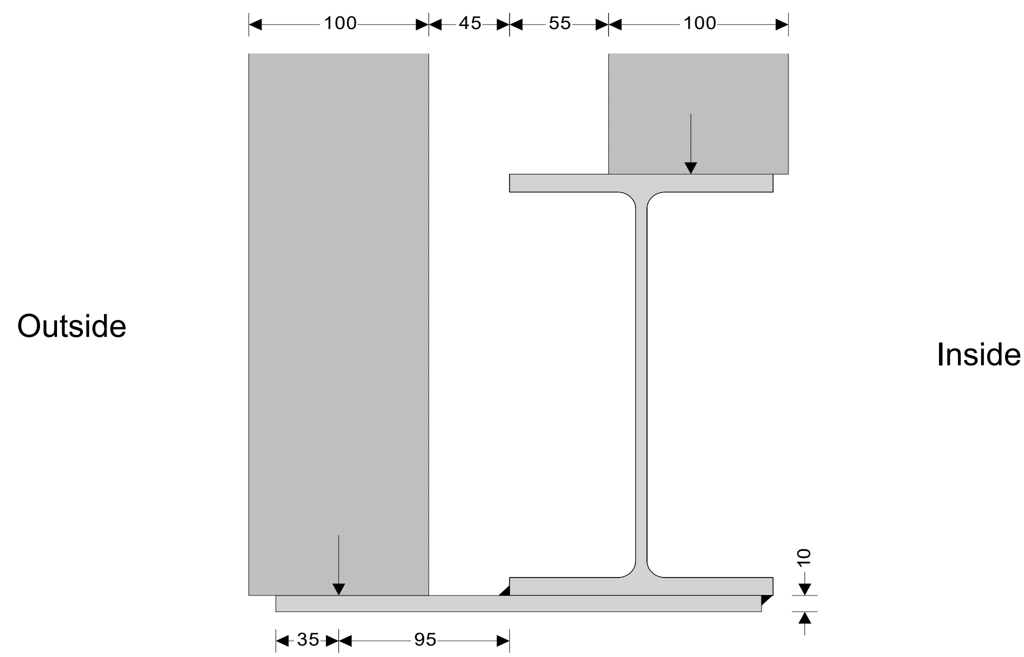
STEEL MASONRY SUPPORT

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 1.0.04



Steel member details

Torsion beam	UB 254x146x37
Masonry support plate	User
Steel grade of support plate	S275
Design strength of support plate	p_{ysb} = **275** N/mm^2
Modulus of elasticity	E = 205000 N/mm^2
Constant	$\varepsilon = \sqrt{(275\text{N/mm}^2 / p_{ysb})}$ = **1.000**
Length of plate beyond beam	l_h = **130** mm
Total length of plate	l_{plate} = **270** mm
Thickness of plate	t_{sb} = **10** mm
Width of main beam	B_{mb} = **146** mm
Area of plate	$A_{sbu} = t_{sb} \times l_{plate}$ = **2700.0** mm^2
Distance from weld position to CoG	$c_{yysb} = l_h / 2 - (l_{plate} - l_h) / 2$ = **-5** mm

Supported materials detail

Density of masonry on main beam	$\rho_{m,mb}$ = **10.0** kN/m^3
Width of masonry on main beam	b_{mmb} = **100** mm
Height of masonry on main beam	h_{mmb} = **3000** mm
Eccentricity of main beam material	e_{mb} = **55** mm
Add dead force main beam (not from masonry)	P_{Gaddmb} = **0.0** kN/m
Add live force main beam (not from masonry)	P_{Qaddmb} = **0.0** kN/m
Density of masonry on support beam	$\rho_{m,sb}$ = **10.0** kN/m^3
Width of masonry on support beam	b_{msb} = **100** mm
Height of masonry on support beam	h_{msb} = **3000** mm

Add dead force support beam (not from masonry) P_{Gaddsb} = **0.0** kN/m
Add live force support beam (not from masonry) P_{Qaddsb} = **0.0** kN/m

