AND DESIGNS LTD
90 MEADROW
GODALMING, SURREY, GU7 3HY
Tel. 01483 418 140
e-mail: info@anddesigns.co.uk

Project: 3 SOUTH TERRACE, SURBITON KT6 6HT				Job Ref. 23.030	
Part of Structure: LOADS				Sheet No./rev. 01	
Calc. by JAL	Date SEPT.2023	Chck'd by	Date	App'd by	Date

LOADINGS

Pitched Roof:		**Flat Roof:**	
Tiles	= 0.70	Chippings	= 0.20
Felt & battens	= 0.05	3 layer felt	= 0.10
Roof timber	= 0.15	Boards	= 0.15
Insulation & ceiling	= 0.20	Joists	= 0.15
	+ 1.10	Insulation + ceiling	= 0.20
			+ 0.80
I.L. Roof	= 0.75	I.L. Roof	= 0.75
I.L Loft	= 0.25		
	+ 2.10 kN/m²		+ 1.55 kN/m²
Upper Floors – Timber:		**Ground Floor:**	
Boards	= 0.15	50 screed	= 1.20
Joists	= 0.15	150 p.c. units	= 2.20
Ceiling	= 0.20		+ 3.40
	0.50		
		I.L. Floor	= 1.50
I.L Floor	+ 1.50	Partitions	= 1.30
	+ 2.00 kN/m²		+ 6.20 kN/m²
External walls:		**Stud partitions:**	
Brickwork	= 2.20		
Blockwork	= 1.00	Studs	= 0.10
Plaster	= 0.20	Plasterboard	= 0.30
	+ 3.40 kN/m²		+ 0.40 kN/m²
Block partitions:			
100 blocks	= 1.00	225 dense blocks	= 4.60
Plaster (both side)	= 0.40	Plaster (both sides)	= 0.40
	+ 1.40 kN/m²		+ 5.00 kN/m²
Upper Floors – PC Conc:		**Tile Hung Stud:**	
Screed	= 1.80	Tiles	= 0.70
PC Planks	= 3.0	Battens	= 0.05
Ceiling	= 0.2	12mm ply	= 0.10
	+ 5.00	50 x 150 Studs	= 0.10
		Plaster Bd + insul.	=0.20
I.L Floor	= 1.50		
Partitions	= 1.30		+ 1.15 kN/m²
	+ 7.80 kN/m²		

Steel design to	B.S. 5950	225 th brick walls	5.0 kN/m²
Concrete design to	B.S. 8110	330 th brick walls	7.5 kN/m²
Masonry design to	B.S. 5628	450 th brick walls	10.0 kN/m²
Timber design to	B.S. 5268		

Foundations are designed for a maximum ground bearing capacity of 100 kN/m², which is to be verified on site.

The Institution of StructuralEngineers

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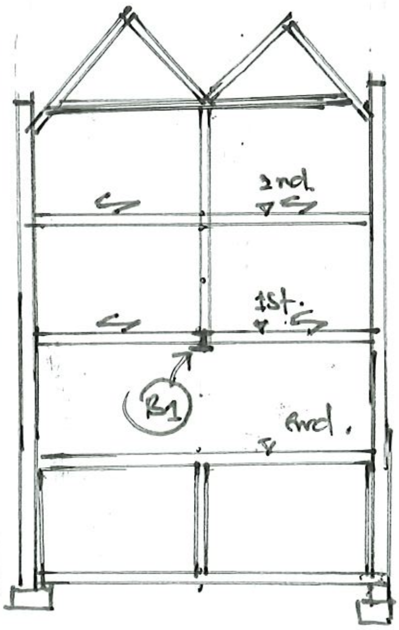
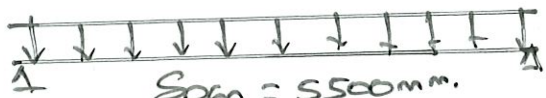
Design of Beam (B1) at 1st Floor Level Span = 5400mm.

Loadings



	DL (kN/m)	IL (kN/m)
Roof Load $= \frac{1}{2} \times 9 \times (1.1 + 0.75)$ (DL, LL) + 1×0.75 ← Additional Snow.	5.0	4.1
Storage $= \frac{1}{2} \times 8 \times (0.5 + 0.25)$	2.0	1.0.
1st & 2nd floor walls $= (3 + 2.8) \times 0.6$. ← Timber studs	3.5	
1st & 2nd Floor $= \frac{1}{2} \times 9 \times (0.5 + 1.5) \times 2$.	2.3	6.8
Total Load	12.8	11.9.



Span = 5500mm.

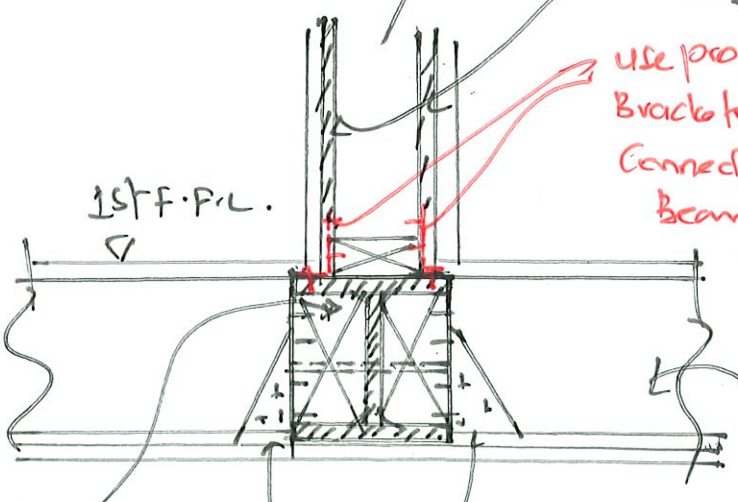
Section.

