# 5. DEVELOPMENT OF PROGRAM FOR COMPOSITE BEAMS

## 5.1 AN OVERVIEW

Composite beam is the most common form of composite element in steel frame building construction and has been the major form for mid range steel bridges. A steel concrete composite beam consists of a steel beam, over which a reinforced concrete slab is cast with shear connectors, as shown in **Fig. 5.1(a)**. The composite beam can also be constructed with profiled sheeting with concrete topping, instead of cast-in place or pre-cast reinforced concrete slab (**Fig 5.1 (b)**).

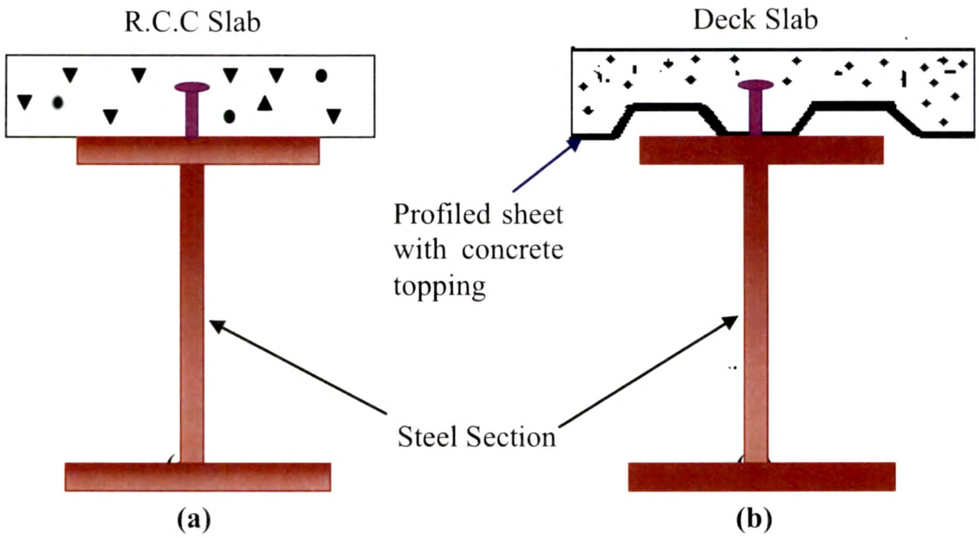


**Fig. 5.1 Typical Section of Composite Beam** [6]

The high bearing capacity and stiffness of composite beams allow the construction of very wide column free rooms with comparatively little construction height. Until now composite beams have only been built single span or continuous; rigid composite connections to the columns have been avoided because of missing knowledge. For higher column spacing castellated beams and trusses are brought into action. In special cases the steel beam sections may be partially encased. The normal method of designing simply-supported beams for strength is by plastic analysis of the cross-section. Full shear connection means that sufficient shear connectors are provided to develop the full plastic capacity of the section. Beams designed for full shear connection result in the lightest beam size. Where fewer shear-connectors are provided (known as partial shear connection) the beam size is heavier but the overall design may be more economical.

Partial shear connection is most attractive where the number of shear-connectors is placed in a standard pattern, such as one per deck trough or one per alternate trough where profiled decking is used. In such cases, the resistance of the shear connectors is a fixed quantity irrespective of the size of the beam or slab. Conventional elastic design of the section results in heavier beams than with plastic design because it is not possible to develop the full tensile resistance of the steel section.

## 5.2 Behaviour of Simply Supported Composite Beam

The behaviour of simply supported composite beams under uniformly distributed load of w/unit length as shown in **Fig. 5.2** is best illustrated by using two identical beams, each having a cross section of b×h and spanning a distance of $\ell$, one placed at the top of the other. For theoretical explanation, two extreme cases of no interaction and 100% (full) interaction are considered and their effect on bending and shear stress distribution is depicted in **Figs. 5.2 (b)** and **(c)** respectively.

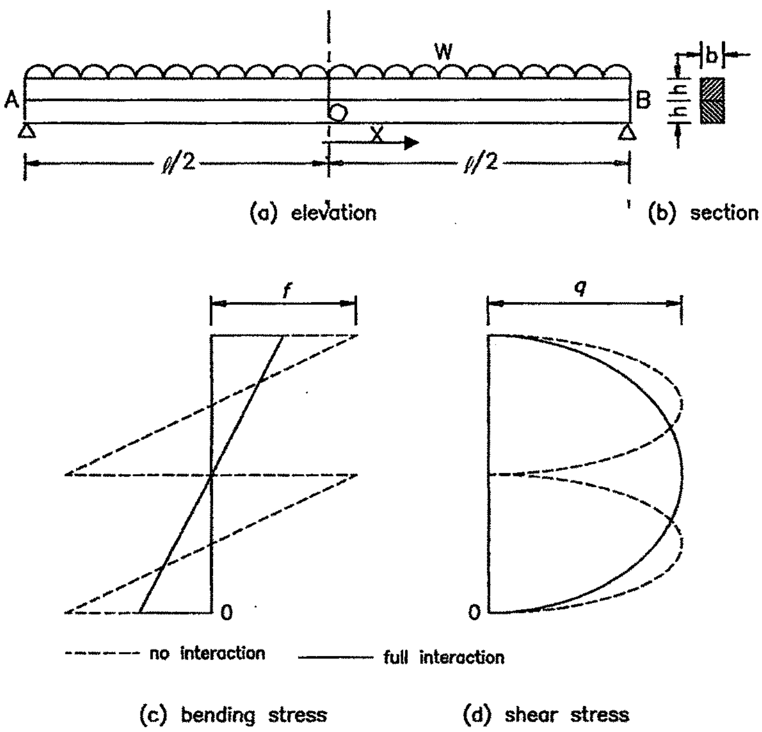


**Fig. 5.2 Effect of Shear Connection on Bending and Shear Stresses**

## 5.3 BEHAVIOUR OF CONTINUOUS COMPOSITE BEAM

Continuous beams offer greater load resistance and greater stiffness which result in a smaller steel section being required to withstand specified loading. However, continuity of structural steel can be achieved economically by running a single length of section across two or more spans. The concrete is cast continuously over the supports and, to control shrinkage and other cracking, the concrete is reinforced. The mid span regions of continuous composite beams behave in the same way as the simple span composite beam. However, the support regions display a considerably different behaviour as shown diagrammatically in **Fig. 5.3** [95]. The concrete in the mid span region is generally in compression and the steel in tension. Over the support this distribution reverses as the moment is now hogging. The concrete cannot carry significant tensile strains and therefore cracks. To avoid cracking longitudinal reinforcements are provided as shown in **Fig. 5.3.**

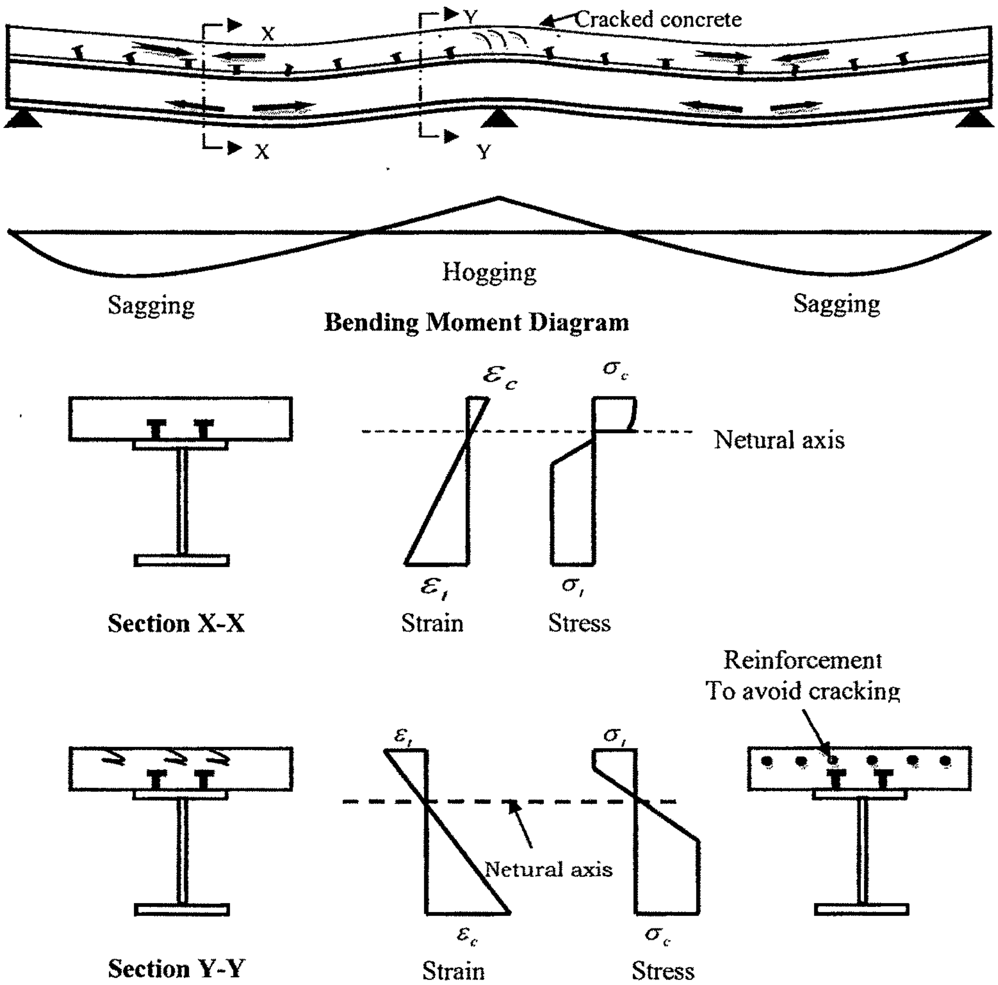


**Fig. 5.3 Behavior of Continuous Composite Beam under Gravity Load**[6]

## 5.4 Basis of The Design

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. Some of the design criteria are also considered as per EC4.

### 5.4.1 Span to Depth Ratio

EC4 specifies the span to depth (total beam and slab depth) ratios as given in **Table 5.1** for which the serviceability criteria will be deemed to be satisfied.

**Table 5.1 Span to Depth Ratio as According to EC4**

| Types of Beam | Eurocode 4 |
|---|---|
| Simply supported | 15-18 (Primary Beams)<br>18-20(Secondary Beams) |
| Continuous | 18-22 (Primary Beams)<br>22-25 (end bays) |

### 5.4.2 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. 5.4**, 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.

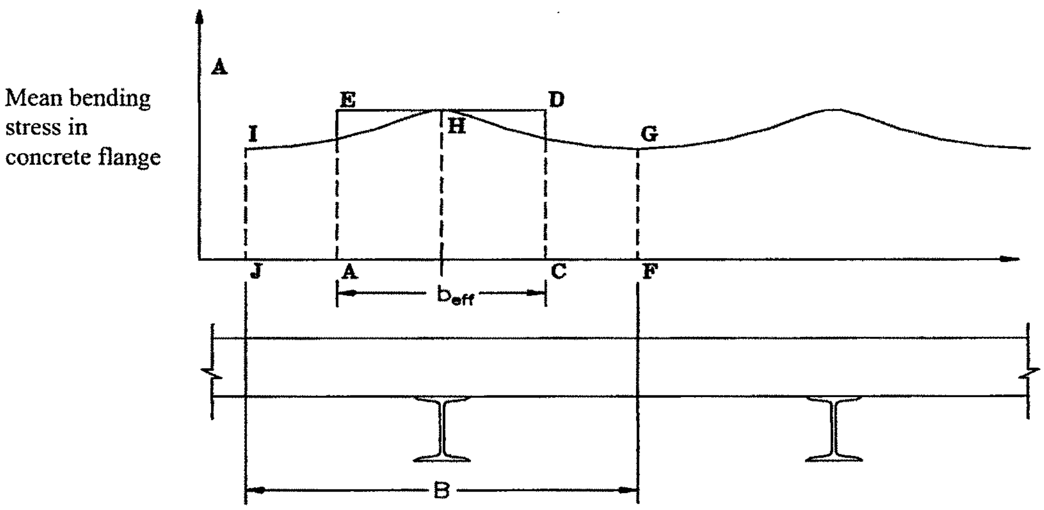


**Fig. 5.4 Use of Effective Width to Allow for Shear Lag**

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

$$b_{eff} = \frac{l}{4}\ but\ \leq B \qquad \text{...(5.1)}$$

where, $l_o$ = The effective span taken as the distance between points of zero moments, l= Actual span and B = Centre to centre distance of transverse spans for slab.

For continuous beams $l_o$ is obtained from **Fig. 5.5** [6].

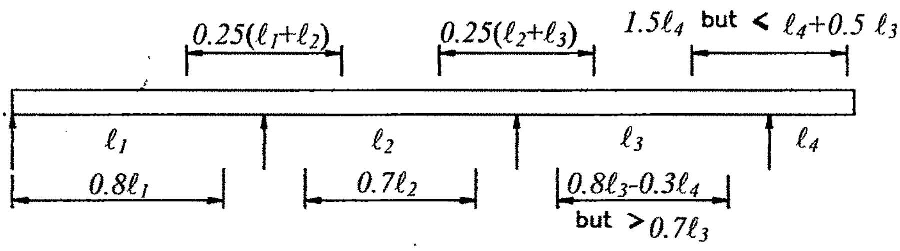


**Fig. 5.5 Value of $\ell_0$ for Continuous Beam as Per EC4**

### 5.4.3 PARTIAL SAFETY FACTOR FOR LOADS AND MATERIALS

The partial safety factors for load $\gamma_f$ and for materials $\gamma_m$ are shown in **Table 5.2.**

**Table 5.2 Partial Safety Factors as per the New IS: 800: 2007**

| Load | Partial safety factor, $\gamma_f$ |
|---|---|
| Dead load | 1.5 |
| Live load | 1.5 |
| **Materials** | **Partial safety factor, $\gamma_m$** |
| Concrete | 1.5 |
| Structural Steel | 1.15 |
| Reinforcement | 1.15 |

### 5.4.4 SECTION CLASSIFICATIONS

Four classes of sections are defined as follows:

a) Plastic – Cross sections, which can develop plastic hinges and have the rotation capacity required for failure of the structure by formation of a plastic mechanism.

b) Compact – Cross sections, which can develop plastic moment of resistance, but have inadequate plastic hinge rotation capacity for formation of a plastic mechanism.

c) Semi-Compact – Cross sections, in which the extreme fibre in compression can reach, yield stress, but cannot develop the plastic moment of resistance, due to local buckling.

d) Slender – Cross sections in which the elements buckle locally even before reaching yield stress.