Geometry

Cavity width c = **100** mm
Supported width of masonry $d_m = l_h + e_{mb} - c$ = **85** mm

Biaxial stress effects in the plate (SCI-P-110)

Maximum overall bending moment M_x = **48.3** kNm
Dist to NA combined section (CoG torsion beam) $y_{e,all} = (D_{mb} + t_{sb}) \times A_{sbu} / (2 \times (A_{mb} + A_{sbu}))$ = **48** mm
Second moment of area of combined section $I_{xx,all} = (I_{xxmb} + A_{mb} \times y_{e,all}^2) + A_{sbu} \times (D_{mb} / 2 + t_{sb} / 2 - y_{e,all})^2$ = **8574** cm^4
Elastic section modulus of combined section $Z_{xx,all} = I_{xx,all} / (D_{mb} / 2 + t_{sb} - y_{e,all})$ = **957.13** cm^3
Section modulus of plate $Z_{xx,plate} = 1m \times t_{sb}^2 / (6 \times 1m)$ = **16.67** cm^3/m
Eccentricity of support beam masonry e_1 = **95** mm
Force of masonry on support plate $P_1 = (b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ}$ = **4.2** kN/m
Bending at heel $M_{x,plate} = P_1 \times e_1$ = **0.4** kNm/m
Moment capacity of plate $M_c = 1.2 \times Z_{x,plate} \times p_{ysb}$ = **5.5** kNm/m

PASS - Design strength exceeds stress at heel

Longitudinal stress due to overall bending $\sigma_1 = M_x / Z_{xx,all}$ = **50.5** N/mm^2
Constant relating to Von Mises curve $c_{fp} = (4 \times p_{ysb}^2 - 3 \times \sigma_1^2)^{0.5}$ = **543.0** N/mm^2
Transverse bending stress ratio limit $\alpha_{ts} = (c_{fp}^2 - \sigma_1^2) / (2 \times c_{fp} \times p_{ysb})$ = **0.979**
Transverse bending stress ratio $\alpha_{ls} = M_{x,plate} / M_c$ = **0.073**

PASS - Transverse bending stress ratio less than allowable limit

Deflection at toe

Unfactored force on support angle $P_{1SLS} = b_{msb} \times h_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb}$ = **3.0** kN/m
Distance from weld to load position $a_m = e_1$ = **95** mm
Length of load resultant to edge of plate $b_m = l_h - e_1$ = **35** mm
Dist from weld to load position as ratio of length $a_l = a_m / (a_m + b_m)$ = **0.731**
Effective second moment of inertia $I_{eff\_def} = t_{sb}^3 / 12$ = **83333** mm^4/m
Deflection at toe $\delta = (a_l^2 \times (3 - a_l) / 6) \times (P_{1SLS} \times (a_m + b_m)^3) / (E_{S5950} \times I_{eff\_def})$ = **0.08** mm
Deflection limit δ_{lim} = **1.85** mm

PASS - Deflection is within specified criteria

Weld details - assume a full length weld and that the plate acts as a propped cantilever with the prop at the weld position and the fixed end at the centre of the torsion beam

Leg length of weld s_{weld} = **6** mm
Throat size of weld $a_{weld} = 1/\sqrt{(2)} \times s_{weld}$ = **4.2** mm
Shear force at weld position $R_A = P_1 \times maX((1 + (3 \times e_1) / (2 \times B_{mb} / 2)), 1.4)$ = **12.4** kN/m
Maximum possible force in plate $R_p = (l_h + B_{mb}) \times t_{sb} \times p_{ysb}$ = **760.1** kN
Longitudinal shear between beam and plate $R_l = 2 \times R_p / L$ = **381.0** kN/m
Horizontal shear between beam and plate $R_h = P_1 \times e_1 / (s_{weld} / 2 + t_{sb} / 2)$ = **49.9** kN/m
Resultant weld force $R_{weld} = (R_A^2 + R_l^2 + R_h^2)^{0.5}$ = **0.384** kN/mm
Strength of weld (Table 37) p_{weld} = **220.0** N/mm^2
Capacity of full length weld $p_{c,weld} = a_{weld} \times p_{weld}$ = **0.933** kN/mm

