External Stairs
Structural Calculations

Calculations for Balcony Structure

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Revision: 0 - Initial issue
Date: 09/03/2023
Reference: G0738-1

Contents

Loadings

Load	Type		SLS
• **Flooring DL**		**Total**	**0.5**
Steel	Dead		0.5
• **Floor LL**	**Imposed**		**4**
• **Balustrade DL**		**Total**	**0.5**
Dead	Dead		0.5
• **Dead DL**		**Total**	**1**
Dead	Dead		1
• **Live LL**		**Total**	**1**
Live	Imposed		1
• **Factored LL**		**Total**	**1**
Live	Imposed		1

The following design codes are used or referred to in these calculations;

BS EN 1991-1-1:2002 Eurocode 1: Actions on Structures (and UK National annex)
BS EN 1993-1-1:2005 Eurocode 3: Design of Steel Structures (and UK National annex)

<u>Stability</u>

The structure is floor-mounted and flat and is therefore inherently stable.

<u>Execution Class</u>

The steelwork will be produced in accordance with Execution Class 2.

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Component Loads

Part	Loading DL/LL	TYPE	SOURCE	DL	LL	Total loads on each part (UDL only) DL	LL
Tread	DL	UDL	Flooring	0.5 x 0.3 = 0.15 kN/m		0.15 kN/m	
	DL	PL	Live	3kN at 0.425m			
Stringer	DL	UDL	Flooring	0.5 x 0.425 = 0.21 kN/m		0.76 kN/m	1.7 kN/m
	LL	UDL	Floor		4 x 0.425 = 1.7 kN/m		
	DL	UDL	Balustrade	0.5 x 1.1 = 0.55 kN/m			
Landing 2	DL	UDL	Flooring	0.5 x 0.4 = 0.2 kN/m		0.2 kN/m	1.6 kN/m
	LL	UDL	Floor		4 x 0.4 = 1.6 kN/m		
	DL	PL	Dead	1.81kN at 0m			
	LL	PL	Live		3.23kN at 0m		
	DL	PL	Dead	1.81kN at 0.85m			
	LL	PL	Live		3.23kN at 0.85m		
	DL	PL	Dead	1.81kN at 0.95m			
	LL	PL	Live		3.23kN at 0.95m		
	DL	PL	Dead	1.81kN at 1.8m			
	LL	PL	Live		3.23kN at 1.8m		
Landing 1	DL	UDL	Balustrade	0.5 x 1.1 = 0.55 kN/m		0.55 kN/m	
	DL	PL	Dead	1.81kN at 0m			
	LL	PL	Live		3.23kN at 0m		
	DL	PL	Dead	1.81kN at 0.85m			
	LL	PL	Live		3.23kN at 0.85m		
Landing 1	DL	UDL	Flooring	0.5 x 0.6 = 0.3 kN/m		0.3 kN/m	2.4 kN/m
	LL	UDL	Floor		4 x 0.6 = 2.4 kN/m		
Landing 1.2	DL	UDL	Flooring	0.5 x 0.6 = 0.3 kN/m		0.85 kN/m	2.4 kN/m
	LL	UDL	Floor		4 x 0.6 = 2.4 kN/m		
	DL	UDL	Balustrade	0.5 x 1.1 = 0.55 kN/m			
Landing 0.8	DL	UDL	Flooring	0.5 x 0.4 = 0.2 kN/m		0.75 kN/m	1.6 kN/m
	LL	UDL	Floor		4 x 0.4 = 1.6 kN/m		
	DL	UDL	Balustrade	0.5 x 1.1 = 0.55 kN/m			

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Component Loads

Part	Loading			DL	LL	Total loads on each part (UDL only)	
	DL/LL	TYPE	SOURCE			DL	LL
Main Beam	DL	UDL	Flooring	0.5 x 0.9 = 0.5 kN/m		1.1 kN/m	3.6 kN/m
	LL	UDL	Floor		4 x 0.9 = 3.6 kN/m		
	DL	UDL	Balustrade	0.5 x 1.1 = 0.6 kN/m			
	DL	PL	Dead	0.8kN at m			
	LL	PL	Live		1.4kN at m		
	DL	PL	Dead	3.9kN at m			
	LL	PL	Live		7.9kN at m		
	DL	PL	Dead	3.3kN at m			
	LL	PL	Live		7.1kN at m		
	DL	PL	Dead	0.9kN at m			
	LL	PL	Live		2.2kN at m		
Trimmers	DL	UDL	Flooring	0.5 x 1 = 0.5 kN/m		0.5 kN/m	4 kN/m
	LL	UDL	Floor		4 x 1 = 4 kN/m		