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 5.3.**

**Table 5.3 Classification of Section**

<table>
<tr><th rowspan="2">Type of Element</th><th rowspan="2" colspan="2">Type of Section</th><th colspan="3">Class of Section</th></tr>
<tr><th>Plastic($\beta_1$)</th><th>Compact($\beta_2$)</th><th>Semi-compact($\beta_3$)</th></tr>
<tr><td rowspan="2">Outstand element of compression flange</td><td colspan="2">Rolled</td><td>$b/t \leq 9.4\varepsilon$</td><td>$b/t \leq 10.5\varepsilon$</td><td>$b/t \leq 15.7\varepsilon$</td></tr>
<tr><td colspan="2">Welded</td><td>$b/t \leq 8.4\varepsilon$</td><td>$b/t \leq 9.4\varepsilon$</td><td>$b/t \leq 13.6\varepsilon$</td></tr>
<tr><td rowspan="2">Internal element of compression flange</td><td colspan="2">Bending</td><td>$b/t \leq 29.3\varepsilon$</td><td>$b/t \leq 33.5\varepsilon$</td><td>$b/t \leq 42\varepsilon$</td></tr>
<tr><td colspan="2">Axial Comp.</td><td colspan="2">Not Applicable</td><td>$b/t \leq 42\varepsilon$</td></tr>
<tr><td rowspan="3">web</td><td colspan="2">N.A. at mid depth</td><td>$d/t \leq 84\varepsilon$</td><td>$d/t \leq 105\varepsilon$</td><td>$d/t \leq 126\varepsilon$</td></tr>
<tr><td rowspan="2">Generally d/t</td><td>If r1 is negative</td><td rowspan="2">$\frac{84\varepsilon}{1+r_1} \leq 42\varepsilon$</td><td>$\frac{105\varepsilon}{1+r_1} \leq 42\varepsilon$</td><td rowspan="2">$\frac{126\varepsilon}{1+2r_1} \leq 42\varepsilon$</td></tr>
<tr><td>If r1 is positive</td><td>$\frac{105\varepsilon}{1+1.5r_1} \leq 42\varepsilon$</td></tr>
</table>

where, b = half width of flange of rolled section, t = Thickness, d = clear depth of web, $\varepsilon = \sqrt{250/f_y}$ , and $f_y$ = 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.

## 5.5 DESIGN OF COMPOSITE BEAMS

The composite beam is designed to have sufficient bending strength and stiffness and secure connection to the slab. The principle aspect of behaviors of the composite beam which need to be considered in this respect are bending strength, adequacy of the connection between slab and beam and its deflection performance.

### 5.5.1 REINFORCED CONCRETE SLAB SUPPORTED ON STEEL BEAMS

Reinforced concrete slab connected to rolled steel section through shear connectors **(Fig. 5.6)** is perhaps the simplest form of composite beam.

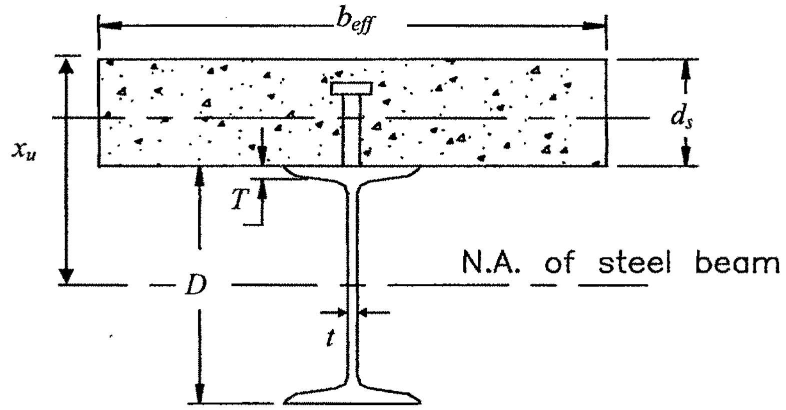


**Fig. 5.6 Notations as per IS: 11384-1985**

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 [1]. In this code a parabolic stress distribution is assumed in the concrete slab. Here a stress factor $a = 0.87f_y/0.36(f_{ck})_{cu}$ is applied to convert the concrete section into steel. The additional assumptions made by the IS: 11384-1985 are given below:

- The maximum strain in concrete at outermost compression member is taken as 0.0035 in bending.
- The total compressive force in concrete is given by $f_{cc} = 0.36\ (f_{ck})_{cu}\ bx_u$ and this acts at a depth of 0.42 $x_u$, not exceeding $d_s$.
- The stress strain curve for steel section and concrete are as per IS: 456-2000.

The notations used here are : $A_f$ = area of top flange of steel beam, $A_s$ = cross sectional area of steel beam, $b_{eff}$ = effective width of concrete slab, $b_f$ = width of top flange of steel section, $d_c$ = distance between centroids of concrete slabs and steel beam in a composite section, $t_f$ = thickness of the top flange of the steel section, $x_u$ = depth of neutral axis at ultimate limit state of flexure, $M_u$ = ultimate bending moment.

The three cases that may arise are given below with corresponding $M_u$.

**Case I: Plastic neutral axis within the slab (Fig. 5.7)**

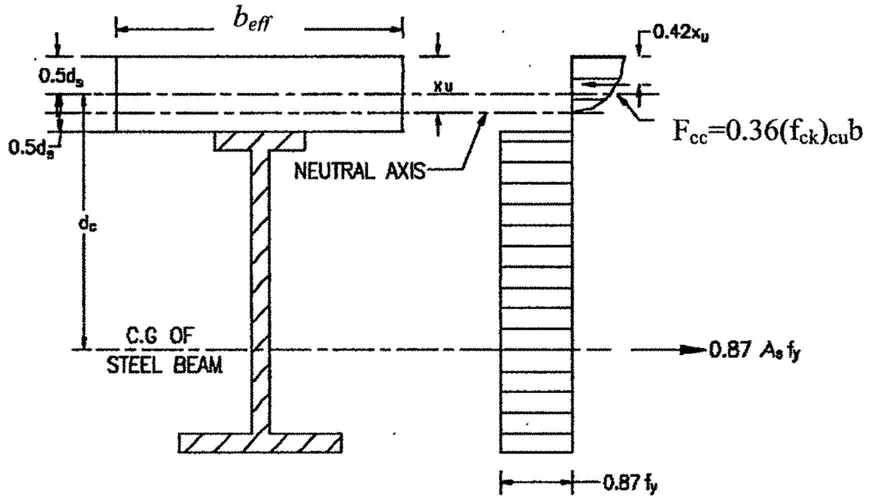


**Fig. 5.7 Stress Distribution with Neutral Axis within Concrete Slab**

This occurs when $b_{eff}\, d_s \geq a\, A_s$

Taking moment about centre of concrete compression

$$M_u = 0.87 A_s f_y (d_c + 0.5 d_s - 0.42 x_u) \qquad ...(5.2)$$

Where, $x_u = aA_s/b_{eff}$ *and* $a = 0.87 f_y / 0.36 (f_{ck})_{cu}$

**Case II: Plastic neutral axis within the top flange of steel section (Fig. 5.8)**

This happens when $b_{eff}\, d_s < aA_s < (b_{eff}\, d_s + 2a\, A_f)$

Equating forces, one gets

$$x_u = d_s + \frac{aA_s - b_{eff} d_s}{2 b_f a} \qquad ...(5.3)$$

Taking moment about centre of concrete compression

$$M_u = 0.87 f_y \left[ A_s (d_c + 0.08 d_s) - b_f (x_u - d_s)(x_u + 0.16 d_s) \right] \qquad ...(5.4)$$

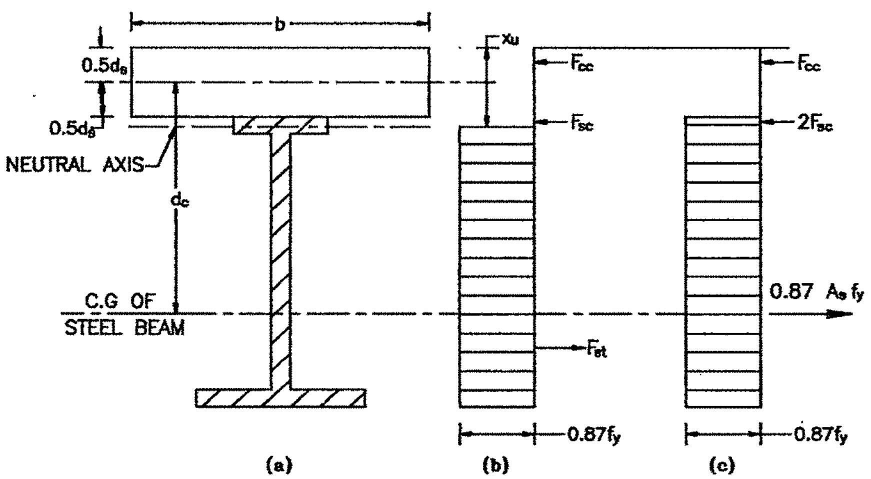


**Fig. 5.8 Stress Distribution with Neutral Axis within Flange of Steel Beam**

**Case III: Plastic neutral axis lies within web (Fig. 5.9)**

This happens when, $a\,(A_s - 2A_f) > b_{eff}\,d_s$

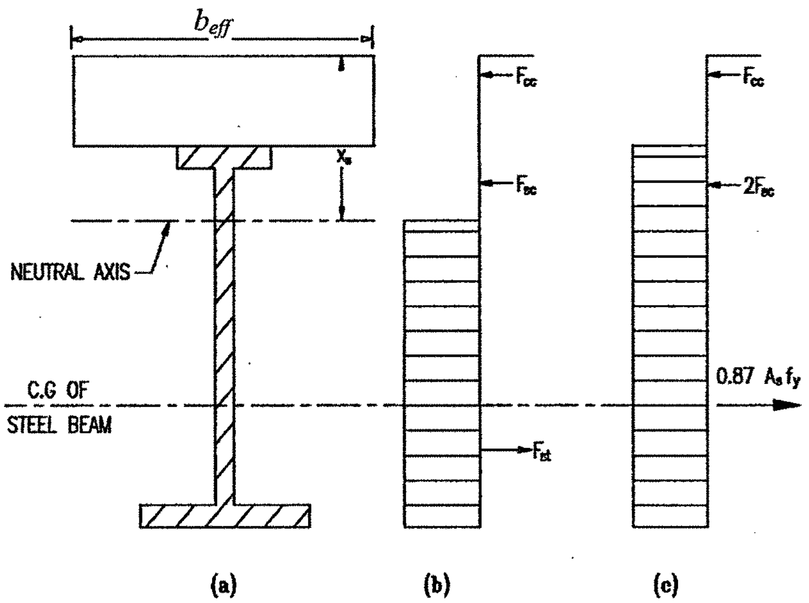


**Fig. 5.9 Stress Distribution with Neutral Axis within the Web of the Steel Beam**

Equating area under tension and compression

$$x_u = d_s + t_f + \frac{a(A_s - 2A_f) - b_{eff}d_s}{2at_w} \qquad \text{... (5.5)}$$

Taking moment about the centre of concrete compression

$$M_u = 0.87 f_y A_s (d_c + 0.08 d_s) - 2A_f(0.5t_f + 0.58d_s)$$
$$-2t_w(x_u - d_s - t_f)(0.5x_u + 0.08d_s + 0.5t_f) \qquad \text{... (5.6)}$$

### 5.5.2 REINFORCED CONCRETE SLABS WITH PROFILED DECK AND STEEL BEAMS

The design procedure of composite beams depends upon the class of the compression flange and web. **Table 5.4** shows the classification of the sections suggested in EC4 based upon the buckling tendency of steel flange or web. The resistance to buckling is a function of width to thickness ratio of compression members. **Table 5.4** shows that for sections falling in Class 1 and 2 [7], plastic analysis is recommended. For simply supported composite beams the steel compression flange is restrained from local as well as lateral buckling due to its connection to concrete slab. Moreover, the plastic neutral axis is usually within the slab or the steel flange for full interaction. So, the web is not in compression. This allows the composite section to be analysed using plastic method.

**Table 5.4 Classification of Sections and Methods of Analysis (EC4)**

| Slenderness class and name | 1 plastic | 2 compact | 3 semi-compact | 4 slender |
|---|---|---|---|---|
| Method of global analysis | plastic | elastic | elastic | elastic |
| Analysis of cross-sections | plastic | plastic | elastic | elastic |
| Maximum ratio of c/t for flanges of rolled I-section | | | | |
| Uncased web | 8.14 | 8.95 | 12.2 | no limit |
| Encased web | 8.14 | 12.2 | 17.1 | no limit |

Where, c is half the width of a flange of thickness t.

The notations used here are as follows:

$A_a$ = area of steel section, $\gamma_a$ = partial safety factor for structural steel, $\gamma_c$ = 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 and $h_g$ = 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.

