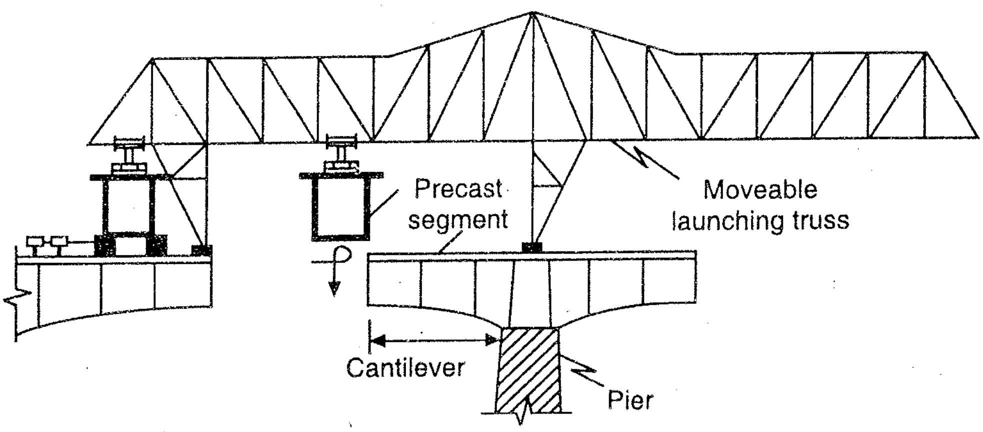
(d) Box Girder, Two Cell (30 to 70 m)

(e) Box Girder, Trapezoidal (30 to 80 m)

Fig. 10.3 Typical Cross Sections of Post-tensioned Prestressed Concrete Bridge Decks.



(a) Using Cast *in-situ* Segments



(b) Using Precast Girder Segments

Fig. 10.4 Cantilever Method of Construction of Prestressed Concrete Bridges.

10.5 DESIGN OF POST-TENSIONED PRESTRESSED CONCRETE SLAB BRIDGE DECK

Design a post tensioned prestressed concrete slab bridge deck for a National highway crossing to suit the following data:—

1. Data

Clear span	10 m
Width of bearing	400 m
Clear width of Roadway	7.5 m
Foot path	1m on either side
Kerbs	600 mm wide
Thickness of wearing coat	80 mm

Live laod	I.R.C. Class AA tracked vehicle
Type of structure	Class 1 type

Materials: M–40 grade concrete and 7 mm diameter High tensile wires with an ultimate tensile strength of 1500 N/mm^2 housed in cables with 12 wires and anchored by Freyssinst anchorages of 150 mm diameter.

For supplementary reinforcement, adopt Fe–415 grade HYSD bars.

Compressive strength at transfer (f_{ci}) = 35 N/mm^2

Loss Ratio = 0.8

The design should conform to the recommendations of the codes IRC–6, IRC–18[10], IRC–21[11] and IS: 1343[12]

2. Permissible stresses

The permissible compressive stresses in concrete at transfer and working loads as recommended in IRC–18 are as follows:

f_{ct} = 15 N/mm^2 < 0.45 f_{ci} = (0.45 × 35) =15.75 N/mm^2

Loss ratio η = 0.80

f_{cw} = 12 N/mm^2 < 0.33 f_{ck} = (0.33 × 40) =13.2 N/mm^2

$f_{tt} = f_{tw}$ = 0.

3. Depth of slab and Effective span

Assuming the thickness of slab at 50 mm per metre of span for high way bridge decks, overall thickness of slab = (10 × 50) = 500 mm.

Width of bearing = 400 mm

Effective span = 10.4 m

The cross section of deck slab is shown in Fig. 10.5.

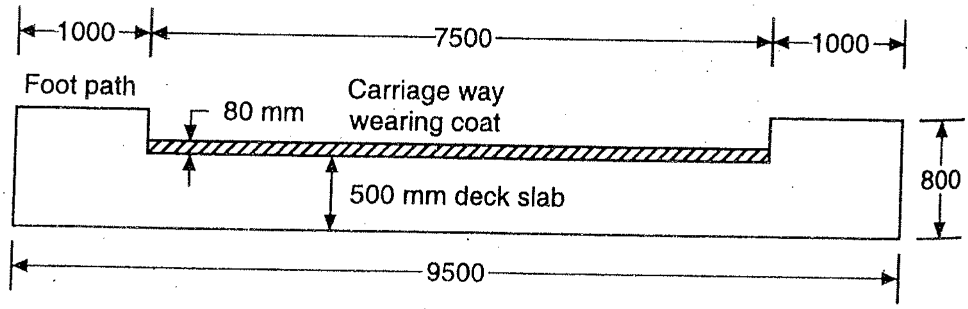


Fig. 10.5 Cross Section of Deck Slab.

4. Dead load Bending Moments

Dead weight of slab = (0.5 × 24) = 12 kN/m^2

Dead weight of W.C. = (0.08 × 22) = 1.76 kN/m^2

Total dead load = 14.00 kN/m^2

Dead load bending moment (M_g) = (14 × 10.4^2)/8 = 190 kN·m

5. Live load Bending Moments

Generally the bending moment due to live load will be maximum for I.R.C. Class AA tracked vehicle. Impact factor for class AA tracked vehicle is 25% for 5 m span, decreasing linearly to 10% for 9 m span.

∴ Impact factor = 10 percent for a span of 10.4 m.

The tracked vehicle is placed symmetrically on the span.
Effective length of load = [3.6 + 2 (0.5 + 0.08)] = 4.76 m
Effective width of slab perpendicular to span is expressed as,

$$b_e = k \cdot x\,(1 - x/L) + b_w$$

Referring to Fig. 10.6.

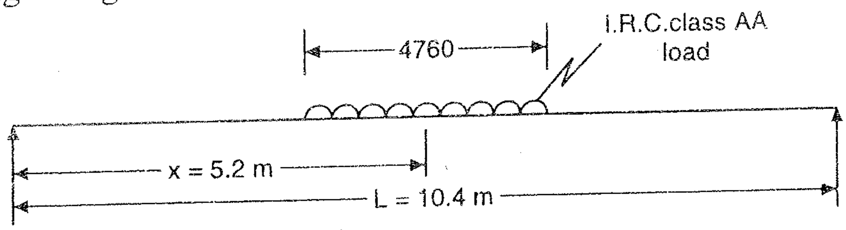


Fig. 10.6 Position of Load for Maximum Bending Moment.

x = 5.2 m, L = 10.4 m, B = 9.5 m

(B/L) = (9.5/0.4) = 0.913

b_w = (0.85 + 2 × 0.08) = 1.01 m

From Table 3.5 for (B/L) = 0.913, simply supported slabs, k = 2.37

∴ b_e = 2.37 × 5.2 (1 − 5.2/10.4) + 1.01 = 7.172 m

The tracked vehicle is placed close to the kerb with the required minimum clearance as shown in Fig. 10.7.
Net effective width of dispersion = 8.261 m
Total load of two tracks with impact = (700 × 1.10) = 770 kN
Average intensity of load = 770/(4.76 × 8.261) = 19.58 kN/m^2
Maximum bending moment due to live load is given by

$$M_q = [(19.58 \times 4.76)\ 0.5 \times 5.2] - [(19.58 \times 4.76)\ 0.5 \times 0.25 \times 4.76]$$
$$= 187 \text{ kN·m}$$

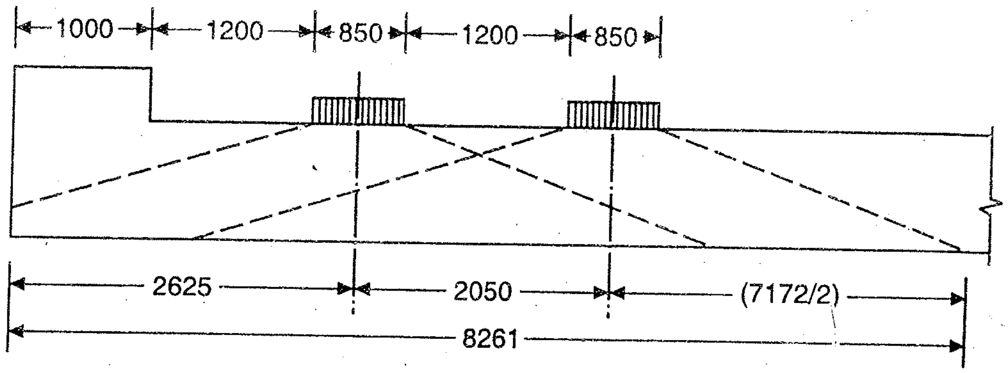


Fig. 10.7 Effective width of Dispersion for IRC Class AA, Tracked Vehicle.

6. Shear due to Class AA Tracked Vehicle

For maximum shear force at support section; the I.R.C. Class AA tracked vehicle is arranged as shown in Fig. 10.8

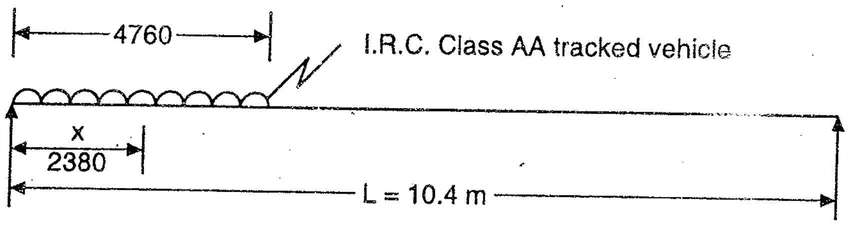


Fig. 10.8 Position of Load for Maximum Shear.

Effective width of dispersion is given by

$b_e = k.x\ (1 - x/L) + b_w$

where $x = 2.38\ m$, $L = 10.4\ m$, $B = 9.5\ m$, $b_w = 1.01\ m$

(B/L) = (9.5/10.4) = 0.913

∴ From table 3.5 for (B/L) = 0.913 the value of $k = 2.37$

∴ $b_e = [2.37 \times 2.38\ (1 - 2.38/10.4) + 1.01] = 5.364$ m

Referring to Fig. 10.7

width of dispersion for two tracks

= [2625 + 2050 + (5364/2)]

= 7357 mm

∴ Intensity of load = [770/(4.76 × 7.357)]

= 22 kN·m²

∴ Shear force V_A = (22 × 4.76 × 8.02)/10.4

= 80.75 kN

Dead load shear = (0.5 × 14 × 10.4)

= 72.8 kN

∴ Total Design shear = (80.75 + 72.80)

= 153.55 kN.