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GHD GRAHAM HUGHES DESIGN

Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
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Stair Tread - Steel

B	H	A	d	Ad	$BH^3/12$	Ad^2	$I_b = BH^3/12+Ad^2$
5	25	125	12.5	1563	6511	19538	26049
5	25	125	12.5	1563	6511	19538	26049
5	25	125	12.5	1563	6511	19538	26049
5	25	125	12.5	1563	6511	19538	26049
5	25	125	12.5	1563	6511	19538	26049
5	25	125	12.5	1563	6511	19538	26049
5	25	125	12.5	1563	6511	19538	26049
		875		10941			182343

Y_b = ΣAd / ΣA
Y_b = 10941 / 875
Y_b = 13mm

$\Sigma A \times Y_b^2$ = 875×12.504^2
$\Sigma A \times Y_b^2$ = 136806.264

I = $I_b - (\Sigma A \times Y_b^2)$
I = 182343 - 136806.264
I = 45537 mm^4
I = 5 cm^4
E = 205000 N/mm^2
EI = 1.025E+10
Z_b = I / Y_b
Z_b = 45537 / 13
Z_b = 3503 mm^3
Z_b = 4 cm^3
Material = Steel - S275
Yield Strength = 275 N/mm^2
Mom. Cap = 1.1 kNm

Dead Load (kN/m) = 0.15 x
Live Load (kN) = 3.00 x

Span (m) = 0.85
Moment (kNm) = 0.97
Deflection (mm) = 4.22

Section is adequate

Short Stringer

B	H	A	d	Ad	$BH^3/12$	Ad^2	$I_b = BH^3/12+Ad^2$
10	220	2200	110	242000	8873334	26620000	35493334
		2200		242000			35493334

Section Weight (kN/m)
0.17

Y_b = ΣAd / ΣA
Y_b = 242000 / 2200
Y_b = 110mm

$\Sigma A \times Y_b^2$ = 2200×110^2
$\Sigma A \times Y_b^2$ = 26620000

I = $I_b - (\Sigma A \times Y_b^2)$
I = 35493334 -
I = 8873334 mm^4
I = 888 cm^4

Z_b = I / Y_b
Z_b = 8873334 / 110
Z_b = 80667 mm^3
Z_b = 81 cm^3
Material = Steel - S275
Yield Strength = 275 N/mm^2
Mom. Cap = 22.275 kNm

Dead Load (kN/m) = 0.76
Live Load (kN/m) = 1.70
Factored UDL (kNm) = 3.58
Span (m) = 3.80
Moment (kNm) = 6.45
Deflection (mm) = 3.67

Section is adequate

Reaction - DL (kN) = 1.77
Reaction - LL (kN) = 3.23
Reaction - ULT (kN) = 7.23

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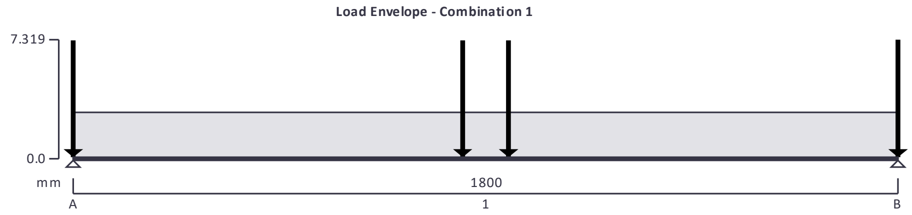
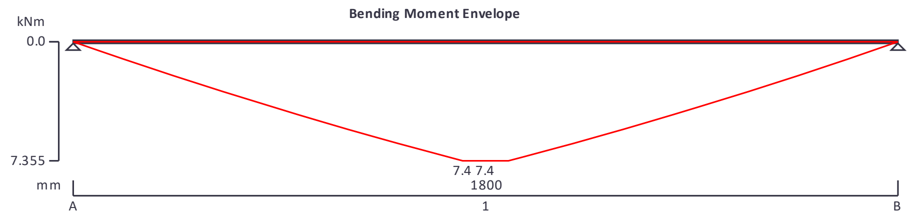
	Calcs for Landing Beam - 2 flights				Start page no./Revision 6	
	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date