**Case I: Neutral axis within the concrete slab (Full shear connection, Fig. 5.10)**

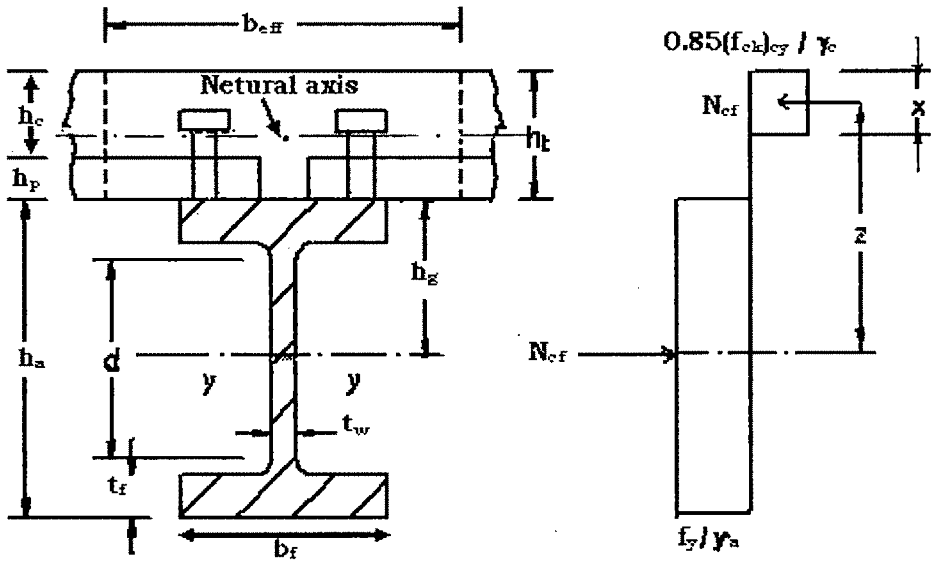


**Fig. 5.10 Distribution with Neutral Axis in Concrete Slab**

This occurs when

$$0.85\frac{(f_{ck})_{cy}}{\gamma_c}b_{eff}h_c \geq \frac{A_a f_y}{\gamma_a} \quad ...(5.7)$$

The depth of plastic neutral axis can be found by using force equilibrium.

$$N_{cf} = \frac{A_a f_y}{\gamma_a} = b_{eff}\, x\, 0.85\frac{(f_{ck})_{cy}}{\gamma_c} \quad ...(5.8)$$

$$\therefore x = \frac{A_a f_y/\gamma_a}{b_{eff}\;0.85\frac{(f_{ck})_{cy}}{\gamma_c}} \quad ...(5.9)$$

This expression is valid for $x \leq h_c$.

The plastic moment of resistance of the section,

$$M_p = \frac{A_a f_y}{\gamma_a}(h_g + h_t - x/2) \quad ...(5.10)$$

**Case II: Neutral axis within the steel top flange (Full shear connection, Fig. 5.11)**

This case arises when

$N_{cf} < N_{a\cdot pl}$

$$i.e.\, b_{eff}\, h_c 0.85\frac{(f_{ck})_{cy}}{\gamma_c} < \frac{A_a f_y}{\gamma_a} \quad ...(5.11)$$

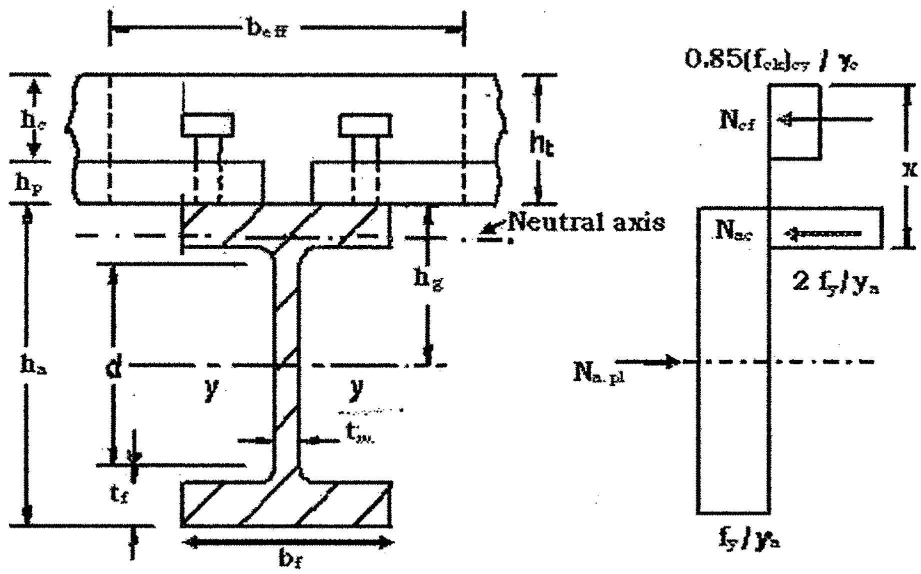


**Fig. 5.11 Stress Distribution with Neutral Axis in Flange of Beam**

To simplify the calculation it is assumed that strength of steel in compression is $2f_y/\gamma_a$, so that, the force $N_{a\cdot pl}$ and its line of action remain unchanged. Note that the compression flange is assumed to have a tensile stress of $f_y/\gamma_a$ and a compressive stress of $2f_y/\gamma_a$, giving a net compressive stress of $f_y/\gamma_a$. So, the plastic neutral axis will be within steel flange if,

$$N_{a\text{-}pl} - N_{cf} \leq 2\, b_f\, t_f\, f_y/\gamma_a$$

Equating tensile force with compressive,

$$N_{a\text{-}pl} = N_{cf} + N_{ac}$$

$$i.e. \frac{A_a f_y}{\gamma_a} = 0.85 \frac{(f_{ck})_{cy}}{\gamma_c}\, b_{eff} h_c + 2b_f\,(x - h_l)\frac{f_y}{\gamma_a} \qquad \text{... (5.12)}$$

The value of x is found from the above equation.

The plastic moment of resistance is found from

$$M_p = N_{a.pl}(h_g + h_l - h_c/2) - \frac{N_{ac}(x - h_c + h_l)}{2} \qquad \text{... (5.13)}$$

**Case III: Neutral axis lies within web (Full shear connection, Fig. 5.12)**

If the value of x exceeds ($h_c$ + $t_f$), then the neutral axis lies in the web. In design this case should be avoided, otherwise the web has to be checked for slenderness.

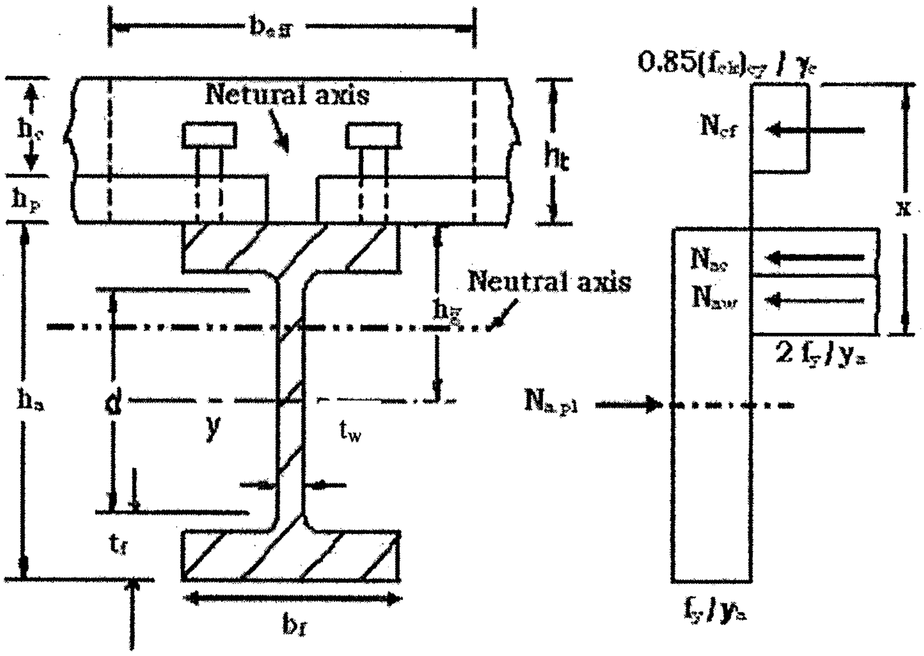


**Fig. 5.12 Stress Distribution with Neutral Axis in the Web of the Beam**

In similar procedure as the previous one, here *x* can be found from

$$N_{a.pl} = N_{cf} + N_{acf} + N_{aw}$$

$$= N_{cf} + 2b_f t_f f_y/\gamma_a + 2t_w(x - h_l - t_f)f_y/\gamma_a \qquad \text{... (5.14)}$$

Plastic moment of resistance

$$M_p = N_{a.pl}(h_g + h_l - h_c/2) - N_{acf}(h_l + t_f/2 - h_c/2) - N_{a.w}(x + h_l + t_f - h_c)/2 \qquad \text{... (5.15)}$$

**Case IV: Resistance to hogging Bending Moment**

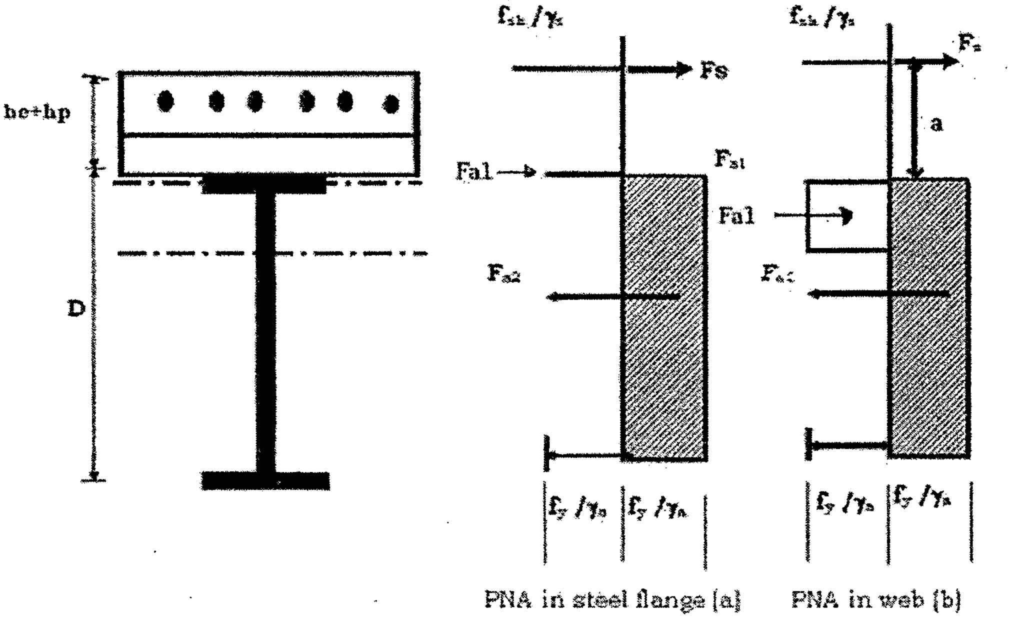


**Fig. 5.13 Resistance to Hogging Bending Moment**

The stress distribution for hogging moment region for neutral axis within flange and for netural axis within web is shown in **Fig. 5.13**. In case of continuous composite beam resistance to hogging moment is calculated by using formula given in **Table 5.5**.

**Table 5.5 Negative Moment Capacity of Section with Full Shear Connection**

| Position of Plastic Neutral Axis | Condition | Moment Capacity $M_p$ |
|---|---|---|
| Plastic neutral axis in steel flange, Fig. 5.13 (a) | $\frac{A_{aw}f_y}{\gamma_a} < \frac{A_s f_{sk}}{\gamma_s} < \frac{A_a f_y}{\gamma_a}$ | $M_p \approx \frac{A_a f_y}{\gamma_a}\frac{D}{2} + \frac{A_s f_{sk}}{\gamma_s} a$ |
| Plastic neutral axis in web, Fig. 5.13 (b) | $\frac{A_s f_{sk}}{\gamma_s} < \frac{A_{aw}f_y}{\gamma_a}$ | $M_p = M_{ap} + \frac{A_s f_{sk}}{\gamma_s}\left(\frac{D}{2} + a\right) + \left(\frac{A_{sk}f_{sk}}{\gamma_s}\right)^2 \Big/ 4 * t * \frac{f_y}{\gamma_a}$ |

## 5.6 OTHER DESIGN ASPECTS

### 5.6.1 VERTICAL SHEAR RESISTANCE

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 was not composite.

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

$$V \leq V_p$$

where, $V_p$ is the plastic shear resistance given by,

$$= 0.6Dt\frac{f_y}{\gamma_a}\ (for\ rolled\ I, H, C\ sections) \quad ...(5.16)$$

$$= dt\frac{f_y}{\gamma_a\sqrt{3}}\ (for\ builtup\ I\ sections) \quad ...(5.17)$$

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

$$\frac{d}{t} \leq 67\varepsilon \qquad for\ web\ not\ encased\ in\ concrete \quad ...(5.18)$$

$$\frac{d}{t} \leq 120\varepsilon \qquad for\ web\ encased\ in\ concrete \quad ...(5.19)$$

where, $\varepsilon = \sqrt{(250/fy)}$ and d is the depth of the web considered in the shear area.

### 5.6.2 RESISTANCE OF SHEAR CONNECTORS

#### 5.6.2.1 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 5.14.** EC 4 suggests the following reduction factor $k$ (relative to solid slab).

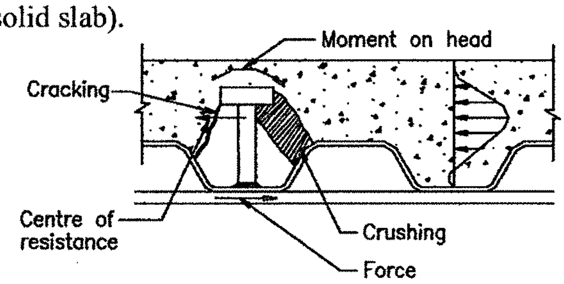


**Fig. 5.14 Behaviour of a Shear Connection Fixed Through Profile**

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

$$K_p = 0.6\frac{b_0}{h_p}\left(\frac{h - h_p}{h_p}\right) \leq 1.0\ where\ h \leq h_p + 75 \quad ...(5.20)$$

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

For studs of diameter not exceeding *20 mm,*

$$k_t = \frac{0.7}{\sqrt{N_r}} \frac{b_0}{h_p} \left( \frac{h - h_p}{h_p} \right) \leq 1.0 \; where \; h_p \leq 85 \; and \; b_0 \geq h_p \qquad \text{... (5.21)}$$

where, $b_0$ = is the average width of trough, h = is the stud height, $h_p$ = is the height of the profiled decking slab, and $N_r$ = is the number of stud connectors in one rib at a beam intersection (Should not be 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 $N_r \geq 2$.

### 5.6.3 LONGITUDINAL SHEAR FORCE

#### 5.6.3.1 Full Shear Connection

Single span beams

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

$$V_l = F_{cf} = \frac{A_a f_y}{\gamma_a} \qquad \text{... (5.22)}$$

Or

$$V_l = 0.85 f_{ck} \frac{(f_{ck})_{cy} b_{eff} h_c}{\gamma_c} \qquad \text{... (5.23)}$$

Whichever is smaller.

Continuous span beams

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

$$V_l = F_{cf} + \frac{A_s f_{sk}}{\gamma_s} + \frac{A_{ap} f_{yp}}{\gamma_{ap}} \qquad \text{... (5.24)}$$

where, $A_s$ is the effective area of longitudinal slab reinforcement and $A_{ap}$ is the effective area of profiled steel sheeting.

