OPENINGS IN COMPOSITE SLAB AT CRANE BRACING
COMPOSITE SECTION CAPACITY EVALUATION
(BS 5950 : Part 3 : Section 3.1)

see also "STEEL DESIGNERS' MANUAL" - 6th edition, Chapter 21

Units: $kN := 10^3 \cdot N$ $\quad MPa := 10^6 \frac{N}{m^2}$

Configuration:

UB 533x210x92 steel beam

Sections properties:

$D := 533.1mm$	depth	$t := 10.1mm$	web thickness
$B := 209.3mm$	width	$T := 15.6mm$	flange thickness
$A := 117cm^2$	cross sectional area	$Z_{x.x} := 2070cm^3$	x - x elastic modulus
$I_{x.x} := 55200cm^4$	x - x second moment of area	$S_{x.x} := 2360cm^3$	x - x plastic modulus

steel grade *S355*

$p_y := 355 \frac{N}{mm^2}$ $\quad p_y = 355\,MPa$

Steel elasticity modulus:

$E_s := 205 \frac{kN}{mm^2}$ $\quad E_s = 2.05 \times 10^5\,MPa$

Compressive strength of concrete

$f_{cu} := 40MPa$ $\quad f_{cu} = 40 \frac{N}{mm^2}$ $\quad f_{cu} = 4 \times 10^4 \frac{kN}{m^2}$

Beam span:

$L := 12.8m$

Distance between beams (centre to centre)
(used to compute the breadth of the concrete flange):

$D_{c.c} := 2.68m$

Concrete thickness (slab depth):

$D_s := 180mm$

Breadth of the concrete rib
(BS 5950 : Part 3 : Section 3.1 - Figure 5)

$b_r := 135mm$

Steel deck rib heigth (profile height):

$D_p := 80mm$

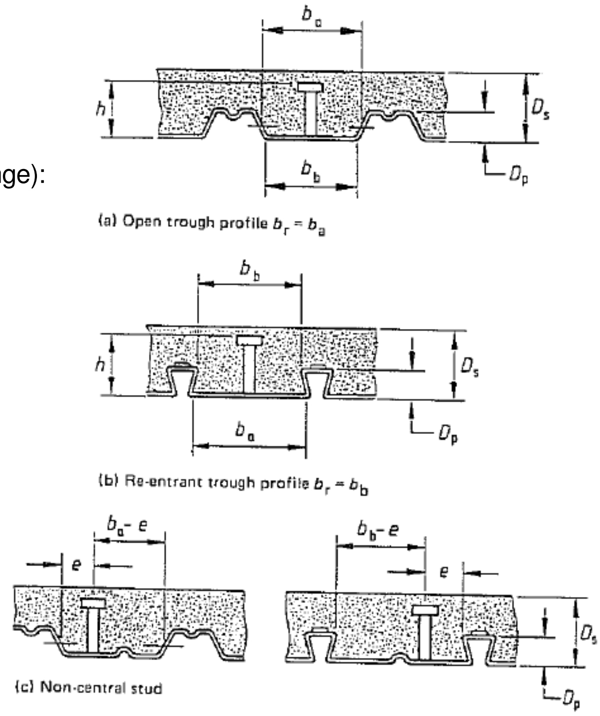


Figure 5. Breadth of concrete rib b_r

Loading:

Beam selfweight:

left end: (at core) $p_{bsw.1} := 1.01 \frac{kN}{m}$

right end: (at diagrid) $p_{bsw.2} := 1.01 \frac{kN}{m}$

Slab selfweight:

left end: (at core) $p_{ssw.1} := 3.6 \frac{kN}{m^2} \cdot 1.8m$

right end: (at diagrid) $p_{ssw.2} := 3.6 \frac{kN}{m^2} \cdot 3.9m$

Finishes load:

left end: (at core) $p_{sdl.1} := 2.5 \frac{kN}{m^2} \cdot 1.8m$

right end: (at diagrid) $p_{sdl.2} := 2.5 \frac{kN}{m^2} \cdot 3.9m$

Live load:

left end: (at core) $p_{ll.1} := 5.0 \frac{kN}{m^2} \cdot 1.8m$

right end: (at diagrid) $p_{ll.2} := 5.0 \frac{kN}{m^2} \cdot 3.9m$

Construction load:

left end: (at core) $p_{cl.1} := 1.5 \frac{kN}{m^2} \cdot 1.8m$

right end: (at diagrid) $p_{cl.2} := 1.5 \frac{kN}{m^2} \cdot 3.9m$

Construction condition (cc):

1.0 * Beam + 1.0 Slab + 1.0 * Construction Load

$p_{cc.1} := 1.0p_{bsw.1} + 1.0p_{ssw.1} + 1.0p_{cl.1}$

$p_{cc.2} := 1.0p_{bsw.2} + 1.0p_{ssw.2} + 1.0p_{cl.2}$

$p_{cc.1} = 10.19 \frac{kN}{m}$

$p_{cc.2} = 20.9 \frac{kN}{m}$

$W_{cc.1} := \frac{(p_{cc.2} - p_{cc.1}) \cdot L}{2}$

$W_{cc.1} = 68.544\,kN$

$W_{cc.2} := p_{cc.1} \cdot L$

$W_{cc.2} = 130.432\,kN$

$$M_{cc}(x) := \frac{W_{cc.1} \cdot x}{3} \cdot \left(1 - \frac{x^2}{L^2}\right) + \frac{W_{cc.2} \cdot x}{2} \cdot \left(1 - \frac{x}{L}\right)$$

$$x_{Mmax} := \text{root}\left[\frac{1}{3} \cdot W_{cc.1} \cdot \left(1 - \frac{x^2}{L^2}\right) - \frac{2}{3} \cdot W_{cc.1} \cdot \frac{x^2}{L^2} + \frac{1}{2} \cdot W_{cc.2} \cdot \left(1 - \frac{x}{L}\right) - \frac{1}{2} \cdot W_{cc.2} \cdot \frac{x}{L}, x\right]$$

$x_{Mmax} = 6.764\,m$ maximum moment location

$M_{cc.max} := M_{cc}(x_{Mmax})$

$M_{cc.max} = 319.404\,kN \cdot m$ design moment in construction condition

Stress in steel (top and lower flange):

$p_{s.cc} := \frac{M_{cc.max}}{Z_{x.x}}$ $p_{s.cc} = 154.302 \frac{N}{mm^2}$ $p_{s.cc} = 154.302\ MPa$

$\frac{p_{s.cc}}{p_y} = 0.435$

Moment resistance of steel section:

$M_{steel.section} := S_{x.x} \cdot p_y$ $M_{steel.section} = 837.8\ kN \cdot m$

$\frac{M_{cc.max}}{M_{steel.section}} = 0.381$ section utilization factor

check_moment_capacity = "OK"

Deflection during construction:

Total load:

$W_{cc} := \frac{p_{cc.1} + p_{cc.2}}{2} \cdot L$ $W_{cc} = 198.976\ kN$

$\delta_{cc} := \frac{5}{384} \cdot \frac{W_{cc} \cdot L^3}{E_s \cdot I_{x.x}}$ $\delta_{cc} = 48.015\ mm$ deflection - steel simply supported beam

