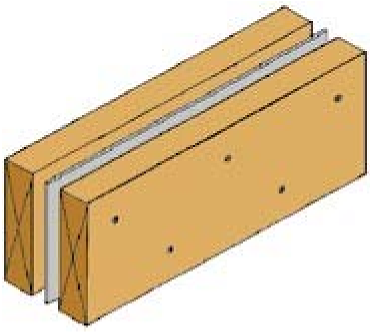
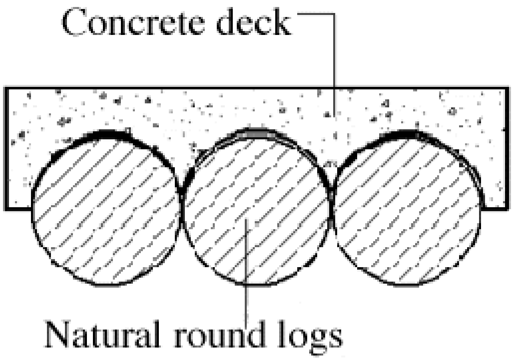
Composite Construction and Design

Introduction

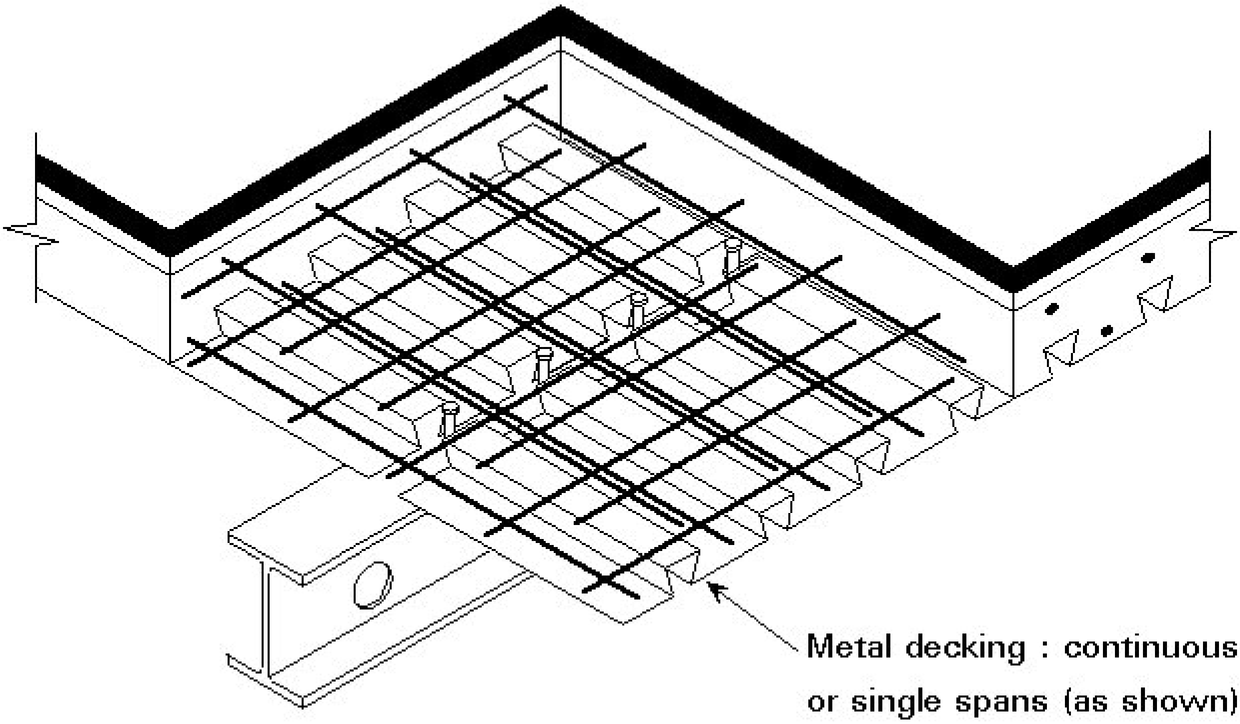
Composite construction refers to any members composed of more than 1 material. The parts of these composite members are rigidly connected such that no relative movement can occur. Examples are:



Timber and steel 'flitch' beams



Timber-reinforced concrete



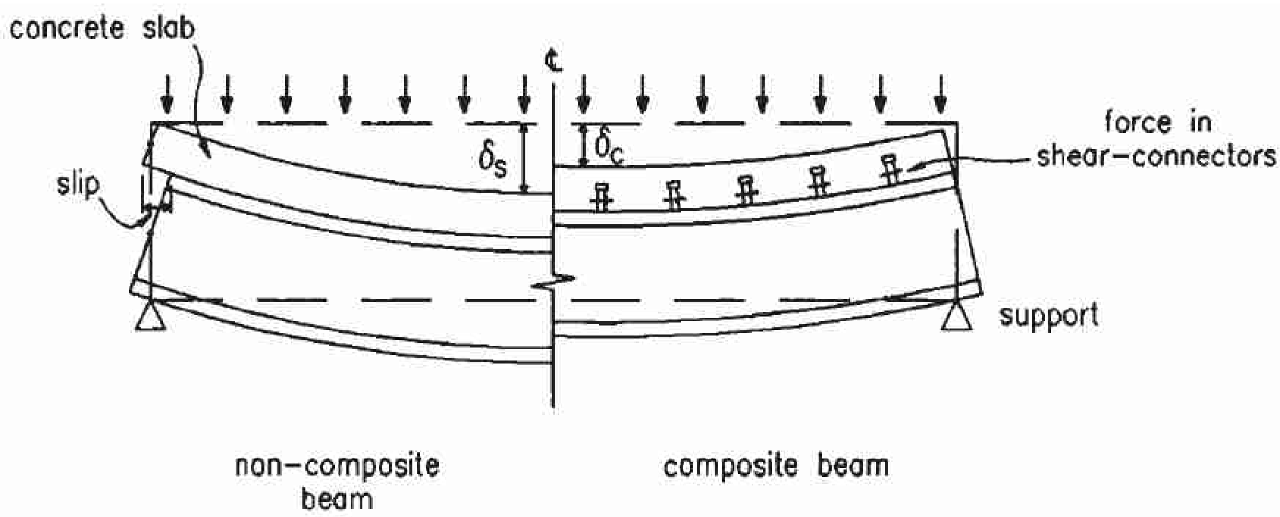
Typical steel and concrete composite construction

Composite construction aims to make each material perform the function it is best at, or to strengthen a given cross section of a weaker material.

Name and explain another form of composite construction.