Numbers of shear connectors

The number of shear connector should be calculated to resist the horizontal shear force to be transmitted at collapse between point of maximum and zero moment. This force is taken as the force in the concrete $F_{cc}$ at ultimate moment (IS: 11384-1985). Number of connectors is calculated by dividing the total load carried by connectors to the design strength of connectors.

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

$$n_f = \frac{V_l}{P} \quad \text{... (5.25)}$$

where, $V_\ell$ is the design longitudinal shear force, and P is the design resistance of the connector. The shear connectors are usually equally spaced.

**5.6.3.2 Minimum degree of shear connectors**

The minimum degree of shear connection for headed studs with an overall length after welding not less than 4 times diameter and shank diameter not less than 16 mm and not exceeding 22 mm is defined by the following equations:

For steel sections with equal flanges :

$$L \leq 5 \qquad n/n_f \geq 0.4$$

$$5 \leq L \leq 25 \qquad \frac{n}{n_f} \geq 0.25 + 0.03L$$

$$L \geq 5 \qquad n/n_f \geq 1.0$$

where, L is the span of the beam in meter, $N_f$ is the number of stud connectors determined for relevant length of beam in accordance of with 5.8.1, and N is the number of stud connectors.

(i) Between the point of maximum bending moment and the end support $V_\ell$ to be resisted by shear connectors is given by;

$$V_l = F_c \quad \text{... (5.26)}$$

(ii) Between the point of maximum positive bending moment and an intermediate support $V_\ell$ to be resisted by shear connectors is given by:

$$V_l = F_c + \frac{A_s f_{sk}}{\gamma_s} + \frac{A_{ap} f_{yp}}{\gamma_{ap}} \quad \text{... (5.27)}$$

where,

$$F_c = \frac{M - M_{ap}}{M_p - M_{ap}} F_{cf} \quad \text{... (5.28)}$$

## 5.6.4 INTERACTION BETWEEN MOMENT AND SHEAR

Interaction between bending and shear can influence the design of continuous beam. **Fig. 5.15** 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. 5.15**), 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

$$M \le M_f + \left(M_p - M_f\right)\left[1 - \left(2\frac{V}{V_p} - 1\right)^2\right] \quad \text{...(5.29)}$$

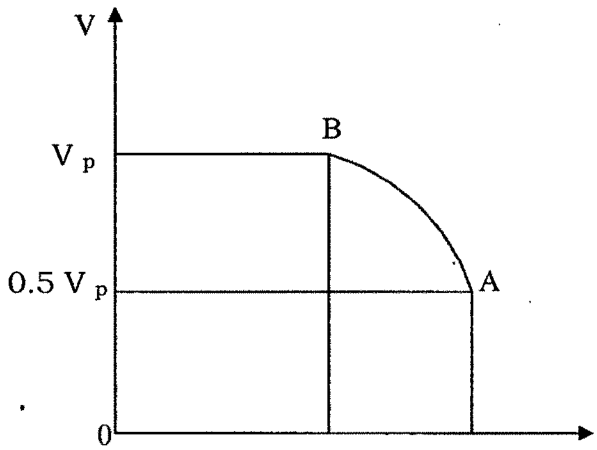


**Fig. 5.15 Resistance to Combined Bending and Vertical Shear**

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 and $V_p$ = plastic shear resistance.

### 5.6.5 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. 5.16**. 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.

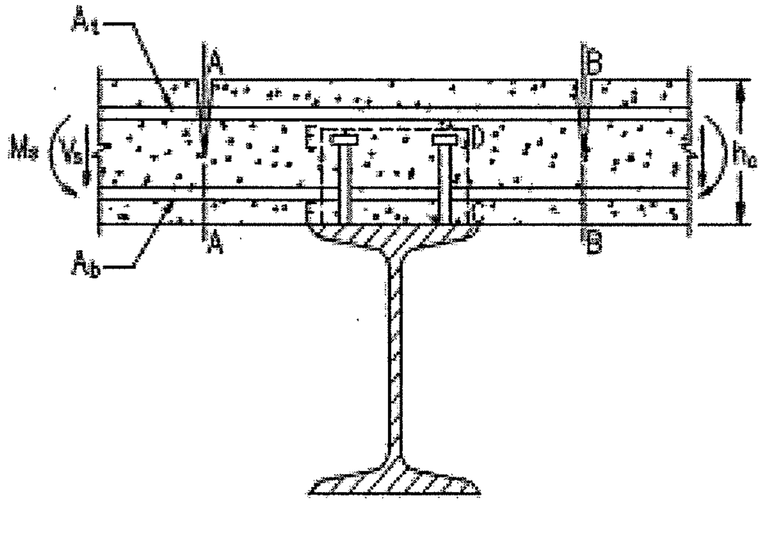


**Fig. 5.16 Surfaces of Potential Shear Failure [90]**

The formulae suggested by EC4 and IS: 11384 – 1985 are given in **Table 5.6** [6].

**Table 5.6 Comparison of Provisions for Transverse Reinforcement**

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

### 5.6.6 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.

In order to determine the distribution of bending moments under the design loads, structural analysis has to be performed. For convenience, the IS: 456-2000 [96] 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.

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. 5.17.**

**Fig. 5.17 Inverted – U Frame Action**

Local-torsional buckling of continuous beams can be neglected if following conditions are satisfied:

- 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.
- The loading on each span is uniformly distributed and the design permanent load exceeds *40%* of the total load.
- The shear connection in the steel-concrete interface satisfies the requirements of section 5.8.
- $h_a \leq 550\ mm.$

### 5.6.7 SERVICEABILITY LIMIT STATES

For simply supported composite beams the most critical serviceability limit state is usually deflection. This would be a governing factor in design for un-propped construction. Besides, the effect of vibration, cracking of concrete, etc. should also be checked under serviceability criteria. Often in exposed condition, it is preferable to design to obtain full slab in compression to avoid cracking in the shear connector region. IS: 11384 – 1985 limits the maximum deflection of the composite beam to L/325. The total elastic stress in concrete is limited to $f_{ck}/3$ while for steel, considering different stages of construction, the elastic stress is limited to 0.87 fy.

### 5.6.7.1 Stresses and deflection in service

As structural steel is supposed to not to yield at service load, elastic analysis is employed in establishing the serviceability performance of composite beam. In this method the concrete area is converted into equivalent steel area by applying modular ratio $m = E_s/E_c$. The analysis is done in terms of equivalent steel section. It is assumed that full interaction exists between steel beam and concrete slab. The effect of reinforcement in compression, the concrete in tension and the concrete between rib of profiled sheeting are ignored.

Refer to **Fig. 5.18**, where a transformed section is shown.

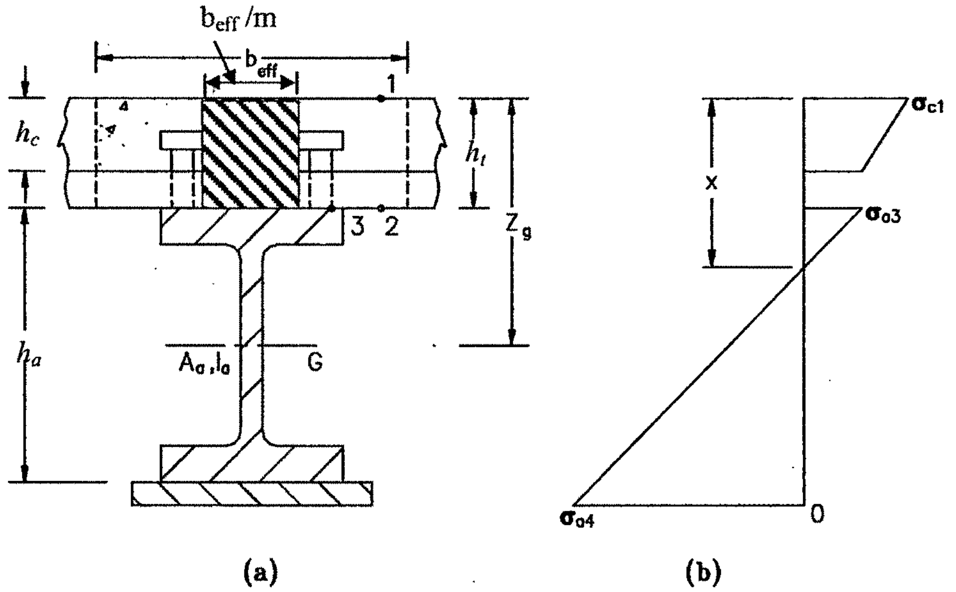


**Fig. 5.18 Elastic Analysis of Composite Beam Section in Sagging Bending**

When neutral axis lies within the slab,

$$A_a(Z_g - h_c) < {}^1\!/_2\, b_{eff}\frac{h_c^{\,2}}{m} \qquad \text{... (5.30)}$$

The actual neutral axis depth can be found from

$$A_a(Z_g - x) = {}^1\!/_2\, b_{eff}\frac{x^2}{m} \qquad \text{... (5.31)}$$

and the moment of inertia of the transformed section is given by

$$I = I_a + A_a(Z_g - x)^2 + \left(\frac{b_{eff}}{m}\right)\frac{x^3}{3} \qquad \text{... (5.32)}$$

When neutral axis depth exceeds $h_c$, its depth x is found from the following equation.

$$A_a(Z_g - x) = \frac{b_{eff}}{m} h_c \left(x - \frac{h_c}{2}\right) \qquad \text{... (5.33)}$$

and moment of inertia of the transformed section is obtained by

$$I = I_a + A_a(Z_g - x)^2 + \frac{b_{eff}}{m} h_c \left(\frac{h_c^2}{12} + \left(x - {}^{h_c}/_2\right)^2\right) \qquad \text{... (5.34)}$$

For distributed load w over a simply supported composite beam, the deflection at mid-span is

$$\delta_c = \frac{5wL^4}{384E_a I} \qquad \text{... (5.35)}$$

where, $E_a$ = Young's Modulus for structural steel, and I = moment of inertia.

The beam can be checked for stresses under service load using the value of 'I' as determined above.

When the shear connection is only partial the increase in deflection occurs due to longitudinal slip. This depends on method of construction. Total deflection is given by the formula,

$$\delta = \delta_c \left(1 + k\left(1 - \frac{N}{N_f}\right)\left(\frac{\delta_a}{\delta_c} - 1\right)\right) \qquad \text{... (5.36)}$$

Where k = 0.5 for propped construction, k = 0.3 for un-propped construction, and $\delta_a$ = deflection of steel beam acting alone.

The expression gives acceptable results when $n_p/n_f \geq 0.4$

The increase in deflection can be disregarded where:

- either $n_p/n_f \geq 0.5$ or
- when the transverse rib depth is less than 80 mm.

### 5.6.7.2 Continuous Beam

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.

### 5.6.7.3 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,

$$p = \frac{A_s}{A_c} = k_c * k * \frac{f_{ct}}{\sigma_s} \quad \text{... (5.37)}$$

where p = is the percentage of steel, $k_c$ = is a coefficient due to the bending stress distribution in the section( kc ≈ 0.9), k = is a coefficient accounting for the decrease in the tensile strength of concrete (k ≈ 8), $f_{ct}$ = is the effective tensile strength of concrete with the minimum value as 3 N/mm$^2$ and $\sigma_s$ = maximum permissible stress in concrete.

## 5.7 ILLUSTRATIVE EXAMPLE

Design a simply supported composite beam with 10 m span shown in the **Fig. 5.19.** 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$^2$.

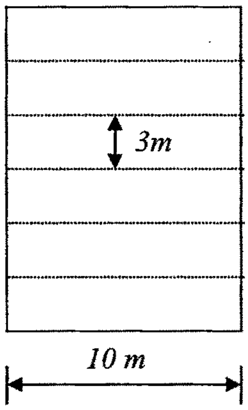


**Fig. 5.19 Layout of Composite Beam**

**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 ( $\gamma_f$ )

for LL - 1.5

for DL - 1.35

**Step 1: Load Calculation**

Construction stage

Self weight of slab = $3 \times 0.125 \times 24 = 9$ kN/m

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

Construction load = $0.75 \times 3 = 2.25$ kN/m

Total design load at construction stage

= {$1.5 \times 2.25 + 1.35 \times (9 + 0.71) = 16.5$ kN/m

Composite stage

Dead Load

Self weight of slab = 9 kN/m

Self weight of beam = 0.71 kN/m

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

Total dead load = 11.2 kN/m

Live Load

Imposed load = $3 \times 3$ = 9.0 kN/m

Load from partition wall = $1.5 \times 3 = 4.5$ 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

$M = 16.5 \times 10^2/8 = 206$ kNm

Composite Stage

$M = 35.4 \times 10^2/8 = 442$ kNm

**Step 3: Classification of Composite Section**

Sectional Properties

T = 17.4mm;

D = 450 mm;

t = 9.4 mm

$I_x = 303.9 \times 10^6$ mm$^4$

$I_y = 8.34 \times 10^6$ mm$^4$

$Z_x = 1350 \times 10^3$ mm;

$r_y = 30.1$ mm

Classification of composite section

$0.5 B/T = 0.5 \times 150/17.4 = 4.3 < 8.9\varepsilon$

$d/t = (450 - 2 \times 17.4)/9.4 = 44.2 < 83\varepsilon$

Therefore the section is a plastic section.

**Step 4: Check for Adequacy of the Section at Construction Stage**

Design moment in construction stage = 206 kNm

Moment of resistance of steel section = $f_{yd} \times Z$

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

= 334.7 kNm > 206 kNm

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

**Step 5: Check for Lateral Buckling of the Top Flange**

Elastic critical stress, $f_{cb}$ is given by

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

$k_1 = 1$ (as $\Psi = 1.0$)

$k_2 = 0$ (as $\phi = 0.5$)

$c_2 = c_1 = 225$ mm;

$T = 17.4$ mm;

$D = 450$ mm;

$\ell = 10{,}000$ mm;

$r_y = 30.1$ mm

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

Therefore the bending compressive stress in beams

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

Moment at construction stage = 206 kNm

Maximum stress at top flange of steel section

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

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

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

$$f_{cb} = \frac{26.5 \times 10^5}{\left(\frac{3330}{30.1}\right)^2} \left[ \sqrt{1 + \frac{1}{20}\left(\frac{10000}{30.1} \times \frac{17.4}{450}\right)^2} \right] = 299.6\ N/mm^2$$

Therefore, the bending compressive stress in beams

$$f_{cb} = \frac{299.6 \times 250}{[(299.6)^{1.4} + (250)^{1.4}]^{\frac{1}{1.4}}} = 165.9\ N/mm^2$$

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

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

**Step 6: 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

<u>Position of neutral axis</u>

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

$A_a$ = 9227 mm$^2$

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

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

Hence PNA lies in concrete.

