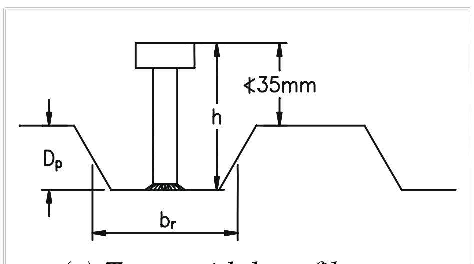
22 COMPOSITE BEAMS – II

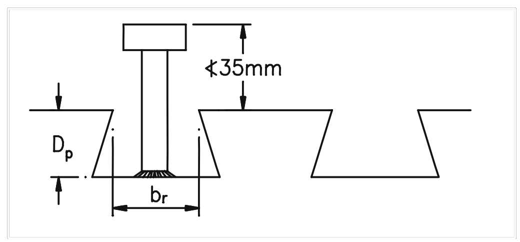
1.0 INTRODUCTION

A steel concrete composite beam consists of a steel beam, over which a reinforced concrete slab is cast with shear connectors, as explained in the previous chapter. Since composite action reduces the beam depth, rolled steel sections themselves are found adequate frequently (for buildings) and built-up girders are generally unnecessary. The composite beam can also be constructed with profiled sheeting with concrete topping, instead of cast-in place or precast reinforced concrete slab. The profiled sheets are of two types

- Trapezoidal profile
- Re-entrant profile



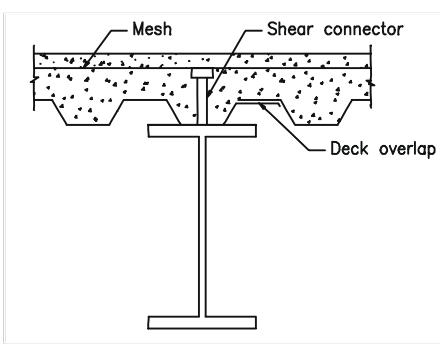
(a) Trapezoidal profile



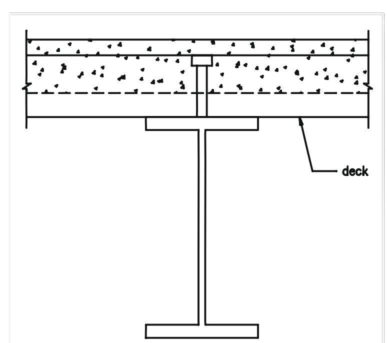
(b) Re-entrant profile

Fig. 1 Types of profile deck

These two types are shown in Fig *1*. The profiled steel sheets are provided with indentations or embossments to prevent slip at the interface. The shape of the re-entrant form, itself enhances interlock between concrete and the steel sheet. The main advantage of using profiled deck slab is that, it acts as a platform and centering at construction stage and also serves the purpose of bottom reinforcement for the slab.



(a) Ribs parallel to the beam



(b) Ribs perpendicular to the beam

Fig. 2. Orientation of Profiled deck slab in a composite beam

The deck slab with profiled sheeting is of two types (see Fig *2*).

- The ribs of profiled decks running parallel to the beam
- The ribs of profiled decks running perpendicular to the beam.

2.0 PROVISION FOR SERVICE OPENING IN COMPOSITE BEAMS

There is now a growing demand for longer spans, either for open plan offices, or to permit greater flexibility of office layout, or for open exhibition and trading floors. For these longer spans, the choice of structural form is less clear cut largely on account of the need for providing for services satisfactorily. Service openings can be easily designed in conventional rolled steel beams. Conventional construction may still be appropriate, but other, more novel, structural forms may offer economy or other overriding advantages, besides easy accommodation of services. Open web joist floor system may be one such solution for longer span (see the chapter on trusses). In fact, many of these were developed in Great Britain and a number of Design Guides have been produced by the Steel Construction Institute.

2.1 Simple Construction with Rolled Sections(1)

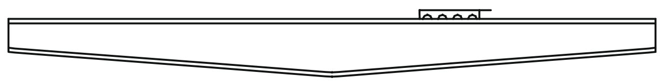
For spans in the range of *6* to *10 m*, perhaps the most appropriate form of construction is rolled sections and simple, shear only connections. Secondary beams at *2.4 m* or *3.0 m* centres support lightweight composite floor slabs and span onto primary beams, which in turn frame directly into the columns. The same form of construction may also be used for longer span floors but beam weights and costs increase to the point where other forms of construction may be more attractive. Of increasing concern to developers is the provision of web openings as these are inflexible and they can create difficulties in meeting the specific needs of tenants or in subsequent reservicing during the life of the structure.

2.2 Fabricated Sections(2)

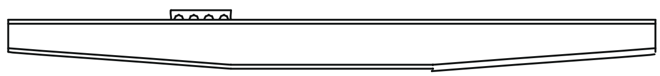
The use of fabricated sections for multi-storey buildings has been explored by some U.K.designers. This usage became economic with advances in the semi-automatic manufacture of plate girder sections. Different approaches to manufacture have been developed by different fabricators. Significant savings in weight can be achieved due to the freedom, within practical limits, to tailor the section to suit its bending moment and shear force envelopes. Depth, taper and shape flange size and web thickness may all be selected independently by the designer.

Fabricated sections are most likely to be economic for spans above *12 m*. Above this span length, rolled sections are increasingly heavy and a fine-tuned fabricated section is likely to be able to save on both flange size and web thickness. With some manufacturing processes, asymmetric sections with narrow top flanges can be adopted, achieving further weight savings.

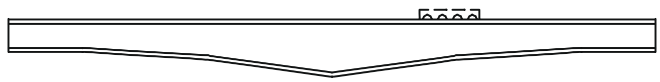
The freedom to tailor the fabrication to the requirements of the designer allows the depth of the girder to be varied along its length and to allow major services to run underneath the shallower regions. A range of shapes is feasible (see Fig.*3*) of which the semi-tapered beam is the most efficient structurally but can only accommodate relatively small ducts. The straight-tapered beams shown in Fig *3(a)* offers significantly more room for ducts, at the expense of some structural efficiency, and has proved to be the most popular shape to date. Cranked taper beams can also be used, providing a rectangular space under the beams at their ends. Fabricated beams are often employed to span the greater distance, and supporting shorter span primary beams of rolled sections.



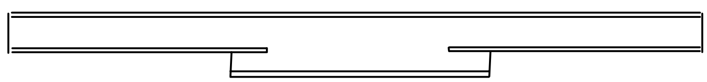
(a) Straight Taper



(b) Semi-Taper



(c)Cranked Taper



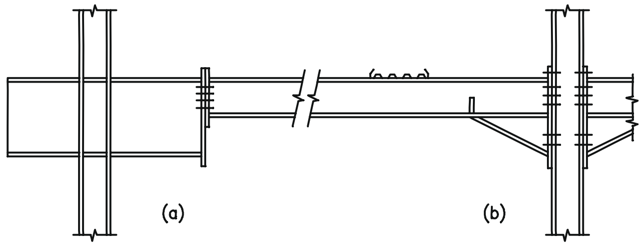
(d) Stepped Beam
(where automatic welding is not crucial)

Fig. 3. Fabricated sections for commercial buildings

2.3 Haunched Beams(3)

In traditional multi-storey steel frames, the conventional way to achieve economy is to use ‘simple’ design. In a long span structure, there is perhaps twice the length of primary beams compared to the columns and for a low rise building their mass/metre will be comparable. In these circumstances the economic balance may shift in favour of sacrificing column economy in order to achieve greater beam efficiency by having moment resisting connections. The benefits of continuity are particularly significant when stiffness rather than the strength governs design, and this is increasingly likely as spans increase. Where fully rigid design is adopted, the beam to column connection is likely to have to develop the hogging bending capacity of the composite section. Until our design concepts on composite connections are more fully developed, designers have to rely on an all-steel connection and this will usually require substantial stiffening and could prove to be expensive.

The most straightforward way to reduce connection costs is to use some form of haunched connection (Fig. *4*); they occupy the region below the beam, which is anyway necessary for the main service ducts. (With haunched beams, the basic section is usually too shallow for holes to be formed in its web that are sufficiently large to accommodate main air-conditioning ducts). Thus the haunches simplify beams of column connection significantly and improve beam capacity and stiffness without increasing the overall floor depth.



(a) sections of different size
(b) haunches cut from main beam
Fig. 4 Haunched beams: Two types of haunches

2.4 Parallel Beam Approach[4]

In the parallel beam approach, it is the secondary beams that span the greater distance. A very simple form of construction results as they run over the primary beams and achieve continuity without complex connections (see Fig. *5*).

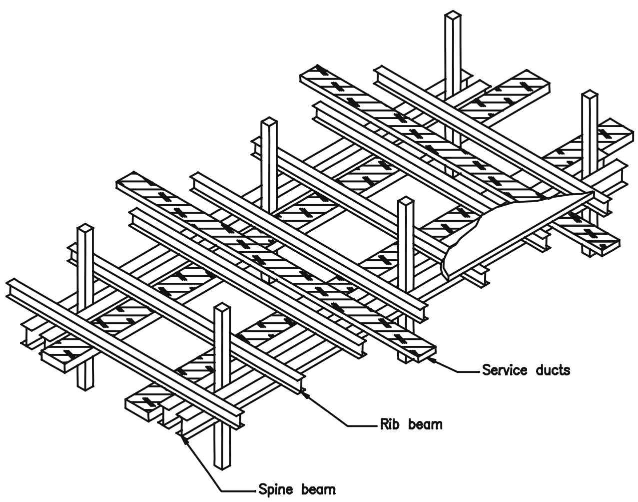


Fig. 5. Parallel beam grillage

The primary or spine beams also achieve continuity by being used in pairs with one beam passing on either side of the columns. Shear is transferred into the columns by means of brackets. This ‘offset’ construction, where members are laid out in the three orthogonal directions deliberately to miss each other enable continuity of the beams to be achieved without the high cost of moment resisting connections; this improves the structural efficiency and (of particular importance for long span construction) stiffness. There is also a considerable saving of both erection time and erection cost. Because continuity is such an integral part of the approach, it is primarily applicable for multi-bay layouts.

Superficially, the approach appears to lead to deeper construction. However, because of continuity, the primary and secondary beams can both be very shallow for the spans and overall depths are comparable with conventional construction. Most importantly, the separation of the two beam directions into different planes creates an ideal arrangement for the accommodation of services.

2.5 Castellated Sections [5]

Castellated beams are made from Rolled Steel beams by fabricating openings in webs, spaced at regular intervals. Castellated sections have been used for many years (see Fig.*6*) as long span roof beams where their attractive shape is often expressed architecturally. The combination of high bending stiffness and strength per unit weight with relatively low shear capacity is ideal for carrying light loads over long spans. As composite floor beams, their usage is limited by shear capacity. These are generally unsuitable for use as primary beams in a grillage, because the associated shears would require either stiffening to or infilling of the end openings, thereby increasing the cost to the point that other types of beams become economical. However, if the castellated sections are used to span the longer direction directly, then the shear per beam drops to the level at which the unstrengthened castellated sections can be used.

