Rob Adaway Structural Solutions

T 01933 276827 M 0794 70 80 766

STRUCTURAL CALCULATIONS

FOR

BASIC LIFTING CRADLE

TO

LIFT AND MOVE

SINGLE STOREY TEMPORARY UNIT

17/1268

AUGUST 2017

R Adaway

Victory House, 26 Denington Road, Wellingborough, Northants NN8 2QH

RASS
ROB ADAWAY STRUCTURAL SOLUTIONS
M 0794 70 80 766 E info@robadaway.co.uk

Project No: 17/1268	Sheet No: 1
Made By: RA	Revision:
Date: AUG '17	Checked By:

Project: LIFTING CRADLE FOR SINGLE STOREY TEMPORARY UNITS

FROM THE INFORMATION WE HAVE RECEIVED IT IS UNDERSTOOD THAT EACH TEMPORARY UNIT IS APPROX 20M LONG x 6.8M WIDE AND WEIGHS APPROX 18 TONNES.

THE SINGLE STOREY TEMPORARY UNITS HAVE A TIMBER GROUND FLOOR CONSTRUCTED AS JOISTED FRAMEWORK WHICH ARE INDIVIDUALLY LEVELLED ABOVE THE GROUND AND SUPPORTED BY TIMBER STILTS. IT IS UNDERSTOOD THAT THE SPAN OF THE JOISTED FRAMES SPAN HALF OF THE UNIT WIDTH THEREBY MAKING EACH JOIST APPROX 3.4M LONG.

ONCE THE JOISTED FRAMEWORK IS LAID & LEVELLED THE WALLS ARE ERECTED AND TOPPED WITH A ROOF

IT IS INTENDED THAT THESE CALCULATIONS SHOW THAT THE TOTAL BUILDING CAN BE MOVED BY THE INSTALLATION OF STEEL BEAMS BENEATH THE FLOOR FRAMEWORK TO FORM A LIFTING CRADLE.

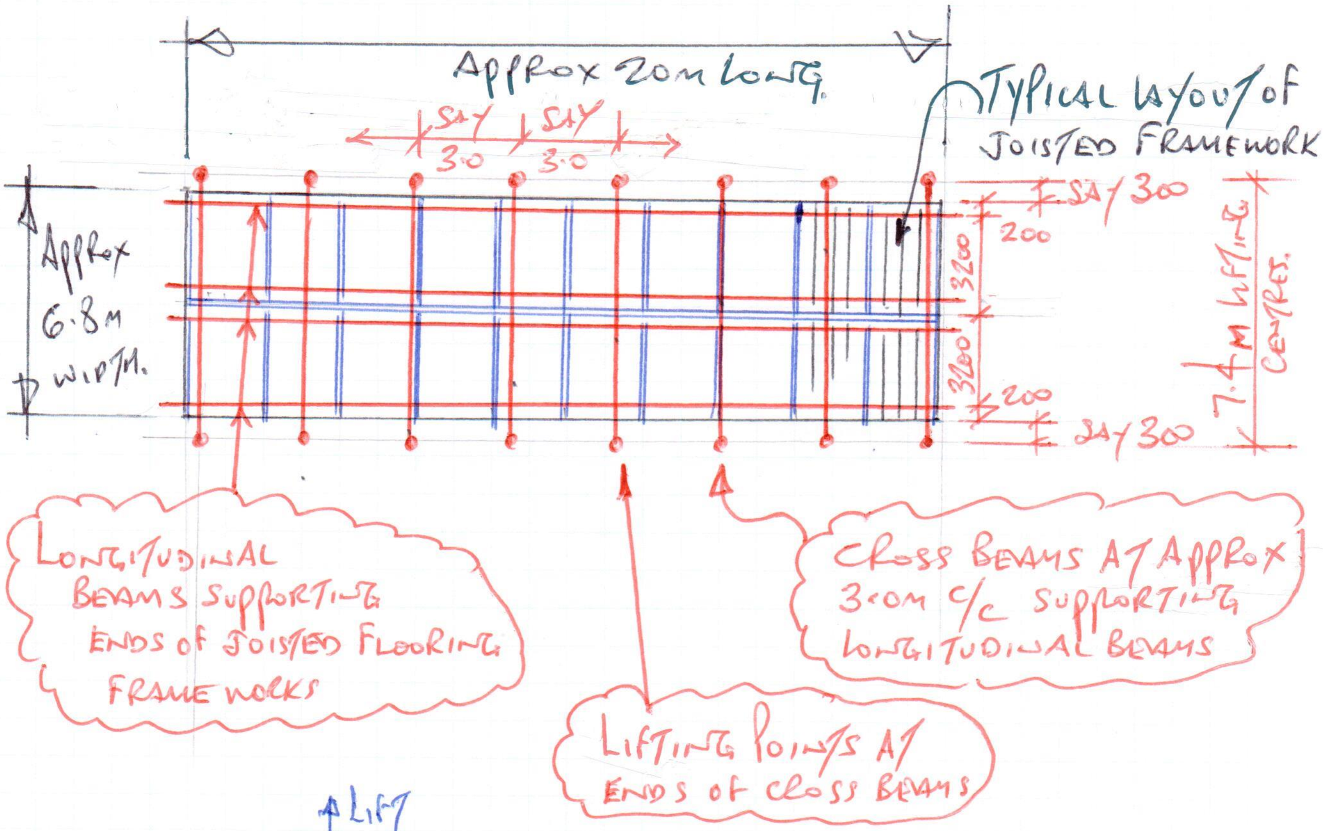
DUE TO THE TEMPORARY USE OF THE LIFTING CRADLE LONG TERM STEEL DESIGN FACTORS ARE NOT CONSIDERED RELEVANT BUT THE PROVISION OF LIMITED BEAM

