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CALCULATION SHEET

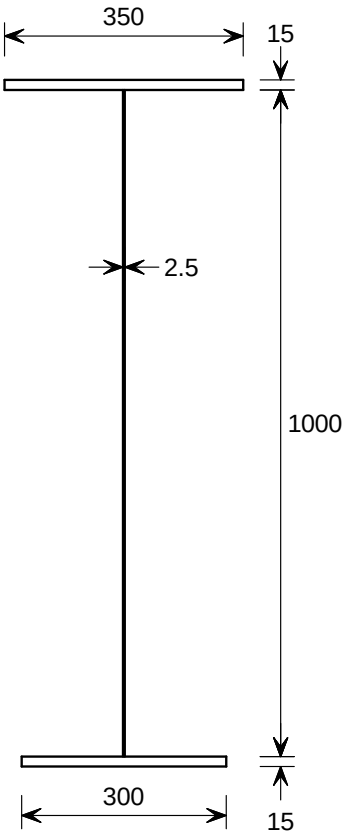
Job No.		Sheet 1 of 7	Rev A
Title			
Subject SINBEAM			
Client	Made by DGB	Date Feb 2018	
	Checked by	Date	

Design in Accordance with EN 1993-1-5 (Annex D) and EN 1993-1-3

Design combinations of actions from EN 1990

UK National Annex provisions adopted in all cases

Beam cross-section



Top flange area = $15 \times 350 = 5250$ mm^2

Bottom flange area = $15 \times 300 = 4500$ mm^2

Web area = $1000 \times 2.5 \times \frac{178}{155} = 2871$ mm^2

Note the 178/155 allows for the corrugations.

Total area = 12621 mm^2

Self weight = $12621 \times 7850 \times 9.81/10^9 = 0.97$ kN/m

Beam loading

Full UDL $g_k = 10$ kN/m

$q_k = 10$ kN/m

Central point load $G_k = 50$ kN

$Q_k = 75$ kN

Total permanent action = 10 + 0.97 = 10.97 kN/m

Design combination of actions using expression 6.10b

ULS UDL = 1.35 × 0.925 × 10.97 + 1.5 × 10 = 28.7 kN/m

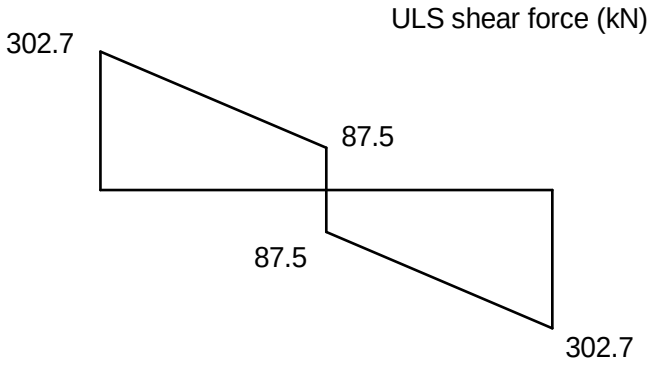
ULS point load = 1.35 × 0.925 × 50 + 1.5 × 75 = 174.94 kN

15 m span

Left reaction

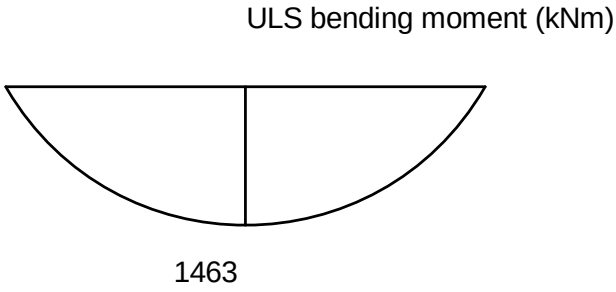
$= 28.7 \times \frac{15}{2} + \frac{174.94}{2} = 302.72$ kN

SF Diagram



Central Bending Moment

$= 28.7 \times 15^2/8 + 174.94 \times \frac{15}{4} = 1463$ kNm



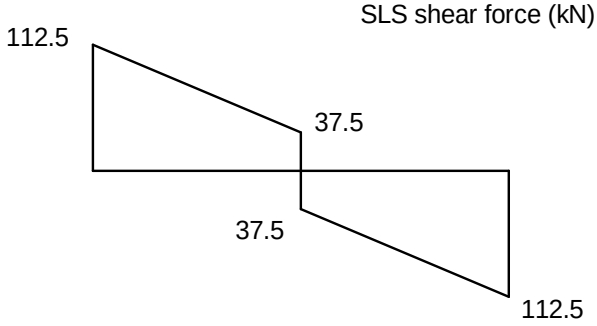
SLS forces and moments

SLS UDL = 10 kN/m

SLS point load = 75 kN

SLS end reaction = $10 \times \frac{15}{2} + \frac{75}{2} = 112.5$ kN

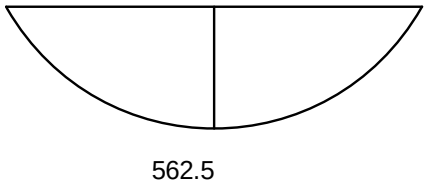
SLS shear force (kN)



