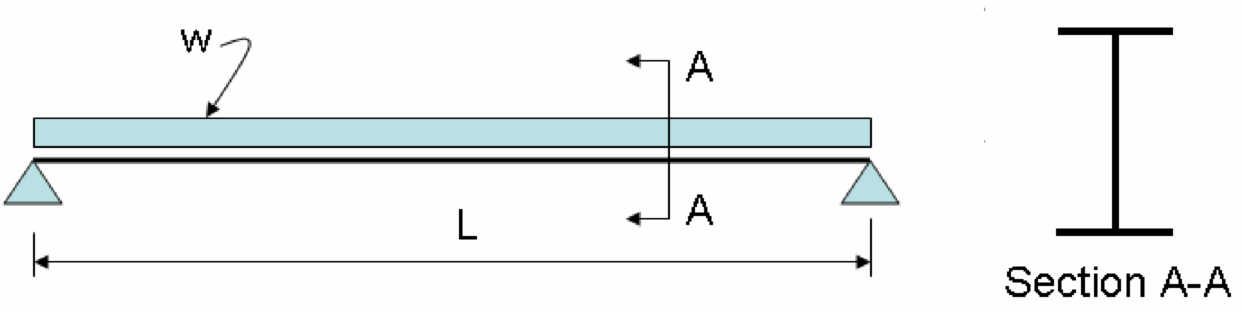
BS-5950-90 Example-001

STEEL DESIGNERS MANUAL SIXTH EDITION - DESIGN OF SIMPLY SUPPORTED COMPOSITE BEAM

EXAMPLE DESCRIPTION

Design a composite floor with beams at 3-m centers spanning 12 m. The composite slab is 130 mm deep. The floor is to resist an imposed load of 5.0 kN/m^2, partition loading of 1.0 kN/m^2 and a ceiling load of 0.5 kN/m^2. The floor is to be un-propped during construction.

GEOMETRY, PROPERTIES AND LOADING



<u>Member Properties</u>
UKB457x191x67
E = 205,000 MPa
F_y = 355 MPa

<u>Loading</u>
w = 6.67kN/m (Dead Load)
w = 1.5kN/m (Construction)
w = 1.5kN/m (Superimposed Load)
w = 18.00kN/m (Live Load)

<u>Geometry</u>
Span, L = 12 m

TECHNICAL FEATURES OF ETABS TESTED

Composite beam design, including:

- Selection of steel section, camber and shear stud distribution
- Member bending capacities, at construction and in service
- Member deflections, at construction and in service

RESULTS COMPARISON

Independent results are referenced from the first example, Design of Simply Supported Composite Beam, in Chapter 21 of the *Steel Construction Institute Steel Designer's Manual*, *Sixth Edition*.

Output Parameter	ETABS	Independent	Percent Difference
Construction Design Moment (kN-m)	211.2	211.3	0.05%
Construction M_s (kN-m)	522.2	522.2	0.00%
Construction Deflection (mm)	29.9	29.9	0.00%
Design Moment (kN-m)	724.2	724.3	0.01%
Full Composite M_{pc} (kN-m)	968.9	968.9	0.00%
Partial Composite M_c (kN-m)	910.8	910.9	0.01%
Shear Stud Capacity Q_n (kN)	57.6	57.6	0.00%
Live Load Deflection (mm)	33.2	33.2	0.00%
Applied Shear Force F_v (kN)	241.4	241.4	0.00%
Shear Resistance P_v (kN)	820.9	821.2	0.00%

COMPUTER FILE: BS-5950-90 EXAMPLE 001.EDB

CONCLUSION

The ETABS results show an excellent comparison with the independent results.

HAND CALCULATION

Properties:

Materials:

S355 Steel:

E = 205,000 MPa, p_y = 355 MPa, γ_s = 7850 kg/m^3

Light-weight concrete:

E = 24,855 MPa, f_{cu} = 30 MPa, γ_c = 1800 kg/m^3

Section:

UKB457x191x67

D = 453.6 mm, b_f = 189.9 mm, t_f = 12.7 mm, t_w = 8.5 mm

A_{steel} = 8,550 mm^2, I_{steel} = 29,380 cm^4

Deck:

D_s =130 mm, D_p = 50 mm, s_r = 300 mm, b_r = 150 mm

Shear Connectors:

d = 19 mm, h = 95 mm, F_u = 450 MPa

Loadings:

Self weight slab	= 2.0 kN/m^2
Self weight beam	= 0.67 kN/m
Construction load	= 0.5 kN/m^2
Ceiling	= 0.5 kN/m^2
Partitions (live load)	= 1.0 kN/m^2
Occupancy (live load)	= 5.0 kN/m^2