Existing studs

Allow 12mm ply glued and screwed

use proprietary Angle Brackets to have mechanical Connection between steel Beam and stud wall/plate. @450c/c

1st F.F.L.



Existing Floor joists T.B.C

Timber packing bolted with M12 bolts @600c/c

Simpsons Timber joist hangers.

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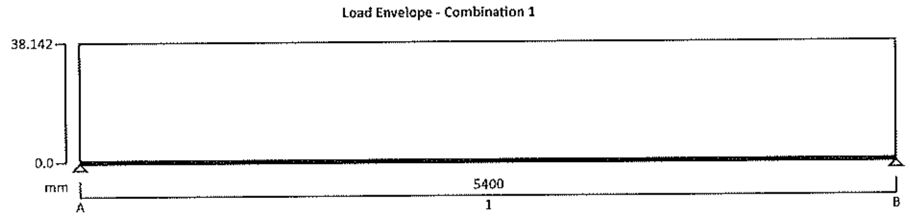
	Section BEAM B1				Sheet no./rev. 3	
	Calc. by JAL	Date	Chk'd by	Date	App'd by	Date

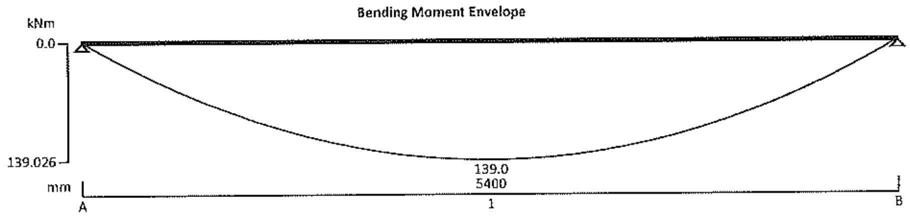
STEEL BEAM ANALYSIS & DESIGN (BS5950)

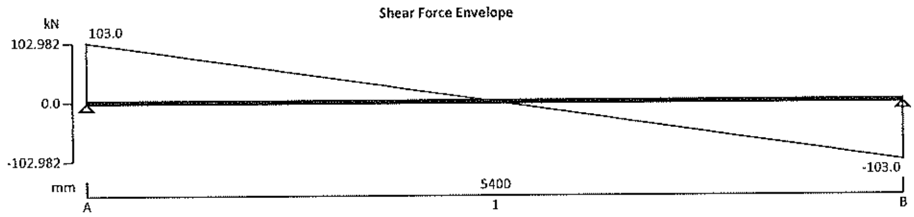
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







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 12.8 kN/m
	Imposed full UDL 11.9 kN/m

Load combinations

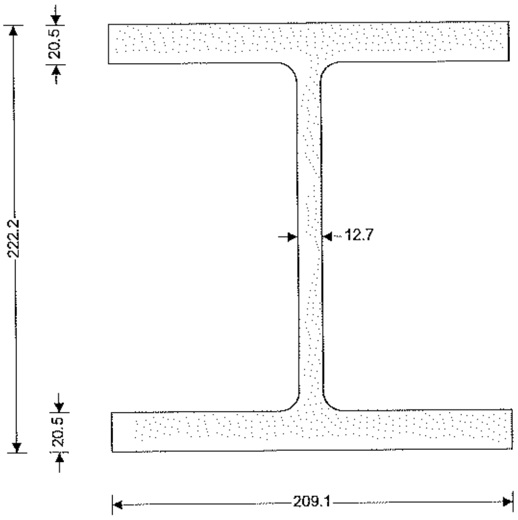
Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40
		Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **139** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **103** kN	V_{min} = **-103** kN
Deflection	δ_{max} = **14.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **103** kN	R_{A_min} = **103** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **36.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **32.1** kN	
Maximum reaction at support B	R_{B_max} = **103** kN	R_{B_min} = **103** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **36.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **32.1** kN	

Section details

Section type **UC 203x203x86 (BS4-1)** Steel grade **S275**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **1.02** Section classification **Plastic**

Shear capacity - Section 4.2.3

Design shear force F_v = **103** kN Design shear resistance P_v = **448.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment M = **139** kNm Moment capacity low shear M_c = **258.8** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **194.3** kNm M_b / m_{LT} = **210.1** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection δ_{lim} = **15** mm Maximum deflection δ = **14.601** mm

PASS - Maximum deflection does not exceed deflection limit

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Design of Beam (B2) Span = 2000mm.

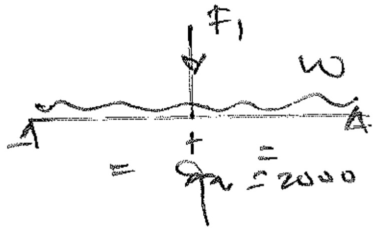
Load from wall over (timber) = 5.8 x 0.6 - 20% opening = 2.8 kN/m.