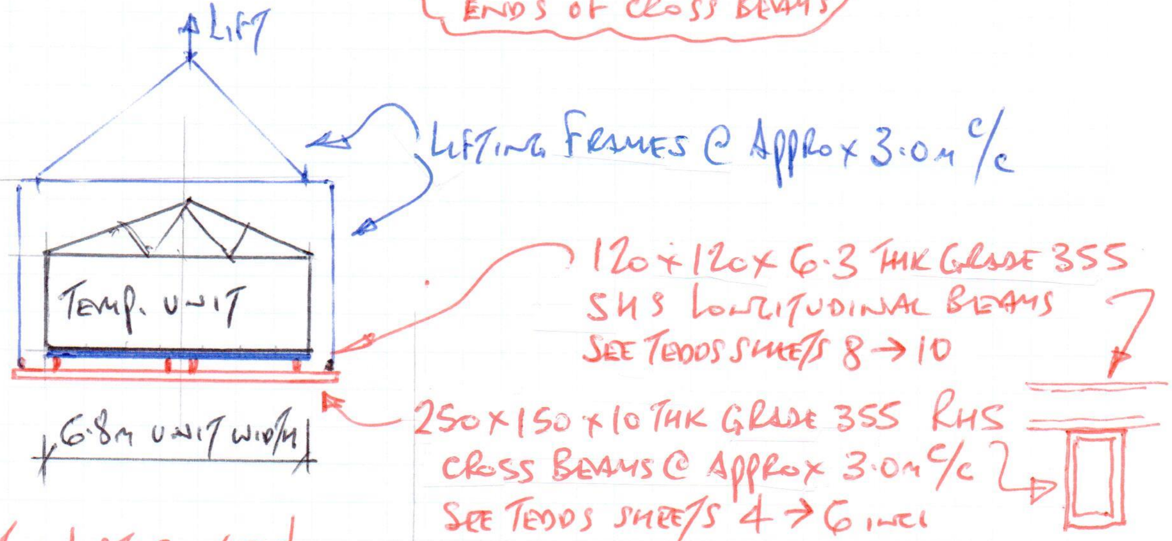
RASS
ROB ADAWAY STRUCTURAL SOLUTIONS
M 0794 70 80 766 E info@robadaway.co.uk

Project No:	17/1268	Sheet No:	2
Made By:	RA	Revision:	
Date:	Aug '17	Checked By:	

Project: Lifting cradle for single storey temporary units

Deflections are considered more onerous that the usual span/360 used in steelwork design.





