PRE-STRESSED CONCRETE BRIDGES

21.2 ADVANTAGES OF PRESTRESSED CONCRETE BRIDGES

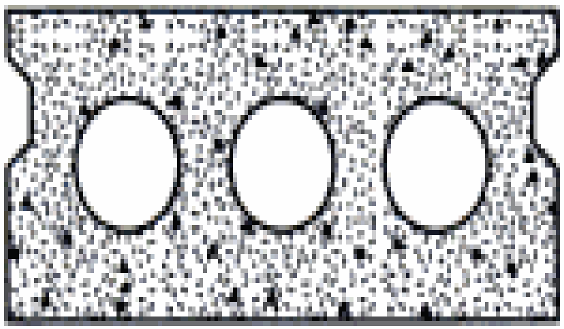
Prestressed concrete which is made up of high-strength concrete and high-tensile steel has distinct advantages in bridge construction. The salient benefits accruing from the use of prestressed concrete in bridges are outlined as follows:

1. High-strength concrete and high-tensile steel, besides being economical, make for slender sections, which are aesthetically superior.
2. Prestressed concrete bridges can be designed as class 1 type structures without any tensile stresses under service loads, thus resulting in a crack-free structure.

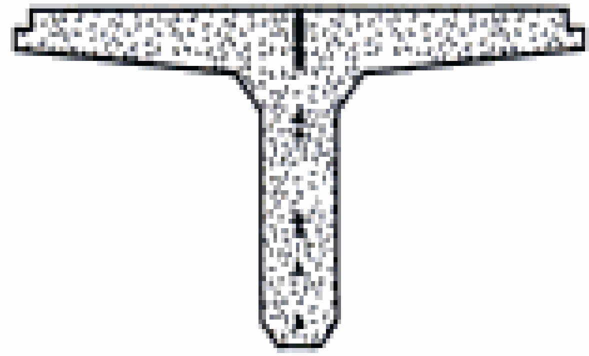
3. In comparison with steel bridges, prestressed concrete bridges require very little maintenance.
4. Prestressed concrete is ideally suited for composite bridge construction in which precast prestressed girders support the cast *in situ* slab deck. This type of construction is very popular since it involves minimum disruption of traffic.
5. Post-tensioned prestressed concrete finds extensive applications in long-span continuous girder bridges of variable cross-section. Not only does it make for sleek structures, but it also effects considerable saving in the overall cost of construction.
6. In recent years, partially prestressed concrete (type-3 structure) has been preferred for bridge construction, because it offers considerable economy in the use of costly high-tensile steel in the girder.

PRETENSIONED PRESTRESSED CONCRETE BRIDGES

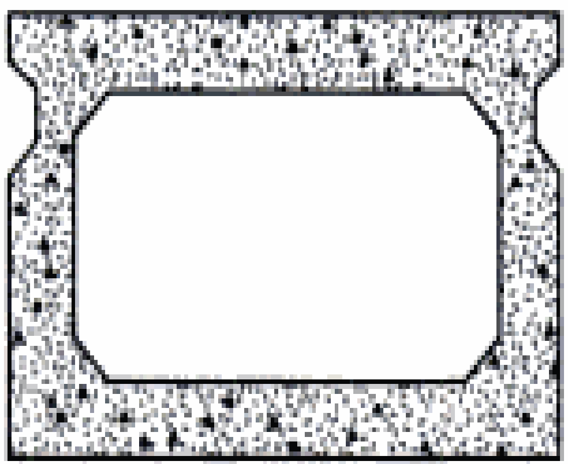
Pretensioned prestressed concrete bridge-decks generally comprise precast pretensioned units used in conjunction with cast *in situ* concrete, resulting in composite bridge decks which are ideally suited for small and medium spans in the range of 20 to 30 m. In general, pretensioned girders are provided with straight tendons. The use of seven-wire strands has been found to be advantageous in comparison with plain or indented wires. In U.S.A.[4], deflected strands are employed in larger girders.