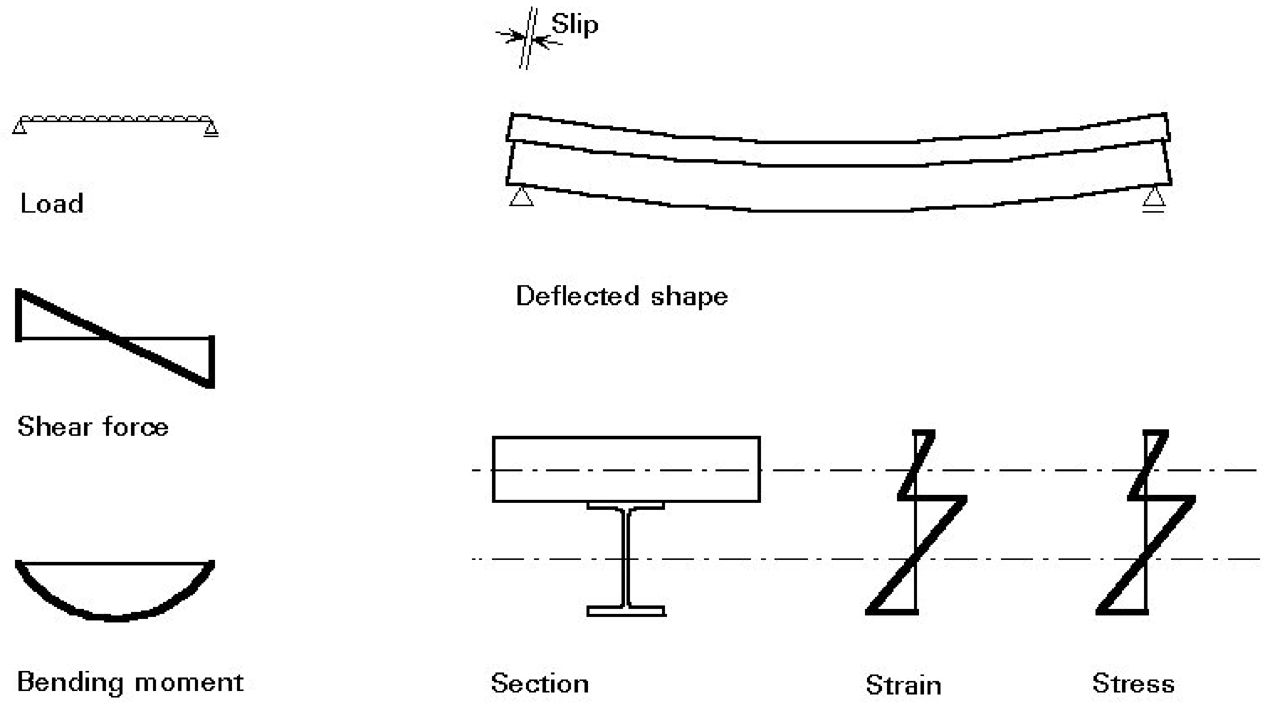
Behaviour of Composite Beams

In the following, we consider only the case of structural steel sections and reinforced concrete slabs. A comparison of behaviours is:



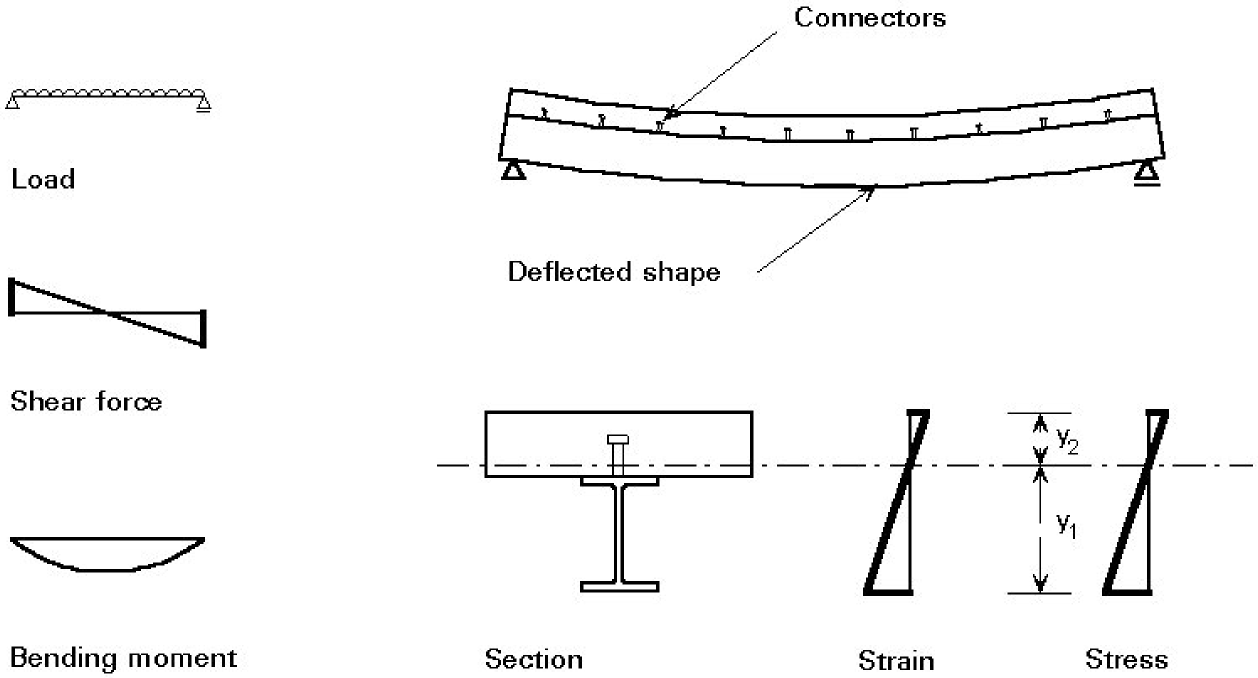
The non-composite beam deflects further, hence it is less stiff. Note that the *E*-value hasn't changed so it is the *I*-value that changes. In addition to the increase in stiffness there is also a large increase in moment capacity leading to reduced section sizes. The metal decking can also be used as permanent formwork, saving construction time.

Non-composite behaviour



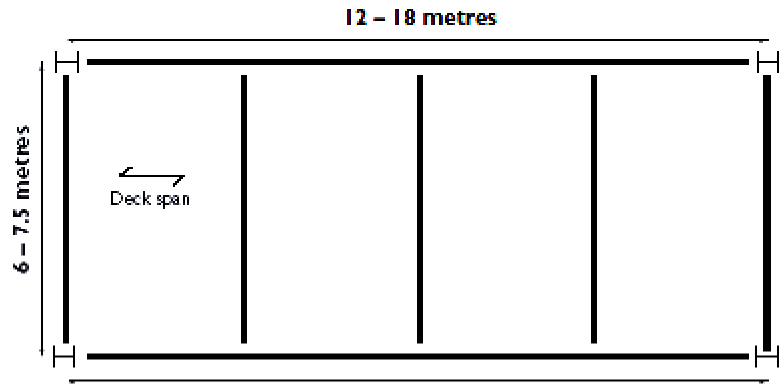
The concrete slab is not connected to the steel section and therefore behaves independently. As it is weak in longitudinal bending, it deforms to the curvature of the steel section and has its own neutral axis. The bottom surface of the concrete slab is free to slide over the top flange of the steel section and slip occurs. The bending resistance of the slab is often so small that it is ignored.

Composite Behaviour

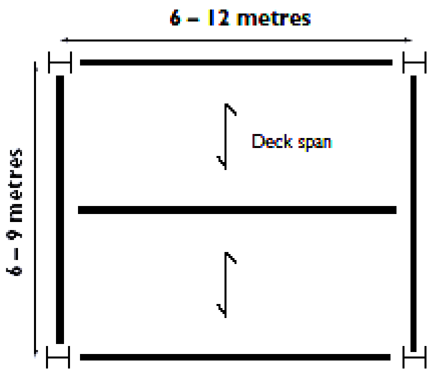


In this case, the concrete slab is connected to the steel section and both act together in carrying the load. Slip between the slab and steel section is now prevented and the connection resists a longitudinal shear force similar in distribution to the vertical shear force shown.

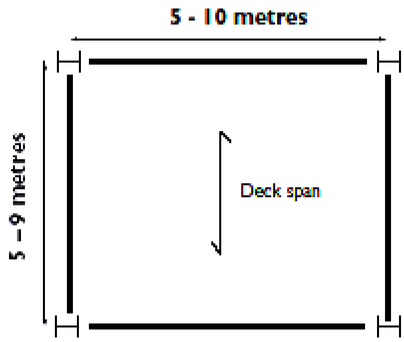
Composite Construction Layout



(A) Long span primary beams (with flexibility for service integration) using shallow decking.



(B) Primary and secondary beams of similar depth using shallow decking.