Stress due to self weight (sw):

1.0 * Beam + 1.0 * Slab

$p_{sw.1} := 1.0 p_{bsw.1} + 1.0 p_{ssw.1}$ $p_{sw.1} = 7.49 \frac{kN}{m}$

$p_{sw.2} := 1.0 p_{bsw.2} + 1.0 p_{ssw.2}$ $p_{sw.2} = 15.05 \frac{kN}{m}$

$W_{sw.1} := \frac{(p_{sw.2} - p_{sw.1}) \cdot L}{2}$ $W_{sw.1} = 48.384\ kN$

$W_{sw.2} := p_{sw.1} \cdot L$ $W_{sw.2} = 95.872\ kN$

$$M_{sw}(x) := \frac{W_{sw.1} \cdot x}{3} \cdot \left(1 - \frac{x^2}{L^2}\right) + \frac{W_{sw.2} \cdot x}{2} \cdot \left(1 - \frac{x}{L}\right)$$

$$x_{Mmax} := root\left[\frac{1}{3} \cdot W_{sw.1} \cdot \left(1 - \frac{x^2}{L^2}\right) - \frac{2}{3} \cdot W_{sw.1} \cdot \frac{x^2}{L^2} + \frac{1}{2} \cdot W_{sw.2} \cdot \left(1 - \frac{x}{L}\right) - \frac{1}{2} \cdot W_{sw.2} \cdot \frac{x}{L}, x\right]$$

$x_{Mmax} = 6.754\ m$ maximum moment location

$M_{sw.max} := M_{sw}(x_{Mmax})$

$M_{sw.max} = 231.526\ kN \cdot m$ ultimate load design moment on composite beam

Stress in steel (top and lower flange):

$p_{s.sw} := \frac{M_{sw.max}}{Z_{x.x}}$ $p_{s.sw} = 111.849 \frac{N}{mm^2}$ $p_{s.sw} = 111.849\,MPa$

$\frac{p_{s.sw}}{p_y} = 0.315$

Deflection due to self weight:

Total load:

$W_{sw} := \frac{p_{sw.1} + p_{sw.2}}{2} \cdot L$ $W_{sw} = 144.256\,kN$

$\delta_{sw} := \frac{5}{384} \cdot \frac{W_{sw} \cdot L^3}{E_s \cdot I_{x.x}}$ $\delta_{sw} = 34.81\,mm$ deflection - steel simply supported beam

Precamber: $camber := 60mm$

Composite condition - ultimate load, plastic (pc):

1.4 * Beam + 1.4 * Slab + 1.4 * Finish Load + 1.6 Live Load

$p_{pc.1} := 1.4p_{bsw.1} + 1.4p_{ssw.1} + 1.4p_{sdl.1} + 1.6 \cdot p_{ll.1}$ $p_{pc.1} = 31.186 \frac{kN}{m}$

$p_{pc.2} := 1.4p_{bsw.2} + 1.4p_{ssw.2} + 1.4p_{sdl.2} + 1.6 \cdot p_{ll.2}$ $p_{pc.2} = 65.92 \frac{kN}{m}$

$W_{pc.1} := \frac{(p_{pc.2} - p_{pc.1}) \cdot L}{2}$ $W_{pc.1} = 222.298\,kN$

$W_{pc.2} := p_{pc.1} \cdot L$ $W_{pc.2} = 399.181\,kN$

$$M_{pc}(x) := \frac{W_{pc.1} \cdot x}{3} \cdot \left(1 - \frac{x^2}{L^2}\right) + \frac{W_{pc.2} \cdot x}{2} \cdot \left(1 - \frac{x}{L}\right)$$

$$x_{Mmax} := root\left[\frac{1}{3} \cdot W_{pc.1} \cdot \left(1 - \frac{x^2}{L^2}\right) - \frac{2}{3} \cdot W_{pc.1} \cdot \frac{x^2}{L^2} + \frac{1}{2} \cdot W_{pc.2} \cdot \left(1 - \frac{x}{L}\right) - \frac{1}{2} \cdot W_{pc.2} \cdot \frac{x}{L}, x\right]$$

$x_{Mmax} = 6.778\,m$ maximum moment location

$M_{pc.max} := M_{pc}(x_{Mmax})$

$M_{pc.max} = 997.875\,kN \cdot m$ ultimate load design moment on composite beam

The ultimate strength of a composite section is determined from its plastic capacity.

Effective breadth of concrete flange - BS 5950 : Part 3 : Section 3.1 : Chapter 4.6
Steel Designers' Manual - Chapter 21.7.1

$B_e := \frac{L}{8} \cdot 2$ simply supported beam $B_e = 3.2\,m$

$D_{c.c} = 2.68\,m$

$B_e := \min(B_e, D_{c.c})$ $B_e = 2.68\,m$

Compressive resistance of the concrete slab:

BS 5950 : Part 3 : Section 3.1 : Chapter 4.4.2
a) Concrete should be assumed to be stressed to a uniform compression of 0.45*fcu over the full depth of concrete on the compression side of the plastic neutral axis.
(formula 21.5 - Steel Designers' Manual)

$R_c := 0.45 \cdot f_{cu} \cdot B_e \cdot (D_s - D_p)$ $\qquad R_c = 4.824 \times 10^3 \, kN$

$(D_p$ is 0 for solid slabs)

Tensile resistance of the steel:

BS 5950 : Part 3 : Section 3.1 : Chapter 4.4.2
b) The structural steel member should be assumed to be stressed to its design stregth py either in tension or in compression. For sections with a semi-compact or slender web, the effective section described in 4.5.3 should be used.

Axial resitance of the steel section:

$R_s := p_y \cdot A$ $\qquad R_s = 4.154 \times 10^3 \, kN$

Axial resistance of the web:

$R_w := t \cdot (D - 2 \cdot T) \cdot p_y$ $\qquad R_w = 1.8 \times 10^3 \, kN$

Axial resistance of the steel flange:

$R_f := \frac{R_s - R_w}{2}$ $\qquad R_f = 1.177 \times 10^3 \, kN$

Stress in steel (top and lower flange):

$p_{s.pc.low.fl} := p_y$ $\qquad p_{s.pc.low.fl} = 355 \frac{N}{mm^2}$

$p_{s.pc.top.fl} := p_y$ $\qquad p_{s.pc.top.fl} = 355 \frac{N}{mm^2}$

Stress in concrete (top):

$f_{c.pc} := 0.45 f_{cu}$ $\qquad f_{c.pc} = 18 \frac{N}{mm^2}$

Moment resistance of composite beam for full shear connection:

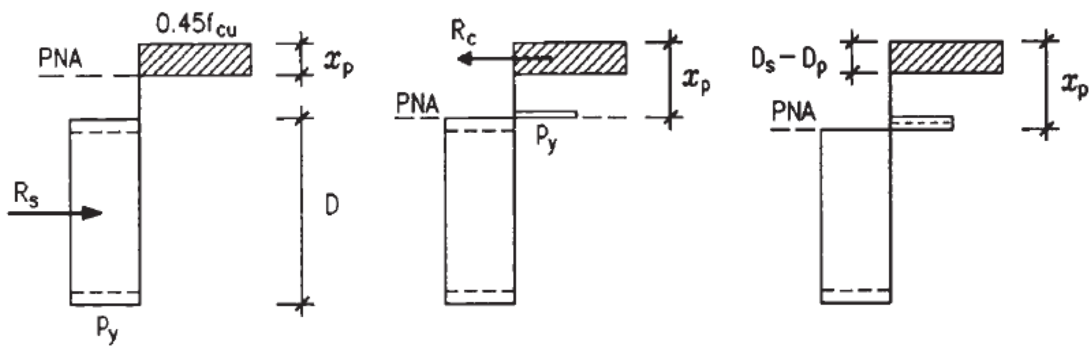


Fig. 21.7 Plastic analysis of composite section under positive (sagging) moment (PNA: plastic neutral axis)

Case 1

$R_c > R_s = 1$ Plastic neutral axis lies in the concrete slab

$$M_{pc.1} := R_s \cdot \left[\frac{D}{2} + D_s - \frac{R_s}{R_c} \cdot \left(\frac{D_s - D_p}{2} \right) \right]$$

$M_{pc.1} = 1.676 \times 10^3 \, kN \cdot m$

Case 2

$R_s > R_c > R_w = 0$ Plastic neutral axis lies in the steel flange

$$M_{pc.2} := R_s \cdot \frac{D}{2} + R_c \cdot \left(\frac{D_s + D_p}{2} \right) - \frac{(R_s - R_c)^2}{R_f} \cdot \frac{T}{4}$$

$M_{pc.2} = 1.733 \times 10^3 \, kN \cdot m$

Case 3

$R_c < R_w = 0$ Plastic neutral axis lies in the web

$M_s := S_{x.x} \cdot p_y$ Plastic moment resistance of the steel section alone

$M_s = 837.8 \, kN \cdot m$

$$M_{pc.3} := M_s + R_c \cdot \left(\frac{D_s + D_p + D}{2} \right) - \frac{R_c^2}{R_w} \cdot \frac{D}{4}$$

for compact web only

$M_{pc.3} = 1.027 \times 10^3 \, kN \cdot m$

Case = "1, plastic neutral axis in concrete slab"

$M_{pc} := \text{if}\left(R_c > R_s, M_{pc.1}, \text{if}\left(R_s > R_c > R_w, M_{pc.2}, M_{pc.3}\right)\right)$

$M_{pc} = 1.676 \times 10^3 \, kN \cdot m$ moment capacity for full shear connection

$$\frac{M_{pc.max}}{M_{pc}} = 0.595$$

Degree of shear connection:

Stud number per trough:	Stud diameter:	Stud Height:	Stud spacing:
$n_s := 2$	$d_s := 19mm$	$h := 125mm$	$D_{stud} := 300mm$

Characteristic resistance of headed studs:

Lightweight concrete factor:

$k_1 := 1.0$

$Q_k := 109kN$ Table 5 BS 5950 : Part 3

$Q_k := k_1 \cdot Q_k$ $Q_k = 109 \, kN$ /one stud

Number of shear connectors in half span:

$N_a := \frac{L}{2} \cdot \frac{1}{D_{stud}} \cdot n_s$ $N_a = 42.667$ (actual number of shear connectors BS 5950 : Part 3 : Section 3.1 : Chapter 5.5.2)

Headed studs in composite slabs (Influence of deck shape on shear connection)
BS 5950 : Part 3 : Section 3.1 : Chapter 5.4.7

For Ribs perpendicular to the beam (5.4.7.2), the capacity of headed studs in composite slabs with the ribs running perpendicular to the beam should be taken as their capacity in a solid slab, multiplied by a **reduction factor k** given by the following expression:

$k := \frac{0.85}{\sqrt{n_s}} \cdot \frac{b_r}{D_p} \cdot \left(\frac{h}{D_p} - 1\right)$ $k = 0.571$

$k_{max} := if(n_s = 1, 1, if(n_s = 2, 0.8, 0.6))$ $k_{max} = 0.8$

$k := min(k, k_{max})$ $k = 0.571$

Capacity of the shear connectors in the solid slab:

To ensure that the shear connection is adequate at all points along the beam, the design resistance of the shear-connectors is taken as 80% of their static resistance
(as per BS 5950 : Part 3 : Section 3.1 : Chapter 5.4.3 letter a)

$Q_p := 0.8 \cdot k \cdot Q_k$

$R_q := Q_p \cdot N_a$ $R_q = 2.123 \times 10^3\ kN$ shear connection capacity

BS 5950 : Part 3 : Section 3.1 : Chapter 5.4.4.1

Longitudinal compressive force in the concrete slab or tension in the steel section at the point of maximum positive moment

$F_p := min(R_c, R_s)$ $F_p = 4.154 \times 10^3\ kN$

Number of shear connectors required to develop the positive moment capacity of the section, i.e. the number of shear connectors each side of the point of maximum moment

$N_p := \frac{F_p}{Q_p}$ $N_p = 83.489$

Degree of shear connection provided:

$K := \frac{N_a}{N_p}$ $K = 0.511$

connection = "partial shear connection"

$K_{min} := if\left(L < 10m, 0.4, if\left(10m < L < 16m, \frac{L - 6m}{10m}, 1\right)\right)$ as per BS 5950 : Part 3 : Section 3.1 : Chapter 5.5.2:

$K_{min} = 0.68$ $\frac{K_{min}}{K} = 1.331$

verif_Kmin = "not OK, increase shear connection capacity"

Moment resistance of composite beam for partial shear connection:

Case 4

$R_q > R_w = 1$ Plastic neutral axis lies in flange

$$M_{c.4} := R_s \cdot \frac{D}{2} + R_q \cdot \left[D_s - \frac{R_q}{R_c} \cdot \left(\frac{D_s - D_p}{2} \right) \right] - \frac{(R_s - R_q)^2}{R_f} \cdot \frac{T}{4}$$

$M_{c.4} = 1.429 \times 10^3 \, kN \cdot m$

Case 5

$R_q < R_w = 0$ Plastic neutral axis lies in web

$$M_{c.5} := M_s + R_q \cdot \left[\frac{D}{2} + D_s - \frac{R_q}{R_c} \cdot \left(\frac{D_s - D_p}{2} \right) \right] - \frac{R_q^{\ 2}}{R_w} \cdot \frac{D}{4}$$

$M_{c.5} = 1.405 \times 10^3 \, kN \cdot m$

$M_c := \text{if}(R_q > R_w, M_{c.4}, M_{c.5})$

$M_c = 1.429 \times 10^3 \, kN \cdot m$

$\frac{M_{pc.max}}{M_c} = 0.698$ $M_{pc} = 1.676 \times 10^3 \, kN \cdot m$

verif_moment_capacity = "OK, but K < Kmin, increase shear connection capacity"

Composite condition - service load, elastic (ec):