PASS - Capacity of weld exceeds resultant force on weld

Torsional loading ULS

Loading of support beam masonry $w_{1ULS} = (h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb}) \times \gamma_{fG} + P_{Qaddsb} \times \gamma_{fQ}$ = **4.20** kN/m
Loading of main beam masonry $w_{2ULS} = (h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb}) \times \gamma_{fG} + P_{Qaddmb} \times \gamma_{fQ}$ = **4.20** kN/m

Self weight of support beam $w_{3ULS} = A_{sbu} \times \rho_{sb} \times \gamma_{fG} = \mathbf{0.30}$ kN/m

Torsional loading SLS

Loading of support beam masonry $w_{1SLS} = h_{msb} \times b_{msb} \times \rho_{m,sb} + P_{Gaddsb} + P_{Qaddsb} = \mathbf{3.00}$ kN/m

Loading of main beam masonry $w_{2SLS} = h_{mmb} \times b_{mmb} \times \rho_{m,mb} + P_{Gaddmb} + P_{Qaddmb} = \mathbf{3.00}$ kN/m

Self weight of support beam $w_{3SLS} = A_{sbu} \times \rho_{sb} = \mathbf{0.21}$ kN/m

Eccentricities

Distance to shear centre of main beam $e_{0mb} = \mathbf{0}$ mm

Eccentricity of support beam masonry $e_{1mb} = (B_{mb} + b_{msb}) / 2 + c - e_{mb} = \mathbf{168}$ mm

Eccentricity of main beam masonry $e_{2mb} = (B_{mb} - b_{mmb}) / 2 - e_{mb} = \mathbf{-32}$ mm

Eccentricity of support beam $e_{3mb} = B_{mb} / 2 + c_{yysb} = \mathbf{68}$ mm

Torsional effects

Applied torque (ULS) $T_{qULS} = abs(w_{1ULS} \times e_{1mb} + w_{2ULS} \times e_{2mb} + w_{3ULS} \times e_{3mb}) = \mathbf{0.59}$ kNm/m

Total torque (ULS) $T_q = T_{qULS} \times L = \mathbf{2.37}$ kNm

Applied torque (SLS) $T_{qSLS} = abs(w_{1SLS} \times e_{1mb} + w_{2SLS} \times e_{2mb} + w_{3SLS} \times e_{3mb}) = \mathbf{0.42}$ kNm/m

Total torque (SLS) $T_{qu} = T_{qSLS} \times L = \mathbf{1.69}$ kNm

STEEL BEAM TORSION DESIGN

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

Tedds calculation version 2.0.02

Section details

Section type UB 254x146x37

Steel grade S275

Design stength $p_{yw} = p_y = \mathbf{275}$ N/mm^2

Constant $\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)} = \mathbf{1.000}$

Geometry - Beam unrestrained against lateral-torsional buckling between supports.

Effective span $L = \mathbf{3990}$ mm

Length of segment for LT buckling $L_{LT} = \mathbf{2550}$ mm

Compression flanges laterally restrained

Both flanges free to rotate on plan

Effective length for LT buckling $L_{E\_LT} = L_{LT} \times 1.0 = \mathbf{2550}$ mm

Loading - Torsional loading comprises only full-length uniformly distributed load(s)

Internal forces & moments on member under factored loading for uls design

Applied shear force $F_{vy} = \mathbf{58.4}$ kN

Maximum bending moment $M_{LT} = M_x = \mathbf{48.34}$ kNm

Applied torque $T_q = \mathbf{2.37}$ kNm

Minor axis bending moment $M_y = 0$ kNm

Compression force $F_c = 0$ kN

Equivalent uniform moment factors

EUM factor (Cl. 4.3.6.6 and T18) $m_{LT} = \mathbf{1.000}$

Torsional deflection parameters

Beam is torsion fixed and warping free at each end. (as defined in SCI-P-057 section 2.1.6) - Appendix B case 4