The openings in the castellated beams allow the accommodation of circular ducts used for many air-conditioning systems. There are, in addition, plenty of openings for all the other services, which can be distributed throughout the span effectively without any consideration of their interaction with the structure. It is also possible, near mid-span, to cut out one post and thereby create a much larger opening encompassing two conventional castellations. The shear capacity of this opening will need careful checking, taking due account of eccentric part span loading and associated midspan shears. If this opening needs strengthening then longitudinal stiffeners at top and bottom are likely to be adequate.

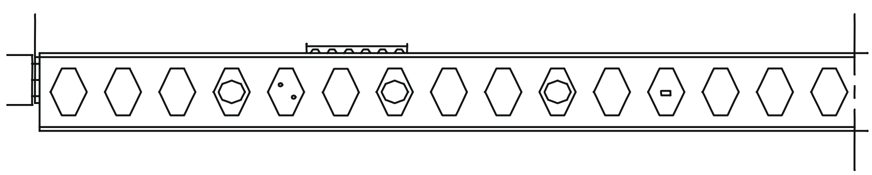


Fig. 6. Castellated beams

2.6 Stub Girders [6]

Stub Girders comprise a steel bottom chord with short stubs connecting it to the concrete or profiled sheet slab (Fig. *7*). Openings for services are created adjacent to the stubs. Bottom chords will need to be propped during construction, if this method is used.

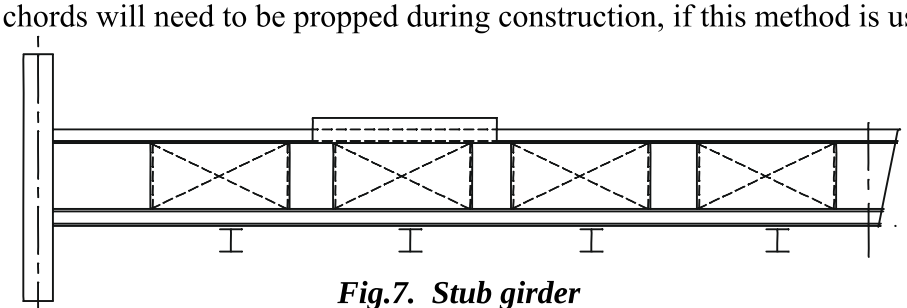


Fig.7. Stub girder

2.7 Composite Trusses [7]

Consider a steel truss acting compositely with the floor slab (Fig. *8*). Bracing members can be generally eliminated in the central part of the span, so that – if needed – large rectangular ducts can pass between bracing members. The chords are fabricated from T sections or cold formed shapes and bracing members from angles. As is obvious from the above discussion, several innovative forms of composite beam using profiled steel deck have been developed in recent years. The designer has, therefore, a wide choice in selecting an appropriate form of flooring using these concepts.

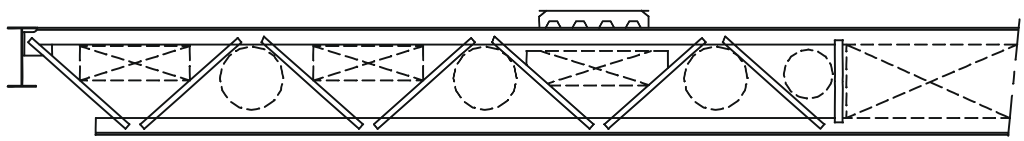


Fig. 8. Composite trusses

3.0 BASIC DESIGN CONSIDERATIONS

3.1 Design Method suggested by Eurocode 4[8]

For design purpose, the analysis of composite section is made using Limit State of collapse method. *IS:11384-1985* Code deals with the design and construction of only simply supported composite beams. Therefore, the method of design suggested in this chapter largely follows *EC4.* Along with this, *IS:11384-1985* Code provisions and its limitations are also discussed.

The ultimate strength of composite section is determined from its plastic capacity, provided the elements of the steel cross section do not fall in the semi-compact or slender category as defined in the section on plate buckling. The serviceability is checked using elastic analysis, as the structure will remain elastic under service loading. Full shear connection ensures that full moment capacity of the section develops. In partial shear connection, although full moment capacity of the beam cannot be achieved, the design will have to be adequate to resist the applied loading. This design is sometimes preferred due to economy achieved through the reduced number of shear connector to be welded at site.

3.2 Span to depth ratio

EC4 specifies the following span to depth (total beam and slab depth) ratios for which the serviceability criteria will be deemed to be satisfied.

Table 1 Span to Depth ratio as according to EC4

	EC4
Simply supported	15-18 (Primary Beams) 18-20 (Secondary Beams)
Continuous	18-22 (Primary Beams) 22-25 (end bays)

3.3 Effective breadth of flange

A composite beam acts as a T-beam with the concrete slab as its flange. The bending stress in the concrete flange is found to vary along the breadth of the flange as in Fig *9*, due to the shear lag effect. This phenomenon is taken into account by replacing the actual breadth of flange *(B)* with an effective breadth *(b_{eff}),* such that the area *FGHIJ* nearly equals the area *ACDE*. Research based on elastic theory has shown that the ratio of the effective breadth of slab to actual breadth (b_{eff}/B) is a function of the type of loading, support condition, and the section under consideration. For design purpose a portion of the beam span *(20% - 33%)* is taken as the effective breadth of the slab.

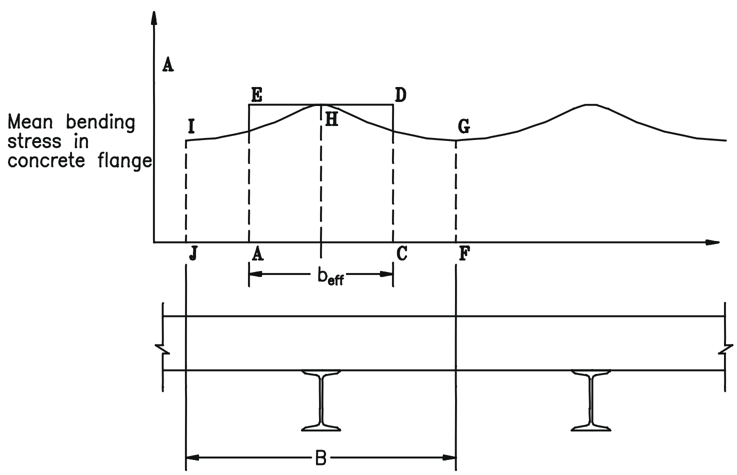


Fig. 9. Use of effective width to allow for shear lag

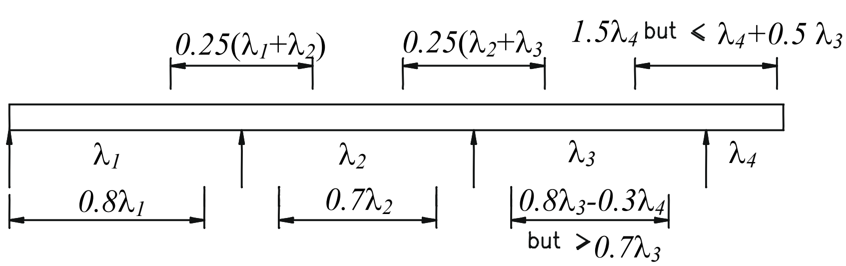


Fig. 10 Value of λ_0 for continuous beam as per EC4

In *EC4,* the effective breadth of simply supported beam is taken as $\lambda_o/8$ on each side of the steel web, but not greater than half the distance to the next adjacent web. For simply supported beam $\lambda_o = \lambda$ Therefore,

$$b_{eff} = \frac{\lambda}{4} \quad but \le B$$

where,

λ_o = The effective span taken as the distance between points of zero moments.
λ = Actual span
B = Centre to centre distance of transverse spans for slab.

For continuous beams λ_o is obtained from Fig *10*.

3.4 Modular ratio

Modular ratio is the ratio of elastic modulus of steel (E_s) to the time dependent secant modulus of concrete (E_{cm}). While evaluating stress due to long term loading (dead load etc.) the time dependent secant modulus of concrete should be used. This takes into account the long-term effects of creep under sustained loading. The values of elastic modulus of concrete under short term loading for different grades of concrete are given in Table *2*.

IS:11384 -1985 has suggested a modular ratio of *15* for live load and *30* for dead load, for elastic analysis of section. It is to be noted that a higher value of modular ratio for dead load takes into account the larger creep strain of concrete for sustained loading. In *EC 4* the elastic modulus of concrete for long-term loads is taken as one-third of the short-term value and for normal weight concrete, the modular ratio is taken as *6.5* for short term loading and *20* for long term loading.

Table 2 Properties of concrete

Grade Designation	*M25*	*M30*	*M35*	*M40*
$(f_{ck})_{cu}$ (N/mm^2)	25	30	35	40
$E_{cm}=5700\sqrt{(fck)_{cu}}(N/mm^2)$	28500	31220	33720	36050

3.5 Shear Connection

The elastic shear flow at the interface of concrete and steel in a composite beam under uniform load increases linearly from zero at the centre to its maximum value at the end. Once the elastic limit of connectors is reached, redistribution of forces occurs towards the less stressed connectors as shown in Fig *11* in the case of flexible shear connectors (such as studs). Therefore at collapse load level it is assumed that all the connectors carry equal force, provided they have adequate shear capacity and ductility. In *EC4,* the design capacity of shear connectors is taken as *80%* of their nominal static strength. Though, it may be considered as a material factor of safety, it also ensures limit condition to be reached by the flexural failure of the composite beam, before shear failure of the interface.

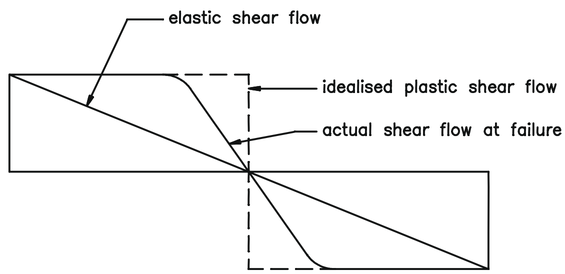


Fig 11. Shear flow at interface

The design strength of some commonly used shear connectors as per *IS:11384-1985* is given in Table *1* of the previous chapter (Composite Beam-I).

3.6 Partial Safety Factor

3.6.1 Partial safety factor for loads and materials **–** The suggested partial safety factors for load, γ_f and for materials, γ_m are shown in Table *3*.