1.0 * Finish Load + 1.0 Live Load

$p_{ec.1} := 1.0 p_{sdl.1} + 1.0 \cdot p_{ll.1}$ $p_{ec.1} = 13.5 \frac{kN}{m}$

$p_{ec.2} := 1.0 p_{sdl.2} + 1.0 \cdot p_{ll.2}$ $p_{ec.2} = 29.25 \frac{kN}{m}$

$W_{ec.1} := \frac{(p_{ec.2} - p_{ec.1}) \cdot L}{2}$ $W_{ec.1} = 100.8 \, kN$

$W_{ec.2} := p_{ec.1} \cdot L$ $W_{ec.2} = 172.8 \, kN$

$$M_{ec}(x) := \frac{W_{ec.1} \cdot x}{3} \cdot \left(1 - \frac{x^2}{L^2} \right) + \frac{W_{ec.2} \cdot x}{2} \cdot \left(1 - \frac{x}{L} \right)$$

$$x_{Mmax} := \text{root}\left[\frac{1}{3} \cdot W_{ec.1} \cdot \left(1 - \frac{x^2}{L^2} \right) - \frac{2}{3} \cdot W_{ec.1} \cdot \frac{x^2}{L^2} + \frac{1}{2} \cdot W_{ec.2} \cdot \left(1 - \frac{x}{L} \right) - \frac{1}{2} \cdot W_{ec.2} \cdot \frac{x}{L}, x \right]$$

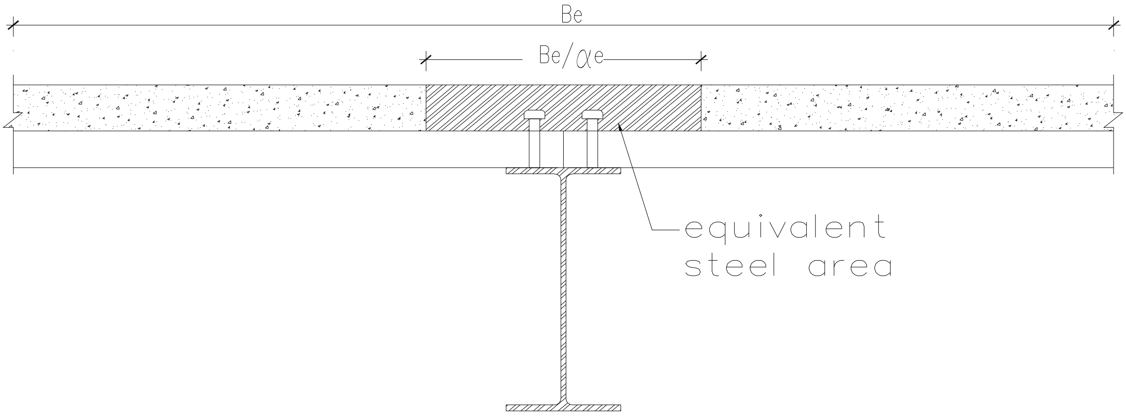
$x_{Mmax} = 6.789 \, m$ maximum moment location

$M_{ec.max} := M_{ec}(x_{Mmax})$ $M_{ec.max} = 439.398 \, kN \cdot m$ service load moment on composite beam

Elastic section properies:

Modular ratio (equivalent steel - concrete component) as per BS 5950 : Part 3 : Section 3.1 : Chapter 4.1:

The elastic section properties of composite members may be expressed in terms of an equivalent steel section by dividing the contributions of the concrete components by the effective modular ratio α_e



Normal concrete, BS 5950 : Part 3 : Section 3.1 : Chapter 4.1 & Table 1

$\alpha_s := 6$ modular ratio for short term loading;

$\alpha_l := 18$ modular ratio for long term loading;

Proportion of the total loading which is long term:

$$\rho_l := \frac{\frac{p_{sdl.1}+p_{sdl.2}}{2}}{\frac{p_{sdl.1}+p_{sdl.2}}{2}+\frac{p_{ll.1}+p_{ll.2}}{2}}$$

$\rho_l = 0.333$

$\alpha_e := \alpha_s + \rho_l \cdot (\alpha_l - \alpha_s)$ $\alpha_e = 10$ element modular ratio

Elastic neutral axis depth:

BS 5950 : Part 3 : Section 3.1 : Chapter 4.4.3
a) The strain distribution in the effective cross section should be linear.
b) The stress distribution in the concrete may be assumed to be linear, based on the appropiate value of the modular ratio from 4.1 and limited to a value of 0.5*fcu. Alternatively the parabolic-rectangular stress distribution recommended in BS 8110 may be used, with a limited compressive strain of 0.0035 and a limiting stress of 0.45fcu

$$r := \frac{A}{(D_s - D_p)\cdot B_e}$$ $r = 0.044$ as per STEEL DESIGNERS' MANUAL 6th Edition equations (21.1) to (21.4)

$$x_e := \frac{\frac{D_s - D_p}{2} + \alpha_e \cdot r \cdot \left(\frac{D}{2} + D_s\right)}{(1 + \alpha_e \cdot r)}$$ $x_e = 0.171\,m$

Second moment of area of the uncracked composite section:

$$I_c := \frac{A\cdot(D + D_s + D_p)^2}{4\cdot(1 + \alpha_e\cdot r)} + \frac{B_e\cdot(D_s - D_p)^3}{12\alpha_e} + I_{x.x}$$

$$I_c = 1.855\times 10^5\ \mathrm{cm}^4$$

Elastic section modulus - lower steel flange:

$$Z_{t.low} := \frac{I_c}{D + D_s - x_e} \qquad Z_{t.low} = 3.419\times 10^3\ \mathrm{cm}^3$$

Elastic section modulus - lower steel flange:

$$Z_{t.top} := \frac{I_c}{\left|x_e - D_s\right|} \qquad Z_{t.top} = 1.955\times 10^5\ \mathrm{cm}^3$$

Elastic section modulus - concrete:

$$Z_c := I_c\cdot\frac{\alpha_e}{x_e} \qquad Z_c = 1.088\times 10^5\ \mathrm{cm}^3$$

Stress in steel (lower flange):

$$p_{s.ec.low} := \frac{M_{ec.max}}{Z_{t.low}} \qquad p_{s.ec.low} = 128.521\ \frac{N}{mm^2} \qquad p_{s.ec.low} = 128.521\ MPa$$

$$p_{s.sw} = 111.849\ \frac{N}{mm^2}$$

$$p_{s.total.low} := p_{s.sw} + p_{s.ec.low} \qquad p_{s.total.low} = 240.369\ \frac{N}{mm^2}$$

$$\frac{p_{s.total.low}}{p_y} = 0.677$$

verif_stress_in_steel_SLS = "OK"

Stress in steel (top flange):

$$p_{s.ec.top} := \frac{M_{ec.max}}{Z_{t.top}} \qquad p_{s.ec.top} = 2.248\ \frac{N}{mm^2} \qquad p_{s.ec.top} = 2.248\ MPa$$

$$p_{s.sw} = 111.849\ \frac{N}{mm^2}$$

$$p_{s.total.top} := p_{s.sw} + p_{s.ec.top} \qquad p_{s.total.top} = 114.096\ \frac{N}{mm^2}$$

$$\frac{p_{s.total.top}}{p_y} = 0.321$$

verif_stress_in_steel_SLS = "OK"

Stress in concrete:

$f_c := \frac{M_{ec.max}}{Z_c}$ $f_c = 4.039 \frac{N}{mm^2}$ $f_c = 4.039\,MPa$

$\frac{f_c}{f_{cu} \cdot 0.5} = 0.202$

verif_stress_in_concrete_SLS = "OK"

Deflection calculation

Simply supported beam:

Total load:

$W_{ec} := \frac{p_{ec.1} + p_{ec.2}}{2} \cdot L$ $W_{ec} = 273.6\,kN$

$\delta_{ec} := \frac{5}{384} \frac{W_{ec} \cdot L^3}{E_s \cdot I_c}$ $\delta_{ec} = 19.646\,mm$ deflection - composite simply supported beam

$\delta_s := \frac{5}{384} \cdot \frac{W_{ec} \cdot L^3}{E_s \cdot I_{x.x}}$ $\delta_s = 66.022\,mm$ deflection - steel (only) simply supported beam

Additional deflection from partial shear connection (STEEL DESIGNERS' MANUAL 6th Edition equation (21.18)

$\delta_{add} := 0.3(1 - K) \cdot (\delta_s - \delta_{ec})$

Total deflection:

$\delta'_c := \delta_{ec} + \delta_{add}$ $\delta'_c = 26.449\,mm$

$L = 483.955\,\delta'_c$

deflection_check = "OK"

Beam natural frequency (nf):

1.0 * Beam + 1.0 * Slab + 1.0 * Finish Load + 0.1 * Live Load

$p_{nf.1} := 1.0 p_{bsw.1} + 1.0 p_{ssw.1} + 1.0 p_{sdl.1} + 0.1 \cdot p_{ll.1}$ $p_{nf.1} = 12.89 \frac{kN}{m}$

$p_{nf.2} := 1.0 p_{bsw.2} + 1.0 p_{ssw.2} + 1.0 p_{sdl.2} + 0.1 \cdot p_{ll.2}$ $p_{nf.2} = 26.75 \frac{kN}{m}$

$W_{nf.1} := \frac{(p_{nf.2} - p_{nf.1}) \cdot L}{2}$ $W_{nf.1} = 88.704\,kN$

$W_{nf.2} := p_{nf.1} \cdot L$ $W_{nf.2} = 164.992\,kN$

$$M_{nf}(x) := \frac{W_{nf.1} \cdot x}{3} \cdot \left(1 - \frac{x^2}{L^2}\right) + \frac{W_{nf.2} \cdot x}{2} \cdot \left(1 - \frac{x}{L}\right)$$

$$x_{Mmax} := \text{root}\left[\frac{1}{3} \cdot W_{nf.1} \cdot \left(1 - \frac{x^2}{L^2}\right) - \frac{2}{3} \cdot W_{nf.1} \cdot \frac{x^2}{L^2} + \frac{1}{2} \cdot W_{nf.2} \cdot \left(1 - \frac{x}{L}\right) - \frac{1}{2} \cdot W_{nf.2} \cdot \frac{x}{L}, x\right]$$

$x_{Mmax} = 6.769\,m$ maximum moment location

$M_{nf.max} := M_{nf}(x_{Mmax})$

$M_{nf.max} = 407.283\,kN \cdot m$ ultimate load design moment on composite beam

Total load:

$W_{nf} := \frac{p_{nf.1} + p_{nf.2}}{2} \cdot L$ $W_{nf} = 253.696\,kN$

$\delta_{nf} := \frac{5}{384} \frac{W_{nf} \cdot L^3}{E_s \cdot I_c}$ $\delta_{nf} = 18.217\,mm$ deflection - composite simply supported beam

Natural frequency

Accurate calculations show that the ratio of the dynamic to static stiffness of a composite beam used in deflection calculations is 1.1 to 1.15. A 1.1 deflection reduction is considered.

$f := 0.18 \cdot \sqrt{\frac{g \cdot 1.1}{\delta_{nf}}}$ $f = 4.38 \frac{1}{s}$

$f_{min} := 4Hz$ minimum accepted value for frequency

$\frac{f_{min}}{f} = 0.913$

frequency_check = "OK"

Summary of results:

[redacted]

$(p_{s.cc}\ \ p_{s.cc}\ \ p_{s.sw}\ \ p_{s.sw}\ \ p_{s.ec.low}\ \ p_{s.ec.top}\ \ f_c\ \ p_y\ \ f_{cu} \cdot 0.5)$

[redacted]

$(M_{cc.max}\ \ M_{steel.section}\ \ M_{pc.max}\ \ M_{pc}\ \ M_{pc.max}\ \ M_c\ \ K)$

[redacted]

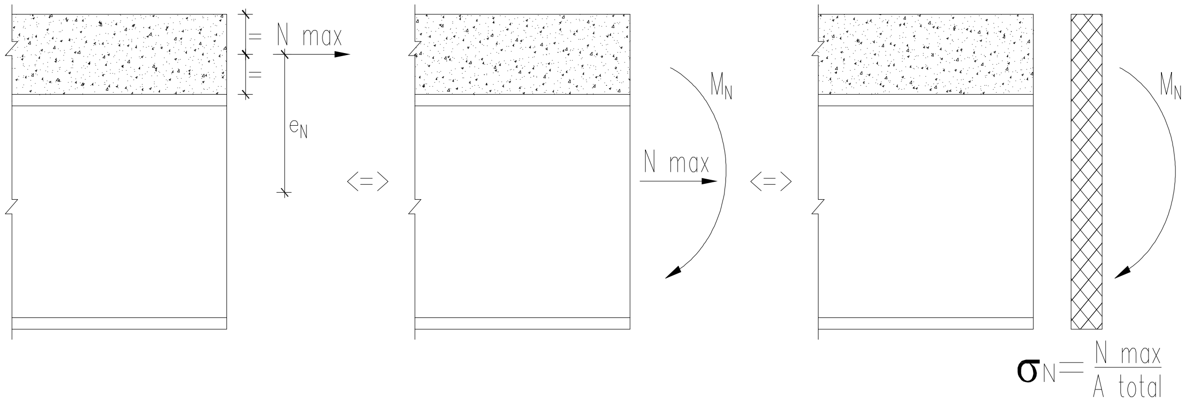
$\left(\delta_{cc} \quad \delta_{sw} \quad camber \quad \delta_{ec} \quad \delta_{add} \quad L\right)$

Natural frequency check: $f = 4.38\,\frac{1}{s}$ $\qquad f_{min} := 4Hz$ $\qquad$ frequency_check = "OK"