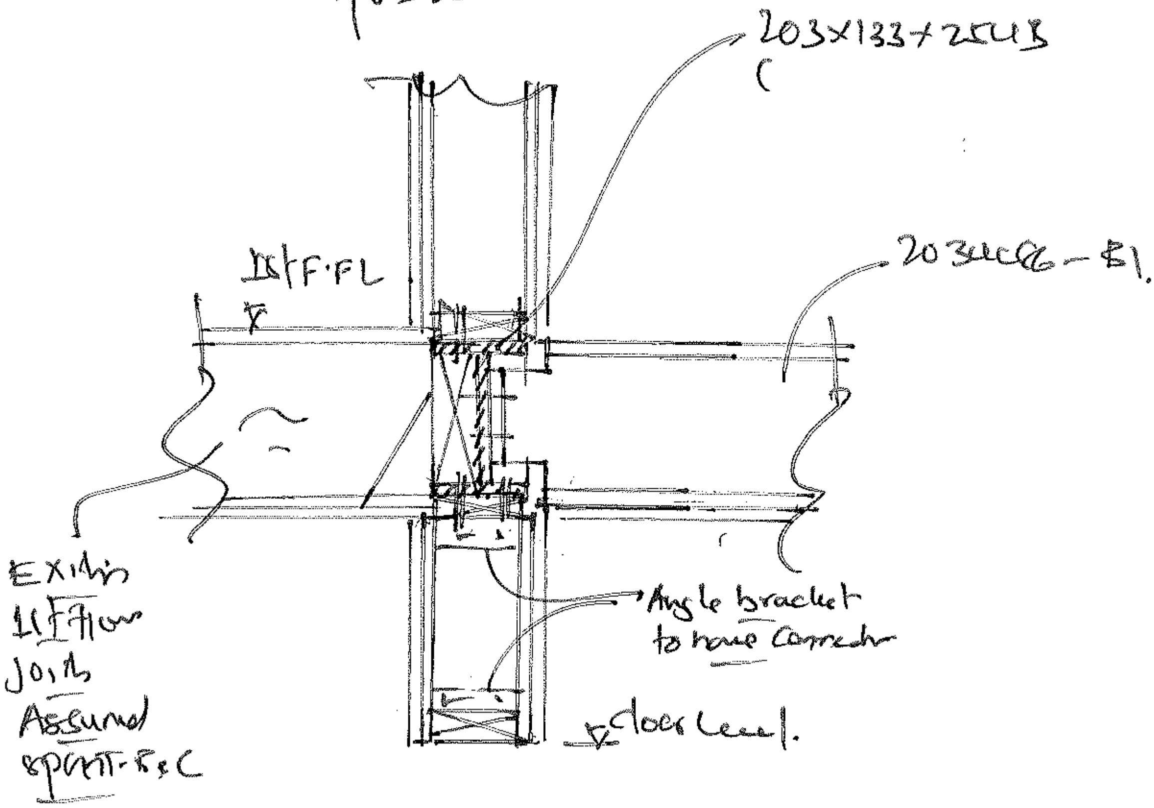
part Roof Load. = 1.2 x (1.1 + 0.75) 2(1.3 + 0.9) kN/m.

1st & 2nd floor (part) = 1.0 x (0.5 + 1.5) x 2 = (1 + 3) kN/m,

Total UDL = (5.1 + 3.9) kN/m = w

pt Load from (B1) = (36.8 + 32.1) kN - F1



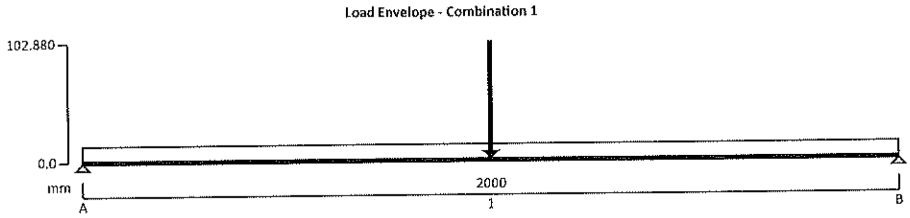


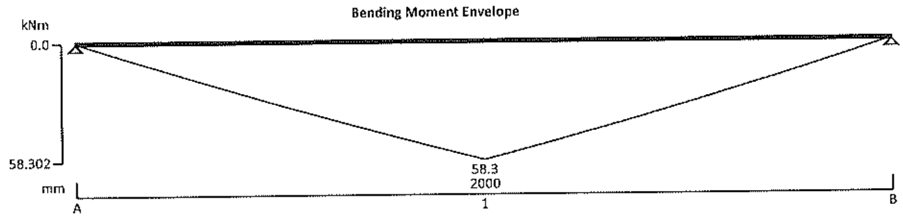
STEEL BEAM ANALYSIS & DESIGN (BS5950)

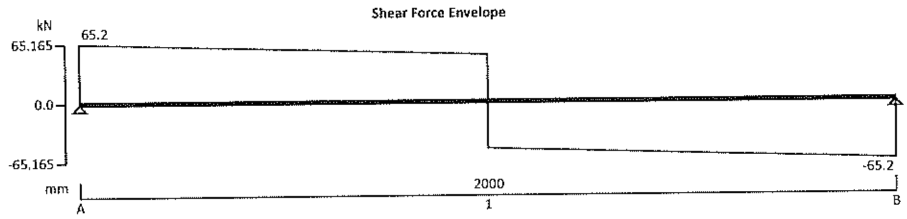
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







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 5.1 kN/m Imposed full UDL 3.9 kN/m Dead point load 36.8 kN at 1000 mm Imposed point load 32.1 kN at 1000 mm

Load combinations

Load combination 1	Support A	Dead × 1.40 Imposed × 1.60
		Dead × 1.40 Imposed × 1.60
	Support B	Dead × 1.40

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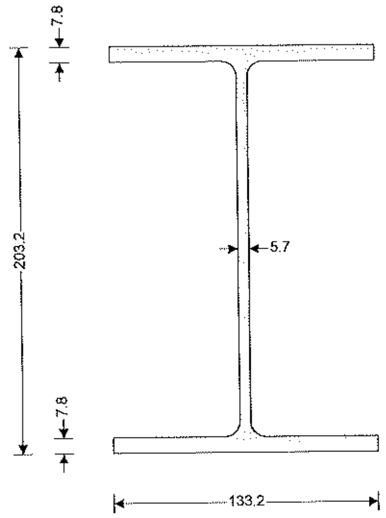
Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **58.3** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **58.3** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **58.3** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum shear	V_{max} = **65.2** kN	V_{min} = **-65.2** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **65.2** kN	$V_{s1\_seg1\_min}$ = **-51.4** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **0** kN	$V_{s1\_seg2\_min}$ = **-65.2** kN
Deflection segment 3	δ_{max} = **2.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **65.2** kN	R_{A_min} = **65.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **23.7** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **20** kN	
Maximum reaction at support B	R_{B_max} = **65.2** kN	R_{B_min} = **65.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **23.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **20** kN	