Note:- Steel lifting eyes & beam to beam connections not part of RASS design brief

RASS
ROB ADAWAY STRUCTURAL SOLUTIONS
M 0794 70 80 766 E info@robadaway.co.uk

Project No: 17/1268	Sheet No: 3
Made By: RA	Revision:
Date: Aug '17	Checked By:

Project: Lifting cradle for single storey temporary units

Approx total weight of unit ≑ 18 Tonnes (Dead load only)
= 180 kN.

Plan size of unit ≑ 20m × 6.8m

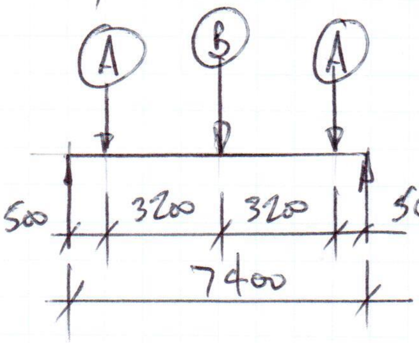
equiv to $180/(20 \times 6.8) = 1.33 \text{ kN/m}^2$

Dead load only call 1.5 kN/m^2.

At time of lifting - no imposed load either within unit or on roof (ie snow)

cross beam design

say c/c lifting points = 7.4m.



Point load 'A' from longitudinal beams
$≑ 1.5 \text{kN/m}^2 \times \frac{6.8\text{m}}{4} \times 3.0\text{m} ≑ \underline{8 \text{kN}}$ - DL

Point load 'B' from longitudinal beams
$≑ 1.5 \text{kN/m}^2 \times \frac{6.8\text{m}}{2} \times 3.0\text{m} ≑ \underline{16 \text{kN}}$ - DL

Using Tedds design package - s/wt will be allowed for within Tedds design

As previously stated - there is no imposed load to allow for plus only consider DL factor of say 1.1 & limit δ to ≑ 15mm

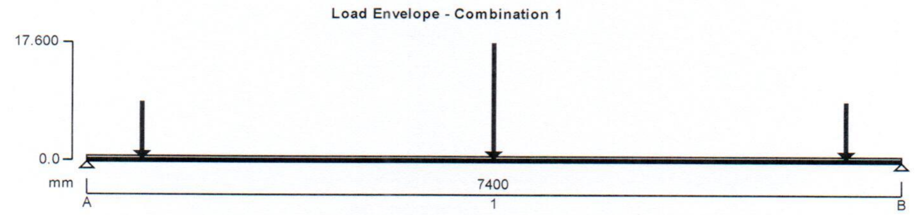
Cont'd sht 7.

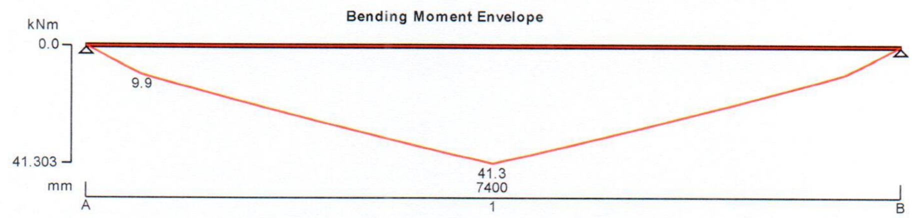
Tekla Tedds Rob Adaway Structural Solutions 26 Denington Road Wellingborough Northants	Project Temporary Unit Lifting Cradle				Job no. 17/1268	
	Calcs for Cross Beams				Start page no./Revision 4	
	Calcs by RA	Calcs date 10/08/2017	Checked by	Checked date	Approved by	Approved date