INFLUENCE OF A CONCENTRATED LONGITUDINAL FORCE APPLIED IN THE SLAB

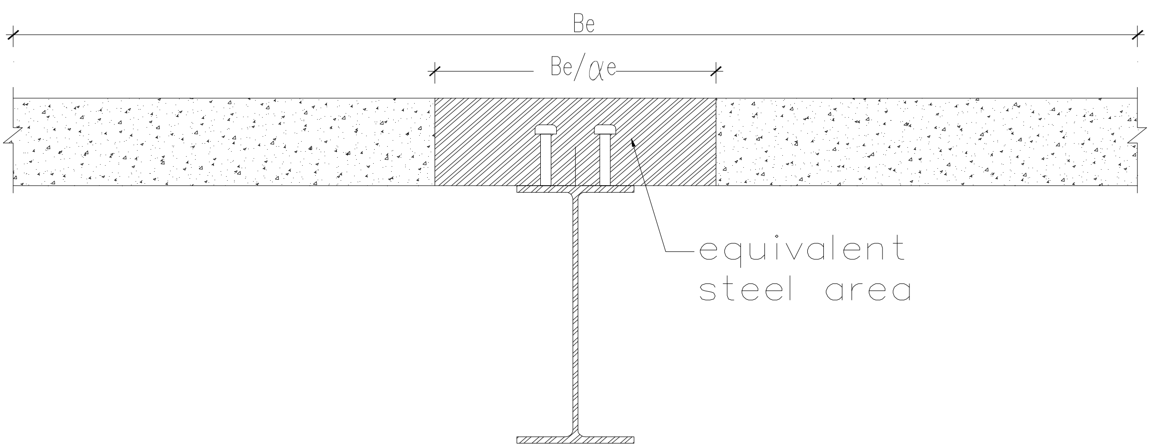
The concentrated force is equivalent with a bending moment and a longitudinal axial force.

$N_{max} := 400kN$ $\qquad$ conservatively $\qquad M_N := N_{max}\cdot\frac{D_s + D}{2}$ $\qquad M_N = 142.62\,kN\cdot m$



Modular ratio (equivalent steel - concrete component) as per BS 5950 : Part 3 : Section 3.1 : Chapter 4.1:

The elastic section properties of composite members may be expressed in terms of an equivalent steel section by dividing the contributions of the concrete components by the effective modular ratio αe



Normal concrete, BS 5950 : Part 3 : Section 3.1 : Chapter 4.1 & Table 1

$\alpha_s := 6$ modular ratio for short term loading;

$\alpha_l := 18$ modular ratio for long term loading;

Proportion of the total loading which is long term:

$$\rho_l := \frac{\dfrac{p_{sdl.1}+p_{sdl.2}}{2}}{\dfrac{p_{sdl.1}+p_{sdl.2}}{2}+\dfrac{p_{ll.1}+p_{ll.2}}{2}}$$

$\rho_l = 0.333$

$\alpha_e := \alpha_s + \rho_l \cdot (\alpha_l - \alpha_s)$ $\alpha_e = 10$ element modular ratio

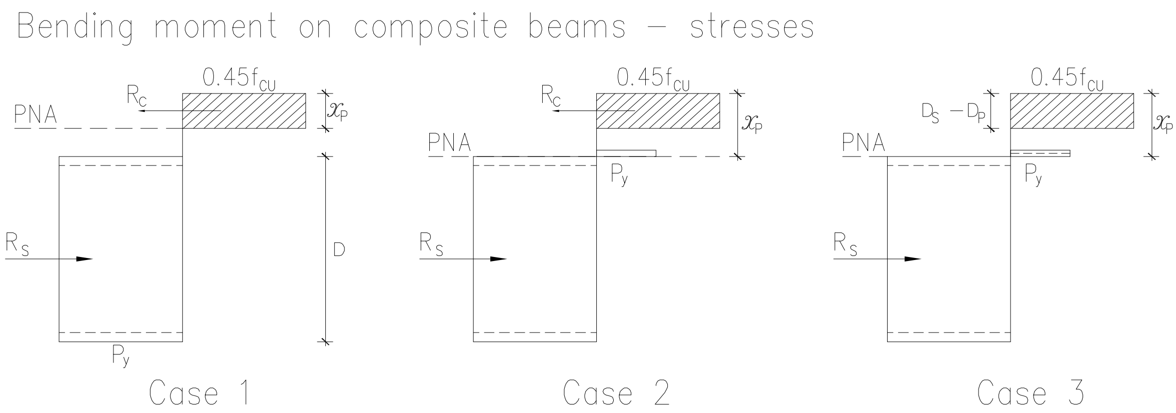
Stresses in steel and concrete from axial force:

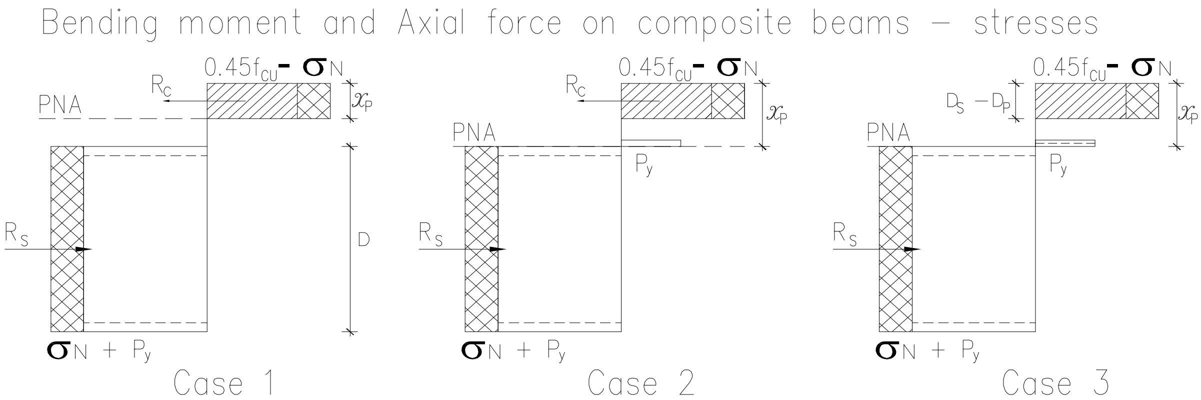
Reenter breadth for full slab portion (one meter on each side of the beam is considered):

$B_e := 2.0\text{m}$

$$\sigma_N := \frac{N_{max}}{A + \dfrac{(B_e \cdot D_s)}{\alpha_e}} \qquad \sigma_N = 8.386\,\text{MPa}$$

Calculation is re-runed, adding σ_N to the strength of concrete and steel:





$D_p := 0m$ - for the massive slab

Compressive resistance of the concrete slab:

BS 5950 : Part 3 : Section 3.1 : Chapter 4.4.2
a) Concrete should be assumed to be stressed to a uniform compression of 0.45*fcu over the full depth of concrete on the compression side of the plastic neutral axis.
(formula 21.5 - Steel Designers' Manual)

$R_c := 0.45 \cdot (f_{cu} - \sigma_N) \cdot B_e \cdot (D_s - D_p)$ $R_c = 5.122 \times 10^3 \, kN$

Tensile resistance of the steel:

BS 5950 : Part 3 : Section 3.1 : Chapter 4.4.2
b) The structural steel member should be assumed to be stressed to its design stregth py either in tension or in compression. For sections with a semi-compact or slender web, the effective section described in 4.5.3 should be used.