Table 3 Partial safety factors as per the proposed revisions to IS: 800

Load	**Partial safety factor, γ_f**
Dead load	1.35
Live load	1.5
Materials	**Partial safety factor, γ_m**
Concrete	1.5
Structural Steel	1.15
Reinforcement	1.15

3.7 Section Classifications

Local buckling of the elements of a steel section reduces its capacity. Because of local buckling, the ability of a steel flange or web to resist compression depends on its slenderness, represented by its breadth/thickness ratio. The effect of local buckling is therefore taken care of in design, by limiting the slenderness ratio of the elements i.e. web and compression flange. The classification of web and compression flange is presented in the Table *4*.

Table 4 Classification of Composite Section

Type of Element	**Type of Section**	**Class of Section**		
		Plastic	**Compact**	**Semi-compact**
Outstand element of compression flange	Built up by welding	$b/T \leq 7.9\in$	$b/T \leq 8.9\in$	$b/T \leq 13.6\in$
	Rolled section	$b/T \leq 8.9\in$	$b/T \leq 9.9\in$	$b/T \leq 15.7\in$
Web, with neutral axis at mid-depth	All sections	$d/t \leq 83\in$	$d/t \leq 103\in$	$d/t \leq 126\in$
Web, generally	All section	$\frac{d}{t} \leq \frac{83\in}{0.4+0.6\alpha}$	$\frac{d}{t} \leq \frac{103\in}{\alpha}$	when $R > 0.5$ for welded section $\frac{d}{t} \leq (109-80R)\in$ for rolled section $\frac{d}{t} \leq (98-57R)\in$ when $R \leq 0.5$ but > -0.45 $\frac{d}{t} \leq \frac{126\in}{1+1.6R}$

where,

b = half width of flange of rolled section
T = Thickness of top flange
d = clear depth of web

$$\alpha = \frac{2\,Y_c}{d} \geq 0$$

where, Y_c is the distance from the plastic neutral axis to the edge of the web connected to the compression flange. But if $\alpha > 2$, the section should be taken as having compression through out.

$$\in = \text{constant} = \sqrt{\frac{250}{f_y}}$$

t = thickness of web

R = is the ratio of the mean longitudinal stress in the web to the design strength.

f_y with compressive stress taken as positive and tensile stress negative.

If the compression flange falls in the plastic or compact category as per the above classification, plastic moment capacity of the composite section is used provided the web is not slender. For compression flange, falling in semi-compact or slender category elastic moment capacity of the section is used.

4.0 DESIGN OF COMPOSITE BEAMS

4.1 Moment Resistance

4.1.1 Reinforced Concrete Slabs, supported on Steel beams

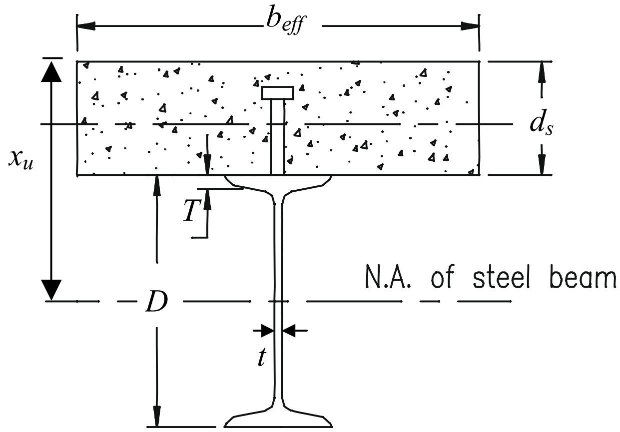


Fig. 12. Notations as per IS: 11384-1985

Reinforced concrete slab connected to rolled steel section through shear connectors is perhaps the simplest form of composite beam. The ultimate strength of the composite beam is determined from its collapse load capacity. The moment capacity of such beams can be found by the method given in *IS:11384-1985*. In this code a parabolic stress distribution is assumed in the concrete slab. The equations used are explained in detail in the previous chapter (Composite Beam-I) and are presented in Table *5*. Reference can be made to Fig. *12* for the notations used in *IS:11384-1985.*

IS: 11384 – 1985, gives no reference to profiled deck slab and partial shear connection. Therefore the equations given in Table *5* can be used only for composite beams without profiled deck sheeting (i.e., steel beam supporting concrete slabs).

Note: 1) Total compressive force in concrete is taken to be $F_{cc}=0.36\,(f_{ck})_{cu}\,b_{eff}\;x_u$ and acting at a depth of $0.42x_u$ from top of slab, where x_u is the depth of plastic neutral axis.

2) $a=\dfrac{0.87\,f_y}{0.36\,(f_{ck})_{cu}}$

Table 5 Moment capacity of composite Section with full shear interaction (according to IS:11384 - 1985)

Position of Plastic Neutral Axis	**Value of x_u**	**Moment Capacity M_p**
Within slab	$x_u = a\,A_a / b_{eff}$	$M_p = 0.87 A_a f_y\,(d_c + 0.5 d_s - 0.42\,x_u)$
Plastic neutral axis in steel flange	$x_u = d_s + \dfrac{(aA_a - b_{eff} d_s)}{2Ba}$	$M_p = 0.87 f_y\,[A_a\,(d_c + 0.08\,d_s) - B(x_u - d_s)(x_u + 0.16\,d_s)]$
Plastic neutral axis in web	$x_u = d_s + T + \dfrac{a(A_a - 2A_f) - b_{eff} d_s}{2at}$	$M_p = 0.87 f_y\,A_s\,(d_c + 0.08\,d_s) - 2A_f\,(0.5T + 0.58\,d_s) - 2t(x_u - d_s - T)(0.5\,x_u + 0.08\,d_s + 0.5\,T)$

4.1.2 Reinforced concrete slabs, with profiled sheeting supported on steel beams

A more advanced method of composite beam construction is one, where profiled deck slabs are connected to steel beams through stud connectors. In this case the steel sheeting itself acts as the bottom reinforcement and influences the capacity of the section. Table *6* presents the equations for moment capacity according to *EC4*. These equations are largely restricted to sections, which are capable of developing their plastic moment of resistance without local buckling problems. These equations are already discussed in the previous chapter. Fig *13* shows the stress distribution diagram for plastic and compact sections for full interaction according to *EC4*. Fig *14* shows the stress distribution for hogging bending moment.

The notations used here are as follows: -

A_a =area of steel section
γ_a = partial safety factor for structural steel
γ_c = partial safety factor for concrete
b_{eff} =effective width of flange of slab
f_y =yield strength of steel
$(f_{ck})_{cy}$ =characteristic (cylinder) compressive strength of concrete
(f_{sk}) = yield strength of reinforcement.
h_c =distance of rib from top of concrete
h_t =total depth of concrete slab
h_g =depth of centre of steel section from top of steel flange

Note: Cylinder strength of concrete $(f_{ck})_{cy}$ is usually taken as *0.8* times the cube strength $(f_{ck})_{cu}$.

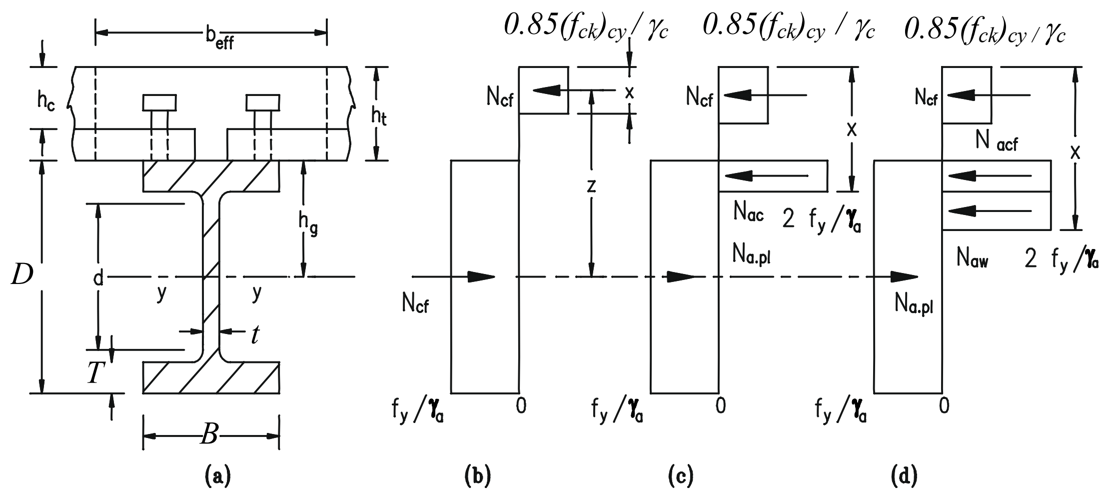


Fig.13. Resistance to sagging bending moment in plastic or compact sections for full interaction.

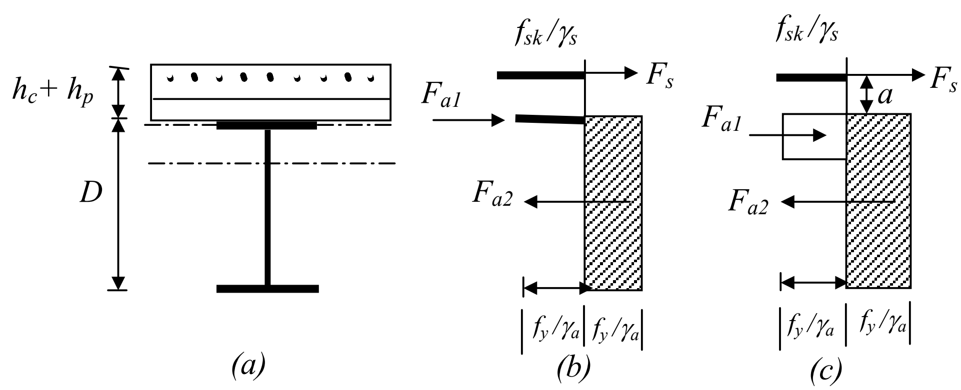


Fig. 14 Resistance to hogging Bending Moment

Table 6 Positive moment capacity of section with full shear connection(According to EC4)

Position of Plastic Neutral Axis	**Condition**	**Moment Capacity M_p**
Plastic neutral axis in concrete slab (Fig.*13b*)	$0.85\dfrac{(f_{ck})_{cy}}{\gamma_c} b_{eff}\, h_c \geq \dfrac{A_a f_y}{\gamma_a}$	$M_p = \dfrac{A_a f_y}{\gamma_a}(h_g + h_t - x/2)$
Plastic neutral axis in steel flange (Fig. *13c)*	$b_{eff}\, h_c \dfrac{0.85(f_{ck})_{cy}}{\gamma_c} < \dfrac{A_a f_y}{\gamma_a}$	$M_p = N_{a.pl}(h_g + h_t - h_c/2) - N_{ac}(x - h_c + h_t)/2$
Plastic neutral axis in web (Fig.*13d)*	$b_{eff}\, h_c \dfrac{0.85(f_{ck})_{cy}}{\gamma_c} + B{*}T{*}f_y/\gamma_a < \dfrac{A_a f_y}{\gamma_a}$	$M_p = N_{a.pl}(h_g + h_t - h_c/2) - N_{acf}(h_t + T/2 - h_c/2) - N_{a.w}(x + h_t + T - h_c)/2$