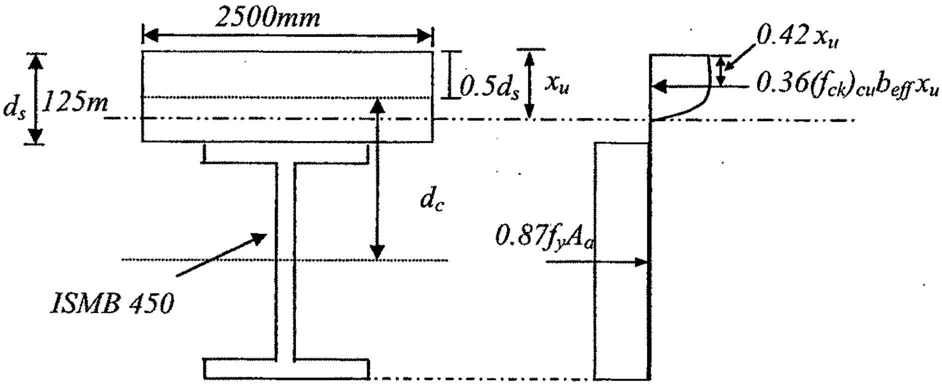


**Fig. 5.20 Cross Section of Composite Beam with Stress Diagram**

**Step 7: Design of Shear Connectors**

The position of neutral axis is within slab.

∴ Total load carried by connectors,

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

The design strength of 20 mm (dia) headed stud for M30 concrete is 58 kN.

∴ Number of shear connectors required for 10/2 m = 5 m length = 2006 /58 ≈ 34

These are spaced uniformly; Spacing = 5000/34 = 147 mm ≈ 145 mm

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

**Step 8: Serviceability Check**

Modular ratio for live load = 15

Modular ratio for deal load = 30

<u>Deflection</u>

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

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

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

To find the moment of inertia of the composite section, one has to first locate the position of neutral axis $x_u$ as

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

Moment of resistance of the section, $M_p$

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

$= 0.87 \times 9227 \times 250(287.5 + 0.5 \times 125 - 0.42 \times 74.3) = 640$ kNm > 442 kNm

Position of neutral axis

$$A(d_g - d_s) < \frac{1}{2}(b_{eff}/\alpha_e) d_s^{\,2}$$

$9227\,(350 - 125) < ½ \times 2500/15 \times 125^2$

$2.08 \times 10^6 < 1.3 \times 10^6$ which is not true

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

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

$$9227\left(\frac{450}{2} + 125 - x_u\right) = \frac{2500}{1500} 125 \left(x_u - \frac{125}{2}\right)$$

$$x_u = 150.75\, mm$$

Moment of inertia of the gross section($I_g$),

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

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

$= 859.6 \times 10^6$

$$\delta_1 = \frac{5 \times 15(1000)^2}{384 \times 2 \times 10^5 \times 859.6 \times 10^6}$$

Total Deflection = $\delta_d + \delta_l = 20.8 + 11.4\, mm$ = 32.2 mm > l/325

The section fails to satisfy the deflection check.

Composite Stage:

<u>Dead load</u>

At 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

$$A(d_g - d_s) < \frac{1}{2} b_{eff} {d_s}^2 / \alpha_e$$

9227 (350 – 125) < ½ × 2500/30 × $125^2$

2.07× $10^6$ > 6.5× $10^5$

∴ N.A. depth exceeds $d_s$.

Location of neutral axis

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

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

$$x_u = 197.5\ mm$$

Moment of area of the section

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

$$= 303.91 \times 10^6 + 9227(350 - 197.5)^2 + \frac{2500 \times 125}{15}\left[\frac{125^2}{12} + \left(197.5 - \frac{125}{2}\right)^2\right]$$

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

Stress in steel flange

$$= \frac{140 \times 10^6(450 + 125 - 197.5)}{721.9 \times 10^6} = 73.2\ N/mm^2.$$

Live load

At composite stage, stress in steel for live load

$W_l$ =13.5 kN/m

M = 13.5× $10^2$/8 = 168.75 kNm

Stress in steel flange

$$= \frac{168.75 \times 10^6(450 + 125 - 150.75)}{859.6 \times 10^6} = 83.29\ N/mm^2$$

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

In a similar manner, the stress in concrete is found.

$$\frac{1}{30}\left\{\frac{140 \times 10^6 \times 197.54}{721.9 \times 10^6}\right\} + \frac{1}{15}\left\{\frac{168.75 \times 10^6 \times 150.75}{859.6 \times 10^6}\right\}$$

$= 3.25 < \frac{(f_{ck})_{cu}}{3} = 10\ N/mm^2$

The section is safe.

**Step 9: Transverse Reinforcement**

Shear force transferred per meter length

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

$\ \ = 400\ kN/m$

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

Or

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

$L_s = 2 \times 125 = 250$ mm

$f_y = 250$ mm

n = 2

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

$= 0.232 \times 250\sqrt{30} + 0.1 \times A_{sv} \times 250 \times 2 = 317.7 + 50A_{sv}$

Or

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

$\therefore 400 = 317 + 50A_{sv} = 165\, mm^2/m$

Minimum Reinforcement

$= 250\, v_{rf}/f_y\, mm^2/mm = 400\ mm^2/mm$

Provide 12Φ @ 280 mm c/c.

## 5.8 PROGRAM FOR COMPOSITE BEAMS

In the present work, a program is developed in Visual Basic for the design of composite beam with R.C.C. slab and design of composite beam with deck slab. A form shown in **Fig. 5.21** is startup screen for design of simply supported or continuous beam. User can tick mark the checkbox to specify the type of design of composite beam. Program is coded in such a way that the calculations of design of floor deck of previous chapter are transferred directly to the selected beam and loading and moments and shear forces are calculated at construction stage and composite stage. Form of **Fig. 5.22** gives the choice of section with available section database. Here whole steel table is interfaced so that the user can choose any section available in the market; even user can change the properties in boxes. Selected section properties are automatically added in the boxes. Form of **Fig. 5.23** calculates the loading for construction and composite stage. For calculation of bending moment and section classification, use of the form of **Fig. 5.24** can be made. A form is also developed for checking the section for the ultimate limit state at the construction and composite stages for composite beam as shown in **Figs. 5.25** and **5.26.** By entering the diameter and height of shear connector, one can get the number of shear connectors required for the section as depicted in **Fig. 5.27.** For the serviceability check of deflection, use the form of **Fig. 5.28.** Check for stresses in material can be verified by using form of **Fig. 5.29.** Finally, **Fig. 5.30** shows the calculation for requirement for transverse reinforcement. Similarly, design program is developed for the design of composite beam with solid slab for the simply supported and continuous beam. For design purpose, the analysis of composite section is made using Limit State of Collapse Method. As IS: 11384-1985 code deals with the design and construction of only simply supported composite beams, for continuous beam design criteria are considered as per EC 4. Various forms developed in the program of design of composite beam with solid slab are shown in **Figs. 5.31 to 5.45** whereas forms developed in the program for the continuous beam are shown in **Figs 5.46 to 5.54.**