Dist along the beam for first derivative of twist $z_1 = 0$ mm

Dist along the beam for second derivative of twist $z_2 = L / 2 = \mathbf{1995}$ mm

First derivative of angle of twist $\phi'_1 = T_q / (G \times J) \times a / L \times [L^2 / (2 \times a) \times (1 / L - 2 \times z_1 / L^2) + \sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1 \text{ rads} = \mathbf{4.29{\times}10^{-2}}$ rads/m

Third derivative of angle of twist $\phi'''_1 = T_q / (G \times J \times a^2) \times a/L \times [\sinh(z_1 / a) - \tanh(L / (2 \times a)) \times \cosh(z_1 / a)] \times 1$ rads = $\mathbf{-3.78 \times 10^{-2}}$ rads/m^3

Angle of twist $\phi_2 = T_q \times a / (G \times J) \times a / L \times [L^2 / (2 \times a^2) \times (z_2 / L - z_2^2 / L^2) + \cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1$ rads = **0.053** rads

Second derivative of angle of twist $\phi''_2 = T_q / (G \times J \times a) \times a / L \times [\cosh(z_2 / a) - \tanh(L / (2 \times a)) \times \sinh(z_2 / a) - 1] \times 1$ rads = $\mathbf{-3.10 \times 10^{-2}}$ rads/m^2

Design parameters

Total angle of twist $\phi = abs(\phi_2) =$ **0.053** rads

First derivative of ϕ $\phi' = abs(\phi'_1) = \mathbf{4.29 \times 10^{-2}}$ rads/m

Second derivative of ϕ $\phi'' = abs(\phi''_2) = \mathbf{3.10 \times 10^{-2}}$ rads/m^2

Third derivative of ϕ $\phi''' = abs(\phi'''_1) = \mathbf{3.78 \times 10^{-2}}$ rads/m^3

Section classification

$b / T =$ **6.7**

$d / t =$ **34.8**

$r_{1s} = \min(1.0, \max(-1.0, F_c / (d \times t \times p_{yw}))) =$ **0.000**

$r_{2s} = F_c / (A_g \times p_{yw}) =$ **0.000**

Section classification is plastic

Shear capacity (parallel to y-axis)

Design shear force $F_{vy} =$ **58.4** kN

Design shear resistance (Cl. 4.2.3) $P_{vy} = 0.6 \times p_y \times A_{vy} =$ **266.1** kN

Pass - Shear

Moment capacity (x-axis)

Design bending moment $M_x =$ **48.3** kNm

Moment capacity $M_{cxu} = p_y \times S_x =$ **132.9** kNm

Moment capacity low shear (Cl. 4.2.5.1) $M_{cx} = \min(p_y \times S_x, 1.2 \times p_y \times Z_x) =$ **132.9** kNm

Pass - Moment capacity exceeds design bending moment

Lateral torsional buckling

Effective length for lateral torsional buckling $L_{E\_LT} =$ **2550** mm

Slenderness ratio $\lambda = L_{E\_LT} / r_y =$**73**

Buckling parameter $u =$ **0.890**

Flange ratio $\eta = 0.5$

Torsional index $x =$ **24.3**

Slenderness factor $v = 1 / (1 + 0.05 \times (\lambda / x)^2)^{0.25} =$ **0.91**

Ratio - cl 4.3.6.9 $\beta_w = 1.0 =$ **1.000**

Equivalent slenderness – cl 4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{(\beta_w)} =$ **59**

Limiting slendernes – Annex B2.2 $\lambda_{L0} = 0.4 \times \sqrt{(\pi^2 \times E_{S5950} / p_y)} =$ **34**

Euler stress $p_E = \pi^2 \times E_{S5950} / \lambda_{LT}^2 =$ **573** N/mm^2

Perry factor $\eta_{LT} = \max(7.0 \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.176**

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **474510235.316**

Bending strength $p_b = p_E \times p_y / (\phi_{LT} + \sqrt{(\phi_{LT}^2 - p_E \times p_y)}) =$ **215** N/mm^2