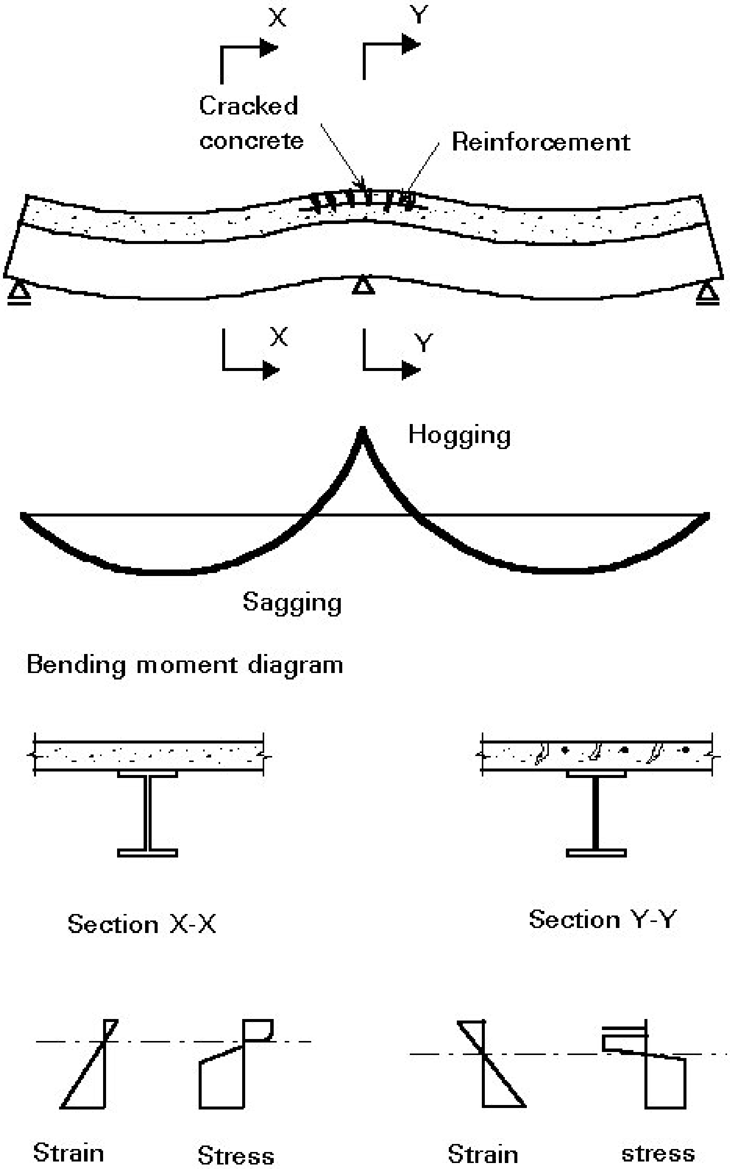


(C) Layout with deep decking using ASB beams.

Composite deck floors using shallow profiles are usually designed to span 2.5 to 4.5 m between supports. When the deck is propped during construction the spans are around 4 to 5 m.

Long span floors (12 to 18 m) are achieved by primary beams at 6 to 9 m centres. Shorter secondary beams support the slab (Diagram *A*). The type of grid shown in Diagram *B* offers services integration within the depth of the floor. Alternatively the secondary beams can be designed to span the longer distance so that the depths of the primary and secondary beams can be optimized.

The Asymmetric Beam (ASB) system from Corus allows a squarer panel (Diagram *C*) and is designed to compete with RC flat-slab construction.



Note that the beam layouts all describe simply-supported spans and this is usual. Continuous spans of composite beams can cause problems, though can be very useful nonetheless.

Over the support the concrete cracks (and these can be large); the steel must take the majority of the bending alone, and so a portion of the section is in compression. Slender sections are prone to local buckling in and any intervening column may need to be strengthened to absorb the compression across its web. Lateral-torsional buckling of the beam may also be a problem.

Propped Construction

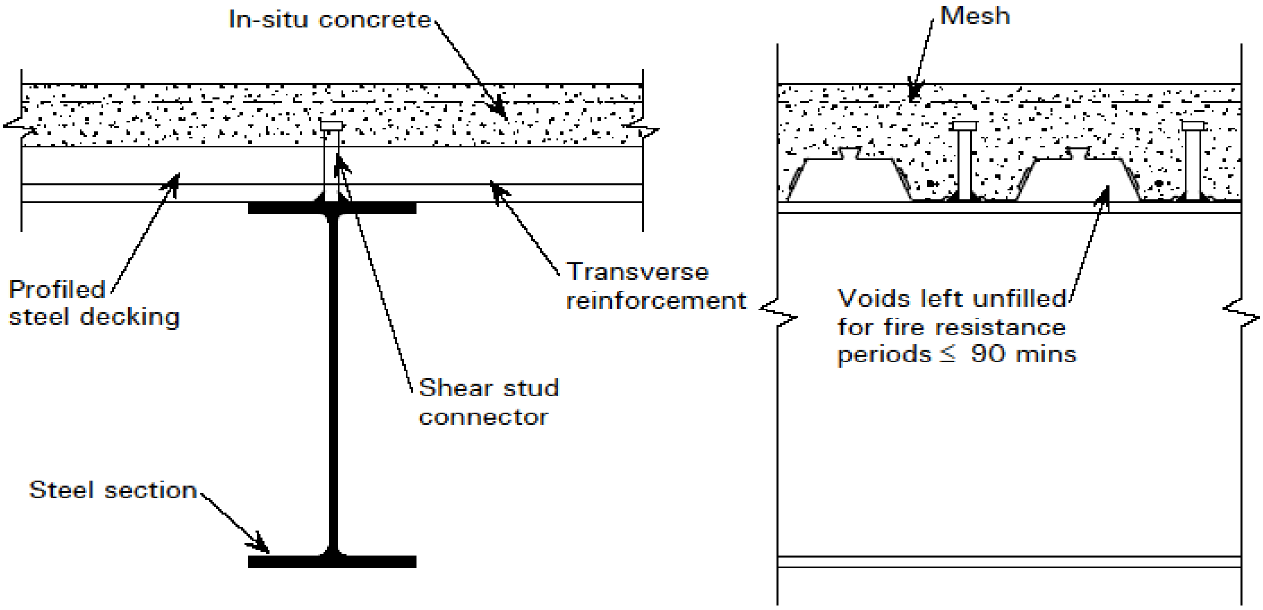
The steel beam is supported at mid- or quarter-span until the concrete slab has hardened sufficiently to allow composite action. Propping affects speed of construction but allows smaller steel sections.

Unpropped Construction

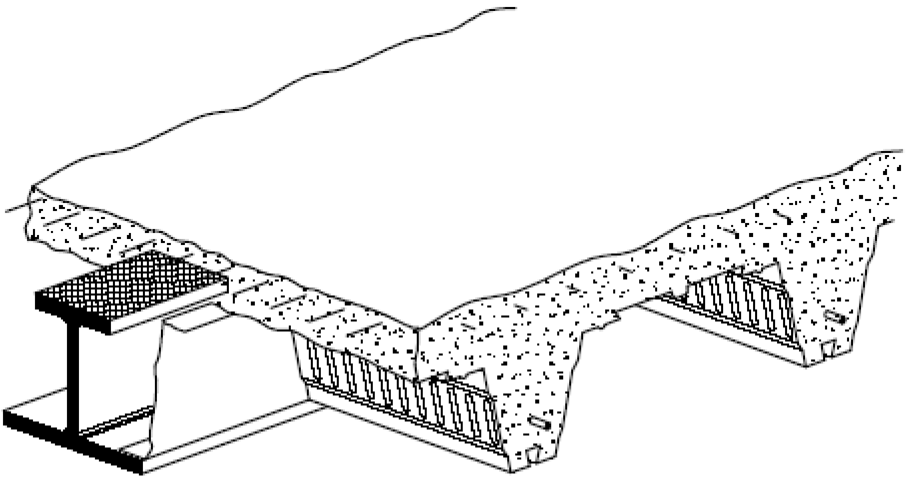
The steel beams must carry the weight of the wet concrete on its own. By the time construction loads can be applied to the slab, some composite behaviour can be used.

Elements of Composite Construction

The elements that make up composite construction are:



There are two main forms of deck: shallow and deep. The figure above illustrates a typical shallow deck (50–100 mm) and below is a deep deck (225 mm) supported on an ASB. The deep deck systems are proprietary; we will only consider the design of shallow deck systems, though the principles are the same.