SLS Bending Moment

$$10 \times \frac{15^2}{8} + 75 \times \frac{15}{4} = 562.5 \text{ kNm}$$

SLS bending moment (kNm)



Cross-section Inertia

Web area = 2.5 × 1000 = 2500 mm^2

Total area = 2500 + 5250 + 4500 = 12250 mm^2

For neutral axis:

$5250 \times 1022.5 + 2500 \times 515 + 4500 \times 7.5 = \bar{y} \times 12250$

$\bar{y}$ = 546 mm from bottom

$$\text{Inertia} = \frac{350 \times 15^3}{12} + 350 \times 15 \times (1022.5 - 546)^2 + \frac{2.5 \times 1000^3}{12} + 2.5 \times 1000 \times (515 - 546)^2 + \frac{300 \times 15^3}{12} + 300 \times 15 \times (7.5 - 546)^2$$

$$= 2.707 \times 10^9 \text{ mm}^4$$

Central Deflections

Bending deflections

$$\text{Point load} = \frac{75 \times 10^3 \times 15000^3}{48 \times 210 \times 10^3 \times 2.707 \times 10^9} = 9.28 \text{ mm}$$

$$\text{UDL} = \frac{5 \times 10 \times 15000^4}{384 \times 210 \times 10^3 \times 2.707 \times 10^9} = 11.60 \text{ mm}$$

Shear deflections

Point load = $\dfrac{75\times10^3\times15000}{4\times81000\times1000\times2.5\times(155/178)} = 1.60$ mm

UDL = $\dfrac{10\times10^3\times15\times15000}{8\times81000\times1000\times2.5\times(155/178)} = 1.60$ mm

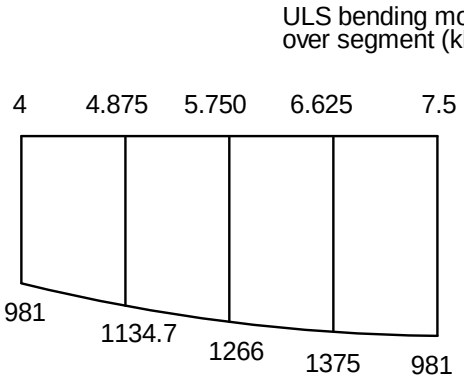
Total = 9.28 + 11.60 + 1.60 + 1.60 = 24.1 mm

Top flange verification

Between restraints at 4 m and 7.5 m

Length = 3500 mm

Bending moment diagram over segment:



At 4 m BM = $302.72 \times 4 - 28.7 \times \dfrac{4^2}{2} = 981$ kNm

At 4.875 m BM = $302.72 \times 4.875 - 28.7 \times \dfrac{4.875^2}{2} = 1134.7$ kNm

At 5.75 m BM = $302.72 \times 5.75 - 28.7 \times \dfrac{5.75^2}{2} = 1266$ kNm

At 6.625 m BM = $302.72 \times 6.625 - 28.7 \times \dfrac{6.625^2}{2} = 1375$ kNm

$C_1 = \sqrt{\dfrac{35\times1463^2}{1463^2+9\times1134.7^2+16\times1266^2+9\times1375^2}} = 1.153$

$k_c = \dfrac{1}{\sqrt{C_1}} = \dfrac{1}{\sqrt{1.153}} = 0.93$

Minor axis inertia of flange

$= \dfrac{15\times350^3}{12} = 53.6\times10^6$ mm^4

Radius of gyration = $\sqrt{\dfrac{53.6\times10^6}{5250}} = 101$ mm

Steel strength – according to UK NA is from product standard

15 mm < 16. so $f_y = 355$ N/mm^2 BS EN 10025-2

$$\varepsilon = \sqrt{\frac{235}{355}} = 0.814$$

$\lambda_1 = 93.9\,\varepsilon = 93.9 \times 0.814 = 76.4$

$$\bar{\lambda}_f = \frac{0.93 \times 3500}{10.1 \times 76.4} = 0.42$$ 6.3.2.4

For minor axis buckling, $f_y < 460$ N/mm^2 and $t < 40$ mm Table 6.2

Use curve 'c'