7. Check for Minimum Section Modulus

Dead load Moment $\quad M_g = 190$ kN·m

Live load moment $\quad M_q = 1\ 87$ kN·m

$$\text{Section Modulus} = Z_t = Z_b = Z = \left(\frac{1000 \times 500^2}{6}\right) = 41.66 \times 10'' \text{ mm}^3$$

The permissible stresses in concrete at transfer (f_{ct}) is obtained from IRC – 18

f_{ct} = 15.0 N/mm², f_{cw} = 12.0 N/mm², f_{tw} = 0,

η = Loss Ratio = 0.8, $f_{br} = (\eta f_{ct} - f_{rw}) = (0.8 \times 15 - 0)$

= 12.0 N/mm²

The minimum section Modulus is given by

$$Z_b \geq \left[\frac{M_q + (1-\eta)M_g}{f_{br}}\right]$$

$$\geq \left[\frac{187 \times 10^6 + (1-0.8)190 \times 10^6}{12}\right]$$

$$\geq 18.75 \times 10^6 \text{ mm}^3 < 41.66 \times 10^6 \text{ mm}^3$$

Hence the section selected is adequate to resist the service loads without exceeding the permissible stresses.

8. Minimum Prestressing Force

The minimum prestressing force required is computed using the relation

$$P = \left[\frac{A(f_{inf} \cdot z_b + f_{sup} \cdot z_t}{z_b + z_t}\right]$$

Where

$$f_{sup} = \left(f_{tt} - \frac{M_g}{z_t}\right) = \left(0 - \frac{190 \times 10^6}{41.66 \times 10^6}\right) = -4.56 \text{ N/mm}^2$$

$$f_{inf} = \left(\frac{f_{tw}}{\eta} + \frac{M_q + M_g}{\eta z_b}\right) = \left[0 + \frac{(187+190)10^6}{0.8 \times 41.66 \times 10^6}\right]$$

$$= 11.31 \text{ N/mm}^2$$

$$\therefore \quad p = \left[\frac{1000 \times 500 \times 41.66 \times 10^6 (11.31 - 4.56)}{2 \times 41.66 \times 10^6}\right]$$

$$= 1687.5 \times 10^3 \text{ N}$$

$$= 1687.5 \text{ kN.}$$

Using Freyssinet cables containing 12 wires of 7 mm diameter stressed to 1200 N/mm², Force in each cable =

$$(12 \times 38.5 \times 1200)/1000 = 554 \text{ kN}$$

$$\therefore \quad \text{Spacings of cables} = \left(\frac{1000 \times 554}{1687.5}\right) = 328 \text{ mm}$$

9. Eccentricity of Cables

The eccentricity of the cables at the centre of span is obtained from the relation:

$$e = \left[\frac{z_t z_b (f_{inf} - f_{sup})}{A(f_{sup} z_t + f_{inf} z_b)}\right]$$

$$= \left[\frac{(41.66)^2 \times 10^{12}(11.31 + 4.56)}{1000 \times 500 \times 41.66 \times 10^6(-4.56 + 11.31)}\right]$$

$$= 195 \text{ mm}$$

The cables are arranged in a parabolic profile with a maximum eccentricity of 195 mm at centre of span reducing to zero eccentricity (Concentric) at supports.

10. Check for Stresses at Service Loads

$P = 1687.5$ kN, $e = 195$ mm

$A = (1000 \times 500) = 5 \times 10^5$ mm²

$Z_t = Z_b = Z = 41.66 \times 106$ mm3

$M_g = 190$ kN·m $\quad M_q = 187$ kN·m

$(P/A) = (1687.5\ 10^3/5 \times 10^5) = 3.375$ N/mm²

$(P_e/Z) = (1687.5 \times 10^3 \times 195/41.66 \times 10^6) = 7.89$ N/mm²

$(M_g/Z) = (190 \times 10^6/41.66 \times 10^6) = 4.56$ N/mm²

$(M_q/Z) = (187 \times 10^6/41.66 \times 10^6) = 4.48$ N/mm²

Stresses at transfer

At top of slab = (3.375 – 7.89 + 4.56) = 0.045 N/mm²

At bottom of slab = (3.375 + 7.89 – 4.56) = 6.705 N/mm²

Stresses at Working loads

At top of slab = 0.8 (3.375 - 7.89) + 4.56 + 4.48 = 5.428 N/mm²

At bottom of slab = 0.8 (3.375 + 7.89) – 4.56 – 4.48

= – 0.028 N/mm²

The actual stresses developed are within the permissible limits.

11. Check for Ultimate Strength (IRC: 18–2000)

Considering 1 m width of slab,

b = 1000 mm

A_p = (12 × 38.5 × 1000)/328 = 1408 mm^2

d = 445 mm, f_p = 1500 N/mm^2

(a) *Failure by yielding of steel*

$M_u = 0.9\ d\ A_p f_p$

= (0.9 × 445 × 1408 × 1500)

= 846 × 10^6 N·mm

= 846 kN·m.

(b) *Failure by crushing of concrete*

$M_u = 0.176\ b\ d^2 f_{ck}$

= 0.176 × 1000 × 445^2 × 40

= 1394 × 10^6 N·mm

= 1394 kN·m

The actual M_u is the lesser of (a) or (b) and is equal to 846 kN·m.

According to IRC: 18 – 2000

Required Ultimate Moment = 1.5 M_g + 2.5 M_q

= (1.5 × 190) + (2.5 × 187)

= 285 + 467.5

= 752.5 kN·m

Hence the ultimate moment capacity of the section (M_u = 846 kN·m) is greater than the required ultimate moment (752.5 kN·m).

12. Check for ultimate shear strength

Ultimate shear force, $V_u = (1.5\ V_g + 2.5\ V_q)$

= (1.5 × 72.8) + (2.5 × 87.72)

= 328.5 kN

According to IRC: 18 – 1985, the ultimate shear resistance of support section uncracked in flexure is given by

$$V_{co} = 0.67\ bh\ \sqrt{f_t^2 + 0.8 f_{cp} f_t} + \eta P \sin\theta$$

When b = width of slab = 1000 mm

h = overall depth = 500 mm

f_t = Principal tensile stress

$= 0.24\ \sqrt{f_{ck}} = 0.24\ \sqrt{40} = 1.51$ N/mm^2

f_{cp} = Compressive prestress at centroidal axis

$$= \left(\frac{\eta P}{A}\right) = \left(\frac{0.8 \times 1687.5 \times 10^3}{1000 \times 500}\right) = 2.7 \text{ N/mm}^2$$

Eccentricity of cables at centre of span, e = 195 mm
The cables are concentric at support section
Slope of cable = $\theta = (4e/L) = (4 \times 195)/(10.4 \times 1000) = 0.075$

$$\therefore \quad V_{co} = 0.67 \times 1000 \times 500 \sqrt{1.51^2 + 0.8 \times 2.7 \times 1.51} + (0.8 \times 1687.5 \times 10^3 \times 0.075)$$

$$= 888500 \text{ N}$$
$$= 888.5 \text{ kN.}$$

Since the ultimate shear V_u is less than 50 percent of the ultimate shear resistance V_c, no shear reinforcement is required.

13. Supplementary Reinforcement

Supplementary Reinforcement
= 0.15 percent of the gross cross sectional area
= (0.0015 × 1000 × 500) = 750 mm^2

Provide 10 mm diameter, Fe-415 HYSD bars at 200 mm centres both at top and bottom faces in the longitudinal and transverse directions.

14. Design of end block Reinforcement

At support section the concentric cables carrying a force of 554 kN are spaced at intervals of 328 mm. The bursting tension is computed using Table 10.2 as recommended in IRC: 18 – 2000.

Table 10.2 Design Bursting Tensile Force in End Blocks (IRC: 18–2000)

(y_{po}/y_o)	0.3	0.4	0.5	0.6	0.7
(F_{bst}/P_k)	0.23	0.20	0.17	0.14	0.11

F_{bst} = Bursting tensile force
P_k = Tendon force
$2y_{po}$ = Side of loaded area
$2y_o$ = side of end block

In the present design problem,
P_k = 554 kN
$2y_{po}$ = 150 mm
$2y_o$ = 328 mm

$$\therefore \quad (y_{po}/y_o) = (150/328) = 0.457$$

Interpolating the value of (F_{bst}/P_k) for $(y_{po}/y_o) = 0.457$ from Table 10.2,

$$F_{bst}/P_k = (0.185 \times 554) = 103 \text{ kN}$$

Using 10 mm diameter, Fe-250 grade mild steel bars as end block reinforcement.

$$\text{Area of steel} = \left(\frac{103 \times 10^3}{0.87 \times 250}\right) = 474 \text{ mm}^2$$

Provide 10 mm diameter bars at 100 mm centres in the vertical and horizontal directions in two planes in front of anchorages at 100 and 200 mm respectively. The longitudinal and cross sections of the deck slab is shown in Figs. 10.9 and 10.10.