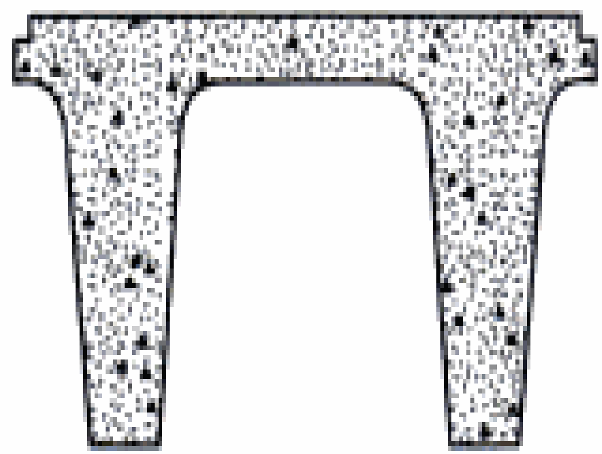


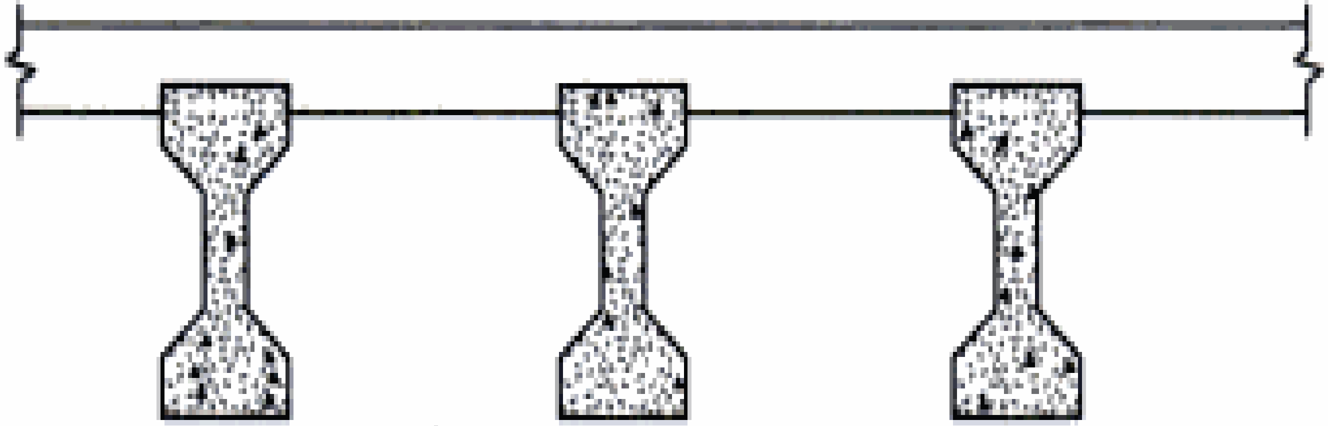
(a) Voided slab



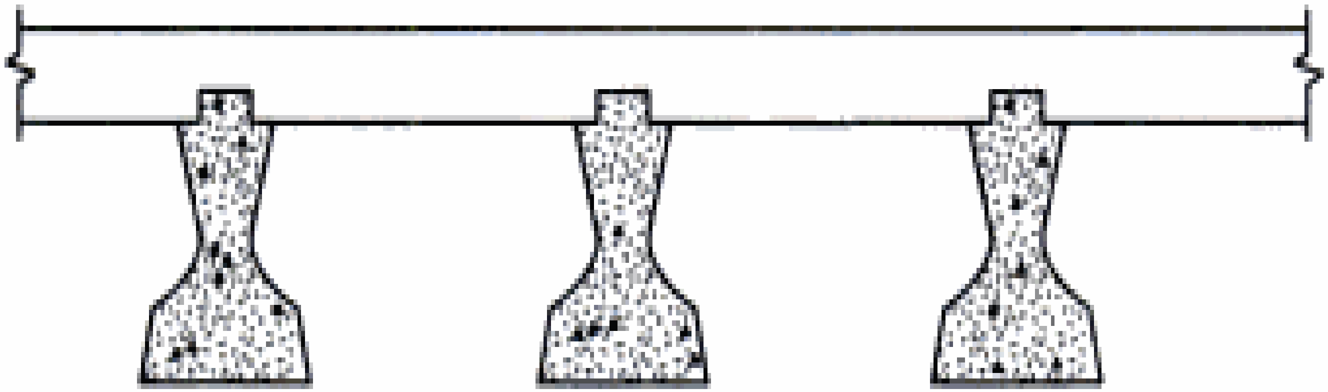
(b) Single tee







(e) AASHO–type girders with slab (U.S.A.)