Table 7 Negative moment capacity of section with full shear connection (according EC4)

Position of Plastic Neutral Axis	**Condition**	**Moment Capacity M_p**
Plastic neutral axis in steel flange (Fig.*14b*)	$\dfrac{A_{aw} f_y}{\gamma_a} < \dfrac{A_s f_{sk}}{\gamma_s} < \dfrac{A_a f_y}{\gamma_a}$	$M_p \approx \dfrac{A_a f_y}{\gamma_a}\dfrac{D}{2} + \dfrac{A_s f_{sk}}{\gamma_s} a$
Plastic neutral axis in web (Fig. *14c)*	$\dfrac{A_s f_{sk}}{\gamma_s} < \dfrac{A_{aw} f_y}{\gamma_a}$	$M_p = M_{ap} + \dfrac{A_s f_{sk}}{\gamma_s}\left(\dfrac{D}{2} + a\right) + \left(\dfrac{A_s f_{sk}}{\gamma_s}\right)^2 \Big/ 4{*}t{*}f_y/\gamma_a$

4.2 Vertical Shear

In a composite beam, the concrete slab resists some of the vertical shear. But there is no simple design model for this, as the contribution from the slab is influenced by whether it is continuous across the end support, by how much it is cracked, and by the local details of the shear connection. It is therefore assumed that the vertical shear is resisted by steel beam alone, exactly as if it were not composite.

The shear force resisted by the structural steel section should satisfy:

$$V \le V_p \qquad (1)$$

where, V_p is the plastic shear resistance given by,

$$V_p = 0.6\, D\, t \frac{f_y}{\gamma_a} \qquad \text{(for rolled I, H, C sections)} \qquad (2)$$

$$= d\, t \frac{f_y}{\gamma_a \sqrt{3}} \qquad \text{(for built up I sections)} \qquad (3)$$

In addition to this the shear buckling of steel web should be checked.

The shear buckling of steel web can be neglected if following condition is satisfied

$$\frac{d}{t} \le 67 \in \qquad \text{for web not encased in concrete} \qquad (4)$$

$$\frac{d}{t} \le 120 \in \qquad \text{for web encased in concrete} \qquad (5)$$

where, $\in = \sqrt{\frac{250}{f_y}}$

d is the depth of the web considered in the shear area.

4.3 Resistance of shear connectors

The design shear resistance of shear connectors for slab without profiled steel decking according to *EC4* and *IS:11384-1985* was already explained in the previous chapter.

4.3.1 Effect of shape of deck slab on shear connection.

The profile of the deck slab has a marked influence on strength of shear connector. There should be a *45°* projection from the base of the connector to the core of the solid slab for smooth transfer of shear. But the profiled deck slab limits the concrete around the connector. This in turn makes the centre of resistance on connector to move up, initiating a local concrete failure as cracking. This is shown in Fig *15*. *EC 4* suggests the following reduction factor *k* (relative to solid slab).

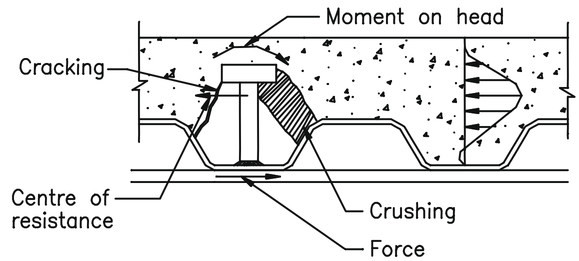


Fig. 15. Behaviour of a shear connection fixed through profile sheeting

(1) Profiled steel decking with the ribs parallel to the supporting beam.

$$k_p = 0.6 \frac{b_0}{h_p}\left(\frac{h-h_p}{h_p}\right) \leq 1.0 \;\; where \;\; h \leq h_p + 75 \tag{6}$$

(2) Profiled steel decking with the ribs transverse to the supporting beam.

For studs of diameter not exceeding *20 mm,*

$$k_t = \frac{0.7}{\sqrt{N_r}} \frac{b_0}{h_p}\left(\frac{h-h_p}{h_p}\right) \leq 1.0 \;\; where \;\; h_p \leq 85 \; and \; b_0 \geq h_p \tag{7}$$

where,

- b_0 is the average width of trough
- h is the stud height
- h_p is the height of the profiled decking slab
- N_r is the number of stud connectors in one rib at a beam intersection (should not greater than *2*).

For studs welded through the steel decking, k_t should not be greater than *1.0* when $N_r = 1$, and not greater than *0.8* when $N_r \geq 2$

4.4 Longitudinal Shear Force

4.4.1 Full Shear Connection

(1) Single span beams

For single span beams the total design longitudinal shear, V_λ to be resisted by shear connectors between the point of maximum bending moment and the end support is given by:

$$V_\lambda = F_{cf} = A_a f_y/\gamma_a \quad \text{or} \quad V_\lambda = 0.85\,(f_{ck})_{cy}\, b_{eff}\, h_c/\gamma_c \qquad (8)$$

whichever is smaller.

(2) Continuous Span Beams

For continuous span beams the total design longitudinal shear, V_λ to be resisted by shear connectors between the point of maximum positive bending moment and an intermediate support is given by:

$$V_\lambda = F_{cf} + A_s f_{sk}/\gamma_s \qquad (9)$$

where, A_s - the effective area of longitudinal slab reinforcement

The number of required shear connectors in the zone under consideration for full composite action is given by:

$$n_f = V_\lambda/P$$

where

V_λ is the design longitudinal shear force as defined in equation (8)
P design resistance of the connector.

The shear connectors are usually equally spaced.

4.4.2 Minimum degree of shear connection

Ideal plastic behaviour of the shear connectors may be assumed if a minimum degree of shear connection is provided, as the opportunity for developing local plasticity are greater in these cases

The minimum degree of shear connection is defined by the following equations:

(1) $n/n_f \geq 0.4 + 0.03\lambda \quad where\ 3A_t \geq A_b$
(2) $n/n_f \geq 0.25 + 0.03\lambda \quad where\ A_t = A_b$
(3) $n/n_f \geq 0.04\lambda \quad where\ A_t = A_b$

where

A_t is the top flange area and
A_b is the bottom flange area.
λ beam span in metres

4.5 Interaction between shear and moment

Interaction between bending and shear can influence the design of continuous beam. Fig. *16* shows the resistance of the composite section in combined bending (hogging or sagging) and shear. When the design shear force, V exceeds $0.5V_p$(point *A* in the Fig.), moment capacity of the section reduces non-linearly as shown by the parabolic curve *AB*, in the presence of high shear force. At point *B* the remaining bending resistance M_f is that contributed by the flanges of the composite section, including reinforcement in the slab. Along curve *AB*, the reduced bending resistance is given by

$$M \leq M_f + \left(M_p - M_f\right)\left[1 - \left(2\frac{V}{V_p} - 1\right)^2\right] \qquad (10)$$

where

M design bending moment
M_f plastic resistance of the flange alone
M_p plastic resistance of the entire section
V design shear force
V_p plastic shear resistance as defined in equation *(2)* and equation *(3)*.

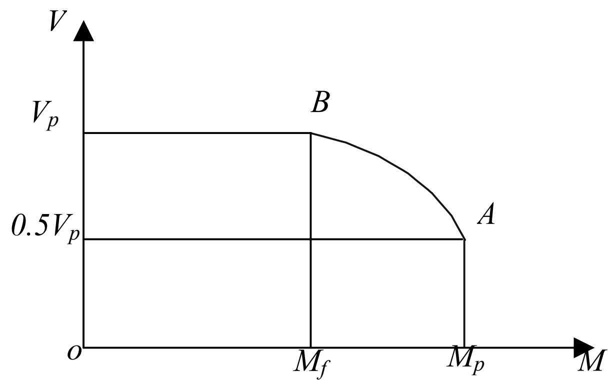


Fig16 Resistance to combined bending and vertical shear

4.6 Transverse reinforcement

Shear connectors transfer the interfacial shear to concrete slab by thrust. This may cause splitting in concrete in potential failure planes as shown in Fig *17*. Therefore reinforcement is provided in the direction transverse to the axis of the beam. Like stirrups in the web of a reinforced T beam, the reinforcement supplements the shear strength of the concrete. A truss model analysis [See Fig. *18*] shows, how the design shear force per unit length V_λ is transferred through concrete struts *AC* and *AB*, causing tension in reinforcement *BC*. Here v_r is the shear resistance of a failure plane as *B-B*. The model gives a design equation of the form

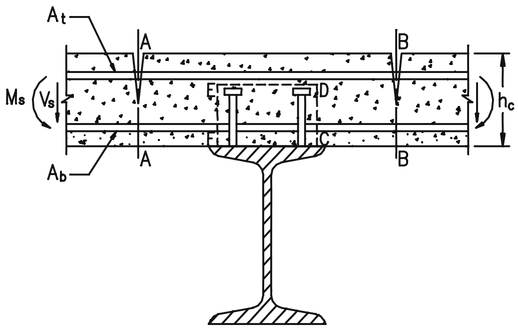


Fig.17. Surfaces of potential shear failure

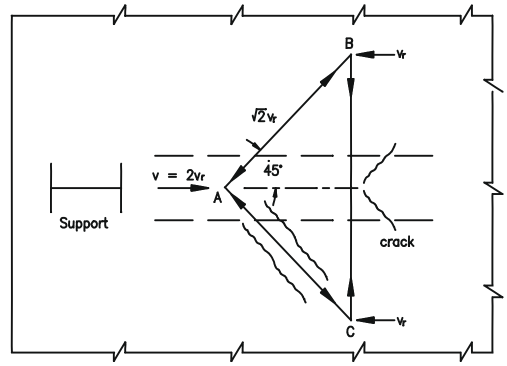


Fig.18. Truss model analysis

$$\frac{V_\lambda}{2} = v_r = A_{cv}\, f\left(\frac{(f_{ck})_{cy}}{\gamma_c}\right) + \frac{A_{sv} f_{sk}}{\gamma_s} \qquad (11)$$

where,

A_{cv} = cross sectional area of concrete shear surface per unit length of beam

A_{sv} = Area of transverse reinforcement.

The formulae suggested by *EC4* and *IS:11384 – 1985* are given in Table 8.