**FORMS DEVELOPED FOR THE DESIGN OF COMPOSITE BEAMS WITH DECK SLAB**

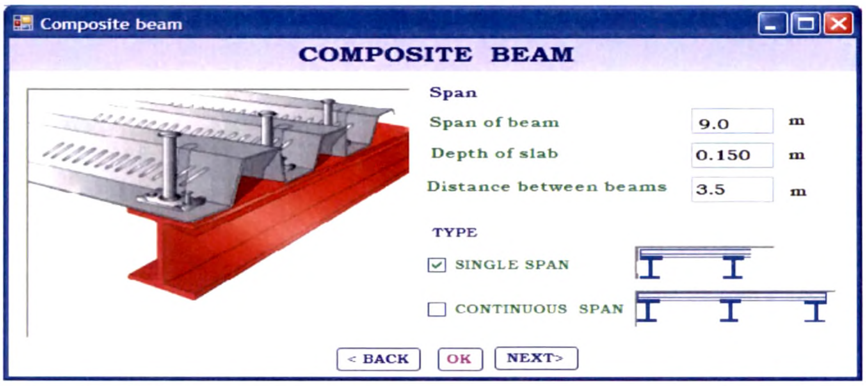


**Fig. 5.21 Form for Composite Beam Design**

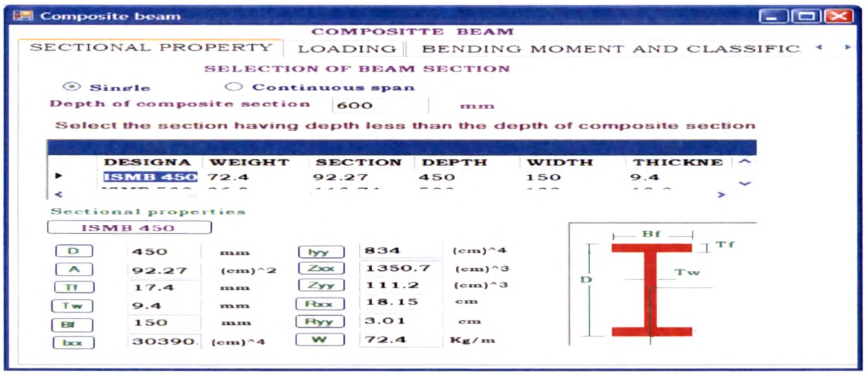


**Fig. 5.22 Form for Selection of Beam Section**

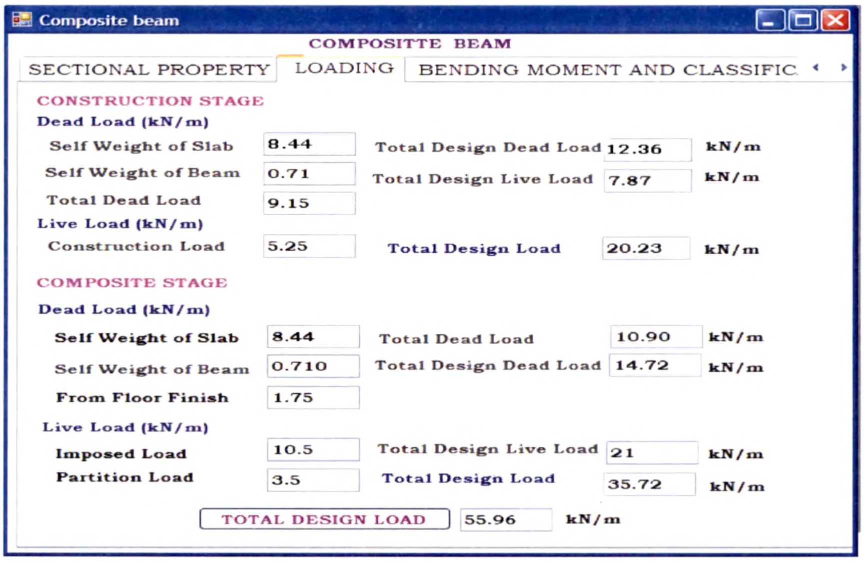


**Fig. 5.23 Form for Calculating Loading**

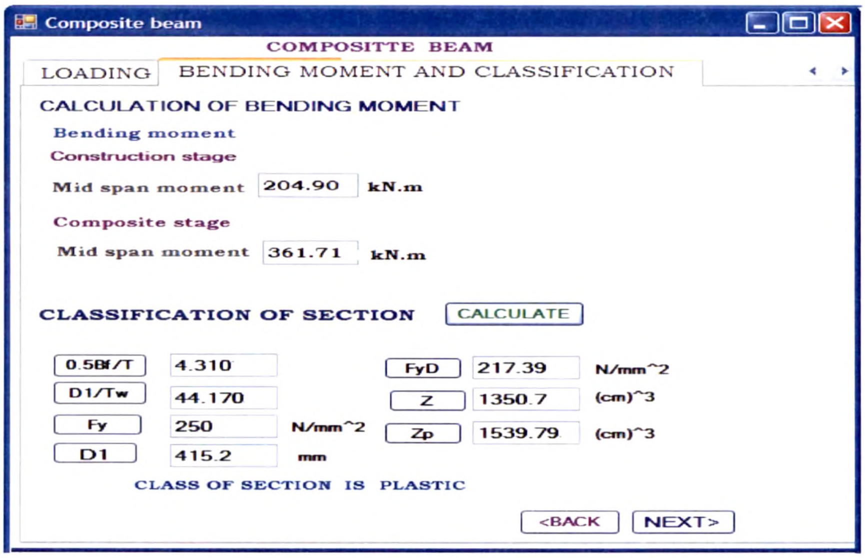


**Fig. 5.24 Form for Bending Moment and Classification of Section**

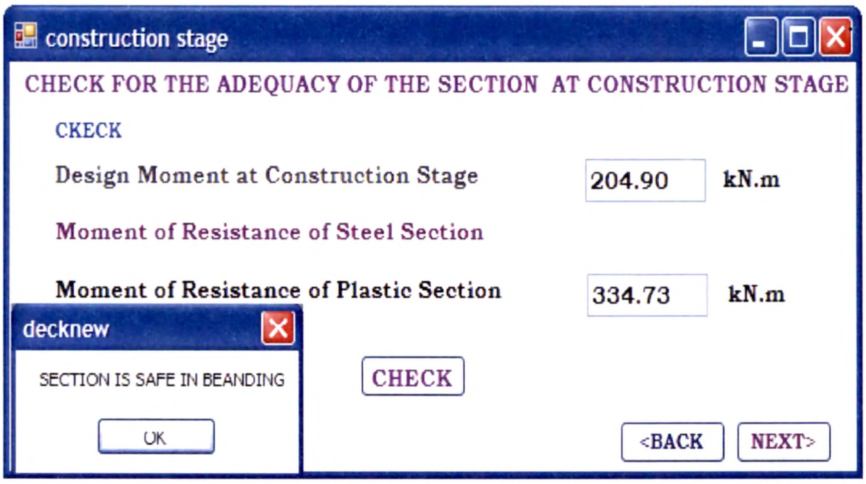


**Fig. 5.25 Form for Check at Construction stage**

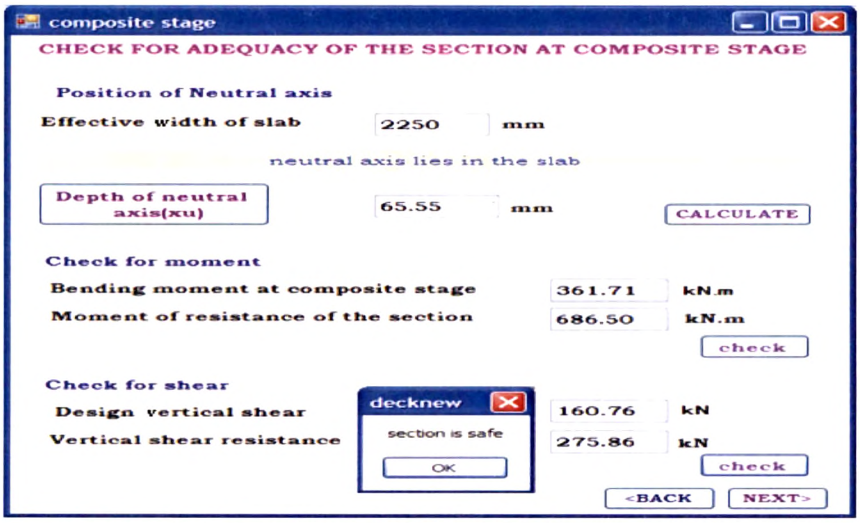


Fig. 5.26 Form for Check at Composite Stage

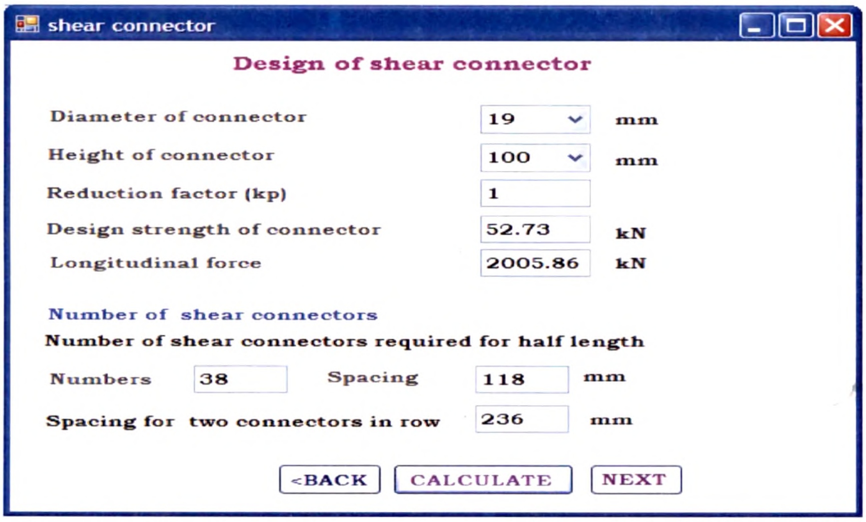


Fig. 5.27 Form for Shear Connectors

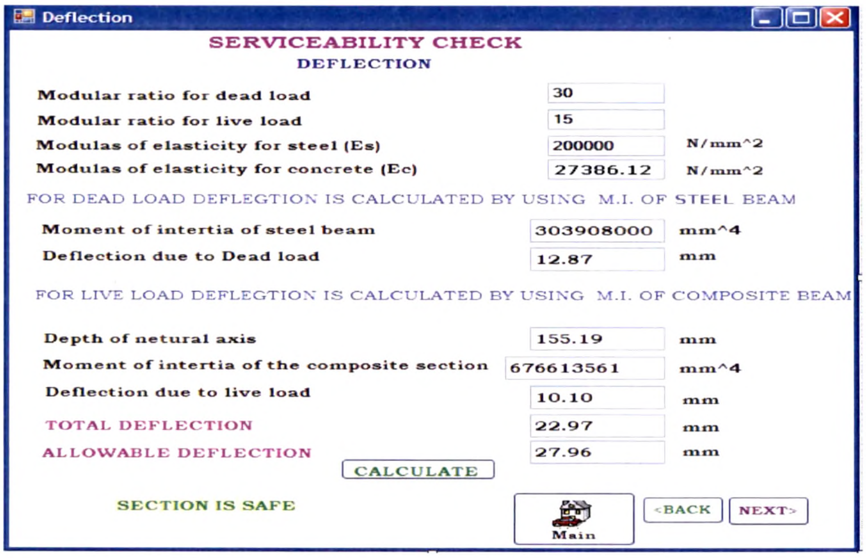


Fig. 5.28 Form for Check for Deflection


**Fig. 5.29 Form for Check for Stress**

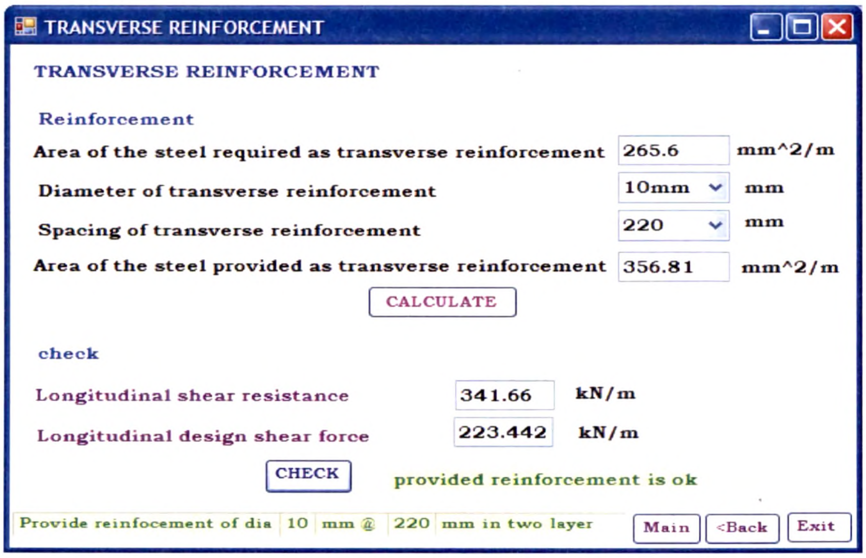


**Fig. 5.30 Form for Check for Transverse Reinforcement**

## FORMS DEVELOPED FOR THE DESIGN OF COMPOSITE BEAMS WITH SOLID SLAB


**Fig. 5.31 Form for Composite Beam**

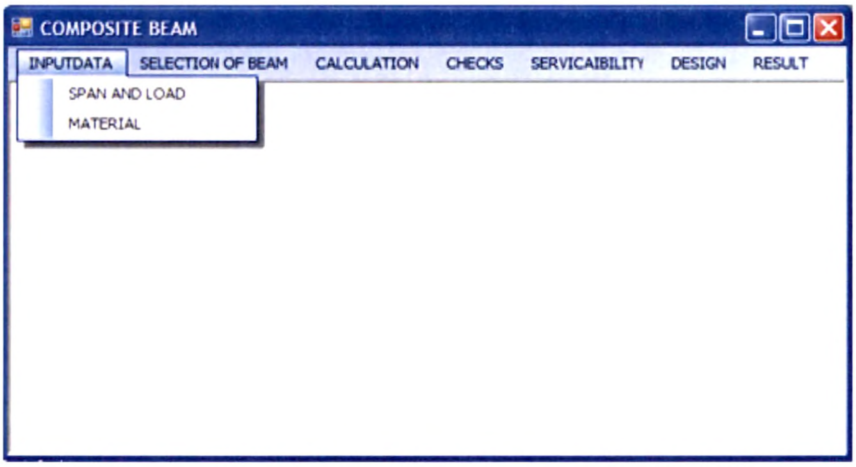