Section details

Section type **UB 203x133x25 (BS4-1)** Steel grade **S355**



Classification of cross sections - Section 3.5

Tensile strain coefficient ε = **0.88** Section classification **Compact**

Shear capacity - Section 4.2.3

Design shear force F_v = **65.2** kN Design shear resistance P_v = **246.7** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 1 - Section 4.2.5

Design bending moment M = **58.3** kNm Moment capacity low shear M_c = **91.5** kNm

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment M_b = **87.2** kNm M_b / m_{LT} = **140.4** 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 δ_{lim} = **5.556** mm Maximum deflection δ = **2.795** mm

PASS - Maximum deflection does not exceed deflection limit

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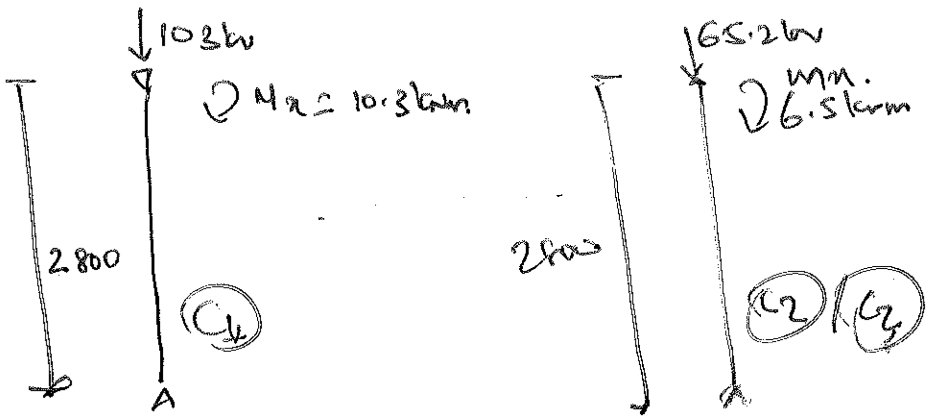
Design of Col^ns C1, C2 & C3

ULS.

Col^n C1 ⇒ Reac^n for C6 B1 = 103 kN (68.9) SLS

Col^n C2/C3 ⇒ Reac^n B2 = 65.2 kN (43.7) kN SLS



Considering Lower Ground Floor wall 215 thick solid wall on which C1, C2 & C3 are supported.

$\sigma = \frac{1.5 f_k}{\gamma_m}$ ⇒ $f_k = 2.2\ N/mm^2$ ⇒ $\sigma = 0.94\ N/mm^2$.

⇒ For Col^n C1 Required spreader length $= \frac{103 \times 10^3}{215 \times 0.94} = 509\ mm$.

Use 600mm long 203 UC 46 spreader beam

For Col^n C2/C3 – Required spreader length $= \frac{65.2 \times 10^3}{215 \times 0.94} = 322$.

⇒ use 440 long × 215 wide and 215 deep RC pad st.

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	Section COLUMN C1				Sheet no./rev. 9	
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STEEL MEMBER DESIGN (BS5950)

Member design checks for a steel member to BS 5950:2000

Section properties

Try **"UC 152x152x30"** section.

D = **158** mm B = **153** mm T = **9.4** mm t = **6.5** mm

d = **123.6** mm b = B / 2 = **76.5** mm u = **0.8485** x = **16.00**

A_g = A = **38.3** cm^2 I_x = **1748** cm^4 I_y = **560** cm^4 r_x = **6.76** cm r_y = **3.83** cm

S_x = **247.7** cm^3 S_y = **111.6** cm^3 Z_x = **221.8** cm^3 Z_y = **73.3** cm^3

$A_{vy} = t \times D$ = **10.2** cm^2 $A_{vx} = 0.9 \times 2 \times T \times B$ = **25.9** cm^2 $d_x = B$ $d_y = d$

$S_{vx} = t \times D^2 / 4$ = **40.4** cm^3

Strut curve (b) for x-axis, curve (c) for y-axis. (Tables 23 & 24) $z_1 = 2.0$ $z_2 = 1.0$

Steel grade **"S355"** p_y = **355** N/mm^2 $p_{yw} = p_y$ $\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)}$ = **0.880** K_e = **1.1**

Cl. 3.1.1 & 3.4.3

Geometry

L = **2800** mm

L_{LT} = **2800** mm K_{LT} = **1.50** $L_{E\_LT} = L_{LT} \times K_{LT}$ = **4200** mm

L_x = **2800** mm K_x = **1.50** $L_{Ex} = L_x \times K_x$ = **4200** mm

L_y = **2800** mm K_y = **1.50** $L_{Ey} = L_y \times K_y$ = **4200** mm

Cl. 4.3.5 & 4.7.3

Loading

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

F_t = 0 kN F_c = **103.00** kN $n = F_c / (A \times p_y)$ = **0.076**

F_{vy} = **5.0** kN

F_{vx} = 0 kN

M_x = **10.30** kNm

M_y = 0 kNm

Moments for member buckling check

M_{LT} = **10.30** kNm

M_X = **10.30** kNm

M_Y = **0.00** kNm

Cl. 4.8.3.3

Equivalent uniform moment factors

m_{LT} = **1.000**

Cl. 4.3.6.6 & T18

m_x = **1.000**

Cl. 4.8.3.3.4 & T26

m_y = 1.0

Cl. 4.8.3.3.4 & T26

Section classification

b / T = **8.1** d / t = **19.0**

$r_1 = \min(1.0, \max(-1.0, F_c / (d \times t \times p_{yw})))$ = **0.361** $r_2 = F_c / (A_g \times p_{yw})$ = **0.076**