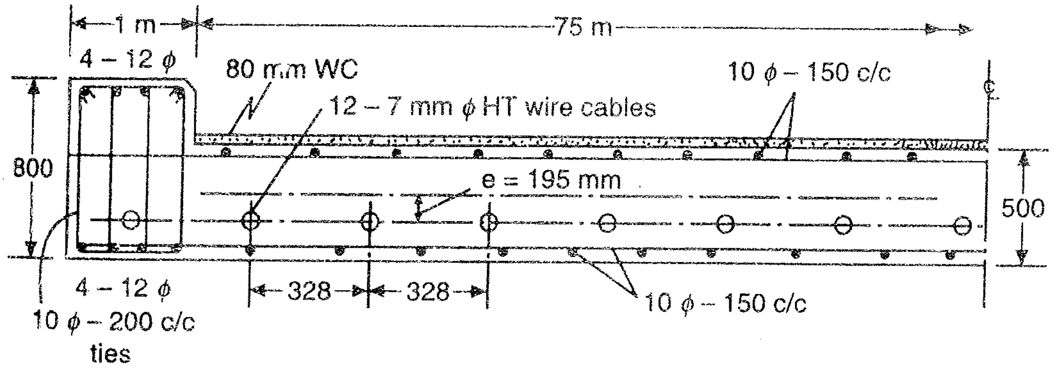


Fig. 10.9 Cross Section of Deck Slab at Centre of Span.

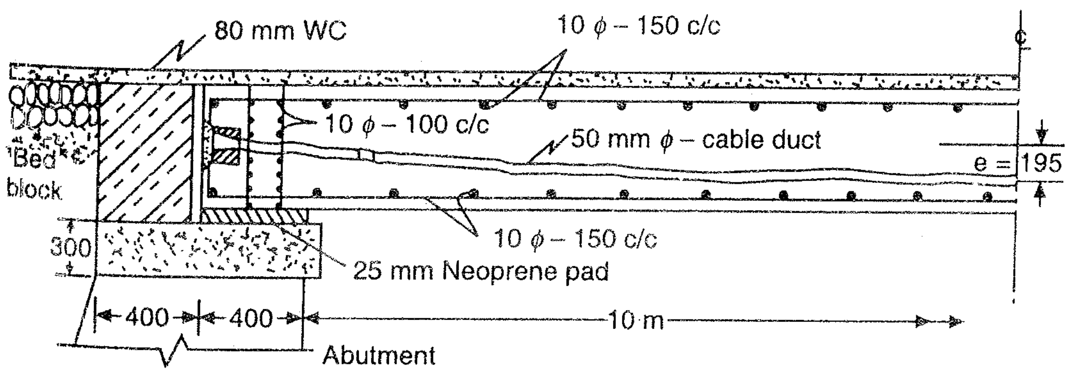


Fig. 10.10 Longitudinal Section of Deck Slab.

10.6 DESIGN OF POST-TENSIONED PRESTRESSED CONCRETE TEE BEAM AND SLAB BRIDGE DECK

Design a post tensioned prestressed concrete Tee beam slab bridge deck for a National highway crossing to suit the following data:

1. Data

Effective span = 30 m

Width of road = 7.5 m

Kerbs = 600 mm on each side

Foot path = 1.5 m wide on each side

Thickness of wearing coat = 80 mm

Live load = I.R.C Class AA tracked vehicle

For deck slab, adopt M-20 grade concrete

For prestressed concrete girders, adopt M-50 grade concrete with cube strength at transfer as 40 N/mm^2.

Loss ratio = 0.85

Spacings of cross girders = 5 m

Adopt Fe-415 grade HYSD bars and strands of 15.2 mm-7 ply conforming to IS: 6006 – 1983 are available for use.

Design the girders as Class I type member.

2. Permissible Stresses

For M-20 grade concrete and Fe-415 grade HYSD bars (IRC: 21-2000)

$\sigma_{cb} = 6.7$ N/mm^2

$\sigma_{st} = 200$ N/mm^2

$m = 10$

$Q = 0.762$

$j = 0.906$

For M-50 grade concrete, (IRC: 18-2000)

$f_{ck} = 50$ N/mm^2

$f_{ci} = 40$ N/mm^2

$f_{ct} = 0.45\, f_{ci} = (0.45 \times 40) = 18$ N/mm^2

$f_{w} = 0.33\, f_{ck} = (0.33 \times 50) = 16$ N/mm^2

$f_{tt} = f_{tw} = 0$ (Class 1 type member)

Modulus of elasticity of concrete in girders

$$E_c = 5700\sqrt{f_{ck}} = 5700\sqrt{50} = 40305 \text{ N/mm}^2 = 40 \text{ kN/mm}^2$$

3. Cross Section of Deck

4 main girders are provided at 2.5 m intervals.

Thickness of deck slab = 250 mm

Wearing coat = 80 mm

Kerbs 600 mm wide by 300 mm deep are provided

The cross section of the deck is shown in Fig. 10.11

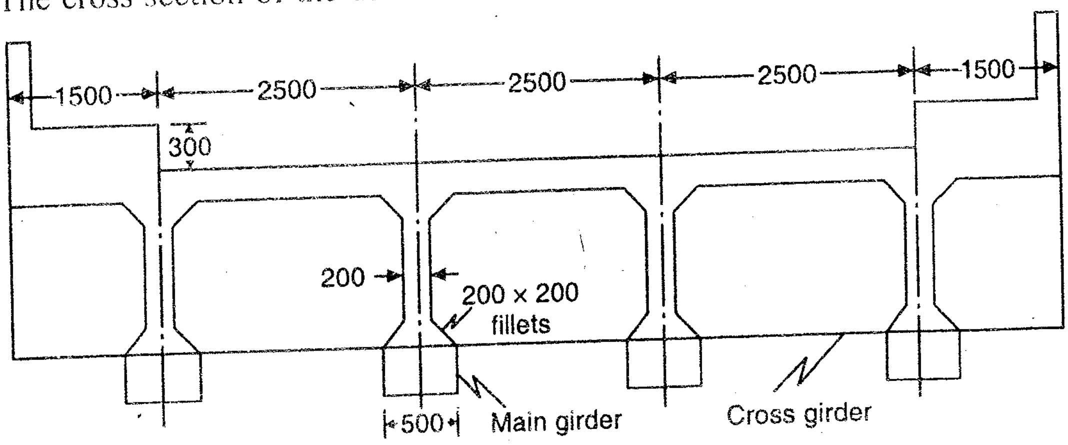


Fig. 10.11 Cross Section of Bridge Deck.

The main girders are precast and the slab connecting the girders is cast *in-situ.*

Spacing of cross girders = 5 m

Spacing of main girders = 2.5 m

4. Design of Interior Slab Panel

(a) *Bending Moments*

Dead weight of slab = $(1 \times 1 \times 0.25 \times 24) = 6.00$ kN/m^2

Death weight of W.C. = (0.08×22) = $\underline{1.76}$

Total dead load = 7.76 kN/m^2

Live load is IRC Class AA tracked vehicle, one wheel is placed at the centre of panel as shown in Fig. 10.12.

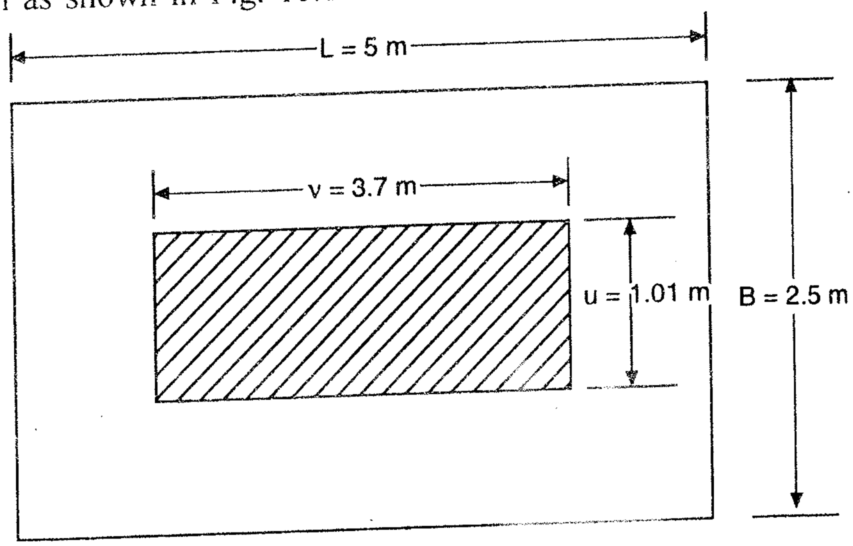


Fig. 10.12 Position IRC Class AA Wheel Load for Maximum Bending Moment.

$$u = (0.85 + 2 \times 0.08) = 1.01 \text{ m}$$
$$v = (3.60 + 2 \times 0.08) = 3.76 \text{ m}$$
$$(u/B) = (1.01/2.5) = 0.404$$
$$(v/L) = (3.76/5.0) = 0.752$$
$$K = (B/L) = (2.5/5.0) = 0.5$$

Referring to Pigeaud's curves (Fig. 10.13)

$$m_1 = 0.098 \text{ and } m_2 = 0.02$$
$$M_B = W(m_1 + 0.15\, m_2)$$
$$= 350\,(0.098 + 0.15 \times 0.02) = 35.35 \text{ kN·m}$$

As the slab is continuous, design B.M = 0.8 M_B. Design B.M, including impact and continuity factor is given by

M_B (Short span) $= (1.25 \times 0.8 \times 35.35) = 35.35$ kN·m

Similarly $M_L = W(m_2 + 0.15\, m_1)$

$= 350\,(0.02 + 0.15 \times 0.098) = 12.14$ kN·m

M_L (Long span) $= (1.25 \times 0.8 \times 12.14) = 12.14$ kN·m

(b) *Shear Forces*

Dispersion in the direction of span

$$= [0.85 + 2\,(0.08 + 0.25)] = 1.51 \text{ m}$$

For maximum shear, load is kept such that the whole dispersion is in span. The load is kept at (1.51/2) = 0.755 m from the edge of the beam as shown in Fig. 10.14

Effective width of slab = $kx\,[1 - (x/L)] + b_W$

Breadth of cross girder = 200 mm

Clear length of panel = (5 – 0.2) = 4.8 m

$$\therefore \quad (B/L) = (4.8/2.3) = 2.08$$

From table 3.5, k for continuous slab is obtained as 2.60

Effective width of slab = $2.6 \times 0.755\,[1 \times (0.755/2.3)] + 3.6 + (2 \times 0.08)$