Design for Pre-Composite Condition:

Construction Required Flexural Strength:

$$w_{ult\ \text{construction}} = 1.4 \bullet 0.67 + \left(1.4 \bullet 2.0 + 1.6 \bullet 0.5\right) \bullet 3.0 = 11.74 \text{ kN/m}$$

$$M_{ult\ \text{construction}} = \frac{w_{ult\ \text{construction}} \bullet L^2}{8} = \frac{11.74 \bullet 12^2}{8} = 211.3\ \text{kN-m}$$

$$M_s = S_z \bullet P_y = 1,471 \bullet 10^3 \bullet 355 \bullet 10^{-6} = 522.2\ \text{kN-m}$$

Pre-Composite Deflection:

$$w_{\text{construction}} = 2.0 \bullet 3.0 + 0.67 = 6.67\ \text{kN/m}$$

$$\delta = \frac{5 \bullet w_{\text{construction}} \bullet L^4}{384 \bullet E \bullet I} = \frac{5 \bullet 6.67 \bullet 12,000^4}{384 \bullet 205,000 \bullet 29,380 \bullet 10^4} = 29.9\ \text{mm}$$

Camber $= 0.8 \bullet \delta = 24$ mm, which is rounded down to 20 mm

Design for Composite Flexural Strength:

Required Flexural Strength:

$$w_{ult} = 1.4 \bullet 0.67 + \left(1.4 \bullet 2.0 + 1.6 \bullet 1 + 1.6 \bullet 5\right) \bullet 3.0 = 40.24\ \text{kN/m}$$

$$M_{ult} = \frac{w_{ult} \bullet L^2}{8} = \frac{40.24 \bullet 12^2}{8} = 724.3\ \text{kN-m}$$

Full Composite Action Available Flexural Strength:

Effective width of slab:

$$B_e = \frac{\text{L}}{4} = \frac{12,000}{4} = 3,000\ \text{mm} \leq 3,000\ \text{mmm}$$

Resistance of slab in compression:

$$R_c = 0.45 \bullet f_{cu} \bullet B_e \bullet (D_s - D_p) = 0.45 \bullet 30 \bullet 3,000 \bullet \left(130 - 50\right) \bullet 10^{-3} = 3,240\ \text{kN}$$

Resistance of steel in tension:

$$R_s = P_y = A_s \bullet p_y = 8,550 \bullet 355 \bullet 10^{-3} = 3,035\ \text{kN}\ \ \textbf{controls}$$

Moment resistance of composite beam for full composite action:

$$M_{pc} = R_s \left[\frac{D}{2} + D_s - \frac{R_s}{R_c} \frac{\left(\text{D}_\text{s} - \text{D}_\text{p}\right)}{2} \right] \text{ for } R_s \leq R_c$$

$$= 3,035 \left[\frac{453.6}{2} + 130 - \frac{3,035}{3,240} \bullet \frac{80}{2} \right] \bullet 10^{-3} = 968.9\ \text{kN-m}$$

Partial Composite Action Available Flexural Strength:

Assume 72% composite action – the 75% assumed in the example requires more shear studs than can fit on the beam given its actual clear length.

$$R_q = 0.72 \bullet R_s = 2{,}189 \text{ kN}$$

Tensile Resistance of web:

$$R_w = t_w \bullet \left(D - 2 \bullet t_f\right) \bullet \mathrm{p_y} = 8.5 \bullet \left(453.6 - 2 \bullet 12.7\right) \bullet 355 \bullet 10^{-3} = 1{,}292 \text{ kN}$$

As $R_q > R_w$, the plastic axis is in the steel flange, and

$$M_c = R_s \frac{D}{2} + R_q \left[D_s - \frac{R_q}{R_c} \frac{\left(D_s - D_p\right)}{2} \right] - \frac{(R_s - R_q)^2}{R_f} \frac{t_f}{4}$$

$$= 3{,}035 \frac{453.6}{2} \bullet 10^{-3} + 2{,}1899 \left[130 - \frac{2{,}189}{3{,}240} \bullet \frac{80}{2} \right] \bullet 10^{-3} - \frac{\left(3{,}035 - 2{,}189\right)^2}{\left(3{,}035 - 1{,}292\right)} \frac{12.7}{4} \bullet 10^{-3}$$

$$= 910.9 \text{ kN-m}$$

Shear Stud Strength:

Characteristic resistance of 19 mm-diameter studs in normal weight 30 MPa concrete:

Q_k = 100 kN from BS 5950: Part 3 Table 5

Adjusting for light-weight concrete:

$$Q_k = 90 \text{ kN}$$

Reduction factor for profile shape with ribs perpendicular to the beam and two studs per rib:

$$k = 0.6 \bullet \frac{b_r}{D_p} \bullet \frac{\left(h - D_p\right)}{D_p} = 0.6 \bullet \frac{150}{50} \bullet \frac{\left(95 - 50\right)}{50} = 1.62 \text{ but } k \le 0.8$$

Design strength:

$$Q_p = k \bullet 0.8 \bullet Q_k = 0.8 \bullet 0.8 \bullet 90 = 57.6 \text{ kN}$$

Shear Stud Distribution:

The example places two rows of shear studs and computes the numbers of deck ribs available for placing shear studs based on the beam center to center span and the deck rib spacing: 12 m / 300 mm = 40

However, the number of deck ribs available for placing shear studs must be based on the beam clear span, and since the clear beam span is somewhat less than the 12 m center to center span there are only 39 deck ribs available.

ETABS selects 72% composite action, which is the highest achievable and sufficient to meet the live load deflection criteria. ETABS satisfies this 72% composite action by placing one stud per deck rib along the entire length of the beam, plus a second stud per rib in all the deck ribs except the mid-span rib since this is the location of the beam zero moment and a stud in that rib would not contribute anything to the total resistance of the shear connectors. The total resistance of the shear connectors is:

$$R_q = 2 \bullet 19 \bullet Q_p = 38 \bullet 57.6 = 2{,}189 \text{ kN}$$

Live Load Deflection:

The second moment of area of the composite section, based on elastic properties, I_c is given by:

$$I_c = \frac{A_{\text{steel}} \bullet \left(\mathrm{D} + D_s + D_p\right)^2}{4 \bullet \left(1 + \alpha_e \bullet r\right)} + \frac{b_{\text{eff}} \bullet \left(D_s - D_p\right)^3}{12 \bullet \alpha_e} + I_{\text{steel}}$$

$$r = \frac{A_{\text{steel}}}{b_{\text{eff}} \bullet (D_s - D_p)} = \frac{8{,}550}{3{,}000 \bullet \left(130 - 50\right)} = 0.0356$$

For light-weight concrete:

$$\alpha_s = 10$$

$$\alpha_l = 25$$

Proportion of total loading which is long term:

$$\rho_l = \frac{w_{dl} + w_{sdl} + 0.33 \bullet w_{\text{live}}}{w_{dl} + w_{sdl} + w_{\text{live}}} = \frac{6.67 + 1.5 + 0.33 \bullet 18}{6.67 + 1.5 + 18} = 0.541$$

$$\alpha_e = \alpha_s + \rho_l \bullet \left(\alpha_l - \alpha_s\right) = 10 + 0.541 \bullet \left(25 - 10\right) = 18.1$$

$$I_c = \frac{8{,}550 \bullet \left(453.4 + 130 + 50\right)^2}{4 \bullet \left(1 + 18.1 \bullet 0.0356\right)} + \frac{3{,}000 \bullet 80^3}{12 \bullet 18.1} + 294 \bullet 10^6$$
$$= \left(521 + 7 + 294\right) \bullet 10^6 = 822 \bullet 10^6 \text{ mm}^4$$

Live load deflection assuming full composite action:

$$\delta_c = \frac{5 \bullet w_{\text{live}} \bullet L^4}{384 \bullet E \bullet I_c} = \frac{5 \bullet 18 \bullet (12,000)^4}{384 \bullet 205,000 \bullet 822 \bullet 10^6} = 28.8 \text{ mm}$$

Adjust for partial composite action:

$$\delta_s = \frac{5 \bullet w_{\text{live}} \bullet L^4}{384 \bullet E \bullet I_c} = \frac{5 \bullet 18 \bullet (12,000)^4}{384 \bullet 205,000 \bullet 294 \bullet 10^6}$$

$$= 80.7 \text{ mm non-composite reference deflection}$$

$$\delta_{\text{partial}} = \delta_c + 0.3 \bullet (1 - K) \bullet (\delta_s - \delta_c)$$

$$= 28.9 + 0.3 \bullet (1 - 0.72) \bullet (80.7 - 28.9) = 33.2 \text{ mm}$$

Design for Shear Strength:

Required Shear Strength:

$$F_v = \frac{w_{ult} \bullet L}{2} = \frac{40.24 \bullet 12}{2} = 241.4 \text{ kN}$$

Shear Resistance of Steel Section:

$$P_V = 0.6 \bullet p_y \bullet D_s \bullet t_w \quad = 0.6 \bullet 355 \bullet 453.4 \bullet 8.5 \bullet 10^{-3} = 820.9 \text{ kN}$$