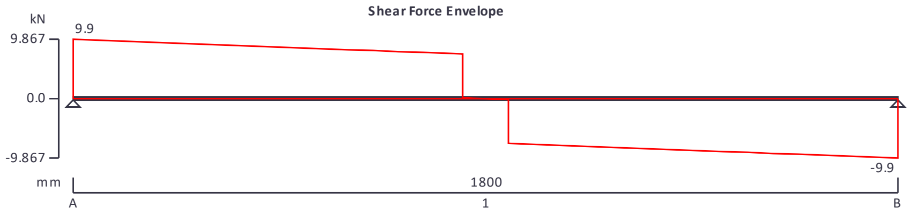
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam * 1
	Permanent full UDL 0.2 kN/m
	Variable full UDL 1.6 kN/m
	Permanent point load 1.81 kN at 0 mm
	Variable point load 3.23 kN at 0 mm
	Permanent point load 1.81 kN at 850 mm
	Variable point load 3.23 kN at 850 mm
	Permanent point load 1.81 kN at 950 mm
	Variable point load 3.23 kN at 950 mm
	Permanent point load 1.81 kN at 1800 mm
	Variable point load 3.25 kN at 1800 mm

Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
		Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

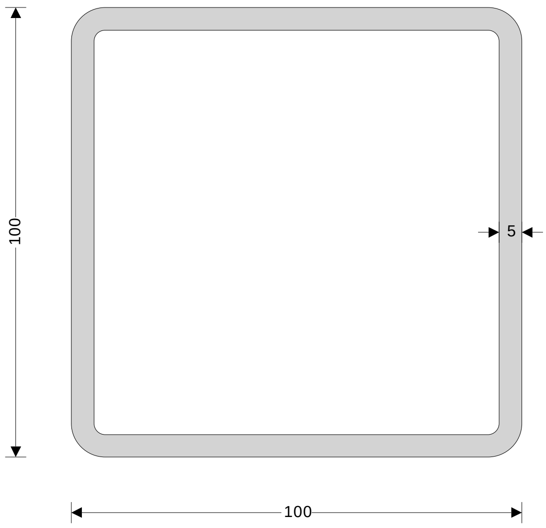
Maximum moment	M_{max} = **7.4** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **9.9** kN	V_{min} = **-9.9** kN
Deflection	δ_{max} = **2.5** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **17.2** kN	R_{A_min} = **17.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **3.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **7.9** kN	
Maximum reaction at support B	R_{B_max} = **17.2** kN	R_{B_min} = **17.2** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **3.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **7.9** kN	

Section details

Section type	**SHS 100x100x5.0 (Tata Steel Celsius)**
Steel grade	**S235H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = **5.0** mm
Nominal yield strength	f_y = **235** N/mm^2
Nominal ultimate tensile strength	f_u = **360** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis K_y = **1.000**

Effective length factor in minor axis K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.200**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **1.00**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = h - 3 * t$ = **85** mm

$c / t = 17.0 * \varepsilon <= 72 * \varepsilon$ Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section $c = b - 3 * t$ = **85** mm

$c / t = 17.0 * \varepsilon <= 33 * \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 * t$ = **90** mm

Shear area factor η = **1.000**

$h_w / t < 72 * \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **9.9** kN

Shear area - cl 6.2.6(3) $A_v = A * h / (b + h)$ = **937** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **127.1** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **7.4** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0}$ = **15.6** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **5** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **2.53** mm