5.0 EFFECT OF CONTINUITY

The above design formulae are applicable to simply supported beams as well as to continuous beams. Besides these, a continuous beam necessitates the check for the stability of the bottom flange, which is in compression due to hogging moments at supports.

5.1 Moment and Shear Coefficients for continuous beam

In order to determine the distribution of bending moments under the design loads, Structural analysis has to be performed. For convenience, the *IS: 456-1978* lists moment coefficients as well as shear coefficients that are close to exact values of the maximum load effects obtainable from rigorous analysis on an infinite number of equal spans on point supports. Table *9* gives the bending moment coefficients and Table *10* gives the shear coefficients according to *IS: 456-1978*. These coefficients are applicable to continuous beams with at least three spans, which do not differ by more than *15* percent of the longest. These values are also applicable for composite continuous beams.

Table8 Comparison of EC4 and IS:11384 – 1985 provisions for transverse reinforcement

EC4	***IS 11384 – 1985***
$v_r = 2.5 A_{cv}\eta\tau + A_e f_{sk}/\gamma_s + v_{pd}$ *or* $v_r = 0.2 A_{cv}\eta(f_{ck})_{cy}/\gamma_c + V_{pd}/\sqrt{3}$	$v_r = N_c.F_c/s < 0.232 L_s \sqrt{(f_{ck})_{cu}}$ + $0.1 A_{sv} f_y n < 0.623 L_s \sqrt{(f_{ck})_{cu}}$ where, N_c is the number of a shear connector at a section
A_e is the sum of the cross sectional areas of transverse reinforcement (assumed to be perpendicular to the beam) per unit length of beam crossing the shear surface under consideration including any reinforcement provided for bending of the slab.	F_c – Load in *kN* on one connector at ultimate load *s* – Spacing of connectors in m L_s - Length of shear surface (mm as shown in Fig.(5d) of previous chapter but $2d_s$ for T - beam d_s for *L* – beam
A_{cv} mean cross sectional area per unit length of the beam of the concrete shear surface under consideration.	A_{sv} = Area of transverse reinforcement in cm per metre of beam.
η = *1* for normal weight concrete η = $0.3+0.7(\rho/24)$ for light weight concrete	$n = 2$ for T beam $n = 1$ for L – beams
τ basic shear strength to be taken as 0.25 f_{ctk}/γ_c, where f_{ctk} is the characteristic tensile strength of concrete.	[*n* is the number of times each lower transverse reinforcement intersects shear surface.]
V_{pd} contribution of profiled steel sheeting, if any $= A_p f_{yp}/\gamma_{ap}$ (for ribs running perpendicular to the beam) $= P_{pb}/s$ but $\leq A_p f_{yp}/\gamma_{ap}$ (for ribs running parallel to the beam)	
P_{pd} design resistance of the headed stud against headed stud against tearing through the steel sheet.	
A_p cross-sectional area of the profile steel sheeting per unit length of the beam	
f_{yp} yield strength of steel sheeting.	

s is the spacing centre to centre of the studs along the beam	

Table9 Bending moment coefficients according to IS: 456-1978

TYPE OF LOAD	SPAN MOMENTS		SUPPORT MOMENTS	
	Near middle span	At middle of interior span	At support next to the end support	At other interior supports
Dead load + Imposed load (fixed)	+ 1/12	+1/24	- 1/10	- 1/12
Imposed load (not fixed)	+1/10	+1/12	- 1/9	- 1/9
For obtaining the bending moment, the coefficient shall be multiplied by the total design load and effective span.				

Table 10 Shear force coefficients

TYPE OF LOAD	At end support	At support next to the end support		At all other interior supports
		Outer side	Inner side	
Dead load + Imposed load(fixed)	0.40	0.60	0.55	0.50
Imposed load(not fixed)	0.45	0.60	0.60	0.60
For obtaining the shear force, the coefficient shall be multiplied by the total design load				

5.2 Lateral Torsional Buckling of Continuous Beams

The concrete slab is usually assumed to prevent the upper flange of the steel section from moving laterally. In negative moment regions of continuous composite beams the lower flange is subjected to compression. Hence, the stability of bottom flange should be checked at that region. The tendency of the lower flange to buckle laterally is restrained by the distortional stiffness of the cross section. The tendency for the bottom flange to displace laterally causes bending of the steel web, and twisting at top flange level, which is resisted by bending of the slab as shown in Fig. *19*.

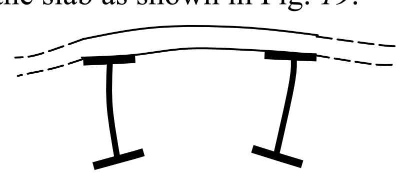


Fig 19 Inverted – U frame Action

Local-Torsional Buckling of Continuous Beams can be neglected if following conditions are satisfied.

1. adjacent spans do not differ in length by more than *20%* of the shorter span or where there is a cantilever, its length does not exceed *15%* of the adjacent span.
2. the loading on each span is uniformly distributed and the design permanent load exceeds *40%* of the total load.

3. the shear connection in the steel-concrete interface satisfies the requirements of section *4.4*

4. $h_a \leq 550\ mm$

6.0 SERVICEABILITY

Composite beams must also be checked for adequacy in the Serviceability Limit State. It is not desirable that steel yields under service load. To check the composite beams serviceability criteria, elastic section properties are used.

IS:11384-1985 limits the maximum deflection of the composite beam to $\lambda/325$. The total elastic stress in concrete is limited to $(f_{ck})_{cu} / 3$ while for steel, considering different stages of construction, the elastic stress is limited to $0.87\, f_y$. Unfortunately this is an error made in the Code as the same limits are applied for steel in determining the ultimate resistance of the cross section. Since *EC4* gives explicit guidance for checking serviceability Limit State, therefore the method described below follows *EC 4.*

6.1 Deflection

The elastic properties relevant to deflection are section modulus and moment of inertia of the section. Applying appropriate modular ratio *m* the composite section is transformed into an equivalent steel section. The moment of inertia of uncracked section is used for calculating deflection. Normally unfactored loads are used for for serviceability checks. No stress limitations are made in *EC 4.*

Under positive moment the concrete is assumed uncracked, and the moment of inertia is calculated as:

$$I = \frac{A_a\left(h_c + 2h_p + h_a\right)^2}{4\left(1 + mr\right)} + \frac{b_{eff}\, h_c^{\ 3}}{12m} + I_a \qquad (12)$$

where

m is the ratio of the elastic moduli of steel to concrete taking into account creep.

$r = \dfrac{A_a}{b_{eff}\, h_c}$

I_a is the moment of inertia of steel section.

6.1.1 Simply supported Beams

The mid-span deflection of simply supported composite beam under distributed load *w* is given by

$$\delta_c = \frac{5w\lambda^4}{384\,E_a I} \tag{13}$$

where, E_a is the modulus of elasticity of steel.
I is the gross uncracked moment of inertia of composite section.

6.1.2 Influence of partial shear connection

Deflections increase due to the effects of slip in the shear connectors. These effects are ignored in composite beams designed for full shear connection. To take care of the increase in deflection due to partial shear connection, the following expression is used.

$$\frac{\delta}{\delta_c} = 1 + 0.5\left(1 - \frac{n_p}{n_f}\right)\left(\frac{\delta_a}{\delta_c} - 1\right) \quad \text{for propped construction} \tag{14}$$

$$\frac{\delta}{\delta_c} = 1 + 0.3\left(1 - \frac{n_p}{n_f}\right)\left(\frac{\delta_a}{\delta_c} - 1\right) \quad \text{for unpropped construction} \tag{15}$$

where
δ_a and δ_c are deflection of steel beam and composite beam respectively with proper serviceability load.

Note: For $\frac{n_p}{n_f} \geq 0.5$, this additional simplification can usually be ignored

6.1.3 Shrinkage induced deflections

For simply supported beams, when the span to depth ratio of beam exceeds *20*, or when the free shrinkage strain of the concrete exceeds 400×10^{-6} shrinkage, deflections should be checked. In practice, these deflections will only be significant for spans greater than *12 m* in exceptionally warm dry atmospheres. The shrinkage induced deflection is calculated using the following formula:

$$\delta_s = 0.125 K_s \lambda^2 \tag{16}$$

where

λ is the effective span of the beam.

K_s is the curvature due to the free shrinkage strain, $\in_s$ given by

$$K_s = \frac{\in_s \left(h + h_c + 2h_p\right)A_a}{2\left(1 + mr\right)I_c} \qquad (17)$$

m modular ratio appropriate for shrinkage calculations (*m=20*)
Note: This formula ignores continuity effects at the supports.

6.1.4 Continuous Beams

In the case of continuous beam, the deflection is modified by the influence of cracking in the hogging moment regions (at or near the supports). This may be taken into account by calculating the second moment of area of the cracked section under negative moment (ignoring concrete). In addition to this there is a possibility of yielding in the negative moment region. To take account of this the negative moments may be further reduced. As an approximation, a deflection coefficient of *3/384* is usually appropriate for determining the deflection of a continuous composite beam subject to uniform loading on equal adjacent spans. This may be increased to *4/384* for end spans. The second moment of area of the section is based on the uncracked value.

6.1.5 Crack Control

Cracking of concrete should be controlled in cases where the functioning of the structure or its appearance would be affected. In order to avoid the presence of large cracks in the hogging moment regions, the amount of reinforcement should not exceed a minimum value given by,

$$p = \frac{A_s}{A_c} = k_c * k * \frac{f_{ct}}{\sigma_s} \qquad (18)$$

where

p is the percentage of steel
k_c is a coefficient due to the bending stress distribution in the section($k_c \approx 0.9$)
k is a coefficient accounting for the decrease in the tensile strength of concrete ($k \approx 8$)
f_{ct} is the effective tensile strength of concrete. A value of $3\ N/mm^2$ is the minimum adopted.
σ_s is the maximum permissible stress in concrete.

7.0 CONCLUSION

This chapter summarises the method of design of composite beams, connected to solid slab, as well as profiled deck slab. Two design examples follow this chapter, where designs of simply supported and continuous composite beams have been presented in detail. The design of simply supported beam follows *IS:11384-1985* whereas, the design of continuous beam follows *EC4*.