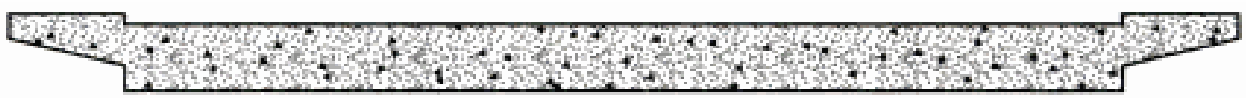


21.5 DESIGN OF POST-TENSIONED PRESTRESSED CONCRETE SLAB BRIDGE DECK

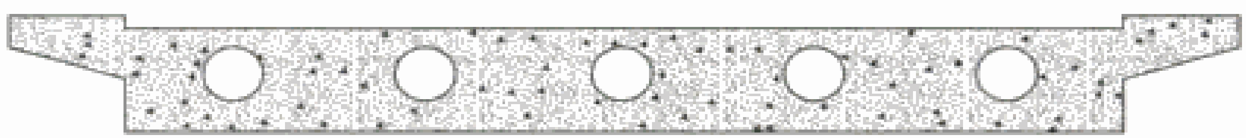
Design a post-tensioned prestressed concrete slab bridge deck for a national highway crossing to suit the following data:

1. Data

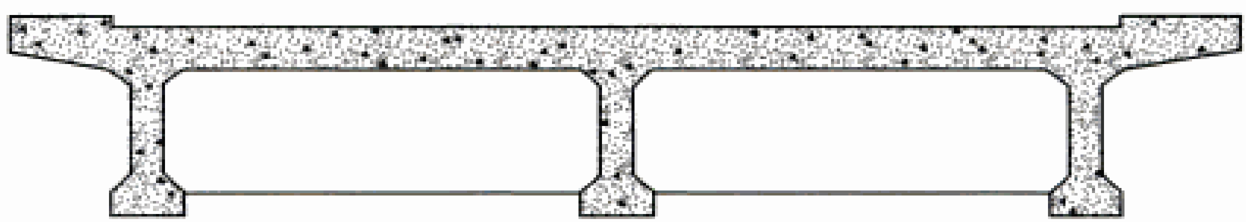
Clear span	10 m
Width of bearing	400 mm
Clear width of roadway	7.5 m



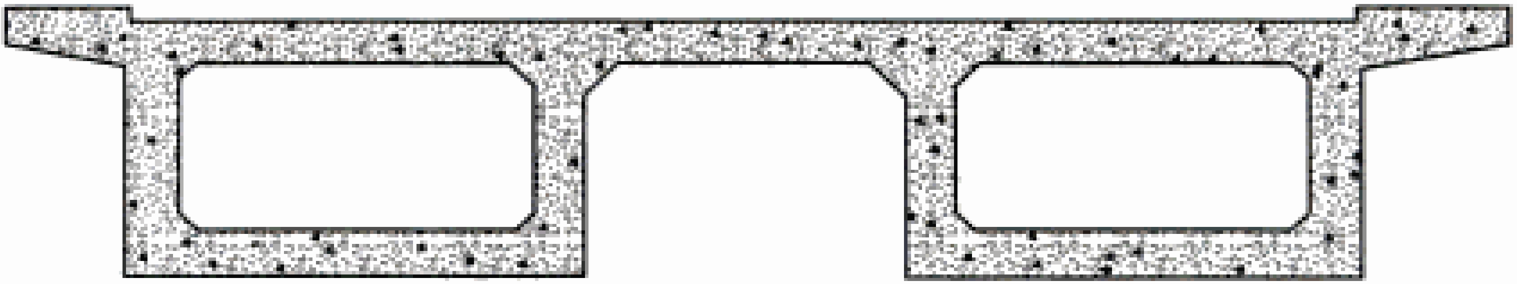
(a) Solid slab (10 to 15 m)