Section classification is Compact

Cl. 3.5.1 - 3.5.5

Shear capacity (parallel to x-axis)

F_{vx} = **0.0** kN $P_{vx} = 0.6 \times p_y \times A_{vx}$ = **551.0** kN abs(F_{vx}) / P_{vx} = **0.00**

Check $F_{vx} \leq P_{vx}$ Pass - Shear
Cl. 4.2.3

Shear capacity (parallel to y-axis)

F_{vy} = **5.0** kN $P_{vy} = 0.6 \times p_y \times A_{vy}$ = **218.2** kN abs(F_{vy}) / P_{vy} = **0.02**

Check $F_{vy} \leq P_{vy}$ Pass - Shear
Cl. 4.2.3

Moment capacity (x-axis)

F_{vy} / P_{vy} = **0.02**

M_{cxu} = if(or(Class == "Plastic", Class == "Compact", Class == "Not Reqd."), $p_y \times S_x$,
if(Class == "Semi-Compact", $p_y \times Z_x$, 0 kNm)) = **87.9** kNm

M_{cx} = min(M_{cxu} , if(BeamType == "general", $1.5 \times p_y \times Z_x$, $1.2 \times p_y \times Z_x$)) = **87.9** kNm

M_x = **10.3** kNm

Check $M_x \leq M_{cx}$ Pass - Moment
Cl. 4.2.5.1

Moment capacity (y-axis)

M_{cyL} = if(or(Class == "Plastic", Class == "Compact", Class == "Not Reqd."), $p_y \times S_y$,
if(Class == "Semi-Compact", $p_y \times Z_y$, 0 kNm)) = **39.6** kNm

M_{cyH} = 0.001 kNm

M_{cyu} = if(F_{vx} / P_{vx} < 0.6, M_{cyL}, M_{cyH}) = **39.6** kNm

M_{cy} = min(M_{cyu} , if(BeamType == "general", $1.5 \times p_y \times Z_y$, $1.2 \times p_y \times Z_y$)) = **31.2** kNm

M_y = **0.0** kNm

Check $M_y \leq M_{cy}$ Pass - Moment
Cl. 4.2.5.1

Lateral torsional buckling

L_{E_LT} = **4200** mm $\lambda = L_{E\_LT} / r_y$ =**110** u = **0.849** η = 0.5 x = **16.0**

$v = 1 / (1 + 0.05 \times (\lambda / x)^2)^{0.25}$ = **0.74**

β_w = if(or(Class == "Plastic", Class == "Compact"), 1.0,
if(Class == "Semi-Compact", Z_x / S_x, 1.0)) = **1.000**

$\lambda_{LT} = u \times v \times \lambda \times \sqrt{(\beta_w)}$ = **69** $\lambda_{L0} = 0.4 \times \sqrt{(\pi^2 \times E_{S5950} / p_y)}$ = **30**

$p_E = \pi^2 \times E_{S5950} / \lambda_{LT}^2$ = **427** N/mm² η_{LT} = max($7.0 \times (\lambda_{LT} - \lambda_{L0}) / 1000$, 0)

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ $p_b = p_E \times p_y / (\phi_{LT} + \sqrt{(\phi_{LT}^2 - p_E \times p_y)})$ = **226** N/mm²

M_b = if(or(Class == "Plastic", Class == "Compact"), $p_b \times S_x$,
if(Class == "Semi-Compact", $p_b \times Z_x$, 0 kNm)) = **55.9** kNm

M_{LT} = **10.3** kNm m_{LT} = **1.00** M_b / m_{LT} = **55.9** kNm

Check $M_{LT} \leq M_b / m_{LT}$ Pass - lat. tors. buckling
Cl. 4.3.6.2 & Annex B

Compression resistance - strut buckling about x-axis

F_c = **103.0** kN L_{Ex} = **4.20** m $\lambda_x = L_{Ex} / r_x$ = **62**

Strut curve (b) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_x = **3.5**

η_x = max(0, $a_x \times (\lambda_x - \lambda_0) / 1000$) = **0.165** $p_{ex} = \pi^2 \times E_{S5950} / \lambda_x^2$ = **524** N/mm²

$\phi_x = (p_y + (\eta_x + 1) \times p_{ex}) / 2$ $p_{cx} = p_{ex} \times p_y / (\phi_x + (\phi_x^2 - p_{ex} \times p_y)^{0.5})$ = **266** N/mm²

p_{cx} = **266** N/mm² $P_{cx} = A \times p_{cx}$ = **1017.9** kN

Check $F_c \leq P_{cx}$ Pass - Compression
Slenderness less than 180
Cl. 4.7.4 & Annex C

Compression resistance - strut buckling about y axis

F_c = **103.0** kN L_{Ey} = **4.20** m $\lambda_y = L_{Ey} / r_y$ = **110**

Strut curve (c) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_y = **5.5**

$\eta_y = \max(0, a_y \times (\lambda_y - \lambda_0) / 1000)$ = **0.521** $p_{ey} = \pi^2 \times E_{S5950} / \lambda_y^2$ = **168** N/mm^2

$\phi_y = (p_y + (\eta_y + 1) \times p_{ey}) / 2$ $p_{cy} = p_{ey} \times p_y / (\phi_y + (\phi_y^2 - p_{ey} \times p_y)^{0.5})$ = **122** N/mm^2

$P_{cy} = A \times p_{cy}$ = **467.3** kN

Check $F_c \leq P_{cy}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Compression member with moments, local capacity (simplified method)

F_{vx} / P_{vx} = **0.00** F_{vy} / P_{vy} = **0.02** **Cross-section capacity not affected by shear**