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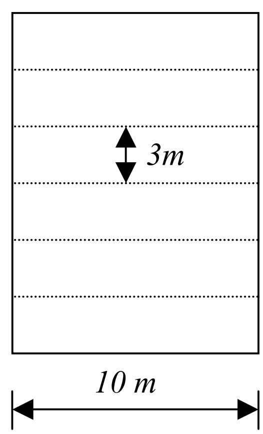
PROBLEM 1

Design a simply supported composite beam with 10m span shown (dotted line) in the figure below. The thickness of slab is 125 mm. The floor is to carry an imposed load of 3.0 kN/m^2, partition load of 1.5 kN/m^2 and a floor finish load of 0.5 kN/m^2



Given Data

Imposed load	*3.0 kN/m^2*
Partition load	*1.5 kN/m^2*
Floor finish load	*0.5 kN/m^2*
Construction load	*0.75 kN/m^2*

Data assumed

$(f_{ck})_{cu}$	*30 N/mm^2*
f_y	*250 N/mm^2*
Density of concrete	*24 kN/m^3*

Partial safety factors

Load Factor, γ_f

for LL	*1.5*
for DL	*1.35*

Material Factor, γ_m

Steel	*1.15*
Concrete	*1.5*
Reinforcement	*1.15*

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Step 1: Load Calculation

Construction stage

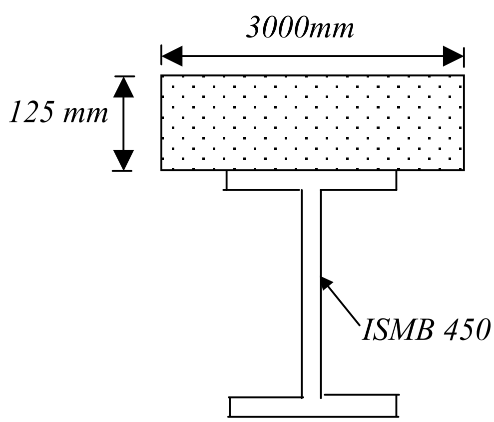
i) Self weight of slab = 3 * 0.125 * 24 = 9 kN/m

ii) Self weight of beam = 0.71 kN/m (assuming ISMB 450)

iii) Construction load = 0.75 * 3 = 2.25 kN/m

Total design load at Construction Stage

= {1.5 * 2.25 + 1.35 * (9 + 0.71) =16.5 kN/m



Composite stage

Dead Load

i) Self weight of slab = 9 kN/m

ii) Self weight of beam = 0.71 kN/m

iii) Load from floor finish = 0.5 * 3 =1.5 kN/m

Total Dead Load = 11.2 kN/m

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Live Load

*i) Imposed load = 3 * 3 = 9.0 kN/m*
*ii) Load from partition wall = 1.5 * 3 = 4.5 kN/m*

Total Live Load = 13.5 kN/m

*Design load carried by composite beam= (1.35 * 11.2 + 1.5 * 13.5) = 35.4 kN/m*

Step 2: Calculation of Bending Moment

Construction Stage

$M = 16.5 * 10^2/8 = 206$ kNm

Composite Stage

$M = 35.4 * 10^2/8 = 442$ kNm

Step 3: Classification of Composite Section

Sectional Properties

$T = 17.4$mm;
$D = 450$ mm;
$t = 9.4$ mm
$I_x = 303.9 * 10^6$ mm^4
$I_y = 8.34 * 10^6$ mm^4
$Z_x = 1350*10^3$ mm;
$r_y = 30.1$ mm

Classification of composite section — *Refer Table 4*

$0.5\ B/T = 0.5*150/17.4 = 4.3 < 8.9\varepsilon$
$d/t = (450-2*17.4)/9.4 = 44.2 < 83\varepsilon$

Therefore the section is a plastic section.

Step 4: Check for the adequacy of the section at construction stage

Design moment in construction stage = 206 kNm

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Moment of resistance of steel section

$=f_{yd} * Z_p$

$=[(250/1.15) * 1.14 * 1350.7 * 10^3]/10^6$ kNm

$=334.7$ kNm > 206 kNm

<u>As the top flange of the steel beam is unrestrained and under compression, stability of the top flange should be checked.</u>

Step 5: Check for Lateral Buckling of the top flange

From clause 6.2.4, IS:800-1984

Elastic critical stress, f_{cb} is given by

$$f_{cb} = k_1 \frac{c_2}{c_1} \frac{26.5*10^5}{\left(\frac{\lambda}{r_y}\right)^2} \left[\sqrt{1 + \frac{1}{20}\left(\frac{\lambda}{r_y}\frac{T}{D}\right)^2} + k_2 \right]$$

$k_1 = 1$ (as $\Psi = 1.0$)
$k_2 = 0$ (as $\phi = 0.5$)
$c_2 = c_1 = 225$ mm;
$T = 17.4$ mm;
$D = 450$ mm;
$\lambda = 10{,}000$ mm;
$r_y = 30.1$ mm

$$f_{cb} = \frac{26.5*10^5}{\left(\frac{10000}{30.1}\right)^2} \left[\sqrt{1 + \frac{1}{20}\left(\frac{10000*17.4}{30.1*450}\right)^2} \right] = 73\,N/mm^2$$

Therefore the bending compressive stress in beams

$$F_{cb} = \frac{f_{cb} * f_y}{\left[(f_{cb})^{1.4} + (f_y)^{1.4}\right]^{\frac{1}{1.4}}} = 64.9\,N/mm^2$$

Moment at construction stage = 206 kNm

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Maximum stress at top flange of steel section

$$F_{cb} = \frac{206*10^6*225}{303.9 * 10^6} = 152.5\ N/mm^2 > 64.9\ N/mm^2$$

So, we have to reduce the effective length of the beam.

Provide 2 lateral restraints with a distance of approximately 3330 mm between them

From clause 6.2.4, IS:800-1984

$$f_{cb} = \frac{26.5*10^5}{\left(\frac{3330}{30.1}\right)^2}\left[\sqrt{1+\frac{1}{20}\left(\frac{3330*17.4}{30.1*450}\right)^2}\right] = 299.6\ N/mm^2$$

Therefore the bending compressive stress in beams

$$F_{cb} = \frac{299.6*250}{\left[(299.6)^{1.4}+(250)^{1.4}\right]^{\frac{1}{1.4}}} = 165.9\ N/mm^2$$

$F_{cb}=165.9 > 152.5\ N/mm^2$

Note: These restraints are to be kept till concrete hardens.

Step6: Check for adequacy of the section at Composite stage

Bending Moment at the composite Stage, M = 442 kNm

Effective breadth of slab is smaller of

I. span /4 = 10000/4 = 2500 mm

II. C/C distance between beams = 3000 mm

Hence, $b_{eff} = 2500$ *mm*

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Position of neutral axis

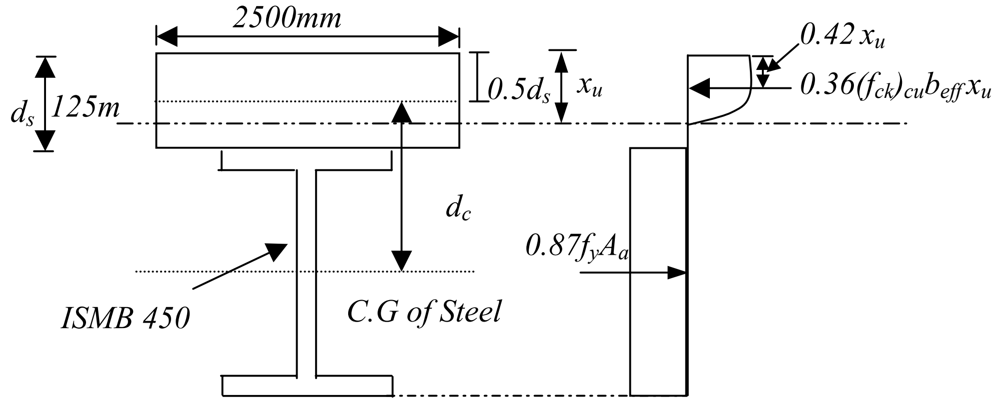
$$a = \frac{0.87 f_y}{0.36\left(f_{ck}\right)_{cu}} = \frac{0.87*250}{0.36*30} = 20.1$$

$A_a = 9227\ mm^2$

$a\ A_a = 20.1*\ 9227 = 1.85 * 10^5\ mm^2$

$b_{eff}\, d_s = 2500 * 125 = 3.13 * 10^5\ mm^2 > aA_a$

Hence PNA lies in concrete



Position of neutral axis

$$x_u = \frac{0.87*9227*250}{0.36*30*2500} = 74.3\ mm \quad \text{from the top of the slab}$$

Moment Resistance of the section, M_p

$M_p = 0.87 A_a f_y (d_c + 0.5 d_s - 0.42 x_u)$

$= 0.87*9227*250(287.5+0.5*125-0.42*74.3)$

$= 640\ kNm > 442\ kNmm$

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Step7 : Design of shear connectors

The position of neutral axis is within slab.

$\therefore$ *Total load carried by connectors*

$$F_{cc} = 0.36(f_{ck})_{cu}\, b_{eff}\, x_u = (0.36 * 30 * 2500 * 74.3)/1000\ kN$$
$$= 2006\ kN$$

As per Table 1(Composite Beam-II), the design strength of 20 mm (dia) headed stud for M30 concrete is 58 kN

$\therefore$ *Number of shear connectors required for 10/2 m = 5 m length*

$$= 2006 / 58 \approx 34$$

These are spaced uniformly

Spacing = 5000/34 = 147 mm $\approx$ *145 mm*

*If two connectors are provided in a row the spacing will be = 145 * 2 = 290 mm*

Step8: Serviceability check

Modular ratio for live load = 15

Modular ratio for deal load = 30

(1) Deflection

For dead load deflection is calculated using moment of inertia of steel beam only

$$\delta_d = \frac{5*9.71*(10000)^4}{384*2*10^5*303.91*10^6} = 20.8\ mm$$

For live load deflection is calculated using moment of inertia of composite section

To find the moment of inertia of the composite section we have to first locate the position of neutral axis.

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Position of neutral axis

$A\,(d_g - d_s) < ½(b_{eff}/\alpha_e)\, d_s^2$
$9227\,(350 - 125) < ½ * 2500/15 * 125^2$

$2.08 * 10^6 < 1.3 * 10^6$ *which is not true*

$\therefore$ *N.A. depth exceeds* d_s

$$A_a\left(d_g - x_u\right) = \frac{b_{eff}}{m} d_s\left(x_u - \frac{d_s}{2}\right)$$

$$9227\left(\frac{450}{2} + 125 - x_u\right) = \frac{2500}{15} * 125 * \left(x_u - \frac{125}{2}\right)$$

$$x_u = 150.75\,mm$$

Moment of inertia of the gross section, I_g

$$I_g = I_x + A_a\left(d_g - x_u\right)^2 + \frac{b_{eff}}{\alpha_e} d_s\left[\frac{d_s^{\,2}}{12} + \left(x_u - d_s\right)^2\right]$$

$$= 303.91*10^6 + 9227\left(350 - 150.75\right)^2 + \frac{2500*125}{15}\left[\frac{125^2}{12} + \left(150.75 - \frac{125}{2}\right)^2\right]$$

$$= 859.6*10^6\ mm^4$$

$$\delta_l = \frac{5*15*\left(10000\right)^4}{384*2*10^5*859.6*10^6} = 11.4\ mm$$

$$\therefore Total\ Deflection = \delta_d + \delta_l = 20.8 + 11.4\ mm$$

$$= 32.2\ mm > \frac{\lambda}{325}$$

The section fails to satisfy the deflection check.