$\alpha = 0.49$ Table 6.1

Then $\phi = 0.5\,[1+0.49\,(0.42 - 0.2) + 0.42^2] = 0.64$

Then $\chi = \dfrac{1}{0.64 + \sqrt{0.64^2 - 0.42^2}} = 0.89$

Shear lag

1993-1-5 3.1

$b_o = 350/2 = 175$ mm

$\ell_e/50 = 15000/50 = 300$

$175 < 300$, so shear lag may be neglected $\beta = 1.0$

Buckling factor k_σ

Note Zeman do not use 1993-1-5 Annex D, Clause D.2.1(2)

Zeman use Table 4.2 of 1993-1-5 for k_σ

Because $\beta = 1.0$

$$\frac{\sigma_2}{\sigma_1} = 1.25\,(1 - 0.2) = 1.25\,(1 - 0.2) = 1.0$$

From Table 4.2, if $\psi = 1.0$, $k_\sigma = 0.43$

then:

$$\bar{\lambda}_p = \frac{\bar{b}/t}{28.4\,\varepsilon\sqrt{k_\sigma}} \qquad \bar{b} = \frac{b - 0.5 \times 9}{2}$$

$$= \frac{350 - 0.5 \times 40}{2}$$

$= 165$mm

$$\text{so } \bar{\lambda}_p = \frac{165/15}{28.4 \times 0.814 \times \sqrt{0.43}} = 0.725$$

because $0.725 < 0.748$, $\rho = 1.0$

$A_{eff} = 1.0 \times A_c = 5250$ mm^2

Flange resistance

$= 0.89 \times 5250 \times 355 \times 10^{-3} = 1659$ kN

Force in top flange

Maximum moment in segment = 1463 kNm

Force = $\frac{1463\times10^3}{1000+7.5+7.5}$ = 1441 kN

ULS axial force

= 135 × 0.925 × 100 + 1.5 × 50 = 199.9 kN

Proportion in top flange

= 199.9 × 5250/(5250 + 4500) = 107.6

Total axial force = 1441 + 107.6 = 1549 kN

Utilisation = 1549/1659 = 0.93, < 1.0, ok

Bottom flange verification

b_o = 300/2 = 150

ℓ_e/50 = 15000/50 = 300

300 > 150, so shear lag may be neglected

Maximum moment = 1463 kNm

Force in flange = $\frac{-1463\times10^3}{1000+7.5+7.5}$ = -1441 kN (tension)

Compressive force = 199.9 × 4500/(5250 + 4500) = 92.3 kN

Nett tension = -1441 + 92.3 = -1349 kN

Resistance = 4500 × 355 × 10^{-3} = 1598 kN

Utilisation = 1349/1598 = 0.84

Web verification

Maximum shear = 302.7 kN

Global buckling

1993-1-5 Annex D.2.2

Separately, the following numerical integrations have determined the following values:

Web	D_x	D_z
1.5	56210.9	6.31 × 10^7
2	133240	8.41 × 10^7
2.5	260236	1.05 × 10^8
3	449687	1.26 × 10^8

For 2.5 mm web

$\tau_{cr,g} = \frac{32.4}{2.5\times1000^2}\sqrt[4]{260236\times1.05\times10^8}$

= 303.6 N/mm^2

In accordance with Note 1 to Table 3.1a of EN 1993-1-3, the yield strength is factored by 0.9.

The design yield strength is therefore 0.9 × 355 = 319.5 N/mm^2

$\bar{\lambda}_{c,g} = \sqrt{\frac{319.5}{303.6\sqrt{3}}} = 0.78$

$$\chi = \frac{1.5}{0.5 + 0.78^2} = 1.35 \text{ but} < 1$$

So $\chi = 1.0$

Local buckling

$w = 155$ mm

$s = 178$ mm

$$\tau_{cr,l} = \left(5.34 + \frac{40 \times 178}{1000 \times 2.5}\right)\left(\frac{\pi^2 \times 210{,}000}{12\left(1 - 0.3^2\right)}\right)\left[\frac{2.5}{178}\right]^2$$

$= 8.188 \times 189800 \times 1.9726 \times 10^{-4}$

$= 306.6$ N/mm^2

$$\bar{\lambda}_{cr,l} = \sqrt{\frac{319.5}{306.6 \times \sqrt{3}}} = 0.78$$

$$\chi_{u,l} = \frac{1.15}{0.9 + 0.78} = 0.68$$

$$V_{bw,Rd} = 0.68 \times \frac{319.5}{\sqrt{3}} \times 1000 \times 2.5 \times 10^{-3} = 316 \text{ kN}$$

Utilisation = 302.7/316 = 0.96 < 1.0, OK