$F_c / (A_g \times p_y) + M_x / M_{cx} + M_y / M_{cy}$ = **0.19**

Pass - Combined compression & moment, local capacity

Cl. 4.8.3.2

Member buckling resistance (simplified method)

M_X = **10.30** kNm m_x = **1.00** M_Y = **0.00** kNm m_y = **1.00**

$P_c = \min(P_{cx}, P_{cy})$ = **467.3** kN

$F_c / P_c + m_x \times M_X / (p_y \times Z_x) + m_y \times M_Y / (p_y \times Z_y)$ = **0.35**

$F_c / P_{cy} + m_{LT} \times M_{LT} / M_b + m_y \times M_Y / (p_y \times Z_y)$ = **0.40**

Pass - Member buckling resistance

Cl. 4.8.3.3.1

Results summary

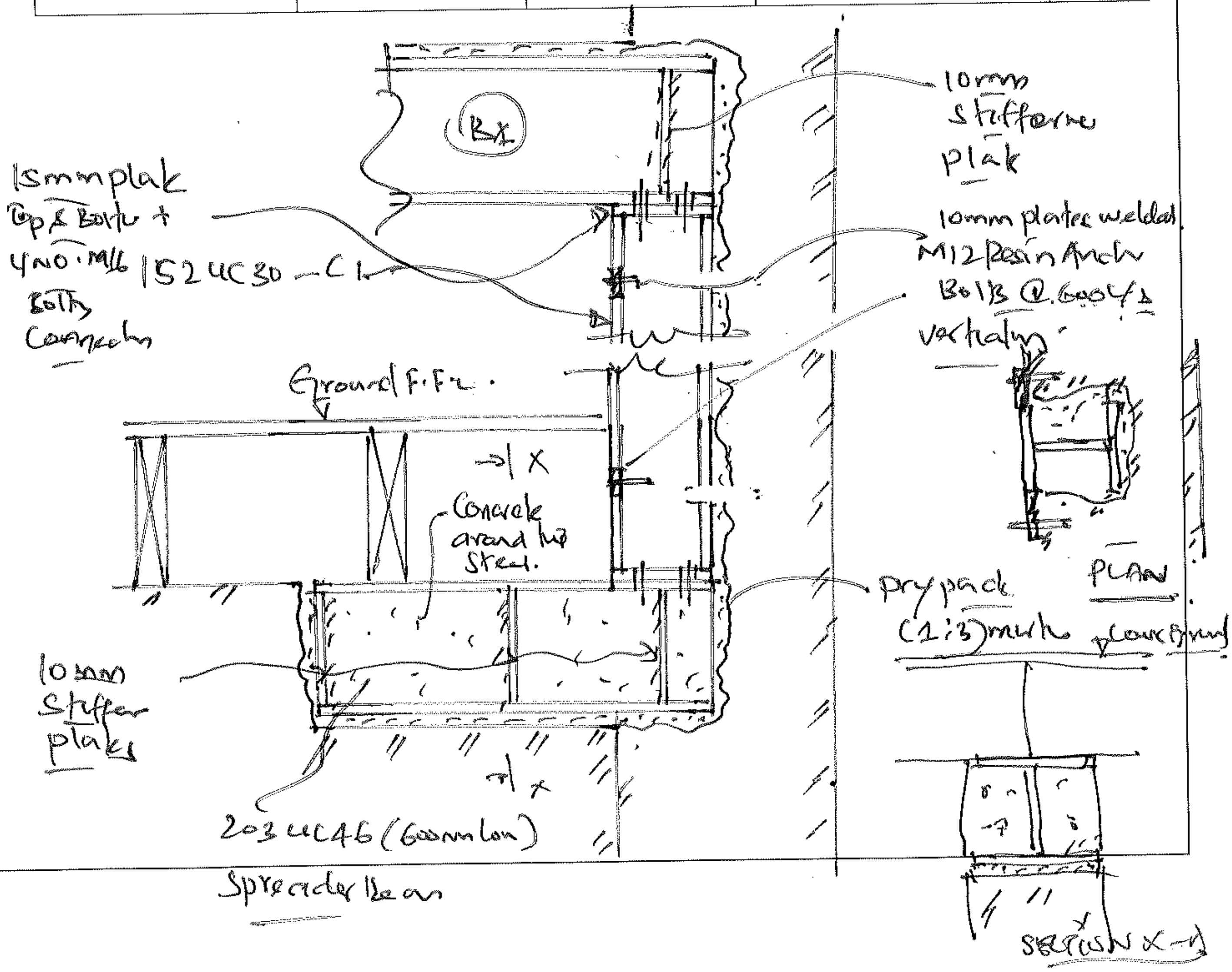
See table overleaf.

AND DESIGNS LTD AND DESIGNS LTD 90 MEADROW, GODALMING SURREY GU7 3HY Tel. No. 01483418140 email:lawrence@anddesigns.co.uk	Project: 3 SOUTH TERRACE, SURBITON KT6 6HT	Job Ref.: 23.030
	Section: COLUMN C1	Sheet no./rev.: 12
	Calc. by: JAL · Date: · Chk'd by: · Date:	App'd by: · Date:

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm^2	
Section	**"UC 152x152x30"**	Classification	**"Compact"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Shear	F_{vy} = **5.0** kN	P_{vy} = **218.2** kN	**Low shear**	**Pass**
Moment	M_x = **10.3** kNm	M_{cx} = **87.9** kNm	**Low shear**	**Pass**
LTB	M_{LT} = **10.3** kNm	M_b / m_{LT} = **55.9** kNm	L_{E_LT} = **4.2** m m_{LT} = **1.00**	**Pass**
Strut buckling (x-axis)	F_c = **103.0** kN	P_{cx} = **1017.9** kN	L_{Ex} = **4.2** m **Slenderness < 180**	**Pass**
Strut buckling (y-axis)	F_c = **103.0** kN	P_{cy} = **467.3** kN	L_{Ey} = **4.2** m **Slenderness < 180**	**Pass**
Compr & moment local	Index **0.19**	Limit = 1.00	**Simplified method. Low shear.**	**Pass**
Member buckling	Index = **0.35** Index (LT) = **0.40**	Limit = 1.0 Limit = 1.0	Simplified method M_x = **10** kNm m_x = **1.00** M_Y = **0** kNm m_y = **1.00**	**Pass**



STEEL MEMBER DESIGN (BS5950)

Member design checks for a steel member to BS 5950:2000

Section properties

Try **"SHS 100x100x10.0"** section.