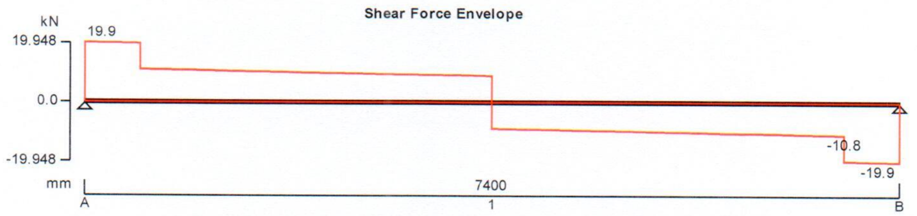
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead point load 8 kN at 500 mm
	Dead point load 16 kN at 3700 mm
	Dead point load 8 kN at 6900 mm

Load combinations

Load combination 1	Support A	Dead × 1.10
		Imposed × 1.00
	Span 1	Dead × 1.10
		Imposed × 1.00
	Support B	Dead × 1.10
		Imposed × 1.00

Analysis results

Maximum moment — M_{max} = **41.3** kNm — M_{min} = **0** kNm

Maximum shear — V_{max} = **19.9** kN — V_{min} = **-19.9** kN

Deflection — δ_{max} = **14.6** mm — δ_{min} = **0** mm

Maximum reaction at support A — R_{A_max} = **19.9** kN — R_{A_min} = **19.9** kN

Unfactored dead load reaction at support A — R_{A_Dead} = **18.1** kN

Maximum reaction at support B — R_{B_max} = **19.9** kN — R_{B_min} = **19.9** kN

Unfactored dead load reaction at support B — R_{B_Dead} = **18.1** kN

Section details

Section type — **RHS 250x150x10.0 (Tata Steel Celsius)**

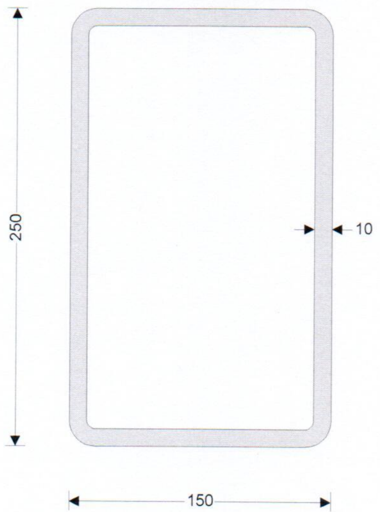
Steel grade — **S355**

From table 9: Design strength p_y

Thickness of element — t = **10.0** mm

Design strength — p_y = **355** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.20** + 2 × D

$K_{LT.B}$ = **1.20** + 2 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Web - major axis - Table 12

Depth of section — d= D - 3 × t = **220** mm

$d / t = 25.0 \times \varepsilon <= 64 \times \varepsilon$ — Class 1 plastic

Flange - major axis - Table 12

Width of section — b = B - 3 × t = **120** mm

$b / t = 13.6 \times \varepsilon <= \min(28 \times \varepsilon, 80 \times \varepsilon - d / t)$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{19.9}$ kN

$(D - 3 \times t) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area $A_v = A \times D / (D + B) = \mathbf{4683}$ mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v = \mathbf{997.5}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{41.3}$ kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{210.4}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.2 \times L_{s1} + 2 \times D = \mathbf{9380}$ mm

Slenderness ratio $\lambda = L_E / r_{yy} = \mathbf{154.693}$

Limiting slenderness ratio - Table 15 $435 \times (275 \text{ N/mm}^2 / p_y) = \mathbf{336.972}$

λ is less than limiting value, no allowance need be made for lateral-torsional buckling

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = \min(15 \text{ mm}, L_{s1} / 360) = \mathbf{15}$ mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{14.603}$ mm

PASS - Maximum deflection does not exceed deflection limit

M 0794 70 80 766 E info@robadaway.co.uk

Project No: 17/1268	Sheet No: 7
Made By: RA	Revision:
Date: AUG '17	Checked By:

Project: LIFTING CRADLE FOR SINGLE STOREY TEMPORARY UNIT

LONGITUDINAL BEAM DESIGN

APPROX SPAN ≑ 3.0M.