$= 5.079$ m

Load per metre width = (350/5.079) = 70 kN

Shear force/metre width = 70 (2.3 – 0.755)/2.3 = 47 kN

Shear force with impact = (1.25 × 47) = 58.75 kN

(c) *Dead load bending moments and shear forces*

Dead load = 7.76 kN/m^2

Total load on panel = (5 × 2.5 × 7.76) = 97 kN

$(u/B) = 1$ and $(v/L) = 1$, as panel is loaded with uniformly distributed load.

$$k = (B/L) - (2.5/5) = 0.5 \text{ and } (1/k) = 2.0$$

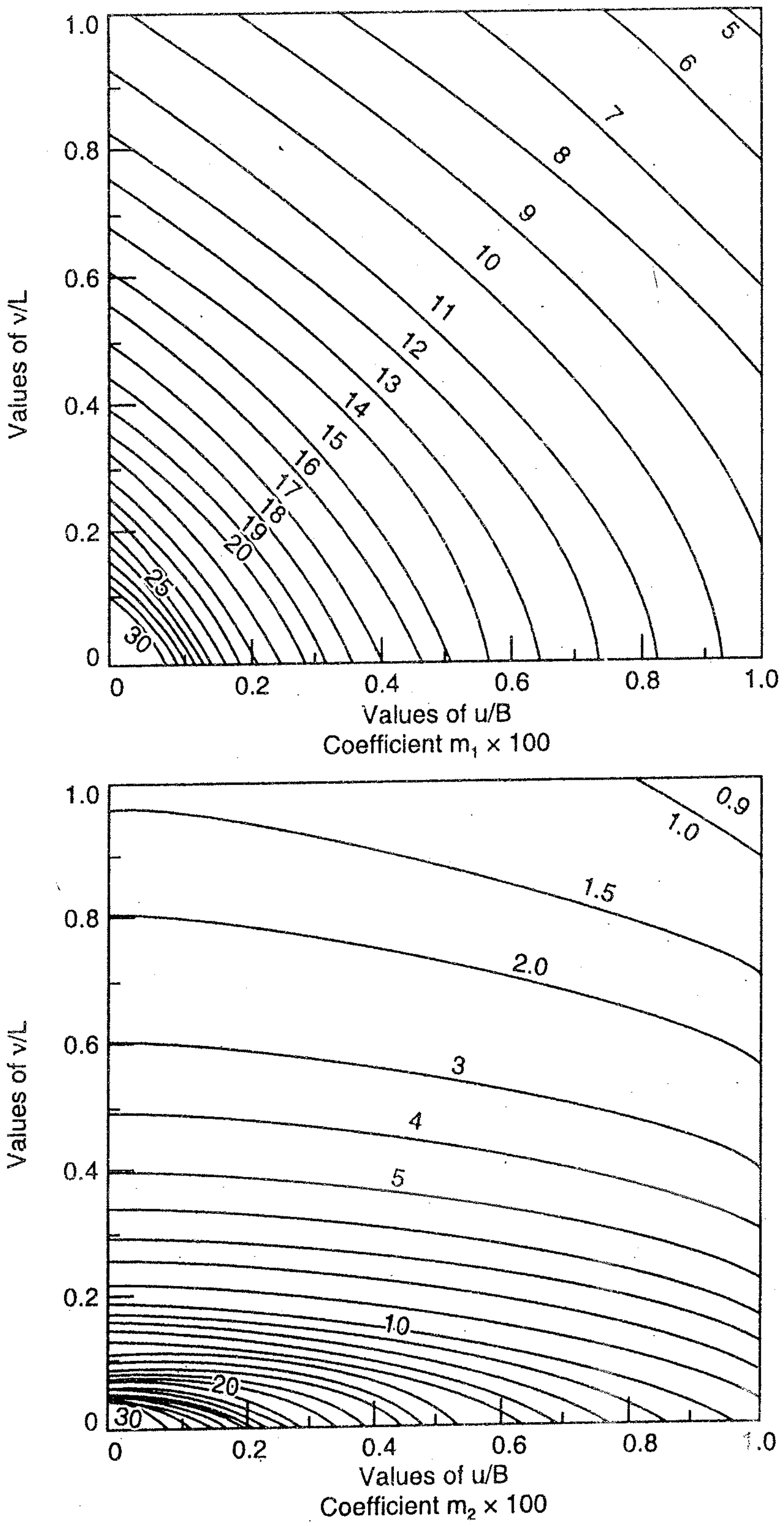


Fig. 10.13 Moment Coefficients m_1 and m_2 for K = 0.5 (Pigeaud's curves).

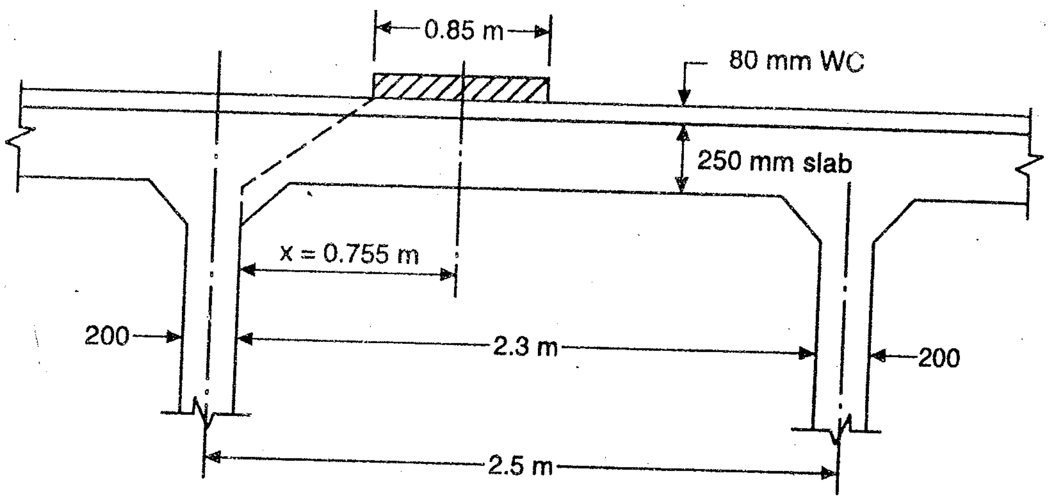


Fig. 10.14 Position of Wheel Loads for Maximum Shear.

From Pigeaud's curve (refer Fig. 10.15)
$m_1 = 0.047$, $m_2 = 0.01$

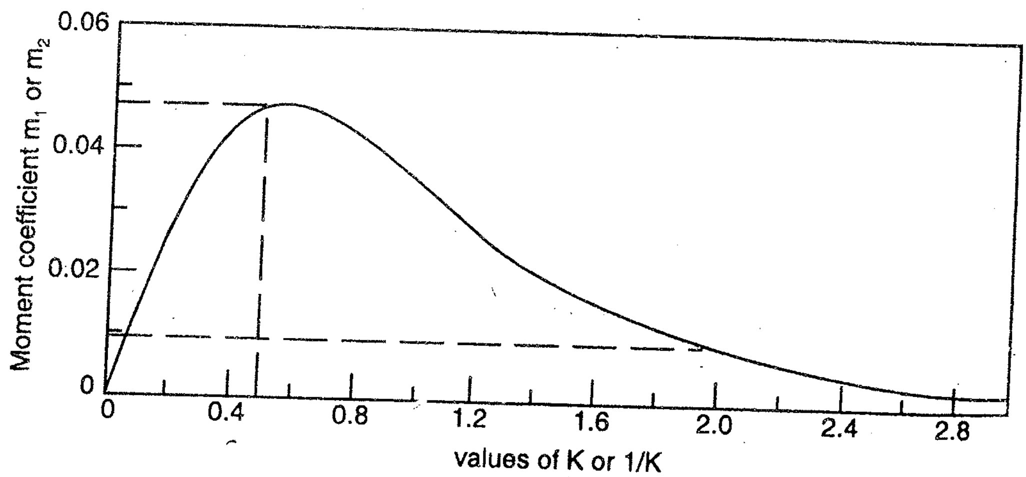


Fig. 10.15 Moment Coefficients for Slabs Completely Loaded with Uniformly Distributed Load, Coefficient is m_1 for K and m_2 for $1/K$.

$M_B = 97\ (0.047 + 0.15 \times 0.01) \doteq 4.70$ kN·m
$M_L = 97\ (0.01 + 0.15 \times 0.047) = 1.65$ kN·m
Design B.M including continuity factor,
$M_B = (0.8 \times 4.7) = 3.76$ kN·m
$M_L = (0.8 \times 1.65) = 1.32$ kN·m
Dead load shear force $= (0.5 \times 7.76 \times 2.3) = 8.924$ kN

(d) *Design Moments and shear forces*
Total $M_B = (35.35 + 3.76) = 39.11$ kN·m
Total $M_L = (12.14 + 1.32) = 13.46$ kN·m

(e) *Design of slab section and reinforcement*

$$\text{Effective depth} = d = \sqrt{\frac{M}{Q.b}} = \sqrt{\frac{39.11\times10^6}{0.762\times1000}} = 226\,\text{mm}$$

Adopt effective depth d = 230 mm

$$A_{st} = \left(\frac{M}{\sigma_{st}\cdot j.d}\right) = \left(\frac{39.11\times10^6}{200\times0.960\times230}\right) = 938\ \text{mm}^2$$

Use 12 mm diameter bars at 120 mm centres (A_{st} = 942 mm²)
Effective depth for long span using 10 mm diameter bars
= (230 – 6 – 5) = 219 mm

$$A_{st} = \left(\frac{13.46\times10^6}{200\times0.906\times219}\right) = 339\ \text{mm}^2$$

But minimum reinforcement using HYSD bars according to IRC: 18 – 2000 is 0.15 percent of cross section, Hence

$$A_{st} = (0.0015 \times 1000 \times 250) = 375\ \text{mm}^2$$

Use 10 mm diameter bars at 150 mm centres (A_{st} = 524 mm²)
For crack control (IRC : 21 – 2000)

(f) *Check for shear stress (As per IRC: 21 – 2003)*

$$\text{Nominal shear stress} = \tau_v = \left(\frac{V}{bd}\right) = \left(\frac{58.75\times10^3}{1000\times230}\right) = 0.255\ \text{N/mm}^2$$

At support section, A_{st} = 942 mm²

$$\text{Hence the ratio} \left(\frac{1000A_{st}}{bd}\right) = \left(\frac{100\times942}{1000\times230}\right) = 0.40$$

Refer Table 3.8 (Table 12B of IRC: 21-2000) and read out the permissible shear Shear stress in concrete corresponding to M-20 Grade Concrete as τ_c = 0.25 N/mm². For a slab of overall depth 250 mm, read out the value of constant K = 1.1 from Table 3.10 (Table 12 C of IRC: 21-2000).