$D = \mathbf{100}$ mm $B = \mathbf{100}$ mm $T = t = \mathbf{10.0}$ mm $A_g = A = \mathbf{34.9}$ cm^2

$d_x = D - 3 \times t = \mathbf{70.0}$ mm $b_x = B - 3 \times t = \mathbf{70.0}$ mm $b_y = d_x$ $d_y = b_x$

$A_{vy} = A \times D / (D + B) = \mathbf{17.5}$ cm^2 $A_{vx} = A \times B / (D + B) = \mathbf{17.5}$ cm^2

$I_x = \mathbf{462}$ cm^4 $I_y = \mathbf{462}$ cm^4 $r_x = \mathbf{3.64}$ cm $r_y = \mathbf{3.64}$ cm

$S_x = \mathbf{116.2}$ cm^3 $S_y = \mathbf{116.2}$ cm^3 $Z_x = \mathbf{92.4}$ cm^3 $Z_y = \mathbf{92.4}$ cm^3

$d_{vx} = A_{vx} / (2 \times t) = \mathbf{87.3}$ mm $d_{vy} = A_{vy} / (2 \times t) = \mathbf{87.3}$ mm

$S_{vx} = 2 \times t \times d_{vy}^2 / 4 = \mathbf{38.1}$ cm^3 $S_{vy} = 2 \times t \times d_{vx}^2 / 4 = \mathbf{38.1}$ cm^3

Strut curve (a) for x-axis and y-axis. (Tables 23 & 24) $z_1 = 5/3$ $z_2 = 5/3$

Steel grade **"S355"** $p_y = \mathbf{355}$ N/mm^2 $p_{yw} = p_y$ $\varepsilon = \sqrt{(275 \text{ N/mm}^2 / p_y)} = \mathbf{0.880}$ $K_e = \mathbf{1.1}$

Cl. 3.1.1 & 3.4.3

Geometry

$L_x = \mathbf{2800}$ mm $K_x = \mathbf{1.50}$ $L_{Ex} = L_x \times K_x = \mathbf{4200}$ mm

$L_y = \mathbf{2800}$ mm $K_y = \mathbf{1.50}$ $L_{Ey} = L_y \times K_y = \mathbf{4200}$ mm

Cl. 4.3.5 & 4.7.3

Loading

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

$F_t = 0$ kN $F_c = \mathbf{65.20}$ kN $n = F_c / (A \times p_y) = \mathbf{0.053}$

$F_{vy} = \mathbf{5.0}$ kN

$F_{vx} = 0$ kN

$M_x = \mathbf{6.50}$ kNm

$M_y = 0$ kNm

Moments for member buckling check

$M_{LT} = \mathbf{6.50}$ kNm

$M_X = \mathbf{6.50}$ kNm

$M_Y = \mathbf{0.00}$ kNm

Cl. 4.8.3.3

Equivalent uniform moment factors

$m_{LT} = \mathbf{1.000}$

Cl. 4.3.6.6 & T18

$m_x = \mathbf{1.000}$

Cl. 4.8.3.3.4 & T26

$m_y = 1.0$

Cl. 4.8.3.3.4 & T26

Section classification

$b_x / t = \mathbf{7.0}$ $d_x / t = \mathbf{7.0}$ $b_y / t = \mathbf{7.0}$ $d_y / t = \mathbf{7.0}$

$r_{1x} = \min(1.0, \max(-1.0, F_c / (2 \times d_x \times t \times p_{yw}))) = \mathbf{0.131}$

$r_{1y} = \min(1.0, \max(-1.0, F_c / (2 \times d_y \times t \times p_{yw}))) = \mathbf{0.131}$

$r_2 = F_c / (A_g \times p_{yw}) = \mathbf{0.053}$

Section classification is Plastic

Cl. 3.5.1 - 3.5.5

Shear capacity (parallel to x-axis)

$F_{vx} = \mathbf{0.0}$ kN $P_{vx} = 0.6 \times p_y \times A_{vx} = \mathbf{372.0}$ kN $\text{abs}(F_{vx}) / P_{vx} = \mathbf{0.00}$

Check $F_{vx} \leq P_{vx}$ Pass - Shear

Cl. 4.2.3

Shear capacity (parallel to y-axis)

$F_{vy} = \mathbf{5.0}$ kN $P_{vy} = 0.6 \times p_y \times A_{vy} = \mathbf{372.0}$ kN $\text{abs}(F_{vy}) / P_{vy} = \mathbf{0.01}$

Check $F_{vy} \leq P_{vy}$ Pass - Shear

Cl. 4.2.3

Moment capacity (x-axis)

$F_{vy} / P_{vy} = \mathbf{0.01}$

$M_{cxu} =$ if(or(Class == "Plastic", Class == "Compact", Class == "Not Reqd."), $p_y \times S_x$,
if(Class == "Semi-Compact", $p_y \times Z_x$, 0 kNm)) = **41.3** kNm

$M_{cx} = \min(M_{cxu}$, if(BeamType == "general", $1.5 \times p_y \times Z_x$, $1.2 \times p_y \times Z_x$)) = **39.4** kNm