Fig. 5.32 Option Form for Composite Beam Design

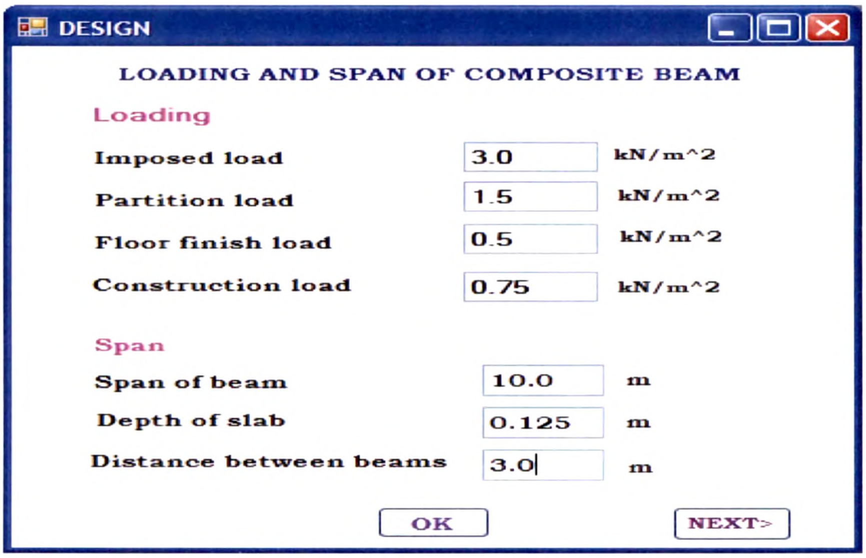


Fig. 5.33 Form for Entering Loading Data

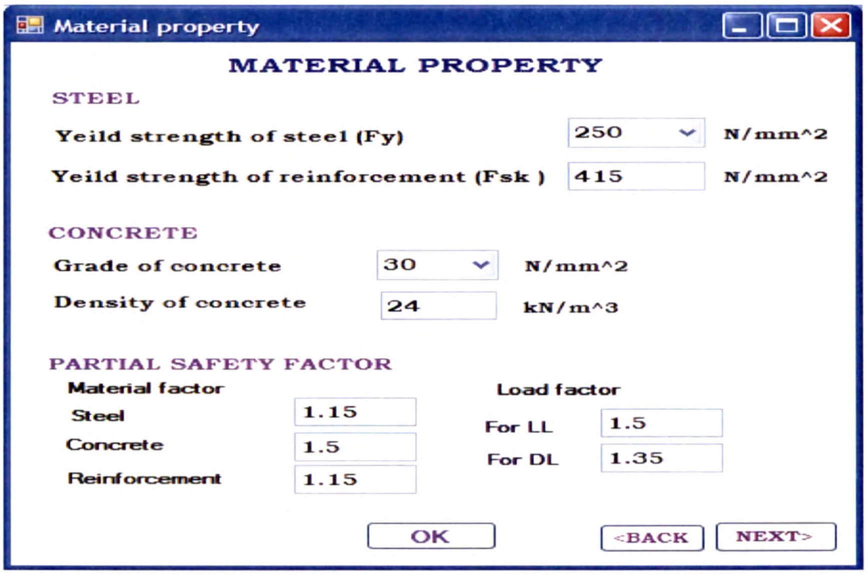


Fig. 5.34 Form for Entering the Material Property

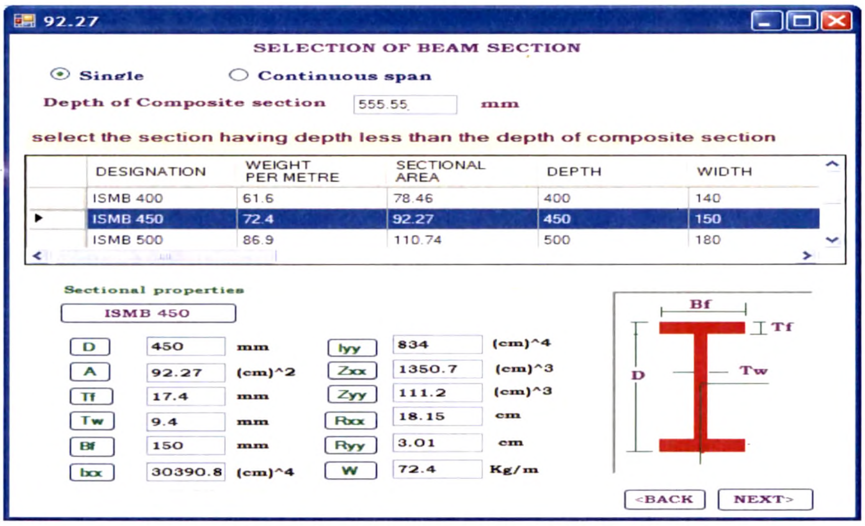


**Fig. 5.35 Form for Sectional Property**

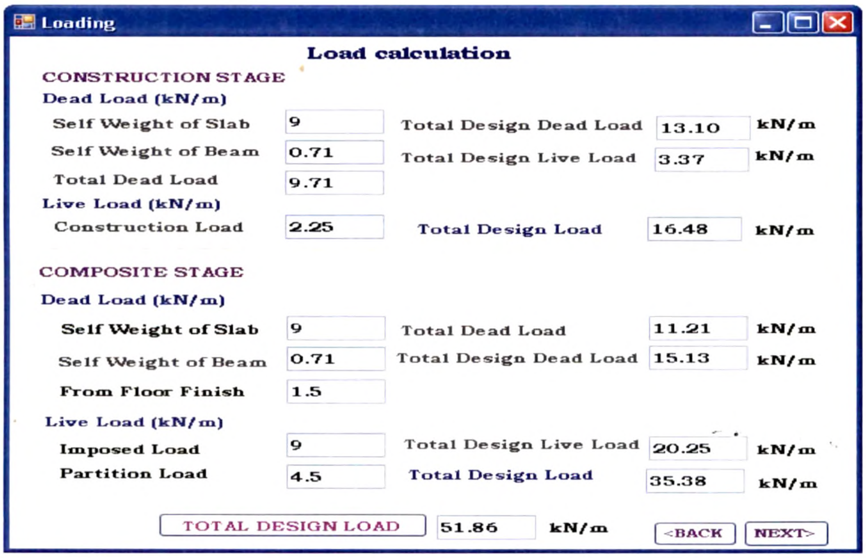


**Fig. 5.36 Form for Calculating the Factored Load**

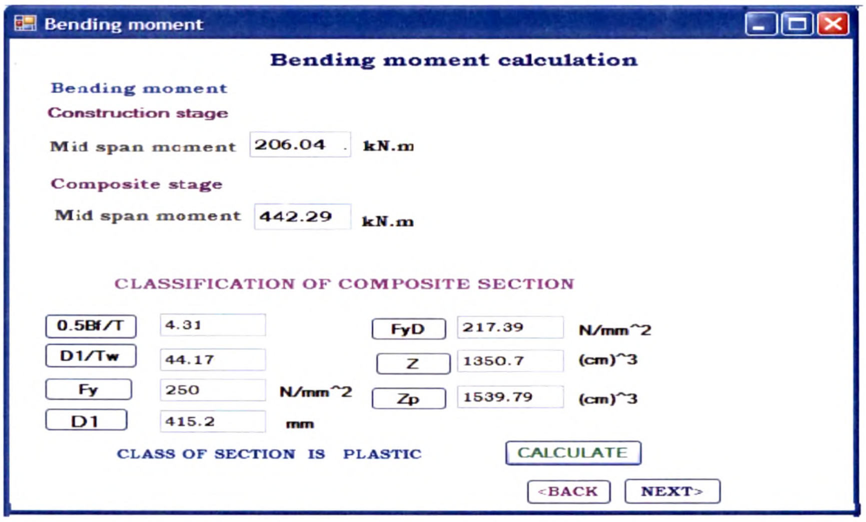


**Fig. 5.37 Form for Bending Moment and Classification of Section**

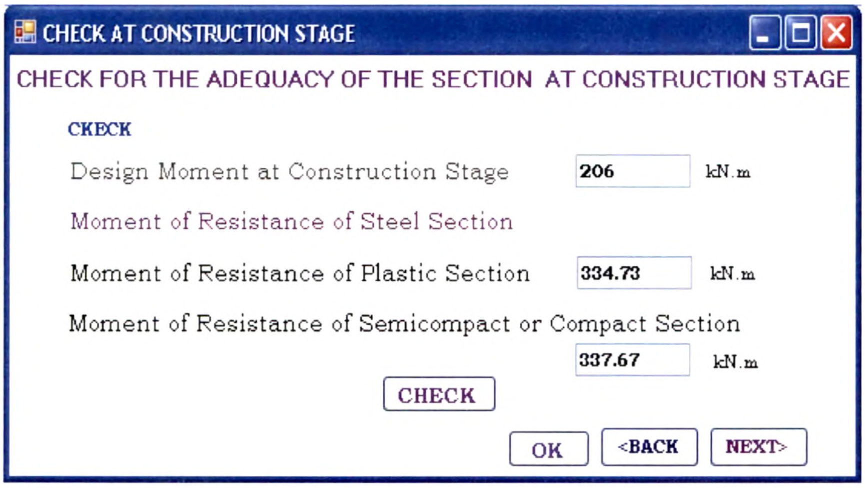


**Fig. 5.38 Form for Check at Construction Stage**

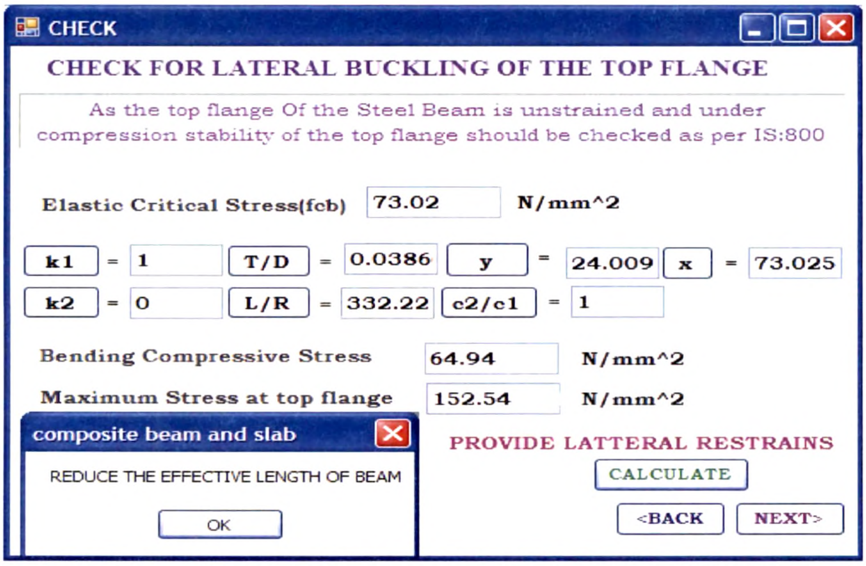


**Fig. 5.39 Form for Checking Lateral Buckling**

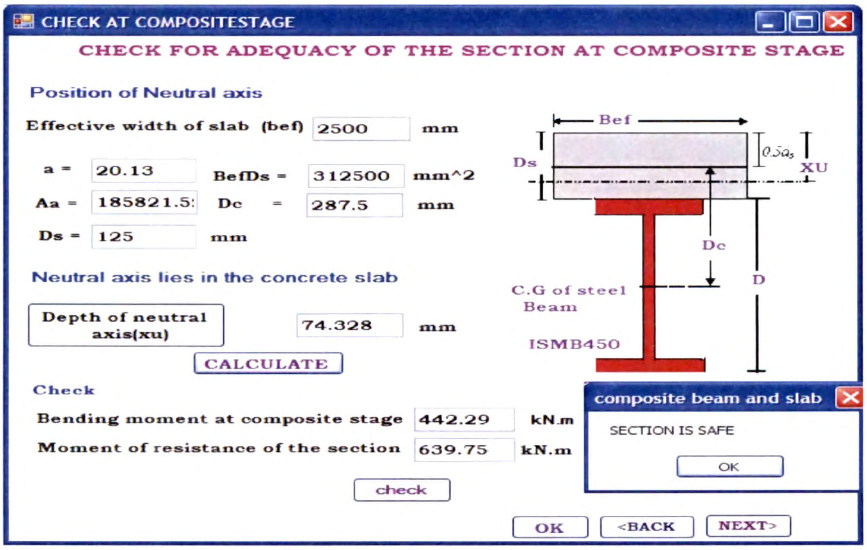


**Fig. 5.40 Form for Check at Composite stage**

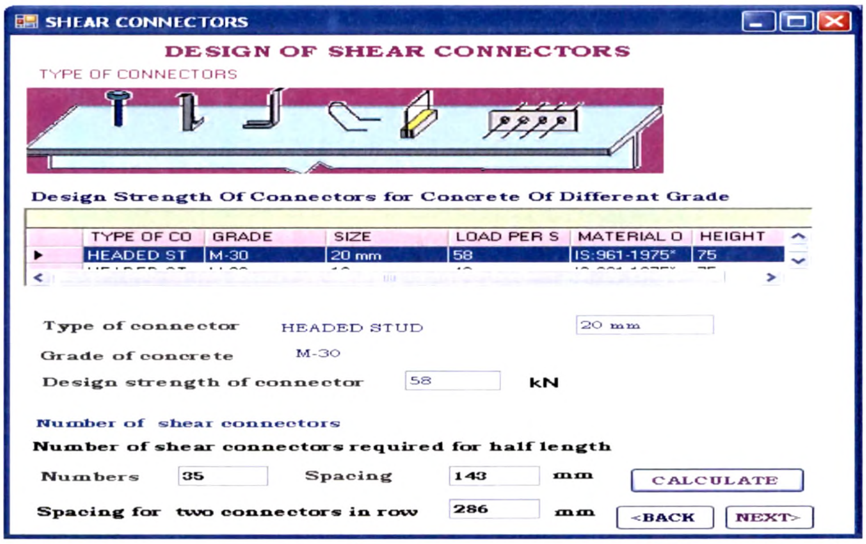


Fig. 5.41 Form for Shear Connectors

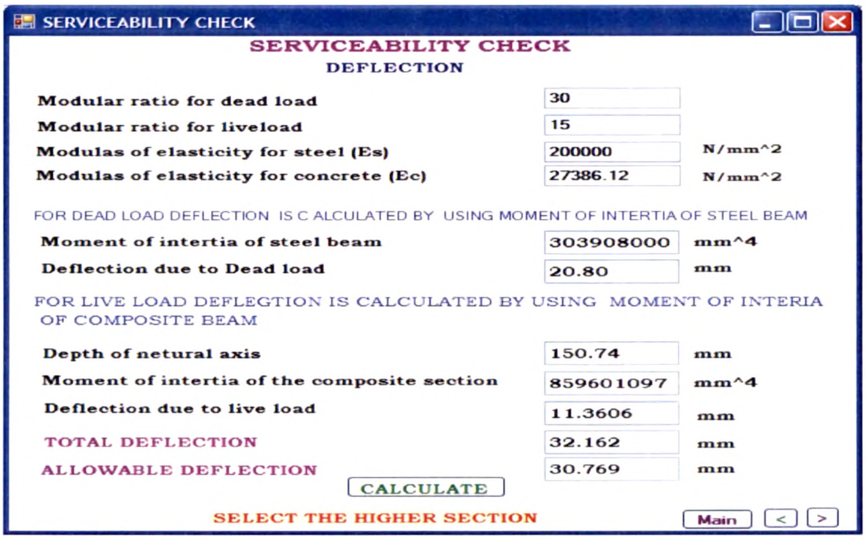


Fig. 5.42 Form for Check for Deflection

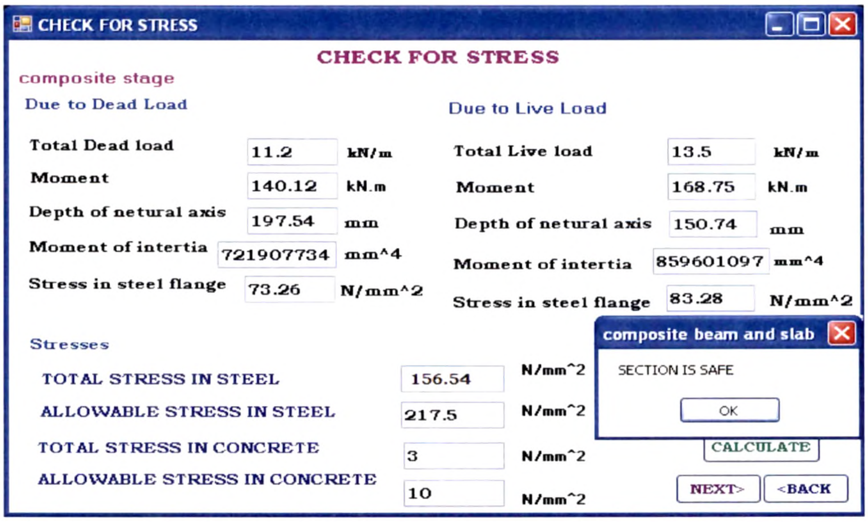


Fig. 5.43 Form for Check for Stress