Axial resitance of the steel section:

$R_s := (p_y + \sigma_N) \cdot A$ $R_s = 4.252 \times 10^3 \, kN$

Axial resistance of the web:

$R_w := t \cdot (D - 2 \cdot T) \cdot (p_y + \sigma_N)$ $R_w = 1.842 \times 10^3 \, kN$

Axial resistance of the steel flange:

$$R_f := \frac{R_s - R_w}{2} \qquad R_f = 1.205 \times 10^3 \, kN$$

Stress in steel (top and lower flange):

$p_{s.pc.low.fl} := p_y + \sigma_N$ $p_{s.pc.low.fl} = 363.386 \frac{N}{mm^2}$

$p_{s.pc.top.fl} := p_y + \sigma_N$ $p_{s.pc.top.fl} = 363.386 \frac{N}{mm^2}$

Stress in concrete (top):

$f_{c.pc} := 0.45(f_{cu} - \sigma_N)$ $f_{c.pc} = 14.226 \frac{N}{mm^2}$

Moment resistance of composite beam for full shear connection:

Case 1

$R_c > R_s = 1$ Plastic neutral axis lies in the concrete slab

$$M_{pc.1} := R_s \cdot \left[\frac{D}{2} + D_s - \frac{R_s}{R_c} \cdot \left(\frac{D_s - D_p}{2} \right) \right] \qquad M_{pc.1} = 1.581 \times 10^3 \, kN \cdot m$$

Case 2

$R_s > R_c > R_w = 0$ Plastic neutral axis lies in the steel flange

$$M_{pc.2} := R_s \cdot \frac{D}{2} + R_c \cdot \left(\frac{D_s + D_p}{2} \right) - \frac{(R_s - R_c)^2}{R_f} \cdot \frac{T}{4} \qquad M_{pc.2} = 1.592 \times 10^3 \, kN \cdot m$$

Case 3

$R_c < R_w = 0$ Plastic neutral axis lies in the web

$M_s := S_{x.x} \cdot p_y$ Plastic moment resistance of the steel section alone

$M_s = 837.8\,kN \cdot m$

$$M_{pc.3} := M_s + R_c \cdot \left(\frac{D_s + D_p + D}{2} \right) - \frac{R_c^{\,2}}{R_w} \cdot \frac{D}{4}$$ for compact web only

$M_{pc.3} = 766.128\,kN \cdot m$

Case = "1, plastic neutral axis in concrete slab"

$M_{pc} := if\left(R_c > R_s, M_{pc.1}, if\left(R_s > R_c > R_w, M_{pc.2}, M_{pc.3}\right)\right)$

$M_{pc} = 1.581 \times 10^3\,kN \cdot m$

The bending moment generated by the axial force in the slab (M_N) is to be substracted from the bending moment capacity of the beam:

$M_{pc} := M_{pc} - M_N$

$M_{pc} = 1.438 \times 10^3\,kN \cdot m$ moment capacity for full shear connection

$\frac{M_{pc.max}}{M_{pc}} = 0.694$

The same degree of shear connection is considered as for the entire composite beam:

$K := \frac{N_a}{N_p}$ $K = 0.511$

connection = "partial shear connection"

$K_{min} = 0.68$ $\frac{K_{min}}{K} = 1.331$

verif_Kmin = "not OK, increase shear connection capacity"

$R_q := Q_p \cdot N_a$ $R_q = 2.123 \times 10^3\,kN$ total shear connection capacity

$p\% := \frac{N_{max}}{R_q}$ $p\% = 18.844\,\%$ Nmax percentage from the shear connection capacity

The total shear connection capacity is decreased by N_{max}:

$R_q := R_q - N_{max}$ $R_q = 1.723 \times 10^3\,kN$

Moment resistance of composite beam for partial shear connection:

Case 4

$R_q > R_w = 0$ Plastic neutral axis lies in flange

$$M_{c.4} := R_s \cdot \frac{D}{2} + R_q \cdot \left[D_s - \frac{R_q}{R_c} \cdot \left(\frac{D_s - D_p}{2} \right) \right] - \frac{(R_s - R_q)^2}{R_f} \cdot \frac{T}{4}$$

$$M_{c.4} = 1.37 \times 10^3 \, kN \cdot m$$

Case 5

$R_q < R_w = 1$ Plastic neutral axis lies in web

$$M_{c.5} := M_s + R_q \cdot \left[\frac{D}{2} + D_s - \frac{R_q}{R_c} \cdot \left(\frac{D_s - D_p}{2} \right) \right] - \frac{R_q^{\ 2}}{R_w} \cdot \frac{D}{4}$$

$$M_{c.5} = 1.34 \times 10^3 \, kN \cdot m$$

$$M_c := \text{if}(R_q > R_w, M_{c.4}, M_{c.5})$$

$$M_c = 1.34 \times 10^3 \, kN \cdot m$$

The bending moment generated by the axial force in the slab (M_N) is to be substracted from the bending moment capacity of the beam:

$$M_c := M_c - M_N$$

$M_c = 1.198 \times 10^3 \, kN \cdot m$ moment capacity for partial shear connection

$$\frac{M_{pc.max}}{M_c} = 0.833$$

verif_moment_capacity = "OK, but K < Kmin, increase shear connection capacity"

Recommandation:
Because already the connection between steel beam and the composite deck was already partial shear connection, it is recommended that for the 2m width where the composite deck is replaced by a solid slab, the number of the connectors to be increased (extra connectors capacity = Nmax)

Stud number per trough:	Stud diameter:	Stud Height:	Stud spacing:
$n_{sv} := 2$	$d_{sv} := 19mm$	$h := 125mm$	$D_{stud} := 300mm$

Characteristic resistance of headed studs:

Lightweight concrete factor:

$k_1 := 1.0$ $Q_k := 109kN$ Table 5 BS 5950 : Part 3

$Q_k := k_1 \cdot Q_k$ $Q_k = 109\,kN$ /one stud

Influence of stud number per trough:

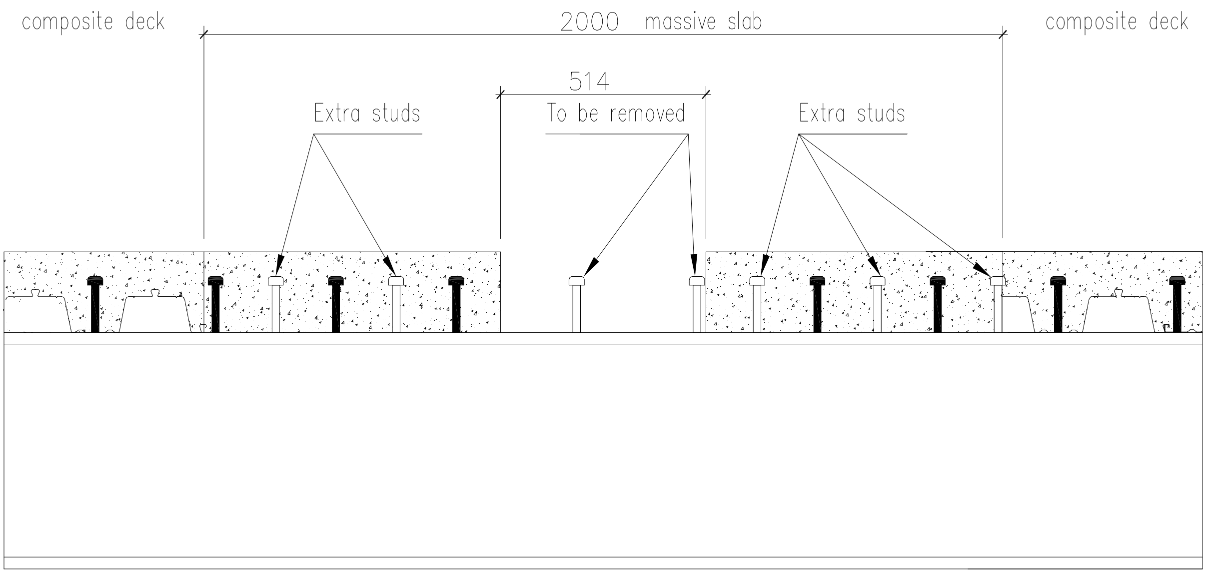
$k := if(n_s = 1, 1, if(n_s = 2, 0.8, 0.6))$ $k = 0.8$