The beams are ordinary structural steel sections (except for the ASB).

The shear studs are normally 19 mm diameter 100 mm high studs, though there are different sizes.

Design of Composite Beams

The design involves the following aspects:

1. Moment capacity:

Design the section such that the moment capacity is greater than that required.

2. Shear capacity

To ensure adequate capacity; this based on the steel section alone – as per usual structural steel design.

3. Shear connector capacity

To enable full composite action to be achieved; these must be designed to be adequate.

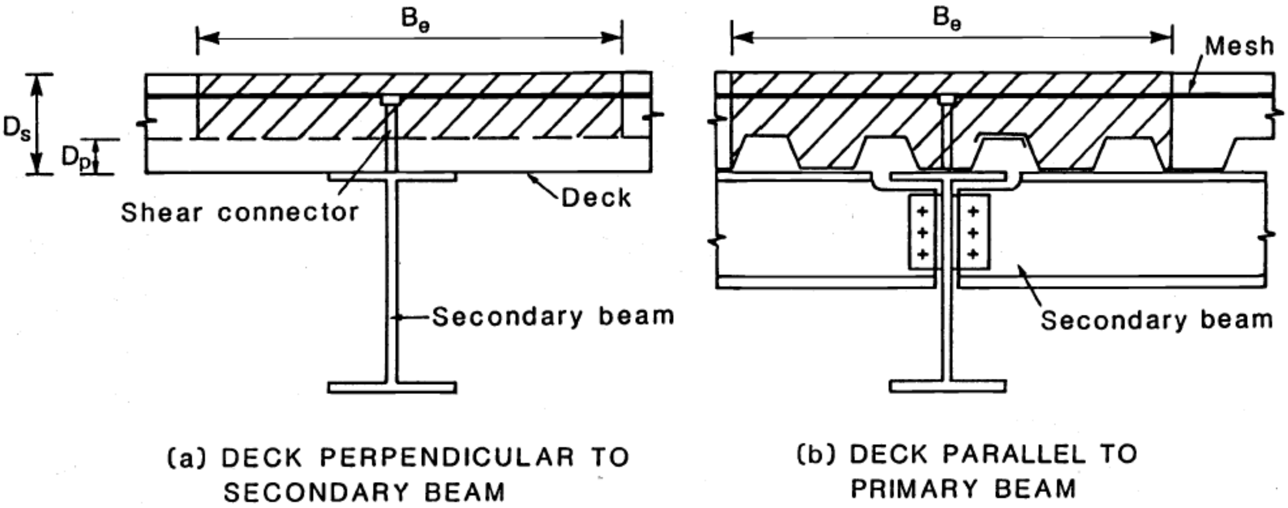
4. Longitudinal shear capacity

Check to prevent possible splitting of the concrete along the length of the beam.

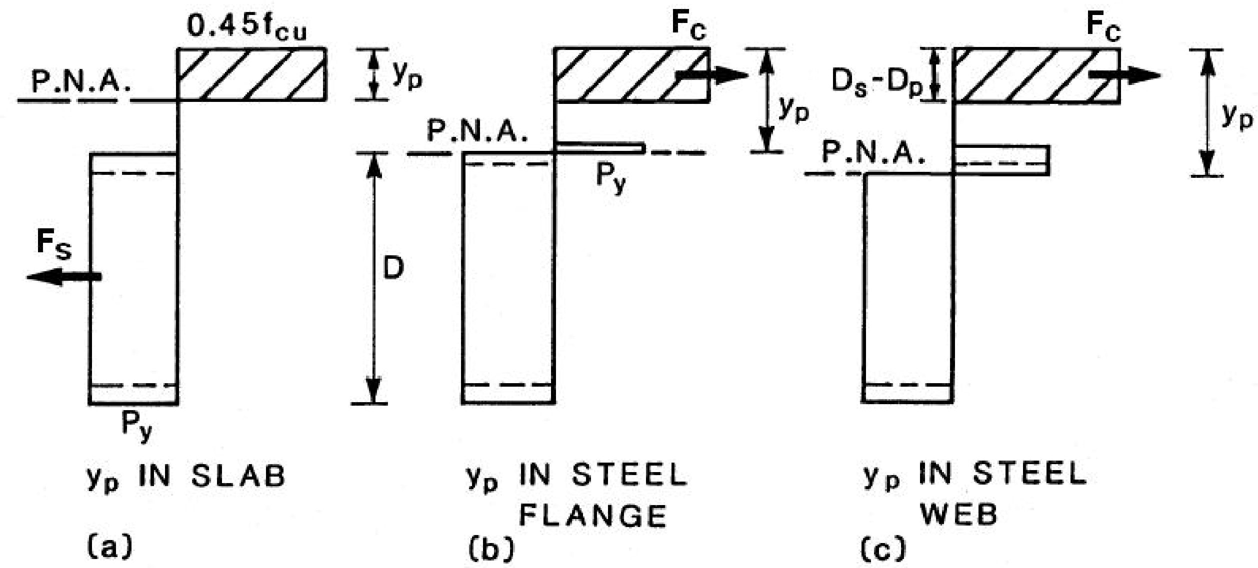
5. Serviceability checks:
    a. Deflection;
    b. Elastic behaviour, and;
    c. Vibration.

These checks are to ensure the safe and comfortable use of the beam in service. We check to ensure it does not cause cracking of ceilings and is not dynamically ‘lively’. Also, we verify that it is always elastic when subjected to service loads to avoid problems with plastic strain (i.e. permanent deflection) of the beam. We will not consider checks on vibration and will only outline the calculations for the elastic check.

Design of Composite Beams: Moment Capacity



Just as in ordinary steel and RC design, the composite moment capacity is derived from plastic theory. There are three cases to consider, based on the possible locations of the plastic neutral axis (PNA), shown below.



When calculating the PNA location, we assume a stress of p_y in the steel and $0.45f_{cu}$ in the concrete. The tensile capacity of the beam of area A is:

$$F_s = p_y A$$

The compression capacity of the slab depends on the orientation of the decking (D_p), and is:

$$F_c = 0.45 f_{cu} \left(D_s - D_p \right) B_e$$

where B_e is the effective breadth of the slab. We also define the axial capacities of the flange and web as:

$$F_f = BTp_y \qquad F_w = F_s - 2F_f \ \text{ or } \ F_w = Dtp_y$$

Using the notation given, where the depth of the PNA is y_p, we have three capacities:

- <u>Case (a):</u> PNA is in the slab; occurs when $F_c > F_s$:

$$M_c = F_s\left[\frac{D}{2} + D_s - \frac{F_s}{F_c}\left(\frac{D_s - D_p}{2}\right)\right]$$

- <u>Case (b):</u> PNA is in the steel flange; occurs when $F_s > F_c$

$$M_c = F_s\frac{D}{2} + F_c\left(\frac{D_s - D_p}{2}\right) - \left\{\frac{(F_s - F_c)^2}{F_f}\cdot\frac{T}{4}\right\}$$

(the term in the braces is small and may be safely ignored).

- <u>Case (c):</u> PNA is in the steel web; occurs when $F_w > F_c$

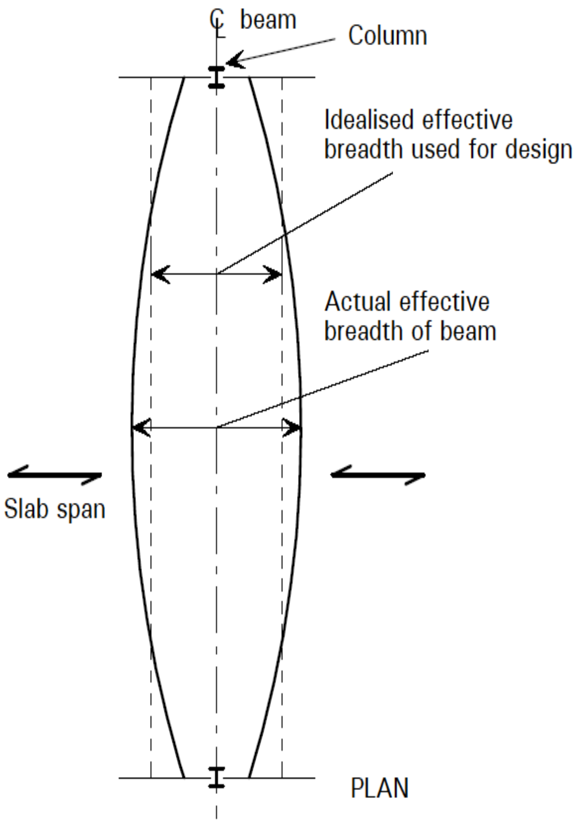
$$M_c = M_s + F_c\left(\frac{D_s + D_p + D}{2}\right) - \frac{F_c^2}{F_w}\cdot\frac{D}{4}$$

where $M_s = p_y S_x$ is the moment capacity of the steel section alone.