(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. 21.3 *Typical Cross-sections of Post-Tensioned Prestressed Concrete Bridge Decks*

Footpath	1 m on either side
Kerbs	600 mm wide
Thickness of wearing coat	80 mm
Live load	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

2. *Permissible stresses* The permissible compressive stresses in concrete at transfer and working loads, as recommended in IRC–18 are as follows:

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

$$\text{Loss ratio, } \eta = 0.80$$

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

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

3. Depth of slab and effective span Assuming the thickness of the slab at 50 mm per metre of span for highway bridge decks, the overall thickness of the slab = $(10 \times 50) = 500$ mm

Width of bearing = 400 mm

Effective span = 10.4 m

The cross-section of the deck slab is shown in Fig. 21.5.

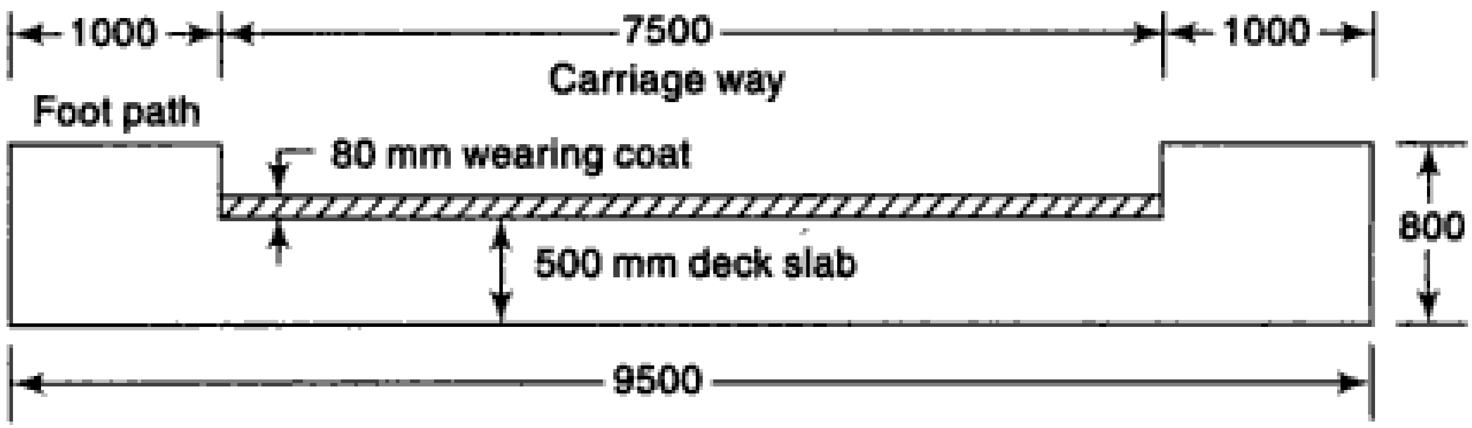


Fig. 21.5 *Cross-section of Deck Slab*

4. Dead load bending moments Dead weight of slab = $(0.5 \times 24) = 12$ kN/m^2

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

Total dead load = 14.00 kN/m^2

Dead load bending moment $(M_g) = (14 \times 10.4^2)/8 = 190$ kN m

5. Live load bending moments Generally, the bending moment due to live load will be maximum for IRC class *AA* tracked vehicle. Impact factor for the class *AA* tracked vehicle is 25 per cent for the 5 m span, which decreases linearly to 10 per cent for the 9 m span.