LOADING ON BEAM = $1.5\,kN/m^2 \times \frac{6.8m \text{ MAX}}{2} \doteqdot 6.0\,kN/m'$

DEAD LOAD ONLY

FOLLOWING TEDDS SHEETS 4 → 6 INDICATES THAT 250 x 150 x 10 THICK GRADE 355 ADEQUATE AS CROSS BEAMS WITHIN LIFTING CRADLE.

PLUS TEDDS SHEETS 8 → 10 INDICATES THAT 120 x 120 x 6.3 THICK SHS GRADE 355 ADEQUATE AS LONGITUDINAL BEAMS SUPPORTED BY CROSS BEAMS

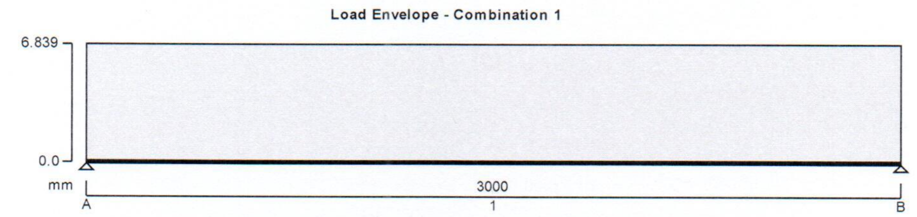
NOTE:- BEAM CONNECTIONS & LIFTING EYE DESIGN NOT PART OF RASS BRIEF.

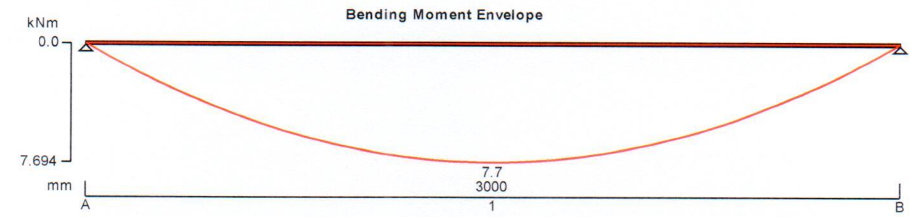
Tekla Tedds Rob Adaway Structural Solutions 26 Denington Road Wellingborough Northants	Project Temporary Unit Lifting Cradle				Job no. 17/1268	
	Calcs for Longitudinal Beams				Start page no./Revision 8	
	Calcs by RA	Calcs date 10/08/2017	Checked by	Checked date	Approved by	Approved date

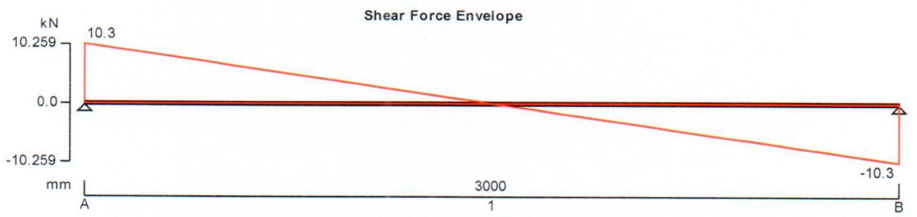
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.05







Support conditions

Support A	Vertically restrained
	Rotationally free
Support B	Vertically restrained
	Rotationally free

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 6 kN/m

Load combinations

Load combination 1	Support A	Dead × 1.10
		Imposed × 1.00
	Span 1	Dead × 1.10
		Imposed × 1.00
	Support B	Dead × 1.10
		Imposed × 1.00