The permissible shear stress in concrete slab = $K\tau_c$ = (1.1 × 0.25) = 0.275 N/mm². Which is greater than the actual shear stress τ_v = 0.255 N/mm². Hence the shear Stresses are within safe permissible limits.

5. Design of Longitudinal Girders

(a) *Reaction Factors*

Using Courbon's theory, the I.R.C. Class AA loads are arranged for maximum eccentricity as shown in Fig. 10.16.

Reaction factor for outer girder A is

$$R_A = \frac{2W_1}{4}\left[1 + \frac{4I \times 3.75 \times 1.1}{(2I \times 3.75^2) + (2I \times 1.25^2)}\right] = 0.764\, W_1$$

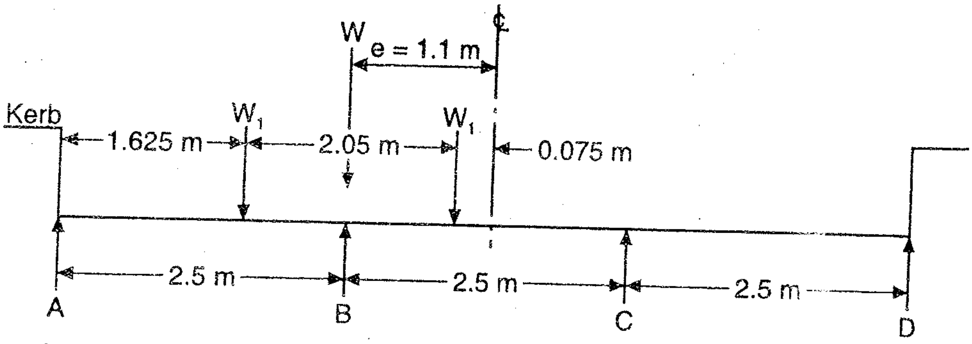


Fig. 10.16 Transverse Disposition of IRC Class AA Tracked Vehicle.

Reaction factor for inner girder B is

$$R_B = \frac{2W_1}{4}\left[1 + \frac{4I \times 1.25 \times 1.1}{(2I \times 3.75^2) + (2I \times 1.25^2)}\right] = 0.588\, W_1$$

If $\quad W$ = Axle load = 700 kN

$W_1 = 0.5\ W$

$\therefore \quad R_A = (0.764 \times 0.5\ W) = 0.382\ W$

$R_B = (0.588 \times 0.5\ W) = 0.294\ W$

(b) *Dead load from slab per girder*

The dead load of deck slab is calculated with reference to Fig. 10.17.

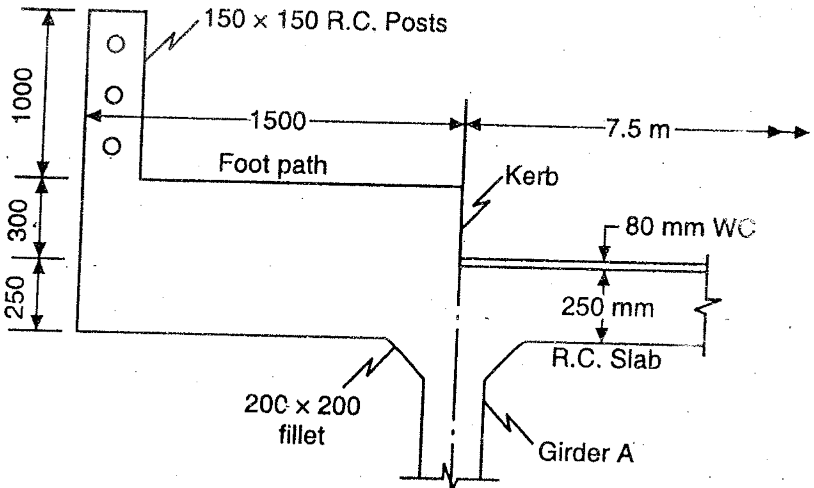


Fig. 10.17 Details of Foot Path, Kerb, Parapet and Deck Slab.

Weight of

1. Parapet railing (Lumpsum) 0.92 kN/m
2. Foot path and kerb = (0.3 × 1.5 × 24) 10.08
3. Deck slab = (0.25 × 1.5 × 24) 9.00

20.00 kN/m

Total dead load of deck = [(2 × 20) + (7.76 × 7.5)] = 98.2 kN/m
It is assumed that the deck load is shared equally by all the four girders

Dead load/girder = (98.2/4) = 24.55 kN/m

(c) *Dead load of Main girder*

The overall depth of the girder is assumed as 1800 mm at the rate of 60 mm for every metre of span.

Span of the girder = 30 m

Overall depth = (60 × 30) = 1800 mm

the bottom flange is selected so that four to 6 cables are easily accommodated in the flange. The section of the main girder selected in shown in Fig. 10.18.

Dead weight of rib = (1.15 × 0.2 × 24) = 5.52 kN/m
Dead weight of bottom flange = (0.5 × 0.4 × 24) = 4.80

10.32 kN/m

Weight of cross girder = (0.2 × 1.25 × 24) = 6 kN/m

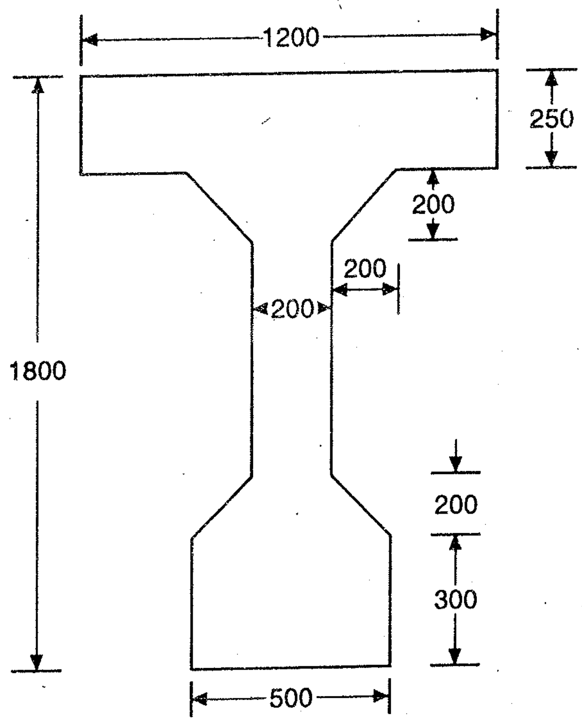


Fig. 10.18 Cross Section of Prestressed Concrete Girder.

(d) *Dead load moments and shears in main girder*

Reaction from deck slab on each girder = 24.55 kN/m

Weight of cross girder = 6 kN/m

Reaction on main girder = (6 × 2.5) = 15 kN/m

Self weight of main girder = 10.32 kN/m

Total dead load on girder = (24.55 + 10.32) = 34.87 kN/m

The maximum dead load bending moment and shear force is computed using the loads shown in Fig. 10.19 thus

$$M_{max} = [(0.125 \times 34.87 \times 30^2) + (0.25 \times 15 \times 30) + (15 \times 10) + (15 \times 5)]$$
$$= 4261 \text{ kN.m}$$

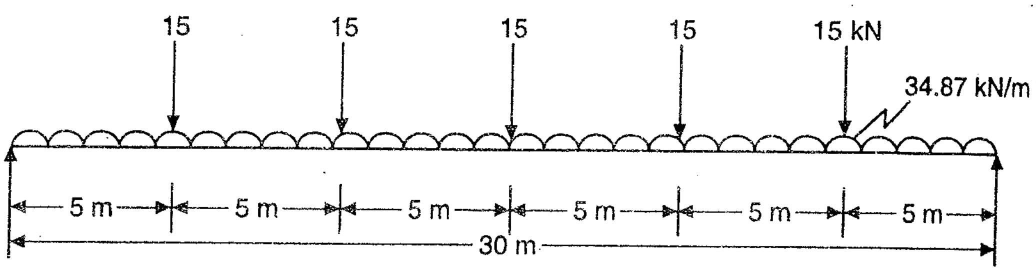


Fig. 10.19 Dead Load on Main Girder.

Dead load shear at support

$$V_{max} = [(0.5 \times 34.87 \times 30) + (0.5 \times 75)]$$
$$= 561 \text{ kN}$$

(e) *Live load bending moments in girder*

Span of the girder = 30 m

Impact factor (Class AA) = 10%

The live load is placed centrally on the span as shown in Fig. 10.20.

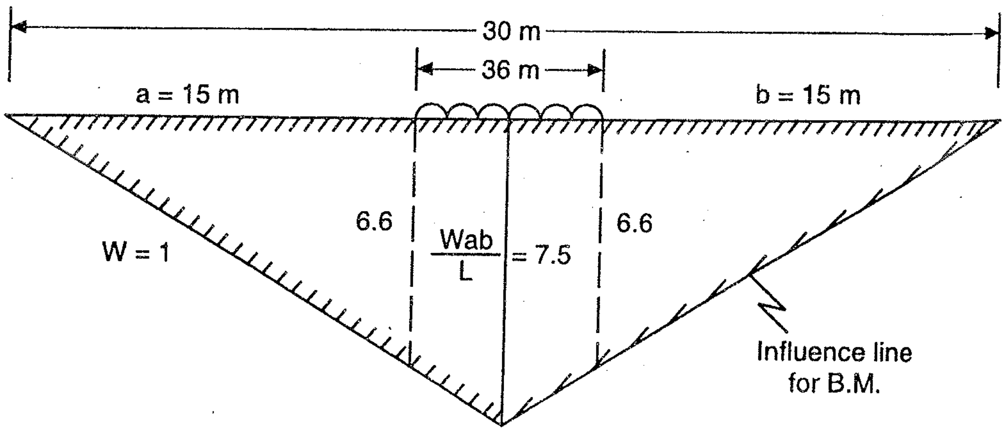


Fig. 10.20 Influence Line for Bending Moment in Girder.