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(2) Stresses

Composite Stage

Dead Load

In composite stage, dead load W_d

$W_d = 11.2$ kN/m

$M = 11.2 * 10^2/8 = 140$ kNm

Position of neutral axis

Assuming neutral axis lies within the slab

$A (d_g - d_s) < ½\, b_{eff}.\, d_s^2/\alpha_e$

$9227 (350 - 125) < ½ * 2500/30 * 125^2$

$2.07 * 10^6 > 6.5 * 10^5$

$\therefore$ *N.A. depth exceeds* d_s

Modular ratio for dead load, $\alpha_e = 30$

Location of neutral axis

$$A_a\left(d_g - x_u\right) = \frac{b_{eff}}{m} d_s\left(x_u - \frac{d_s}{2}\right)$$

$$9227\left(\frac{450}{2} + 125 - x_u\right) = \frac{2500}{30} * 125 * \left(x_u - \frac{125}{2}\right)$$

$x_u = 197.5$mm

Moment of Area of the section

$$I_g = I_x + A_a\left(d_g - x_u\right)^2 + \frac{b_{eff} d_s}{m}\left[\frac{d_s^2}{12} + \left(x_u - d_s\right)^2\right]$$

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$$I_g = 303.91*10^6 + 9227(350 - 197.5)^2 + \frac{2500*125}{30}\left[\frac{125^2}{12} + \left(197.5 - \frac{125}{2}\right)^2\right]$$

$$= 721.9*10^6 \ mm^4$$

$$\text{Stress in steel flange} = \frac{140*10^6(450 + 125 - 197.5)}{721.9*10^6} = 73.2\ N/mm^2$$

Live load

In composite stage stress in steel for live load

W_l =13.5 kN/m

$M = 13.5 * 10^2/8 = 168.75\ kNm$

$$\text{Stress in steel flange} = \frac{168.75*10^6(450 + 125 - 150.75)}{859.6*10^6} = 83.29\ N/mm^2$$

$\therefore$ *Total stress in steel* $= 73.2 + 83.29 = 156.5\ N/mm^2 <$ *allowable stress in steel*

In a similar procedure the stress in concrete is found.

$$\frac{1}{30}\left\{\frac{140*10^6*197.54}{721.9*10^6}\right\} + \frac{1}{15}\left\{\frac{168.75*10^6*150.75}{859.6*10^6}\right\} = 3.25 < \frac{(f_{ck})_{cu}}{3} = 10\ N/mm^2$$

The section is safe.

Since the section does not satisfy the deflection check, therefore trial can be made with higher steel section

Step 9: Transverse reinforcement

Shear force transferred per metre length

$$v_r = \frac{2*58}{0.29} kN/m \quad (n = 2, \text{Since there are two shear studs})$$

$$= 400\ kN/m$$

Refer Table 6

$$v_r \le 0.232 L_s \sqrt{(f_{ck})_{cu}} + 0.1 A_{sv} f_y n$$

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or

$0.632 L_s \sqrt{(f_{ck})_{cu}}$

$L_s = 2*125 = 250\,mm$

$f_y = 250\,mm$

$n = 2$

$\therefore 0.232 L_s \sqrt{(f_{ck})_{cu}} + 0.1 A_{sv} f_y n = 0.232*250\sqrt{30} + 0.1*A_{sv}*250*2$

$= 317.7 + 50 A_{sv}$

or

$0.632*250\sqrt{30} = 865\,kN/m$

$\therefore 400 = 317.7 + 50 A_{sv}$

$= 165\ mm^2/m$

Minimum reinforcement

$= 250 v_r / f_y\ mm^2/m$

$= 400\ mm^2/m$

Provide $12\,mm\ \varphi\ @280\,mm\ c/c.$

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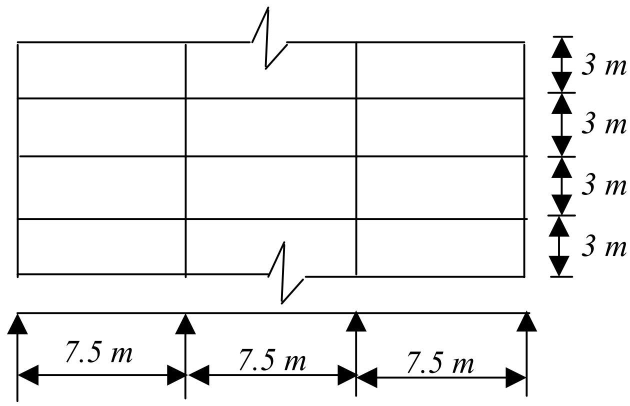
PROBLEM 2

A composite floor slab is supported on three span continuous composite beams spaced at 3 m centres. The effective length of each span being 7.5 m. The thickness of composite slab is 130 mm. The floor has to carry an imposed load of 3.5 kN/m², partition load of 1.0 kN/mm² and a floor finish load of 0.5 kN/m². Design the continuous beam.



Step 1: List of Datas

Given:

Imposed Load $=3.5\ kN/m^2$
Partition Load $=1.0\ kN/m^2$
Floor finish Load $=0.5\ kN/m^2$
Construction Load $=0.5\ kN/m^2$

Assumed:

$(f_{ck})_{cu} = 30\ N/mm^2$; $f_y = 250\ N/mm^2$; $f_{sk} = 415\ N/mm^2$

Density of concrete $=24\ kN/m^2$

Partial Safety factors:

Load Factor γ_f
for LL 1.5; for DL 1.35

Material Factor, γ_m

Steel, $\gamma_a = 1.15$; *Concrete,* $\gamma_c = 1.5$; *Reinforcement,* $\gamma_s = 1.15$

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Step2: Load Calculation
<u>Construction stage</u>

<u>Dead Load</u>

*Self weight of slab=3*0.13*24=9.36 kN/m*

Self weight of beam =0.44 kN/m (assuming ISMB 300)

Total dead load = 9.8 kN/m

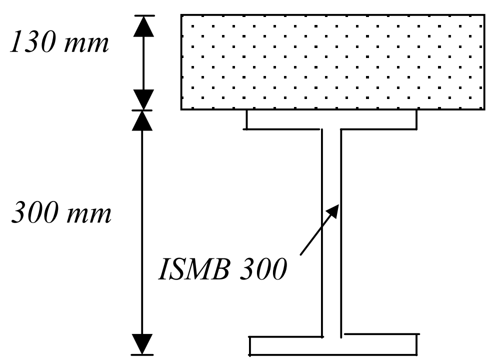
Total design dead load =1.35(9.8)=13.2 kN/m*

<u>Live Load</u>

Construction Load =0.5 3=1.5 kN/m*

*Total design live load =1.5*1.5=2.25 kN/m*

<u>Composite Stage</u>



<u>Dead load</u>

*Self weight of slab=3*0.13* 24=9.36 kN/m*

Self weight of beam=0.44 kN/m

Load from floor finish =0.5 3=1.5 kN/m*

Total dead load = 11.3 kN/m

*Total design dead load =1.35*11.3=15.3 kN/m*

<u>Live Load</u>

*Imposed Load =3.5*3=10.5 kN/m*

*Partition Load =1.0*3=3.0 kN/m*

Total design live load =1.5(13.5)=20.3 kN/m*

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Step3: Bending Moment and Shear Force Calculation

Construction Stage

$$\text{Maximum Positive Moment} = \frac{w_d\lambda^2}{12} + \frac{w_l\lambda^2}{10}$$

$$= \frac{13.2*7.5^2}{12} + \frac{2.25*7.5^2}{10} = 74.5\,kNm$$

$$\text{Maximum Negative Moment} = -\left(\frac{w_d\lambda^2}{10} + \frac{w_l\lambda^2}{9}\right)$$

$$= -\left(\frac{13.2*7.5^2}{10} + \frac{2.25*7.5^2}{9}\right) = -88.3\,kNm$$

$$\text{Maximum Shear force} = 0.6(w_d\lambda + w_l\lambda)$$

$$= 0.6*7.5*(13.2 + 2.25) = 69.5\ kN$$

Composite Stage

$$\text{Maximum Positive Moment} = \left(\frac{w_d\lambda^2}{12} + \frac{w_l\lambda^2}{10}\right)$$

$$= \frac{15.3*7.5^2}{12} + \frac{20.3*7.5^2}{10} = 185.9\,kNm$$

$$\text{Maximum Negative Moment} = -\left(\frac{w_d\lambda^2}{10} + \frac{w_l\lambda^2}{9}\right)$$

$$= -\left(\frac{15.3*7.5^2}{10} + \frac{20.3*7.5^2}{9}\right) = -212.9\ kNm$$

$$\text{Maximum Shear force} = 0.6(w_d\lambda + w_l\lambda)$$

$$= 0.6*7.5*(15.3 + 20.3) = 160.2kN$$

Step4: Selection of steel section

Assuming span/depth =22

Depth of Composite Section=7500 /22=341

Let us take ISMB 300 @ 0.44 kN/m

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Section Properties:

$T = 12.4$ mm; $B=140$ mm
$D = 300$ mm; $t = 7.5$ mm
$I_x = 86*10^6\ mm^4$; $I_y = 4.53*10^6\ mm^4$
$r_x = 123.7$ mm ; $r_y = 28.4$ mm
$Z_x=573.6 * 10^3\ mm^3$; $Z_y=64.8 * 10^3\ mm^3$

Classification of composite section

$0.5\ B/T=0.5*140/12.4 = 5.65< 8.9\in$

$d/t=(300-2*12.4) / 7.5 = 36.7< 83\in$ — *Refer Table 4*

Here, $\in=\sqrt{\dfrac{250}{f_y}}$

Therefore the section is a plastic section.

Step 5:Ultimate Limit State

[A] Construction Stage

(1) Plastic Moment Resistance of the Steel Section

$$M_{ap} = \frac{f_y}{\gamma_a} Z_{px}$$
$$= \left(\frac{250}{1.15}*1.14*573.6*10^3\right) \Big/ 10^{-6} = 142.2\ kNm > 88.3\ kNm$$

$Z_{px}=1.14 * Z_x$

(2) Plastic Shear Resistance

$$V_p = 0.6*D*t*\frac{f_y}{\gamma_a}$$

Refer Section 4.2

$$=\left[0.6*300*7.5*\frac{250}{1.15}\right]/1000 = 293.5\ kN > 69.5\,kN$$

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Bending Moment and Vertical Shear Interaction

Bending Moment and Vertical Shear Interaction can be neglected if

Refer Section 4.5

$V < 0.5\ V_p$

$69.5 < 0.5 * 293.5$

< 146.7 *kN*

Therefore, vertical shear has no effect on the plastic moment resistance.