Buckling resistance moment $M_b = p_b \times S_x =$ **103.7** kNm

Max moment governing buckling resistance $M_{LT} =$ **48.3** kNm

Equiv uniform moment factor for LTB $m_{LT} =$ **1.00**

$M_b / m_{LT} =$ **103.7** kNm

Pass - lat. tors. buckling

Buckling under combined bending & torsion -SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Span factor	L / a = **3.31**
Angle of twist	ϕ = **0.053** rads
Second derivative of ϕ	ϕ'' = **31.0×10^{-3}** rads/m^2
Induced minor axis moment	$M_{yt} = M_x \times \phi$ / 1 rad = **2.54** kNm
Normal stress at flange tip due to M_{yt}	$\sigma_{byt} = M_{yt} / Z_y$ = **33** N/mm^2
Normal stress at flange tip due to warping	$\sigma_w = E_{S5950} \times W_{n0} \times \phi''$ / 1 rad = **57** N/mm^2
Interaction index	$i_b = M_x \times m_{LT} / M_b + (\sigma_{byt} + \sigma_w) / p_y \times (1 + 0.5 \times M_x \times m_{LT} / M_b)$ = **0.87**

Pass - Combined bending and torsion check satisfied

Local capacity under combined bending & torsion

For simplicity, a conservative check is applied using the maximum stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max. direct stress due to M_x	$\sigma_{bx} = M_x / Z_x$ = **112** N/mm^2
Combined stress - eqn 2.22	$\sigma_{bx} + \sigma_{byt} + \sigma_w$ = **201** N/mm^2
Design strength	p_y = **275** N/mm^2

Pass - Local capacity

Combined shear stresses - SCI-P-057 section 2.3

For simplicity, a conservative check is applied using the maximum shear stresses due to each of the separate load effects, even though these do not necessarily all occur at the same section along the member.

Max shear stresses due to bending in web	$\tau_{bw} = F_{vy} \times Q_w / (I_x \times t)$ = **40** N/mm^2
Max shear stresses due to bending in flange	$\tau_{bf} = F_{vy} \times Q_f / (I_x \times T)$ = **9** N/mm^2
Max shear stresses due to torsion in web	$\tau_{tw} = abs(G \times t \times \phi'$ / 1rad) = **21** N/mm^2
Max shear stresses due to torsion in flange	$\tau_{tf} = abs(G \times T \times \phi'$ / 1 rad) = **37** N/mm^2
Max shear stresses due to warping in flange	$\tau_{wf} = abs(-E_{S5950} \times S_{w1} \times \phi'''$ / 1 rad / T) = **3** N/mm^2
Amp shear stress torsion & warping in web	$\tau_{vtw} = \tau_{tw} \times (1 + 0.5 \times M_x \times m_{LT} / M_b)$ = **26** N/mm^2
Amp shear stress torsion & warping in flange	$\tau_{vtf} = (\tau_{tf} + \tau_{wf}) \times (1 + 0.5 \times M_x \times m_{LT} / M_b)$ = **49** N/mm^2

Combined shear stresses due to bending, torsion & warping:

Combined shear stresses in web	$\tau_w = \tau_{bw} + \tau_{vtw}$ = **67** N/mm^2
Combined shear stresses in flange	$\tau_f = \tau_{bf} + \tau_{vtf}$ = **58** N/mm^2
Shear strength	$p_v = 0.6 \times p_y$ = **165** N/mm^2

Pass - Combined shear stresses

Twist check

Total applied torque (unfactored)	T_{qu} = **1.69** kNm
Maximum twist under sls loading	$\phi_{sls} = \phi \times T_{qu} / T_q$ = **2.15** deg
Twist limit	ϕ_{lim} = **2.50** deg

Pass - Twist

Deflection

Maximum y-axis deflection	δ_{y_max} = **1.9** mm
Deflection limit - cl. 2.5.2	$\delta_{lim} = \min(L/ k_\delta, \delta_{lim\_abs})$ = **10.0** mm

Pass - Deflection within specified limit