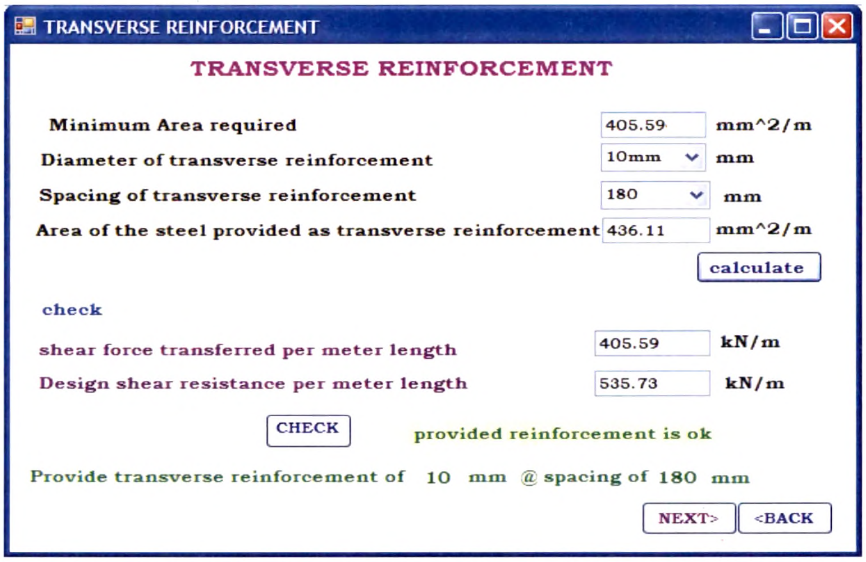


**Fig. 5.44 Form for Check for Transverse Reinforcement**

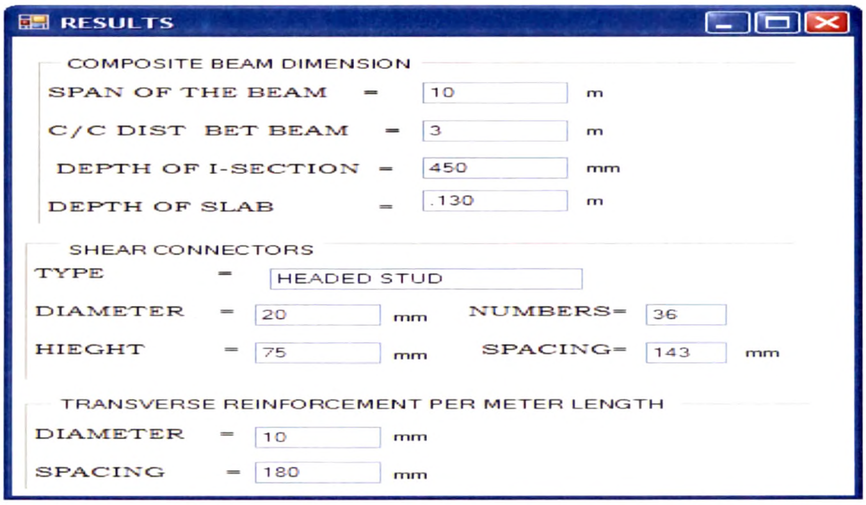


**Fig. 5.45 Form for Displaying Result**

## <u>STEPS FOR CONTINUOUS COMPOSITE BEAM</u>

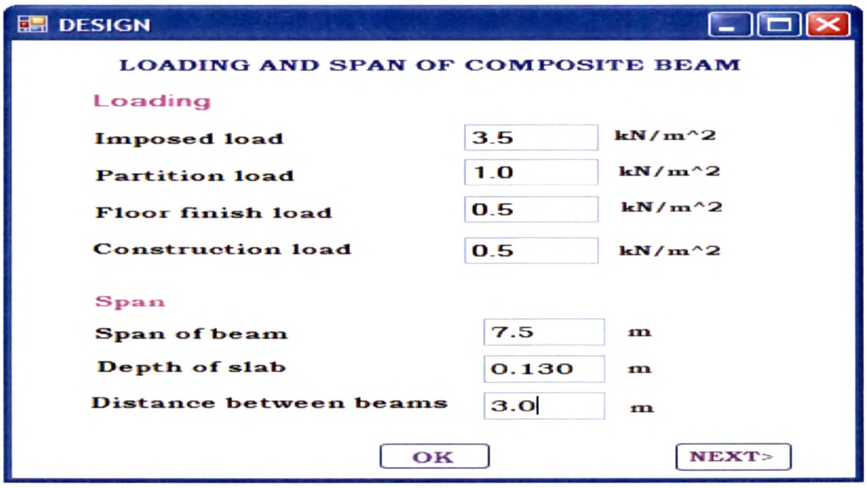


**Fig. 5.46 Form for Entering Loading Data**

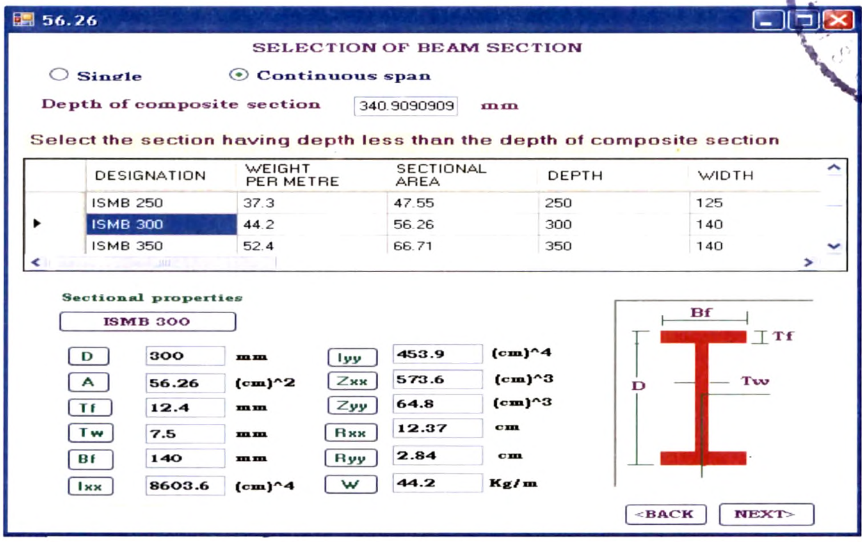


**Fig. 5.47 Form for Sectional Property**

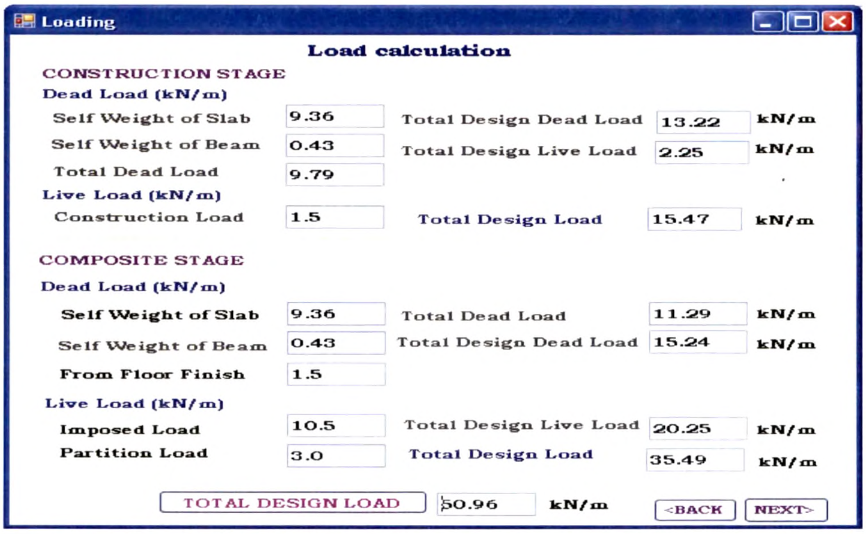


**Fig. 5.48 Form for Calculating the Factored Load**

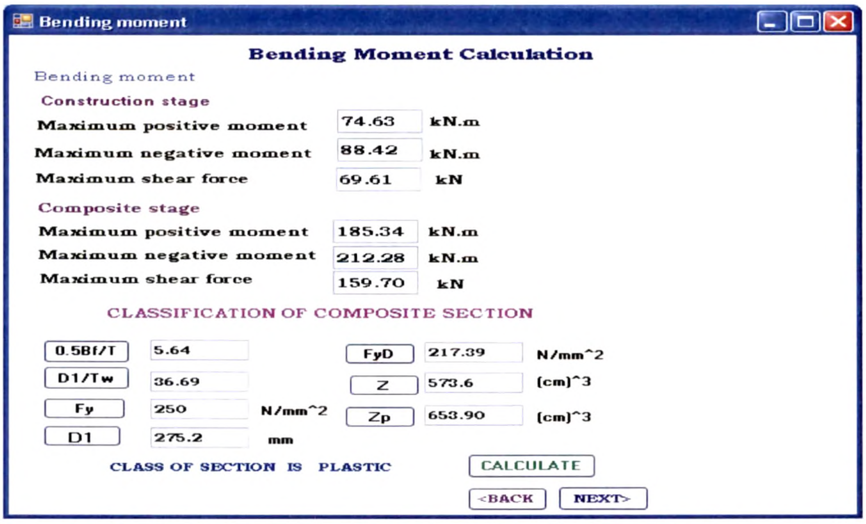


**Fig. 5.49 Form for Bending moment and Classification**

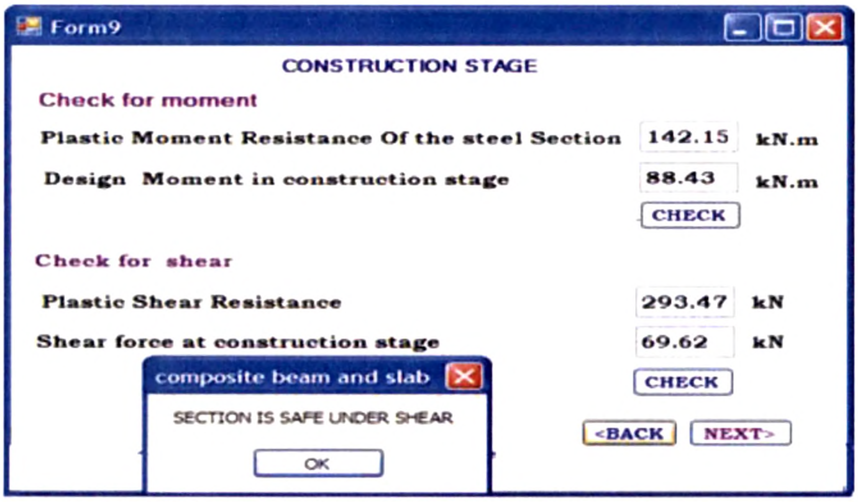


Fig. 5.50 Form for Check at Construction Stage

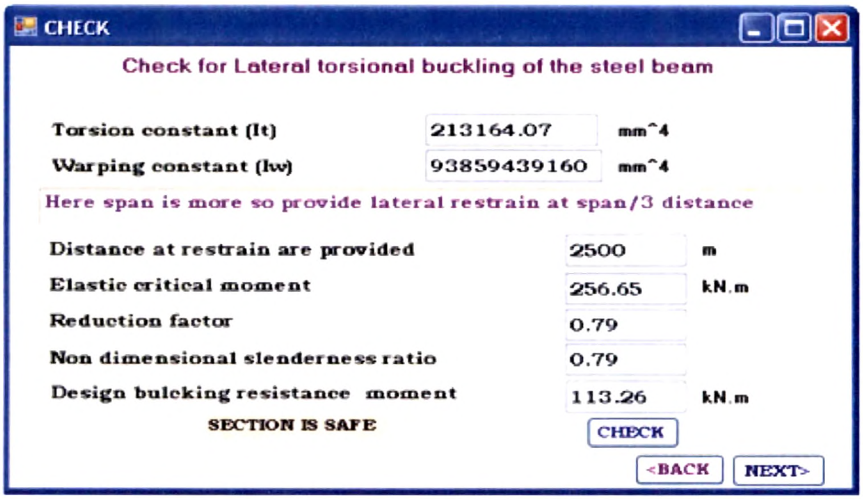


Fig. 5.51 Form for Check for Lateral Buckling

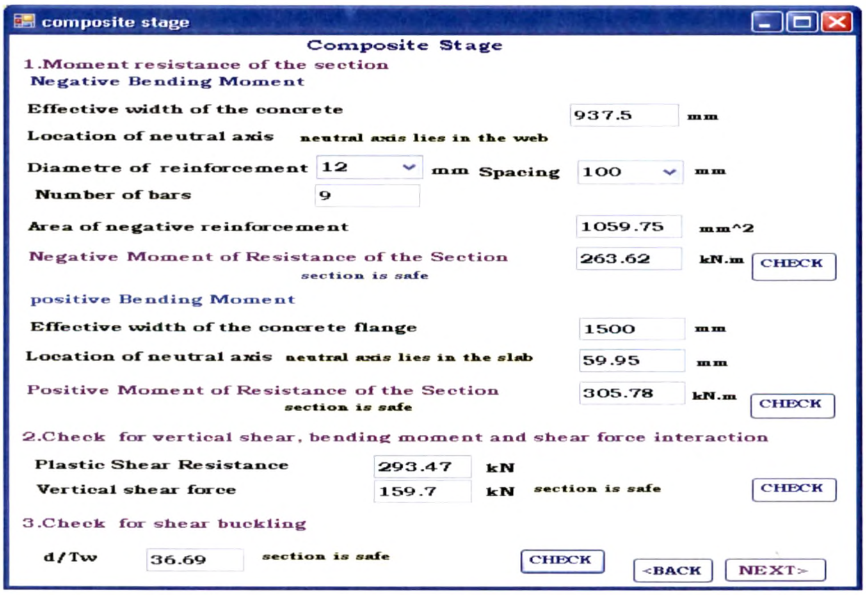


Fig. 5.52 Form for Check at Composite stage

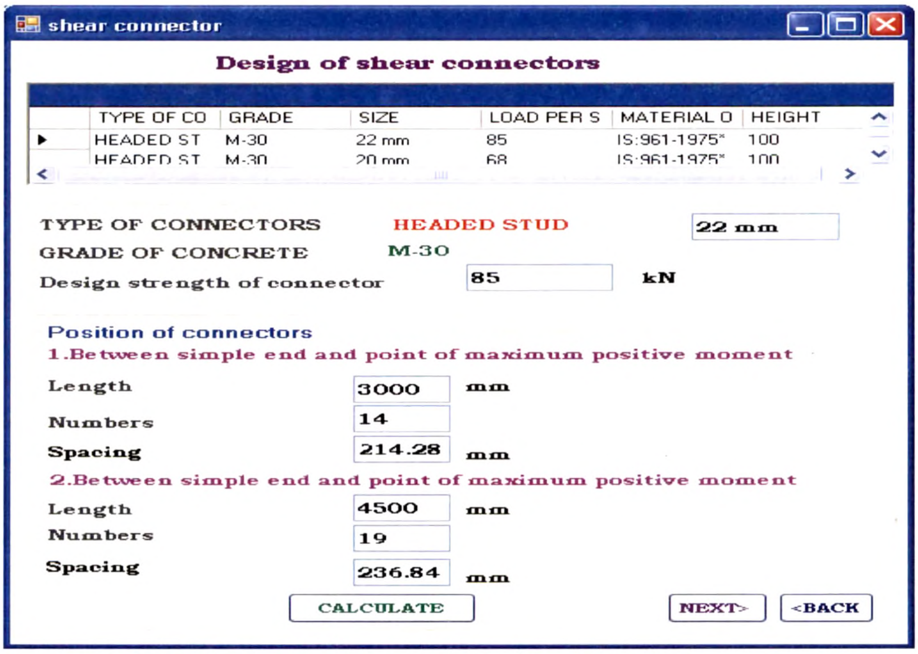


Fig. 5.53 Form for Shear Connectors

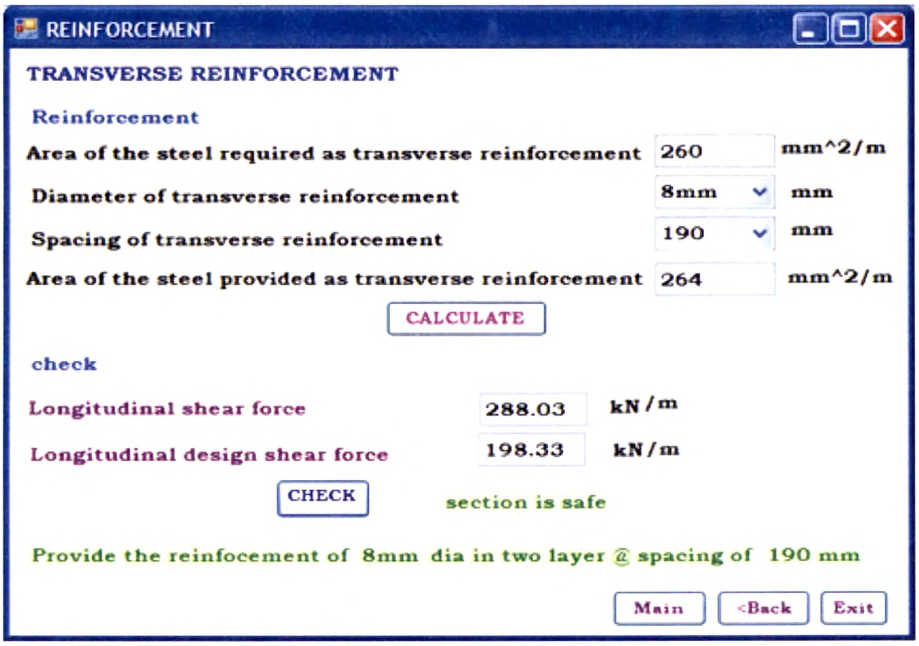


Fig. 5.54 Form for Check for Transverse Reinforcement