$M_x = \mathbf{6.5}$ kNm

Check $M_x \leq M_{cx}$ Pass - Moment

Cl. 4.2.5.1

Moment capacity (y-axis)

$M_{cyL} =$ if(or(Class == "Plastic", Class == "Compact", Class == "Not Reqd."), $p_y \times S_y$,
if(Class == "Semi-Compact", $p_y \times Z_y$, 0 kNm)) = **41.3** kNm

$M_{cyH} = 0.001$ kNm

$M_{cyu} =$ if($F_{vx} / P_{vx} < 0.6$, M_{cyL}, M_{cyH}) = **41.3** kNm

$M_{cy} = \min(M_{cyu}$, if(BeamType == "general", $1.5 \times p_y \times Z_y$, $1.2 \times p_y \times Z_y$)) = **39.4** kNm

$M_y = \mathbf{0.0}$ kNm

Check $M_y \leq M_{cy}$ Pass - Moment

Cl. 4.2.5.1

Lateral torsional buckling

LT buckling check not required for this section (cl. 4.6.3.1)

$M_b = M_{cx} = \mathbf{39.4}$ kNm

LT buckling check not required for this section

Cl. 4.3.6.1

Compression resistance - strut buckling about x-axis

$F_c = \mathbf{65.2}$ kN $L_{Ex} = \mathbf{4.20}$ m $\lambda_x = L_{Ex} / r_x = \mathbf{115}$

Strut curve (a) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5} = \mathbf{15}$ $a_x = \mathbf{2.0}$

$\eta_x = \max(0, a_x \times (\lambda_x - \lambda_0) / 1000) = \mathbf{0.201}$ $p_{ex} = \pi^2 \times E_{S5950} / \lambda_x^2 = \mathbf{152}$ N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{ex}) / 2$ $p_{cx} = p_{ex} \times p_y / (\phi_x + (\phi_x^2 - p_{ex} \times p_y)^{0.5}) = \mathbf{133}$ N/mm^2

$p_{cx} = \mathbf{133}$ N/mm^2 $P_{cx} = A \times p_{cx} = \mathbf{466.0}$ kN

Check $F_c \leq P_{cx}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Compression resistance - strut buckling about y axis

F_c = **65.2** kN L_{Ey} = **4.20** m $\lambda_y = L_{Ey} / r_y$ = **115**

Strut curve (a) applies $\lambda_0 = 0.2 \times (\pi^2 \times E_{S5950} / p_y)^{0.5}$ = **15** a_y = **2.0**

$\eta_y = \max(0, a_y \times (\lambda_y - \lambda_0) / 1000)$ = **0.201** $p_{ey} = \pi^2 \times E_{S5950} / \lambda_y^2$ = **152** N/mm²

$\phi_y = (p_y + (\eta_y + 1) \times p_{ey}) / 2$ $p_{cy} = p_{ey} \times p_y / (\phi_y + (\phi_y^2 - p_{ey} \times p_y)^{0.5})$ = **133** N/mm²

$P_{cy} = A \times p_{cy}$ = **466.0** kN

Check $F_c \le P_{cy}$ Pass - Compression

Slenderness less than 180

Cl. 4.7.4 & Annex C

Compression member with moments, local capacity (simplified method)

F_{vx} / P_{vx} = **0.00** F_{vy} / P_{vy} = **0.01** **Cross-section capacity not affected by shear**

$F_c / (A_g \times p_y) + M_x / M_{cx} + M_y / M_{cy}$ = **0.22**

Pass - Combined compression & moment, local capacity

Cl. 4.8.3.2

Member buckling resistance (simplified method)

M_X = **6.50** kNm m_x = **1.00** M_Y = **0.00** kNm m_y = **1.00**

$P_c = \min(P_{cx}, P_{cy})$ = **466.0** kN

$F_c / P_c + m_x \times M_X / (p_y \times Z_x) + m_y \times M_Y / (p_y \times Z_y)$ = **0.34**

$F_c / P_{cy} + m_{LT} \times M_{LT} / M_b + m_y \times M_Y / (p_y \times Z_y)$ = **0.31**

Pass - Member buckling resistance

Cl. 4.8.3.3.1

Results summary

See table overleaf.

AND DESIGNS LTD 90 MEADROW, GODALMING SURREY GU7 3HY Tel. No. 01483418140 email:lawrence@anddesigns.co.uk	Project 3 SOUTH TERRACE, SURBITON KT6 6HT				Job Ref. 23.030	
	Section COLUMNS C2&C3				Sheet no./rev. 16	
	Calc. by JAL	Date	Chk'd by	Date	App'd by	Date

Summary of results

Material	Grade = **"S355"**	p_y = **355** N/mm^2	
Section	**"SHS 100x100x10.0"**	Classification	**"Plastic"**

Check	*Load*	*Capacity*	*Notes*	*Result*
Shear	F_{vy} = **5.0** kN	P_{vy} = **372.0** kN	**Low shear**	**Pass**
Moment	M_x = **6.5** kNm	M_{cx} = **39.4** kNm	**Serviceability governs**	**Pass**
LTB		M_b equal to M_{cx}	LTB check not req'd	**Pass**
Strut buckling (x-axis)	F_c = **65.2** kN	P_{cx} = **466.0** kN	L_{Ex} = **4.2** m **Slenderness < 180**	**Pass**
Strut buckling (y-axis)	F_c = **65.2** kN	P_{cy} = **466.0** kN	L_{Ey} = **4.2** m **Slenderness < 180**	**Pass**
Compr & moment local	Index **0.22**	Limit = 1.00	**Simplified method.** **Low shear.**	**Pass**
Member buckling	Index = **0.34** Index (LT) = **0.31**	Limit = 1.0 Limit = 1.0	Simplified method M_X = **7** kNm m_x = **1.00** M_Y = **0** kNm m_y = **1.00**	**Pass**