∴ Impact factor = 10 per cent 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 the slab perpendicular to the span is expressed as,

$$b_e = kx\left(1 - \frac{x}{L}\right) + b_w$$

Referring to Fig. 21.6,

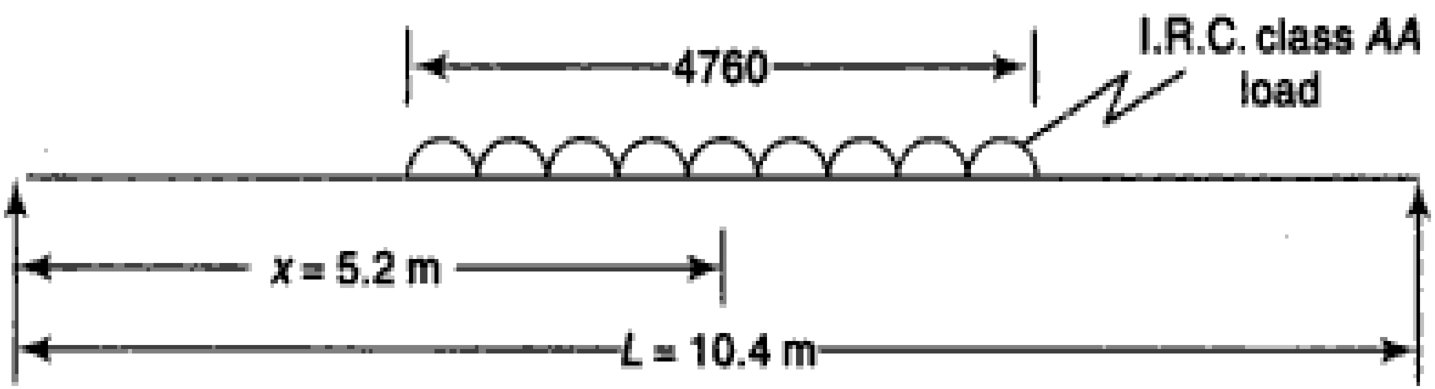


Fig. 21.6 *Position of Load for Maximum Bending Moment*

$$x = 5.2\text{ m},\quad L = 10.4\text{ m},\quad B = 9.5\text{ m}$$

$$\therefore \quad \left(\frac{B}{L}\right) = \left(\frac{9.5}{10.4}\right) = 0.913$$

and $$b_w = (0.85 + 2 \times 0.08) = 1.01\text{ m}$$

From Table 21.2 for $\left(\frac{B}{L}\right) = 0.913$,

and simply supported slabs, $k = 2.37$

$$\therefore \quad b_e = 2.37 \times 5.2\left(1 - \frac{5.2}{10.4}\right) + 1.01 = 7.172\text{ m}$$

The tracked vehicle is placed close to the kerb with the required minimum clearance as shown in Fig. 21.7.