The effective breadth B_e is taken as:

$$B \le B_e = 0.25L \le S$$

where B is the width of the steel section and S is the centre-to-centre spacing of the composite beams (2.5 to 4.5 m) and L is the (simply-supported) span of the beam.

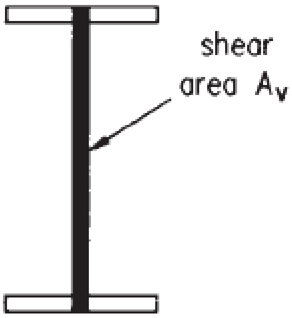


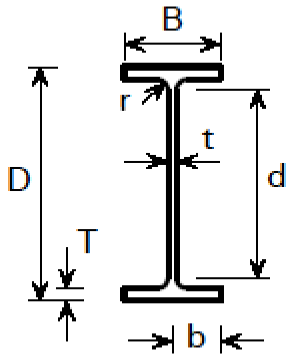
Don't Panic!

Case (a) is frequent; (b) less so, but (c) is very rare. Therefore, for usual design, only F_c and F_s are required (ignoring the term in the braces). Note that if $F_s > F_c$, check that $F_w \not> F_c$ to ensure that you are using Case (b).

Design of Composite Beams: Shear Capacity

The shear capacity is based on the capacity of the steel section only.

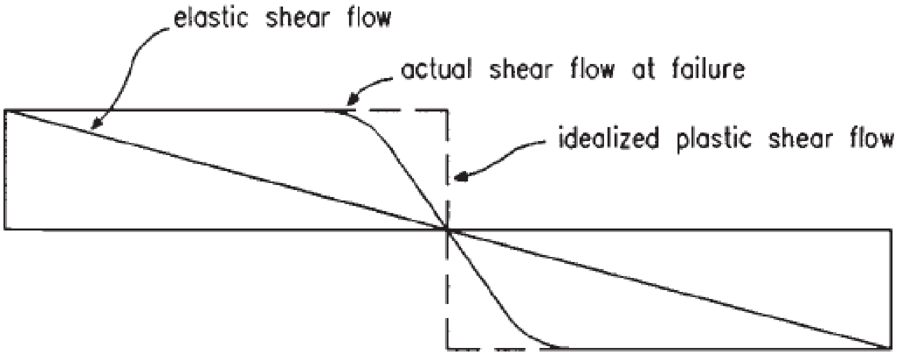


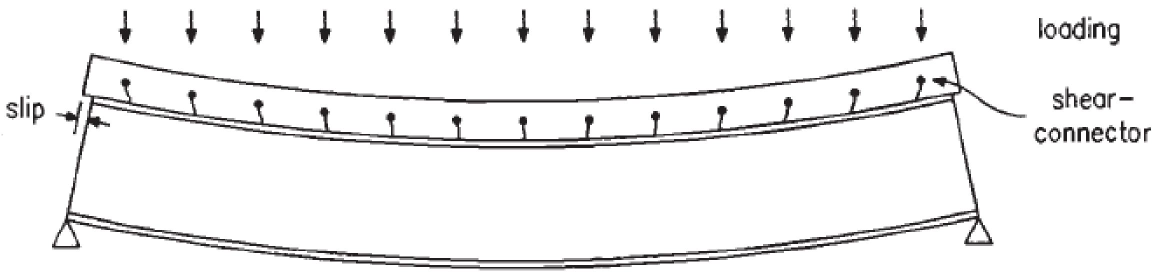


The capacity is: $P_v = 0.6 p_y A_v$ where $A_v = tD$.

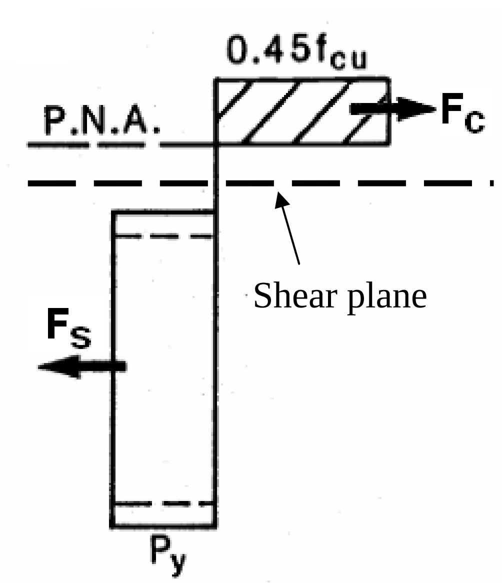
Design of Composite Beams: Shear Connector Capacity

The shear connectors used in ordinary composite construction are dowel-type studs. Other forms used to be used, but headed-studs are now standard. They allow easy construction as they can be shot fixed or welded through the deck onto the beam, after the deck has been laid. In addition to the shear strength, the headed studs prevent the vertical separation, or uplift, of the concrete from the steel.





Note that although some slip does occur (which reduces the capacity slightly) we usually design for full shear connection, though partial interaction is also possible.



The shear force to be transmitted is the smaller of F_c and F_s as calculated earlier. We only need to transfer shear in the zones between zero and maximum moment. Therefore the number of shear connectors required <u>in each half of the span</u> (see diagram above) is:

$$N_p = \frac{\min\left(F_c, F_s\right)}{Q_p}$$

Where Q_p is the force in each shear connector, and

$$Q_p \nless 0.8Q_k$$

where Q_k is the (empirical) characteristic strength of the shear studs, and is given in the following table.

Shear Stud Strength, Q_k (kN)

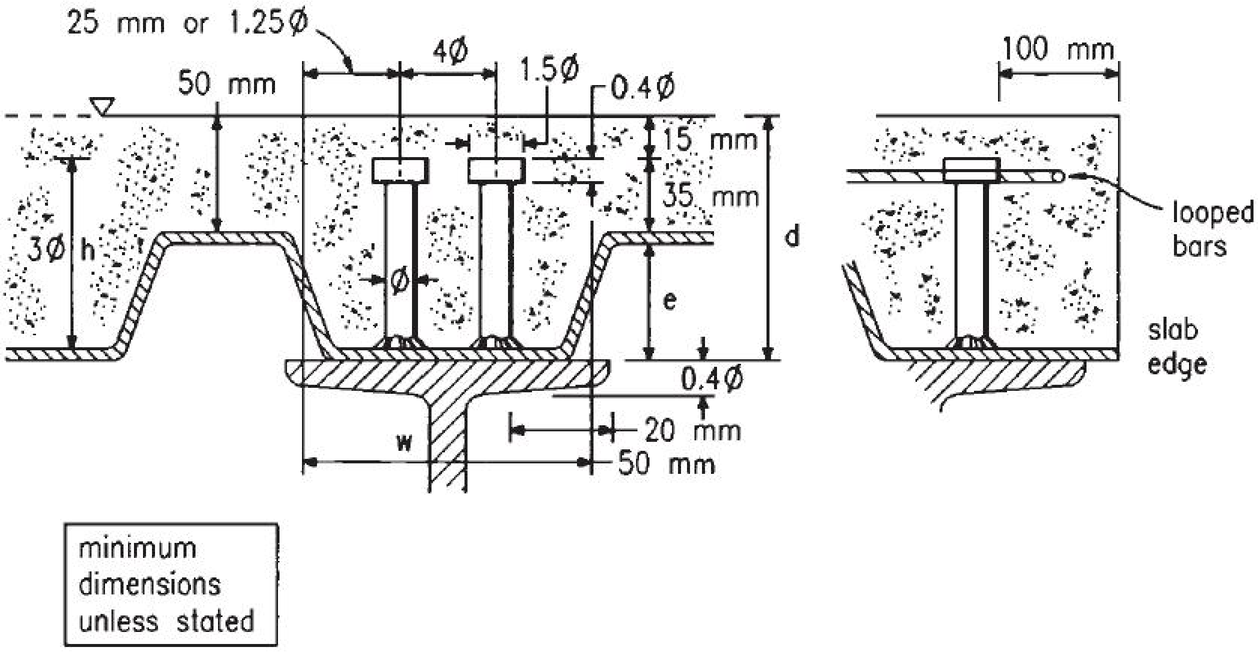
Stud Diameter (mm)	Stud Height (mm)	Concrete strength, f_{cu} (N/mm^2)		
		30	35	40
22	100	126	132	139
19	100	100	104	109
16	75	74	78	82