Capacity of the shear connectors in the solid slab:

To ensure that the shear connection is adequate at all points along the beam, the design resistance of the shear-connectors is taken as 80% of their static resistance
(as per BS 5950 : Part 3 : Section 3.1 : Chapter 5.4.3 letter a)

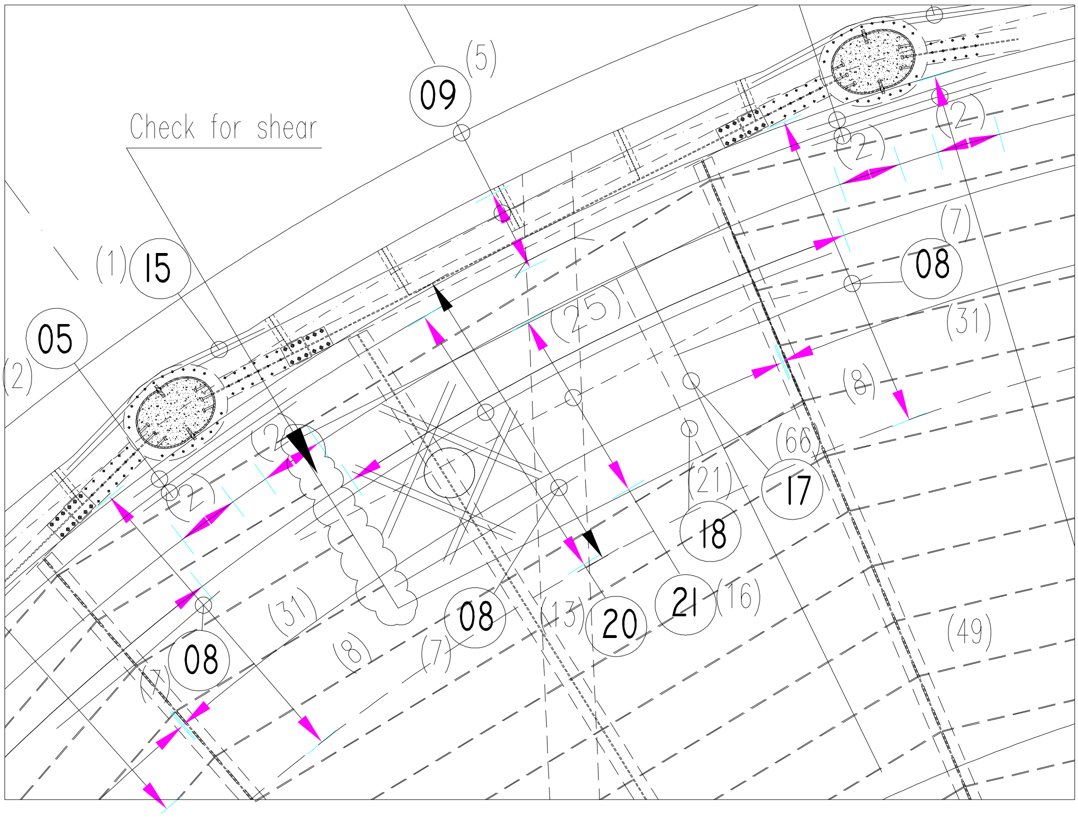
$Q_p := 0.8 \cdot k \cdot Q_k$ $Q_p = 69.76\,kN$

$n_{extra} := \dfrac{N_{max}}{Q_p}$ $n_{extra} = 5.734$



SHEAR CAPACITY AT BOUDARY BETWEEN MASSIVE SLAB AND COMPOSITE SLAB

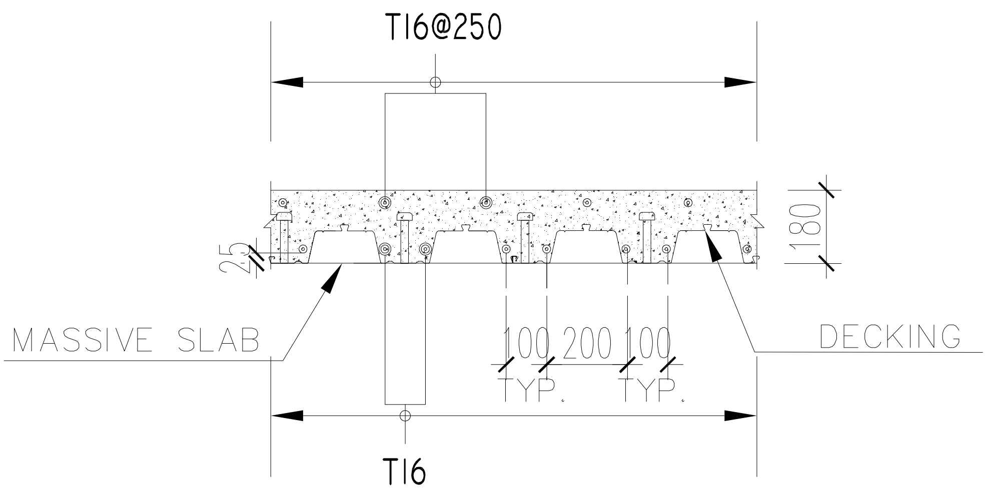
Crane load: $N_{max} := 400kN$ conservatively



$L_c := 2.0m$ Length of boundary (width of massive slab)

$f_{c.shear} := 0.9\frac{N}{mm^2}$ reinforced concrete design shear stress

$L_s := 0.1m$ Slab depth considered for shear resistance (CONSERVATIVE)



Shear capacity of the concrete alone:

$Q_{concrete} := L_c \cdot L_s \cdot f_{c.shear}$

$Q_{concrete} = 180\,kN$

Shear capacity of the reinforcement:

$f_y := 460 \frac{N}{mm^2}$ (Characteristic strength of reinforcement)

Only top reinforcement is considered (CONSERVATIVE ASSUMPTION)

T16@250 => 8 x T16 in the 2.0 m wide section

$A_r := 8 \cdot \pi \cdot \frac{(16 \cdot mm)^2}{4}$ $A_r = 1.608 \times 10^3\,mm^2$ Area of the reinforcement bars in the section

$Q_{reinf} := 0.7 \cdot A_r \cdot f_y$ $Q_{reinf} = 517.936\,kN$

Total capacity of the section:

$Q := Q_{concrete} + Q_{reinf}$

$Q = 697.936\,kN$

$\frac{N_{max}}{Q} = 0.573$ **full crane load is 1/2 of the shear capacity of the section**