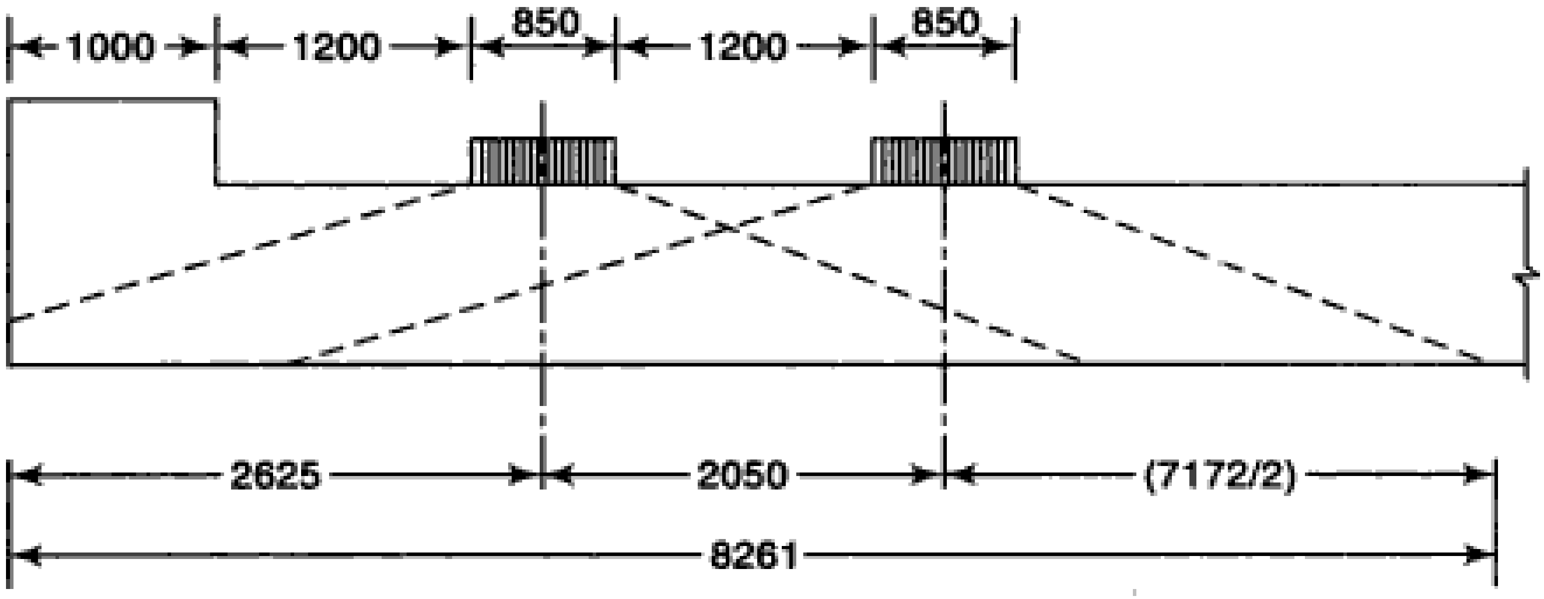


Fig. 21.7 *Effective Width of Dispersion for IRC Class AA Tracked Vehicle*

Net effective width of dispersion = 8.261 m

Total load of two tracks with impact = (700 × 1.10) = 770 kN

$$\text{Average intensity of load} = \frac{770}{(4.76 \times 8.261)} = 19.58 \text{ kN/m}^2$$

Maximum bending moment due to live load is given by

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

6. Shear due to class AA tracked vehicle For maximum shear force at the support section the IRC class *AA* tracked vehicle is arranged as shown in Fig. 21.8.

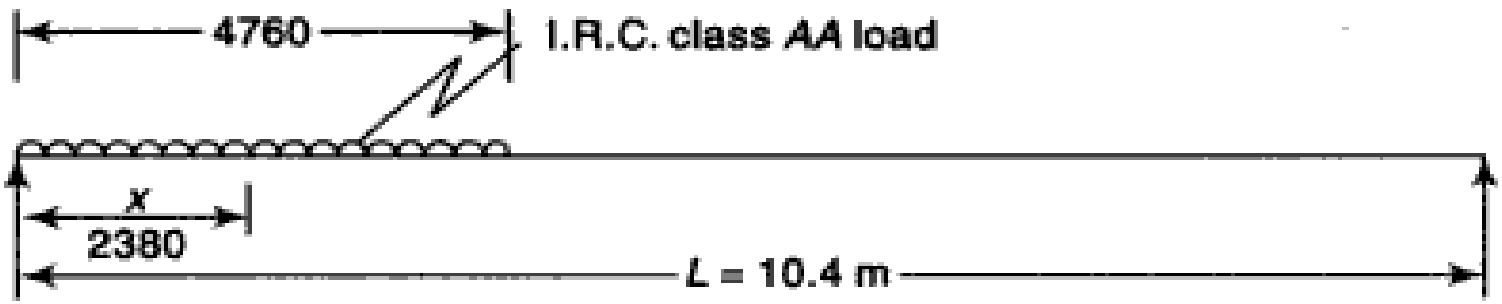


Fig. 21.8 *Position of Load for Maximum Shear*

Effective width of dispersion is given by

$$b_e = kx\left(1 - \frac{x}{L}\right) + b_w$$

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

$$\left(\frac{B}{L}\right) = \left(\frac{9.5}{10.4}\right) = 0.913$$

$\therefore$ From Table 21.2, for $\left(\frac{B}{L}\right) = 0.913$, the value of $k = 2.37$.