Bending Moment at centre of span

$$= 0.5\ (6.6 + 7.5)\ 700 = 4935 \text{ kN·m}$$

B.M including impact and reaction factors for outer girder is,
Live load B.M = (4935 × 1.1 × 0.382) = 2074 kN·m
For Inner girder, B.M = (4935 × 1.1 × 0.294) = 1596 kN·m

(f) *Live load shear forces in girders*

For estimating the maximum live load shear in the girders, the I.R.C. Class AA loads are placed as shown in the Fig. 10.21.

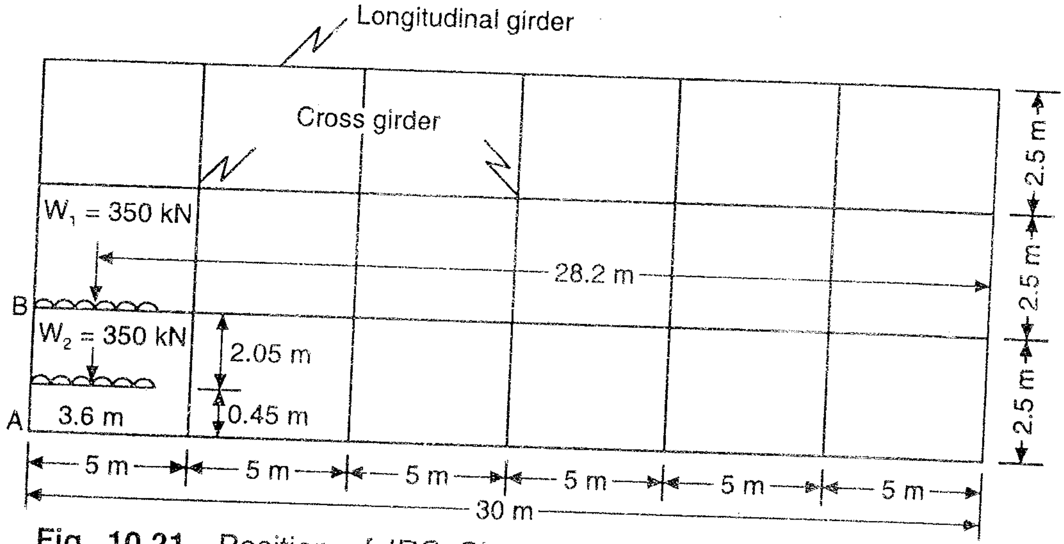


Fig. 10.21 Position of IRC Class AA Loads for Maximum Shear.

Reaction of W_2 on girder B = (350 × 0.45)/2.5 = 63 kN
Reaction of W_2 on the girder A = (350 × 2.05)/2.5 = 287 kN
Total load on girder B = (350 + 63) = 413 kN
Maximum reaction in girder B = (413 × 28.2)/30 = 388 kN
Maximum reaction in girder A = (287 × 28.2)/30 = 270 kN
Maximum live load shear with impact factor in inner girder
= (388 × 1.1) = 427kN
Outer girder = (270 × 1.1) = 297 kN

(g) *Design Bending Moments and shear forces*

The design moments and shear forces are compiled in Table 10.4.

Table 10.4 Abstract of Design Moments and Shear Forces in Main Girders

Bending Moment	D.L.B.M	L.L.B.M	Total B.M	Units
Outer Girder	4261	2074	6335	kN·m
Inner Girder	4261	1596	5857	kN·m
Shear Force	**D.L.S.F**	**L.L.S.F.**	**Total S.F**	**Units**
Outer Girder	561	297	858	kN
Inner Girder	561	427	988	kN

(h) *Properties of Main Girder Section*

The main girder section is as shown in Fig. 10.22 for computational purposes. The properties of the section are:

$A = 73 \times 10^4$ mm^2

$y_1 = 750$ mm, $y_b = 1050$ mm, $I = 2924 \times 10^8$ mm^4

$z_1 = (I/y_t) = (2924 \times 10^8)/750 = 3.89 \times 10^8$ mm^4

$z_b = (I/y_b) = (2924 \times 10^8)/1050 = 2.78 \times 10^8$ mm^3

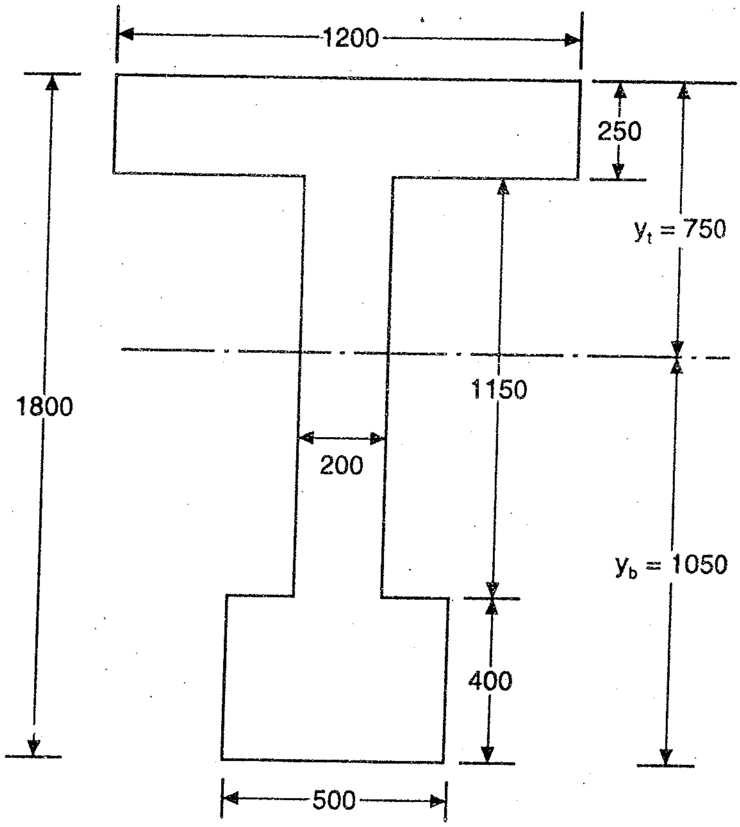


Fig. 10.22 Cross Section of Main Girder.

(i) *Check for Minimum Section Modulus*

$f_{ck} = 50$ N/mm^2 $\qquad\qquad \eta = 18$ N/mm^2

$f_{ct} = 18$ N/mm^2 $\qquad\qquad M_g = 4261$ kN·m

$f_{ci} = 40$ N/mm^2 $\qquad\qquad M_q = 2074$ kN·m

$f_{tt} = f_{tw} = 0 M_d = (M_g + M_q) = 6335$ kN·m

$f_{cw} = 16$ N/mm^2

$f_{br} = (\eta f_{ct} - f_{tw}) = (0.85 \times 18 - 0) = 15.3$ N/mm^2

$f_{tr} = (f_{cw} - \eta f_{tt}) = 16$ N/mm^2

$$f_{inf} = (f_{tw}/\eta) + (M_d/\eta z_b)$$
$$= 0 + (6335 \times 10^6)/(0.85 \times 2.78 \times 10^8)$$
$$= 26.80 \text{ N/mm}^2$$

$$z_b = \left[\frac{M_q + (1-\eta)M_g}{f_{br}}\right]$$

$$= \left[\frac{(2074 \times 10^6) + (1-0.85)4261 \times 10^6}{15.3}\right]$$

$$= 1.77 \times 10^8 \text{ mm}^3 < 2.78 \times 10^8)$$

Hence the section provided is adequate.

(j) *Prestressing Force*

Allowing for two rows of cables, cover required = 200 mm

Maximum possible eccentricity e = (1050 – 200) = 850 mm

Prestressing force is obtained as

$$P = (A.f_{inf}.Z_b)/(Z_b + A.e)$$
$$= [(0.73 \times 10^6 \times 26.80 \times 2.78 \times 10^8)/(2.78 \times 10^8) + (0.73 \times 10^6 \times 850)]$$
$$= 6053 \times 10^3 \text{ N}$$
$$= 6053 \text{ kN.}$$

Using Freyssinet system, anchorage type 7K-15 (7 strands of 15.2 mm diameter) in 65 mm cables ducts, (IS: 6006-1983)

Force in each cable = (7 × 0.8 × 260.7) = 1459 kN

∴ Number of cables = (6053/1459) ≈ 5

Area of each strand = 140 mm^2

Area of 7 strands in each cable = (7 × 140) = 980 mm^2

Area of strands in 5 cables = A_p = (5 × 980) = 4900 mm^2

The cables are arranged at centre of span section as shown in Fig. 10.23.