From these figures, the spacing along the full length is:

$$s = \frac{L}{2N_p - 1}$$

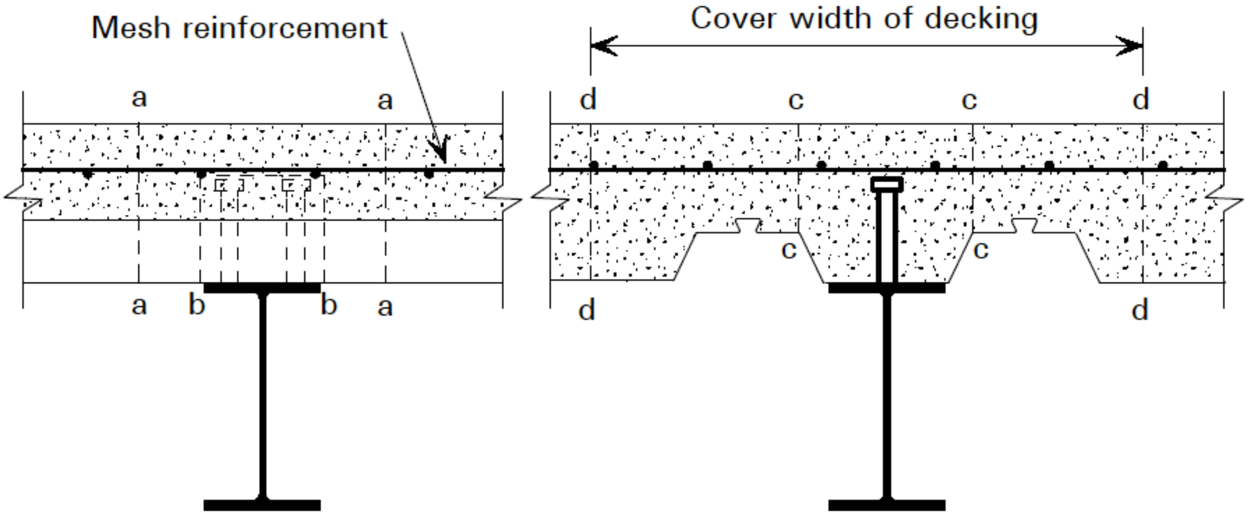
Note:

- The stud should project 25 mm into the compression zone;
- Spacing limits are: $\ngtr 4D_s; \ngtr 600$ mm; longitudinally and as shown in the figure:



Design of Composite Beams: Longitudinal Shear Capacity

The force transmitted by the shear studs can potentially split the concrete along the weakest failure plane. Some such planes are shown:



Perpendicular Deck *Parallel Deck*

Failure planes *a-a*, *b-b* and *c-c* are usually critical; *d-d* has no strength contribution from the decking itself (which is possible, though we will always ignore this safely). Any reinforcement in the slab that crosses these planes is taken to contribute. The force per unit length to be resisted is:

$$v = \frac{N_T Q_p}{s}$$

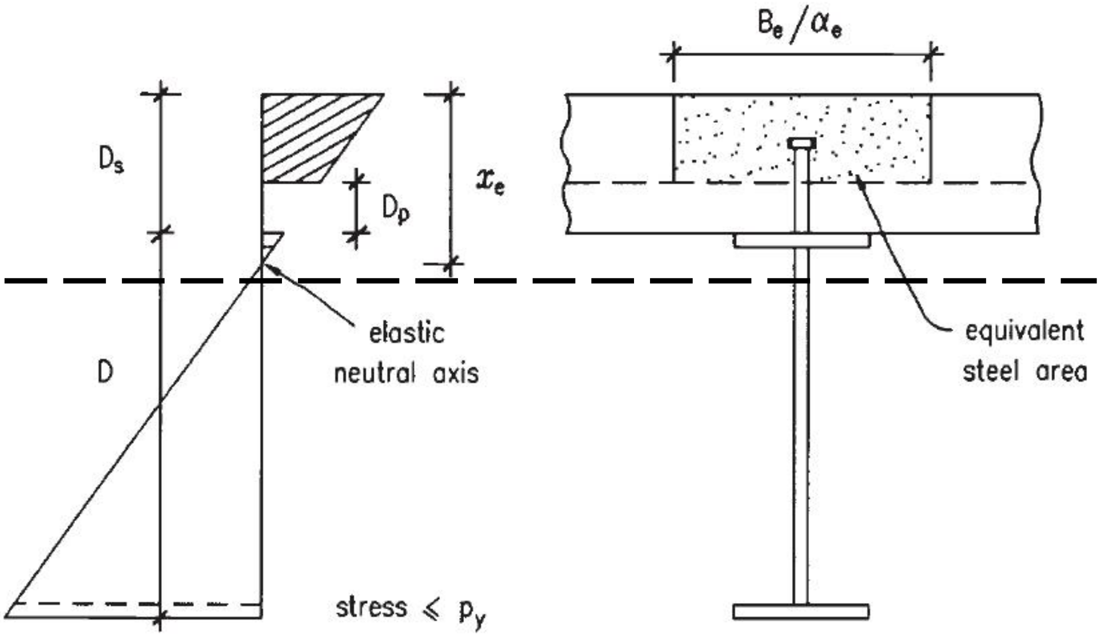
where s is the shear-stud spacing and N_T is the number of studs across the width of the beam (1 or 2). This must be less than the capacity which is:

$$\begin{aligned} v_r &= \left(0.03 f_{cu}\right) L_s + 0.7 A_{sv} f_y \\ &< \left(0.8\sqrt{f_{cu}}\right) L_s \end{aligned}$$

where A_{sv} is the area of reinforcement, per unit length, crossing the failure plane and L_s is the length of the failure plane:

- Plane *a-a* and *c-c*: $L_s = 2D_p$ and $A_{sv} = 2A_s$
- Plane *b-b*: $L_s = 2h + d + s_t$ where s_t is the transverse spacing of the 2 studs and $s_t = 0$ for only 1 stud. Also, $A_{sv} = 2\left(A_s + A_{sc}\right)$

Design of Composite Beams: Serviceability checks



(a) Elastic stress distribution (b) Transformed section

For these checks we define the following:

- *the depth to the elastic neutral axis:*

$$x_e = \frac{\frac{D_s - D_p}{2} + \alpha_e r\left(\frac{D}{2} + D_s\right)}{\left(1 + \alpha_e r\right)}$$

$$r = \frac{A}{B_e\left(D_s - D_p\right)}$$

- *the second moment of area of the un-cracked composite section:*

$$I_g = I_x + \frac{A\left(D + D_s + D_p\right)^2}{4\left(1 + \alpha_e r\right)} + \frac{B_e\left(D_s - D_p\right)^3}{12\alpha_e}$$

where α_e is the effective modular ratio which can be taken as 10 for most purposes; I_x is the second moment of area of the steel section alone; and the other symbols have their previous meanings.