$\therefore$

$$b_e = \left[2.37 \times 2.38\left(1 - \frac{2.38}{10.4}\right) + 1.01\right] = 5.364 \text{ m}$$

Referring to Fig. 21.7,
Width of dispersion for two tracks

$$= \left[2625 + 2050 + \left(\frac{5364}{2}\right)\right] = 7357 \text{ mm}$$

$\therefore$ Intensity of load $= \left[\dfrac{770}{(4.76 \times 7.357)}\right] = 22 \text{ kN/m}^2$

$\therefore$ Shear force, $V_A = \dfrac{(22 \times 4.76 \times 8.02)}{10.4} = 80.75 \text{ kN}$

Dead load shear $= (0.5 \times 14 \times 10.4) = 72.8 \text{ kN}$

$\therefore$ Total design shear $= (80.75 + 72.80) = 153.55 \text{ kN}$

7. Check for minimum section modulus

Dead load moment, $M_g = 190$ kN m

Live load moment, $M_q = 187$ kN m

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

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

$$f_{ct} = 15.0 \text{ N/mm}^2, f_{cw} = 12.0 \text{ N/mm}^2, f_{tw} = 0$$

Loss ratio, $\eta = 0.8$

$$f_{br} = (\eta f_{ct} - f_{tw}) = (0.8 \times 15 - 0)$$
$$= 12.0 \text{ N/mm}^2$$

The minimum section modulus is given by

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

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

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

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

$$P = \left[\frac{A(f_{inf} Z_b + f_{sup} Z_t)}{Z_b + Z_t}\right]$$

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

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

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

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

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

$$= 1687.5 \text{ kN}$$

Using Freyssinet cables containing 12 wires of 7 mm diameter which are stressed to 1200 N/mm^2,

$$\text{Force in each cable} = \frac{(12 \times 38.5 \times 1200)}{1000} = 554 \text{ kN}$$

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

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 the centre of span which reduces to zero (concentric) at supports.

10. Check for stresses at service loads

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

$A = (1000 \times 500) = 5 \times 10^5 \text{ mm}^2$

$Z_t = Z_b = Z = 41.66 \times 10^6 \text{ mm}^3$

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

$$\left(\frac{P}{A}\right) = \left(\frac{1687.5 \times 10^3}{5 \times 10^5}\right) = 3.375 \text{ N/mm}^2$$

$$\left(\frac{P_e}{Z}\right) = \left(\frac{1687.5 \times 10^3 \times 195}{41.66 \times 10^6}\right) = 7.89 \text{ N/mm}^2$$

$$\left(\frac{M_g}{Z}\right) = \left(\frac{190 \times 10^6}{41.66 \times 10^6}\right) = 4.56 \text{ N/mm}^2$$

$$\left(\frac{M_q}{Z}\right) = \left(\frac{187 \times 10^6}{41.66 \times 10^6}\right) = 4.48 \text{ N/mm}^2$$

Stresses at transfer

At top of slab $= (3.375 - 7.89 + 4.56) = 0.045$ N/mm^2

At bottom of slab $= (3.375 + 7.89 - 4.56) = 6.705$ N/mm^2

Stresses at working loads

At top of slab $= 0.8\,(3.375 - 7.89) + 4.56 + 4.48 = 5.428$ N/mm^2

At bottom of slab $= 0.8\,(3.375 + 7.89) - 4.56 - 4.48 = -0.028$ N/mm^2

The actual stresses developed are within the permissible limits.

11. Check for ultimate strength (IRC: 18–2000)

Considering 1 m width of the slab,

$$b = 1000 \text{ mm}$$

$$A_p = \frac{(12 \times 38.5 \times 1000)}{328} = 1408 \text{ mm}^2$$

$$d = 445 \text{ mm} \qquad f_p = 1500 \text{ N/mm}^2$$

(a) Failure by yielding of steel

$$M_u = 0.9\, dA_p f_p$$

$$= (0.9 \times 445 \times 1408 \times 1500)$$

$$= 846 \times 10^6 \text{ Nmm} = 846 \text{ kNm}$$

(b) Failure by crushing of concrete

$$M_u = 0.176\, bd^2 f_{ck}$$
$$= (0.176 \times 1000 \times 445^2 \times 40)$$
$$= 1394 \times 10^6 \text{ N mm} = 1394 \text{ kNm}$$

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 $= 1.5\, M_g + 2.5\, M_q$
$$= (1.5 \times 190) + (2.5 \times 187)$$
$$= 285 + 467.5 = 752.5 \text{ kN m}$$

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