(k) *Permissible tendon zone*

At support section,

$$e \le (Z_b \cdot f_{ct}/P) - (Z_b/A)$$
$$\le (2.78 \times 10^8 \times 18)/(6053 \times 10^3) - (2.78 \times 10^8)/(0.73 \times 10^6)$$
$$\le 445 \text{ mm}$$

and

$$e \ge (Z_b \cdot f_{tw}/\eta P) - (Z_b/A)$$
$$\ge 0 - (2.78 \times 10^8)/(0.73 \times 10^6)$$
$$\ge -380 \text{ mm}$$

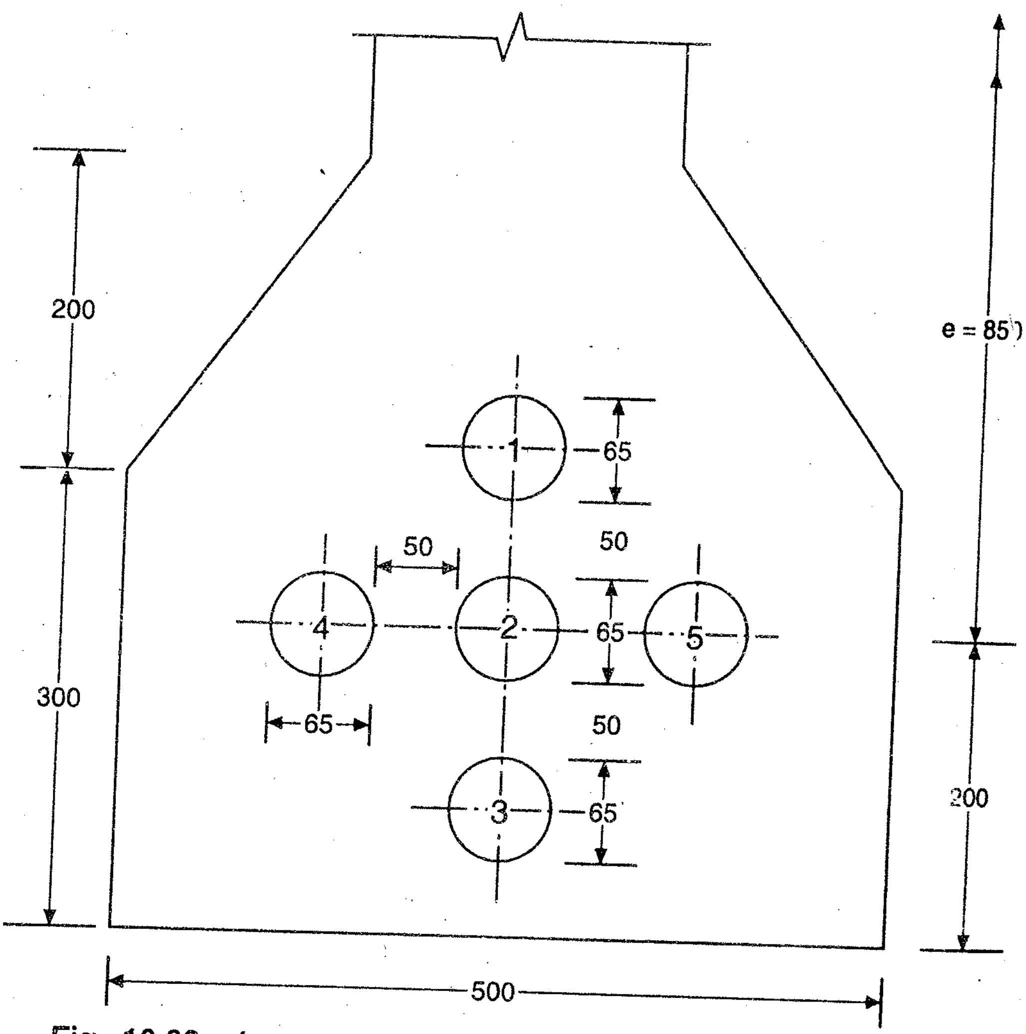


Fig. 10.23 Arrangement of Cable at Centre of Span Section.

The 5 cables are arranged to follow a parabolic profile with the resultant force having an eccentricity of 180 mm towards the ffit at the support section. The position of cables at support section is shown in Fig. 10.24.

6. Check for Stresses

For the centre of span section, we have

$P = 6053$ kN $\qquad z_t = 3.89 \times 10^8$ mm^3

$e = 850$ mm $\qquad h = 0.857? = 0.85$

$A = 0.73 \times 10^6$ mm^2 $\qquad M_g = 4261$ kN·m

$Z_b = 2.78 \times 10^8$ mm^3 $\qquad M_q = 2074$ kN·m

$(P/A) = (6053 \times 10^3)/(0.73 \times 10^6) = 8.29$ N/mm^2

$(Pe/z_t) = (6053 \times 10^3 \times 850)/(3.89 \times 10^8) = 13.22$ N/mm^2

$(Pe/Z_b) = (6053 \times 10^3 \times 850)/(2.78 \times 10^8) = 18.50$ N/mm^2

$(M_g/Z_t) = (4261 \times 10^6)/(3.89 \times 10^8) = 10.95$ N/mm^2

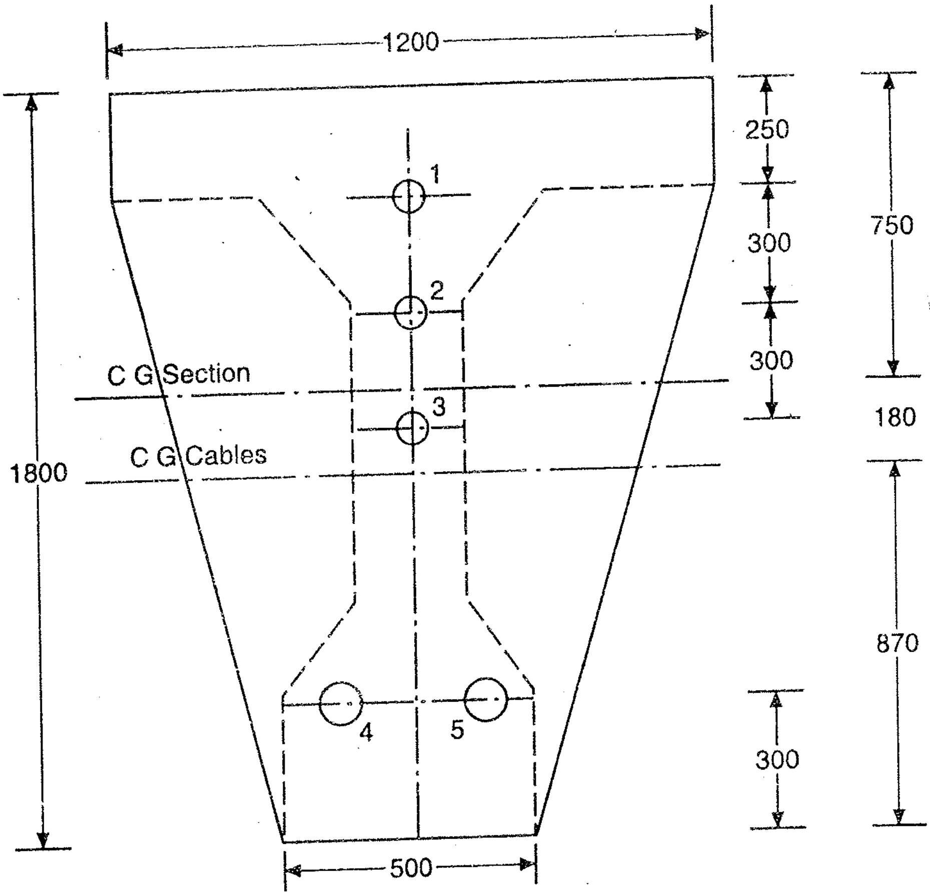


Fig. 10.24 Arrangement of Cables at Support Section.

$(M_g/z_b) = (4261 \times 10^6)/(2.78 \times 10^8) = 15.32\ \text{N/mm}^2$
$(M_q/Z_t) = (2074 \times 10^6)/(3.89 \times 10^8) = 5.33\ \text{N/mm}^2$
$(M_q/Z_b) = (2074 \times 10^6)/(2.78 \times 10^8) = 7.46\ \text{N/mm}^2$

At transfer stage:

$$\sigma_t = [(P/A) - (Pe/Z_t) + (M_g/Z_t)]$$
$$= (8.29 - 13.22 + 10.95)$$
$$= 6.02\ \text{N/mm}^2$$
$$\sigma_b = [(P/A) + (Pe/Z_b) - (M_g/Z_b)]$$
$$= [8.29 + 18.50 - 15.32]$$
$$= 11.47\ \text{N/mm}^2$$

At working load stage:

$$\sigma_t = [\eta\ (P/A) - \eta\ (Pe/Z_b) - (M_g/Z_t) + (M_q/Z_t)]$$
$$= [0.85\ (8.29 - 13.22) + 10.95 + 5.33]$$
$$= 12.09\ \text{N/mm}^2\ \text{(Compression)}$$
$$\sigma_b = [\eta\ (P/A) - \eta\ (Pe/Z_b) - (M_g/Z_b) + (M_q/Z_b)]$$
$$= [0.85\ (8.29 + 18.50) - 15.32 - 7.46]$$
$$= -\ 0.01\ \text{N/mm}^2\ \text{(Tension)}$$

All the stresses at top and bottom fibres at transfer and service loads are well within the safe permissible limits.

7. Check for Ultimate Flexural Strength

For the centre of span section

$A_p = (5 \times 7 \times 1400) = 4900 \text{ mm}^2$

$b = 1200 \text{ mm}$

$d = 1600 \text{ mm}$

$b_w = 200 \text{ mm}$

$f_{ck} = 50 \text{ N/mm}^2$

$f_p = 1862 \text{ N/mm}^2$

$D_f = 250 \text{ mm}$

According to IRC: 18-2000

$M_u = (1.5\, M_g + 2.5\, M_q)$

$= (1.5 \times 4261 + 2.5 \times 2074)$

$= 11577 \text{ kN·m}.$

According to IRC: 18-2000, the ultimate flexural strength is computed as follows:

(i) *Failure by yielding of steel*

$M_u = 0.9\, d\, A_p f_p$

$= (0.9 \times 1600 \times 4900 \times 1862)$

$= 13138 \times 10^6 \text{ N·mm}$

$= 13138 \text{ kN·m}$

(ii) *Failure by crushing of concrete*

$$M_u = 0.176\, b_w\, d^2 f_{ck} + \frac{2}{3} \times 0.8\,(b - b_w)\left(d - \frac{D_f}{2}\right) D_f \times f_{ck}$$

$$= (0.176 \times 200 \times 1600^2 \times 50) + \frac{2}{3} \times 0.8\,(1000)$$

$$\left(1600 - \frac{250}{2}\right)(250 \times 250)$$

$= (4505 \times 10^6 + 9838 \times 10^6) \text{ N·mm}$

$= 14343 \text{ kN·m}$

According to IS: 1343-1980, the ultimate flexural strength of the centre span section is computed as follows:

$A_p = (A_{pw} + A_{pf})$

$A_{pf} = 0.45\, f_{ck}\,(b - b_w)\,(D_f/f_p)$

$= 0.45 \times 50\,(1200 - 200)\,(250/1862)$

$= 3021 \text{ mm}^2$

$\therefore A_{pw} = (4900 - 3021) = 1879 \text{ mm}^2$

Ratio $\left(\frac{A_{pw}.f_p}{b_w.d.f_{ck}}\right) = \left(\frac{1879 \times 1862}{200 \times 1600 \times 50}\right) = 0.218$

From Table 11 of IS: 1343, we have for post tensioned beams with effective bond,

$(f_{pu}/0.87\ f_p) = 0.93$ and $(x_u/d) = 0.43$

$\therefore\ f_{pu} = (0.93 \times 0.87 \times 1862)$ and $x_u = (0.43 \times 1600)$

$= 1506\ \text{N/mm}^2 = 688\ \text{mm}$

$\therefore\ M_u = [f_{pu}\ A_{pw}\ (d - 0.42\ x_u) + 0.45\ f_{ck}\ (b - b_w)\ D_f\ (d - 0.5\ D_f)]$

$= [1506 \times 1879\ (1600 - 0.42 \times 688)]$

$+ 0.45 \times 50 \times 1000 \times 250\ (1600 - 0.5 \times 250)]$

$= 12006 \times 10^6$ N·mm

$= 12006$ kN·m

Require ulimate moment = 11577 kN.m < 12006 kN·m, Hence safe.