- *the section modulus for the steel and concrete:*

$$Z_s = \frac{I_g}{D + D_s - x_e} \qquad Z_c = \frac{\alpha_e I_g}{x_e}$$

The composite stiffness can be 3–5 times, and the section modulus 1.5–2.5 times that of the steel section alone.

a. Deflection

Deflection is checked similarly to ordinary steel design, the allowable deflection is:

$$\delta_{allow} = \frac{L}{360}$$

Assuming a uniformly distributed load, the deflection is:

$$\delta = \frac{5w_q L^4}{384EI_g}$$

where w_q is the imposed UDL only and E = 205 kN/mm^2.

b. Elastic behaviour

We check that the stresses in the steel or concrete remain elastic under the service loads, that is, under $w_{ser} = w_g + w_q$:

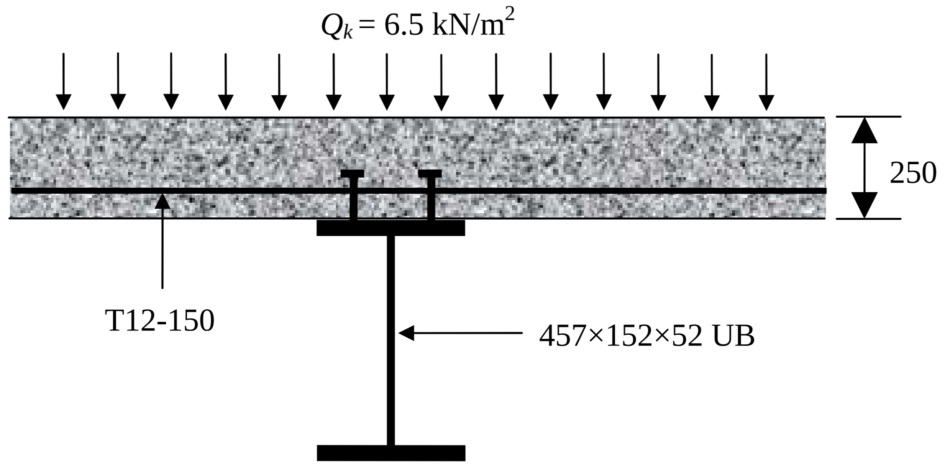
$$\sigma_{s,ser} = \frac{M_{ser}}{Z_s} < p_y \qquad\qquad \sigma_{c,ser} = \frac{M_{ser}}{Z_c} < 0.45 f_{cu}$$

where $M_{ser} = \frac{w_{ser} L^2}{8}$.

c. Vibration

We will not check this.

Design Example



Check that the proposed scheme shown is adequate.

Design Data

Beam:

- 457×152×52 UB Grade 43A (p_y = 275 N/mm^2)
- Span: 7 m simply supported; beams at 6 m centre to centre.
- Section properties:

  A = 66.5 cm^2; D = 449.8 mm; t_w = 7.6 mm;

  I_x = 21345×10^4 mm^4; t_f = 10.9 mm; b = 152.4 mm

Slab:

- D_s = 250 mm
- Grade 30N concrete (f_{cu} = 30 N/mm^2)
- Reinforcement T12-150: A_{sv} = 754 mm^2/m = 0.754 mm^2/mm

Solution

<u>Loading:</u>

The dead load of slab is:

$$G_k = 0.25 \times 24 = 6 \text{ kN/m}^2$$

Hence, the UDL to beam, including self weight:

$$w_g = 6 \times 6 = 36 \text{ kN/m}$$

$$w_{sw} = \frac{52 \times 9.81}{10^3} = 0.5 \text{ kN/m} \qquad w_q = 6 \times 6.5 = 39 \text{ kN/m}$$

So the serviceability and ultimate loads are:

$$w_{ser} = 36 + 0.5 + 39 = 75.5 \text{ kN/m} \qquad w_{ult} = 1.4(36 + 0.5) + 1.6(39) = 113.5 \text{ kN/m}$$

<u>Design moments and shear:</u>

$$M_{ser} = \frac{w_{ser}L^2}{8} = \frac{75.5 \times 7^2}{8} = 462.4 \text{ kNm} \qquad M_{ult} = \frac{w_{ult}L^2}{8} = \frac{113.5 \times 7^2}{8} = 695.2 \text{ kNm}$$

$$V_{ult} = \frac{w_{ult}L}{2} = \frac{113.5 \times 7}{2} = 397.3 \text{ kN}$$

<u>Moment Capacity:</u>

Effective width; $B_e = 0.25L = 0.25 \times 7000 = 1750 \text{ mm}$

$$\begin{aligned} F_c &= 0.45 f_{cu}\left(D_s - D_p\right)B_e \\ &= \frac{0.45(30)(250)(1750)}{10^3} \\ &= 5906.25 \text{ kN} \end{aligned} \qquad \begin{aligned} F_s &= p_y A \\ &= \frac{275\left(66.5 \times 10^2\right)}{10^3} \\ &= 1828.75 \text{ kN} \end{aligned}$$

Thus we have Case (a): PNA is in the slab because $F_c > F_s$:

$$\begin{aligned} M_c &= F_s\left[\frac{D}{2} + D_s - \frac{F_s}{F_c}\left(\frac{D_s - D_p}{2}\right)\right] \\ &= (1828.75)\left[\frac{449.8}{2} + 250 - \frac{1828.75}{5906.25}\left(\frac{250 - 0}{2}\right)\right] \times 10^{-3} \\ &= 797.7 \text{ kNm} \end{aligned}$$

Thus:

$$M_c > M_{ult} \qquad \therefore OK$$

Shear Capacity:

$$P_v = 0.6 p_y A_v$$
$$= 0.6(275)(449.8)(7.6)\times 10^{-3}$$
$$= 564 \text{ kN}$$

Thus:

$$P_v > V_{ult} \qquad \therefore OK$$

Shear Connector Capacity:

Assuming a 19 mm × 100 mm high connector:

$Q_k = 100 \text{ kN}$ from the table of characteristic stud strengths

$$Q_p \not< 0.8 Q_k = 0.8(100) = 80 \text{ kN}$$

Hence the number required in each half of the span is:

$$N_p = \frac{\min(F_c, F_s)}{Q_p}$$
$$= \frac{\min(5906.25, 1828.75)}{80}$$
$$= \frac{1828.75}{80}$$
$$= 22.8$$

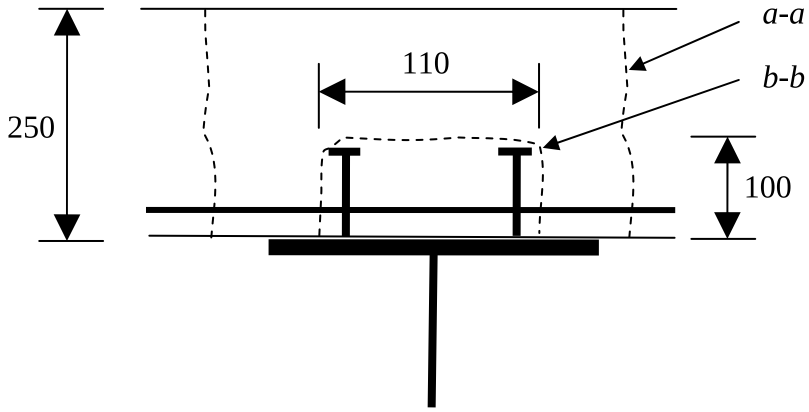
We will use 24 studs as we are putting N_T = 2 studs at each position along the beam.

$$s = \frac{L}{2N_p - 1}$$
$$= \frac{7000}{2(12)-1}$$
$$= 304.3 \text{ mm}$$

Hence use 300 mm c/c evenly spaced along the length of the beam.