Analysis results

Maximum moment	M_{max} = **7.7** kNm	M_{min} = **0** kNm

Maximum shear	V_{max} = **10.3** kN	V_{min} = **-10.3** kN
Deflection	δ_{max} = **5.3** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **10.3** kN	R_{A_min} = **10.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **9.3** kN	
Maximum reaction at support B	R_{B_max} = **10.3** kN	R_{B_min} = **10.3** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **9.3** kN	

Section details

Section type — **SHS 120x120x6.3 (Tata Steel Celsius)**

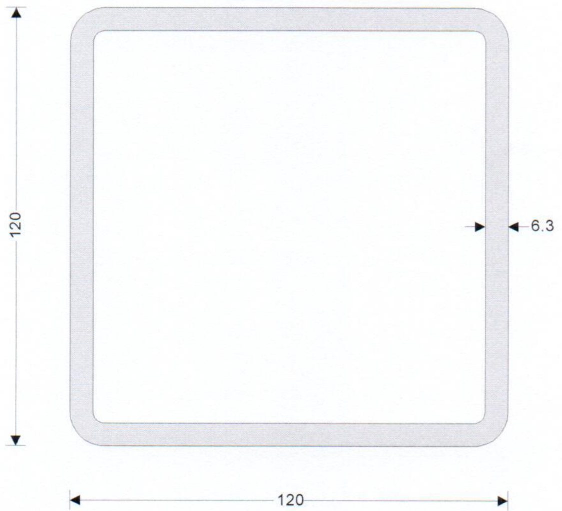
Steel grade — **S355**

From table 9: Design strength p_y

Thickness of element — t = **6.3** mm

Design strength — p_y = **355** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis — K_x = **1.00**

Effective length factor in minor axis — K_y = **1.00**

Effective length factor for lateral-torsional buckling — $K_{LT.A}$ = **1.20** + 2 × D

$K_{LT.B}$ = **1.20** + 2 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Web - major axis - Table 12

Depth of section — d= D - 3 × t = **101.1** mm

$d / t = 18.2 \times \varepsilon <= 64 \times \varepsilon$ — Class 1 plastic

Flange - major axis - Table 12

Width of section — b = B - 3 × t = **101.1** mm

$b / t = 18.2 \times \varepsilon <= \min(28 \times \varepsilon, 80 \times \varepsilon - d / t)$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{10.3}$ kN

$(D - 3 \times t) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = A \times D / (D + B) = \mathbf{1411}$ mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v = \mathbf{300.6}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{7.7}$ kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S, 1.2 \times p_y \times Z) = \mathbf{42.5}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.2 \times L_{s1} + 2 \times D = \mathbf{3840}$ mm

Slenderness ratio — $\lambda = L_E / r_{yy} = \mathbf{83.091}$

Equivalent slenderness - Annex B.2.6.1

Torsion constant — $J = \mathbf{9502289}$ mm^4

$\gamma_b = (1 - I_{yy} / I_{xx}) \times (1 - J / (2.6 \times I_{xx})) = \mathbf{0.000}$

$\phi_b = [S_{xx}^2 \times \gamma_b / (A \times J)]^{0.5} = \mathbf{0.000}$

Ratio - cl.4.3.6.9 — $\beta_W = \mathbf{1.000}$

Equivalent slenderness — $\lambda_{LT} = 2.25 \times \sqrt{[\phi_b \times \lambda \times \beta_W]} = \mathbf{0.000}$

Limiting slenderness - Annex B.2.2 — $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{30.198}$

$\lambda_{LT} < \lambda_{L0}$ - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength — $p_b = p_y = \mathbf{355}$ N/mm^2

Buckling resistance moment — $M_b = p_b \times S = \mathbf{42.5}$ kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 500 = \mathbf{6}$ mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{5.306}$ mm

PASS - Maximum deflection does not exceed deflection limit