8. Check for Ultimate Shear Strength

Ultimate shear force $V_u = (1.5\ V_g + 2.5\ V_q)$

$= (1.5 \times 561) + (2.5 \times 427)$

$= 1909$ kN

According to IRC: 18–2000, the ultimate shear resistance of support section uncracked in flexure is given by,

$$V_{cw} = 0.67 b_w h \sqrt{f_t^2 + 0.8 f_{cp}.f_t} + \eta P.Sin\,\theta$$

Where b_w = width of web = 200 mm

h = ovzerall depth of girder = 1800 mm

f_t = Maximum principal tensile stress at centroidal axis

$f_t = 0.24\ \sqrt{f_{ck}} = 0.24\ \sqrt{50} = 1.7\ \text{N/mm}^2$

f_{cp} = Compressive stress at centroidal axis due to prestress

$$= \left(\frac{\eta P}{A}\right) = \left(\frac{0.85 \times 6053 \times 10^3}{0.73 \times 10^6}\right) = 7.04\ \text{N/mm}^2$$

Eccentricity of cables at centre of span = 850 mm

Eccentricity of cables at support = 180 mm

Net eccentricity = e = (850 – 180) = 670 mm

Slope of cable = θ = (4e/L) = (4 × 670)/(30 × 1000) = 0.089

$$\therefore\ V_{cw} = (0.67 \times 200 \times 1800)\sqrt{(1.7^2 + 0.8 \times 7.04 \times 1.7)} + (0.85 \times 6053 \times 10^3 \times 0.089)$$

$= 1309103$ N

$= 1309$ kN

Shear resistance required = 1909 kN
Shear capacity of section = 1309 kN
Balance Shear = V = 600 kN

Using 10 mm diameter 2 legged stirrups of Fe 415 HYSD bars, the spacings S_v is obtained as,

$$S_v = \left(\frac{0.87 f_y . A_{sv} . d_t}{V}\right) = \left(\frac{0.87 \times 415 \times 2 \times 79 \times 1750}{600 \times 10^3}\right)$$

$= 166$ mm

Provide 10 mm diameter stirrups at 150 mm centres near support and gradually increased to 300 mm towards the centre of span.

9. Supplementary Reinforcements

Longitudinal reinforcements of not less than 0.15 percent of gross cross sectional are to be provided to safeguard against shrinkage cracking.

$$A_{st} = [(0.15 \times 0.73 \times 10^6)/100] = 1095 \text{ mm}^2$$

20 mm diameter bars are provided and distributed in the compression flange as shown in Fig. 10.25.

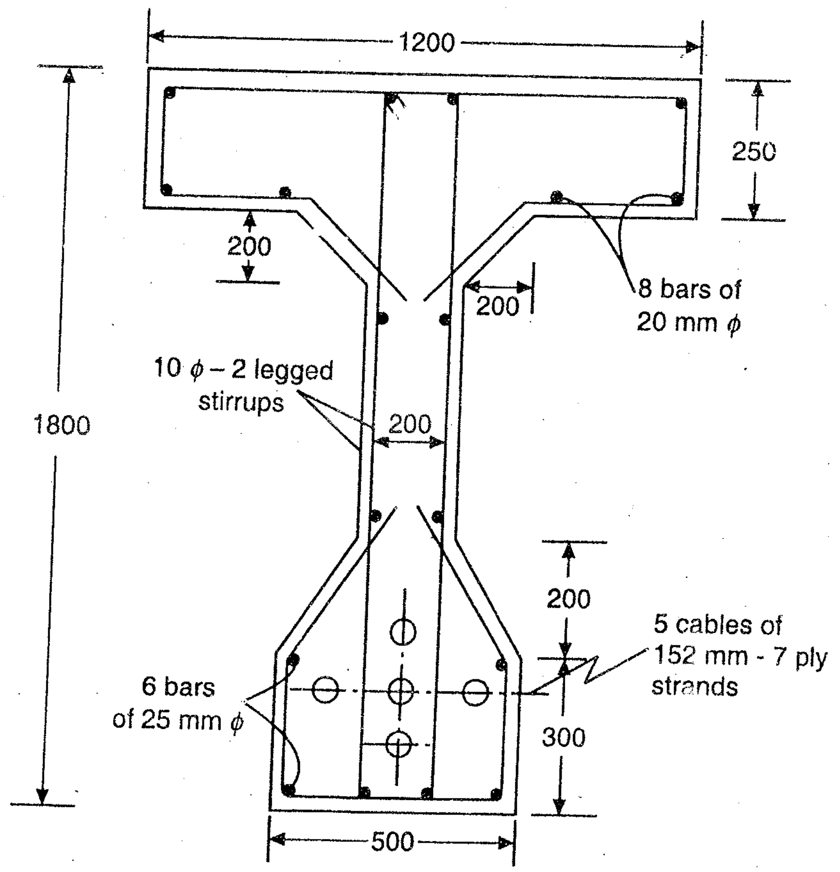


Fig. 10.25 Reinforcement Details at Centre of Span Section.

10. Design of End Block

Solid end blocks are provided at end supports over a length of 1.5 m. Typic equivalent prisms on which the anchorage forces are considered to be effective are detailed in Fig. 12. The bursting tension is computed using the data given in Table 10.2.

In the horizontal plane we have the data,

$$P_k = 1459 \text{ kN}, \quad 2y_{po} = 225 \text{ mm}, \quad \text{and} \quad 2y_0 = 900 \text{ mm}.$$

$\therefore$ The ratio $(y_{po}/y_o) = (112.5/450) = 0.25$

Bursting Tension, $F_{bst} = (0.23 \times 1459) = 336$ kN

Area of steel required to resist this tension is obtained as

$$A_s = (336 \times 10^3)/(0.87 \times 415) = 931 \text{ mm}^2$$

Provide 10 mm diameter bars at 100 mm centres in the horizontal direction. In the vertical plane, the ratio of (y_{po}/y_o) being higher the magnitude of bursting tension is smaller. However the same reinforcements are provided in the form of a mesh both in the horizontal and vertical directions as shown in Fig. 10.26.

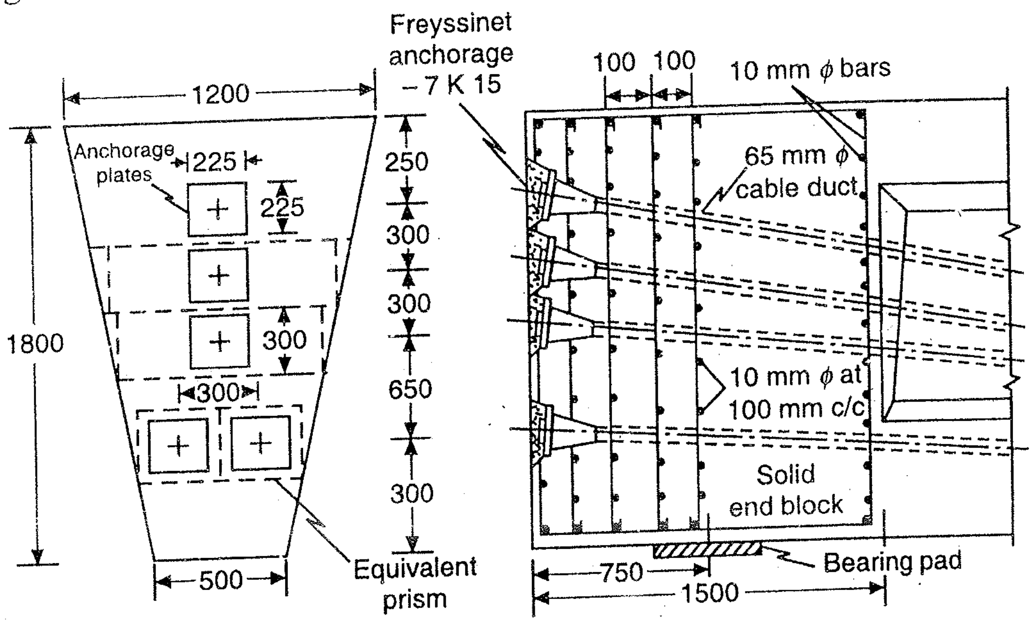


Fig. 10.26 Equivalent Prisms and Anchorage Zone Reinforcement.

(1) *Cross Girders*

The cross girders of width 200 mm and depth 1250 mm are provided with nominal reinforcement of 0.15 percent of the cross section, consisting of 12 mm diameter bars spaced two at top, two at mid depth and two at bottom. Also provide nominal stirrups made up of 10 mm diameter two legged links at 200 mm centers. Two cables consisting of 12 numbers of 7 mm high