(3) Check for Lateral torsional buckling of the steel Beam

The design buckling resistance moment of a laterally unrestrained beam is given by

$$M_b = \chi_{LT} \beta_w Z_{px} \frac{f_y}{\gamma_m}$$

where

χ_{LT} *is the reduction factor for lateral torsional buckling.*

$$= \frac{1}{\left(\phi_{LT} + \sqrt{\phi_{LT}^2 - \bar{\lambda}_{LT}^2}\right)} \le 1.0$$

where

$$\phi_{LT} = 0.5\left(1 + \alpha_{LT}\left(\bar{\lambda}_{LT} - 0.2\right) + \bar{\lambda}_{LT}^2\right)$$

Here

α_{LT} = *imperfection factor (= 0.21for rolled section)*

$$\bar{\lambda}_{LT} = \sqrt{\frac{\beta_w * Z_{px} * f_y}{M_{cr}}}$$ *(non dimensional slenderness ratio)*

where

β_w *is a constant which is equal to 1.0 for plastic section*

M_{cr} *is the elastic critical moment for lateral torsional buckling given by*

$$M_{cr} = \frac{\pi^2 EI_y}{\lambda_e^2}\left[\frac{I_w}{I_y} + \frac{\lambda_e^2 * G * I_t}{\pi^2 EI_y}\right]^{0.5}$$

where

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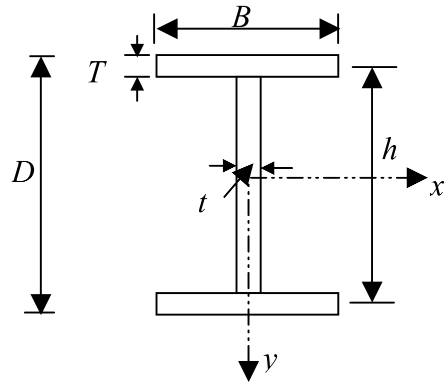
$$G = \frac{E}{2(1+m)} = \frac{2*10^5}{2(1+0.3)} = 76.9*10^3\ N/mm^2$$

$$I_t = \text{torsion constant} = \frac{1}{3}\left(2BT^3 + (D-2T)t^3\right)$$

$$= \frac{1}{3}\left(2*140*12.4^3 + (300-2*12.4)*7.5^3\right)$$

$$= 216.6*10^3\ mm^4$$



$$I_w = \text{warping constant} = \frac{I_y h^2}{4}$$

$$= \frac{4.53*10^6*(287.6)^2}{4}$$

$$= 93.9*10^9\ mm^6$$

Assuming two lateral supports @ 2500 mm

$$M_{cr} = \frac{\pi^2*2*10^5*4.53*10^6}{(2500)^2}\left[\frac{93.9*10^9}{4.53*10^6} + \frac{(2500)^2*76.9*10^3*216.6*10^3}{\pi^2*2*10^5*4.53*10^6}\right]^{0.5}$$

$$= 257.7\ kNm$$

$\bar{\lambda}_{LT} = 0.796;\ \phi_{LT} = 0.879;\ \chi_{LT} = 0.79$

M_b=[0.79* 1.0*1.14 * 573.6 *10^3 * 250/1.15]*10^{-6}

=112.3kNm > 88.3 kNm

[B] Composite Stage

(1) Moment Resistance of the cross section

<u>Negative Bending Moment</u>

At internal support negative bending moment of resistance is obtained by considering the tensile resistance of the reinforcement. Concrete area is neglected.

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a) effective width of the concrete flange $b_{eff} = \dfrac{\lambda_o}{4}$ $= \dfrac{1}{4}\left(0.25\left(\lambda_1 + \lambda_2\right)\right) = \dfrac{1}{4}\left(0.25\left(7.5 + 7.5\right)\right) * 1000$ $\approx 935\ mm$ *Let us provide 12mm ϕ bar @ 100 mm c/c* $A_s = 1050\ mm^2$ *(b) Location of neutral axis* $F_a = A_a \dfrac{f_y}{\gamma_a} = \left(5626 * \dfrac{250}{1.15}\right) \Big/ 1000 = 1223\,kN$ $F_s = A_s \dfrac{f_{sk}}{\gamma_s} = \left(1050 * \dfrac{415}{1.15}\right) \Big/ 1000 = 379\,kN < F_a$ $Depth\,of\ \ web\ in\ tension = \dfrac{D}{2} - \dfrac{F_s}{2t_w * f_y/\gamma_a} \quad = \dfrac{300}{2} - \dfrac{379*1000}{2*7.5*250/1.15}$ $= 33.8\ mm$ *Therefore NA lies in the web.* *Negative Moment of resistance of the section* $M_p = p_y * Z_{px} + \dfrac{A_s f_{sk}}{\gamma_s}\left(\dfrac{D}{2} + a\right) - \left(\dfrac{A_s f_{sk}}{\gamma_s}\right)^2 \Big/ 4t_w f_y/\gamma_a$ $= \dfrac{250}{1.15} * 1.14 * 573.6 * 10^3 + \dfrac{1050 * 415}{1.15}\left(\dfrac{300}{2} + 109\right) - \left(\dfrac{1050 * 415}{1.15}\right)^2 \Big/ 4 * 7.5 * \dfrac{250}{1.15}$ $= 218.3\,kN > 212.9\ kNm$ *(Assuming clear cover to reinforcement 15 mm)*	*Refer section 3.3* *Refer Table 6*

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Positive Bending Moment

(a) Effective width of the concrete flange — Refer Section 3.3

$$b_{eff} = \frac{\lambda_o}{4}$$

$$= \frac{1}{4}(0.8(\lambda)) = \frac{1}{4}(0.8*7500)$$

$$= 1500\,mm$$

b) Location of neutral axis

$$F_a = A_a \frac{f_y}{\gamma_a} = \left(5626*\frac{250}{1.15}\right)\Big/1000 = 1223\ kN$$

$$F_c = 0.85\frac{(0.8*(f_{ck})_{cu})}{\gamma_c}*b_{eff}*h_c = \left(0.85\frac{25}{1.5}*1500*130\right)\Big/1000 = 2763\,kN$$

$$F_c > F_a, \text{Hence neutral axis lies in the slab}$$

Depth of neutral axis

$$x_u = A_a\frac{f_y}{\gamma_a}\Big/0.85\frac{(0.8*(f_{ck})_{cu})}{\gamma_c}*b_{eff} = 5626*\frac{250}{1.15}\Big/0.85\frac{25}{1.5}*1500 = 57.6\ mm$$

Positive Moment of resistance of the section — Refer Table 6

$$M_p = \frac{A_a f_y}{\gamma_a}\left(\frac{D}{2} + h_c - \frac{x_u}{2}\right)$$

$$= \frac{5626*250}{1.15}\left(\frac{300}{2} + 130 - \frac{57.6}{2}\right)*10^{-6}$$

$$= 307.3\,kNm > 185.9\,kNm$$

(2) Check for vertical shear and bending moment and shear force interaction — Refer section 4.5

Vertical shear force, V=160.2 kN < 293.5 kN

Hence safe.

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Bending Moment and Vertical Shear Interaction can be neglected if
$V < 0.5\, V_p$

$V = 162 > 0.5 * 293.5$ kN

$$M \le M_f + (M_p - M_f)\left[1 - \left(2\frac{V}{V_p} - 1\right)^2\right]$$

refer section 4.4

$$= 2*B*T\left(\frac{D-T}{2}\right)\frac{f_y}{\gamma_a} + \left(M_p - 2*B*T\left(\frac{D-T}{2}\right)\frac{f_y}{\gamma_a}\right)*\left[1 - \left(2\frac{V}{V_p} - 1\right)^2\right]$$

$$= \left(2*140*12.4*\left[\frac{300-12.4}{2}\right]*\frac{250}{1.15}\right)*10^{-6} + (236.9 - 108.5)*\left[1 - \left(2*\frac{162}{293.5} - 1\right)^2\right]$$

$212.9 < 235.5$ kNm

(2) Check for shear buckling

refer section 4.2

$d/t_w = (300-2*12.4)/7.5 = 36.7 < 67\varepsilon$, *Hence safe*

[C] Design of shear connectors

Longitudinal shear force

(a) Between simple end support and point of maximum positive moment

Length$=0.4\lambda = 0.4*7500 = 3000$ mm

$V_\lambda = F_a = 1223$ kN

(b) Between point of maximum positive moment and internal support

Length$=7500-3000 = 4500$ mm

$V_\lambda = F_a + A_s f_{sk}/\gamma_s = 1223 + 379 = 1602$ kN

Design resistance of shear connectors

Table 1 (composite Beam-I)

Let us provide 22 mm dia. studs 100 mm high, P = 85 kN

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<u>No. of shear connectors</u>

(a) Between simple end support and point of maximum positive moment

Assuming full shear connection,

No. of shear connectors, $n_f = 1223/85 = 15$

$\therefore$ *Spacing* $= 3000 / 15 = 200$ *mm*

b) Between point of maximum positive moment and internal support.

Assuming full shear connection,

No. of shear connectors, $n_f = 1602/85 = 19$

$\therefore$ *Spacing* $= 4500 / 19 = 230$ *mm*

Let us provide 22 mm dia. Shear Studs @ 200 mm c/c throughout the span.

[D] Transverse reinforcement

Refer Table 6

Assuming a 0.2% reinforcement (perpendicular to the beam) for solid slab

$$A_e = 0.002A_c = 0.002*130*1000 = 265\, mm^2/m$$

Provide 8 mm dia. bar @ 190 mm c/c in 2 layers

$A_e = 2 * 265\ mm^2/m$

Longitudinal shear force in the slab

$v_r = 2.5\ A_{cv}\eta\tau + A_e f_{sk}/\gamma_s + v_p$

or

$v_r = 0.2\ A_{cv}\eta(f_{ck})_{cy}/\gamma_c + v_p/\sqrt{3}$ *, whichever is smaller.*

$A_{cv} = 130*1000 = 130*10^3\ mm^2$

$\eta = 1.0$

$\tau = 0.3\ N/mm^2$

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f_{sk}=415 N/mm^2
γ_s=1.15
A_e =2*265 mm^2/m
v_p=0

v_r=2.5*130*10^3*1*0.3+2*265*415/1.15=288.76 kN/m
or
v_r=0.2*130*10^3*1*25/1.5=433.3 kN/m

Therefore, v_r=288.76 kN/m

The longitudinal design shear force

V_λ = 85*1000 / 200=425 kN/m

For each shear plane

v_r =425 / 2= 212.5 < 288.76 kN/m

Hence safe.