Longitudinal Shear Capacity:

Consider these failure planes:



The 110 mm transverse spacing is $\nless 5d$ and is made as large as possible to help prevent this type of failure. The lengths of these planes are:

- Plane *a-a*: $L_s = 2D_p = 2 \times 250 = 500 \text{ mm}$
- Plane *b-b*: $L_s = 2h + d + s_t = 2(100) + 19 + 110 = 329 \text{ mm}$

Hence, the shortest and most critical length is $L_s = 329 \text{ mm}$. Therefore, on this plane, the shear force per unit length is:

$$\begin{aligned} v &= \frac{N_T Q_p}{s} \\ &= \frac{2(80)}{300} \times 10^3 \\ &= 533.3 \text{ N/mm} \end{aligned}$$

The capacity is:

$$\begin{aligned} v_r &= (0.03 f_{cu}) L_s + 0.7 A_{sv} f_y < \left(0.8\sqrt{f_{cu}}\right) L_s \\ &= 0.03(30)(329) + 0.7(2 \times 0.754)(460) < \left(0.8\sqrt{30}\right) 329 \\ &= 781.7 < 1441.6 \\ &= 781.7 \text{ N/mm} \end{aligned}$$

Thus:

$$v_r > v \quad \therefore OK$$

Serviceability: Deflection

Calculate the area ratio:

$$r = \frac{A}{B_e\left(D_s - D_p\right)} = \frac{66.5\times10^2}{1750\left(250-0\right)} = 0.0152$$

Hence the second moment of area of the section is:

$$\begin{aligned} I_g &= I_x + \frac{A\left(D + D_s + D_p\right)^2}{4\left(1+\alpha_e r\right)} + \frac{B_e\left(D_s - D_p\right)^3}{12\alpha_e} \\ &= 21345\times10^4 + \frac{\left(66.5\times10^2\right)\left(449.8+250+0\right)^2}{4\left(1+10\times0.0152\right)} + \frac{1750\left(250-0\right)^3}{12\left(10\right)} \\ &= 1148\times10^6 \text{ mm}^4 \end{aligned}$$

Therefore the deflection is:

$$\begin{aligned} \delta &= \frac{5w_q L^4}{384EI_g} \\ &= \frac{5\left(39\right)\left(7000\right)^4}{384\left(205\times10^3\right)\left(1148\times10^6\right)} \\ &= 5.2 \text{ mm} \end{aligned}$$

And the allowable is:

$$\begin{aligned} \delta_{allow} &= \frac{L}{360} \\ &= \frac{7000}{360} \\ &= 19.4 \text{ mm} \end{aligned}$$

Thus

$$\delta < \delta_{allow} \qquad \therefore OK$$

Serviceability: Elastic behaviour

In addition to our previous calculations, we need:

$$M_{ser} = \frac{w_{ser}L^2}{8}$$
$$= \frac{75.5\times 7^2}{8}$$
$$= 462.4 \text{ kNm}$$

And the section properties, first the depth to the elastic neutral axis:

$$x_e = \frac{\frac{D_s - D_p}{2} + \alpha_e r\left(\frac{D}{2} + D_s\right)}{\left(1+\alpha_e r\right)}$$
$$= \frac{\frac{250-0}{2} + 10\left(0.0152\right)\left(\frac{449.8}{2} + 250\right)}{\left(1+10\left(0.0152\right)\right)}$$
$$= 171.2 \text{ mm}$$

And the elastic section modulii;

$$Z_s = \frac{I_g}{D + D_s - x_e}$$
$$= \frac{1148\times 10^6}{449.8 + 250 - 171.2}$$
$$= 2.17\times 10^6 \text{ mm}^3$$

$$Z_c = \frac{\alpha_e I_g}{x_e}$$
$$= \frac{10\left(1148\times 10^6\right)}{171.2}$$
$$= 67\times 10^6 \text{ mm}^3$$

And the stresses;

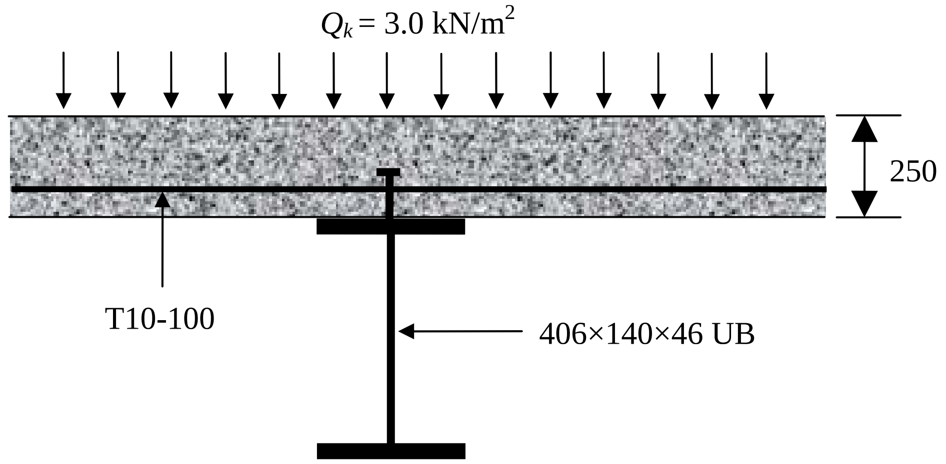
$$\sigma_{s,ser} = \frac{M_{ser}}{Z_s} < p_y$$
$$= \frac{462.4\times 10^6}{2.17\times 10^6} < 275$$
$$= 213.1 \text{ N/mm}^2 \quad \therefore \text{OK}$$

$$\sigma_{c,ser} = \frac{M_{ser}}{Z_c} < 0.45 f_{cu}$$
$$= \frac{462.4\times 10^6}{67\times 10^6} < 0.45\left(30\right)$$
$$= 6.9 < 13.5 \text{ N/mm}^2 \quad \therefore \text{OK}$$

Hence both the steel and concrete stresses remain elastic under the service loads, and so not permanent plastic deformations will occur.

This design has passed all requirements and is therefore acceptable.

Problem 1



Check that the proposed scheme shown is adequate.

Design Data

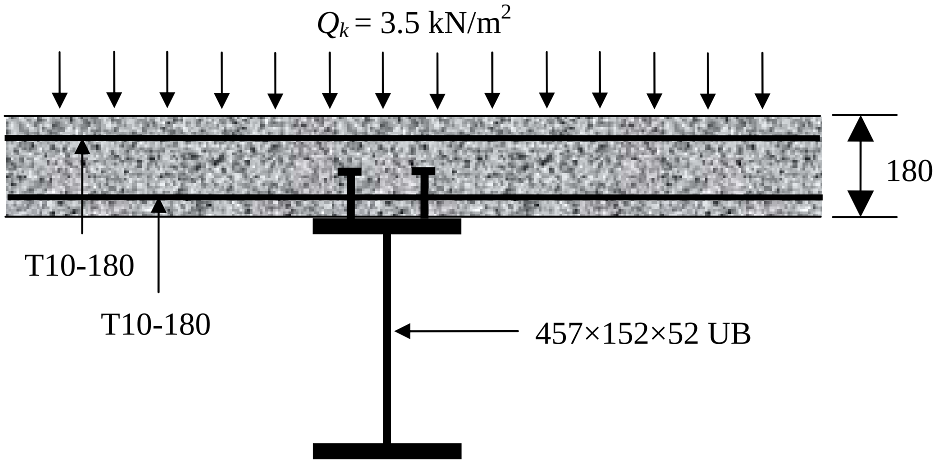
Beam:

- UB is Grade 43A
- Span: 7.5 m simply supported; beams at 6 m centre to centre;
- Use 1 shear stud at each location.

Slab:

- Grade 30N concrete (f_{cu} = 30 N/mm^2)

Problem 2



An allowance of 2.7 kN/m^2 extra dead load is required for ceilings/services and floor tiles.

Check that the proposed scheme shown is adequate.

<u>Design Data</u>

Beam:

- UB is Grade 43A
- Span 8 m simply supported; beams at 5 m centre to centre;
- Use 2 shear studs at each location.

Slab:

- Grade 30N concrete (f_{cu} = 30 N/mm^2)