PASS - Maximum deflection does not exceed deflection limit

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	Calcs for Landing Beam - 1 flight				Start page no./Revision 9	
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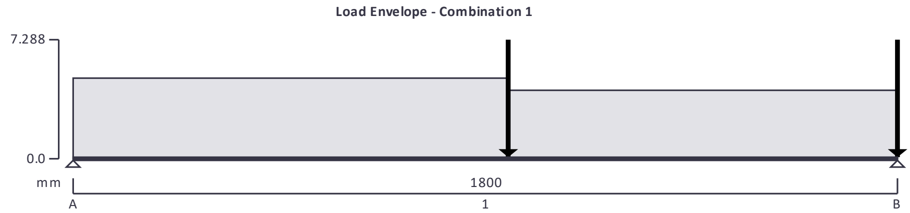
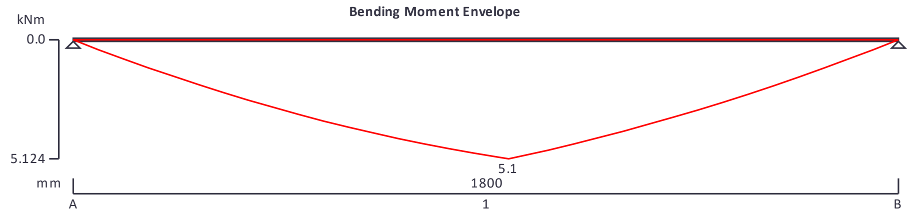
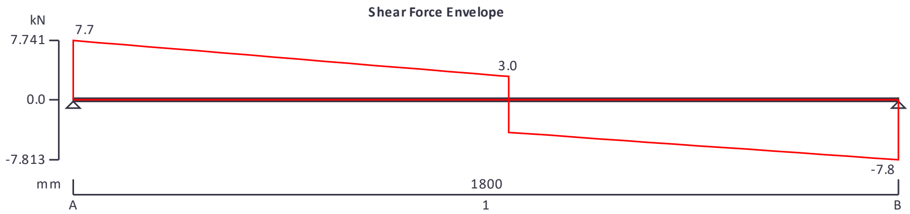
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam * 1
	Permanent full UDL 0.3 kN/m
	Variable full UDL 2.4 kN/m
	Permanent point load 1.81 kN at 950 mm
	Variable point load 3.23 kN at 950 mm
	Permanent point load 1.81 kN at 1800 mm
	Variable point load 3.23 kN at 1800 mm
	Permanent partial UDL 0.55 kN/m from 0 mm to 950 mm

Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50

		Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

Maximum moment	M_{max} = **5.1** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **7.7** kN	V_{min} = **-7.8** kN
Deflection	δ_{max} = **1.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **7.7** kN	R_{A_min} = **7.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **1.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.7** kN	
Maximum reaction at support B	R_{B_max} = **15.1** kN	R_{B_min} = **15.1** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **3.3** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **7.1** kN	

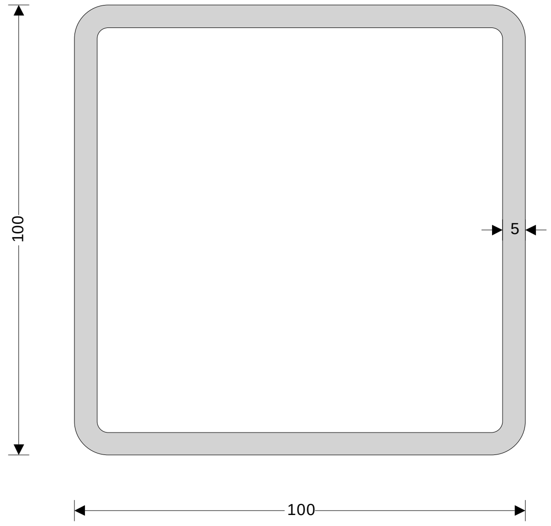
Section details

Section type **SHS 100x100x5.0 (Tata Steel Celsius)**

Steel grade **S235H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = **5.0** mm
Nominal yield strength	f_y = **235** N/mm^2
Nominal ultimate tensile strength	f_u = **360** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**

Effective length factor for torsion $K_{LT.A}$ = **1.200**

$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **1.00**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = h - 3 * t$ = **85** mm

$c / t = 17.0 * \varepsilon <= 72 * \varepsilon$ Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section $c = b - 3 * t$ = **85** mm

$c / t = 17.0 * \varepsilon <= 33 * \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 * t$ = **90** mm

Shear area factor η = **1.000**

$h_w / t < 72 * \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **7.8** kN

Shear area - cl 6.2.6(3) $A_v = A * h / (b + h)$ = **937** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **127.1** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **5.1** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0}$ = **15.6** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360$ = **5** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **1.771** mm

PASS - Maximum deflection does not exceed deflection limit

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	Calcs for Landing Beam - 1.2m				Start page no./Revision 12	
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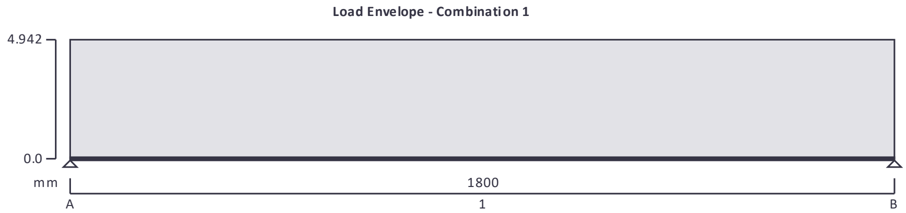
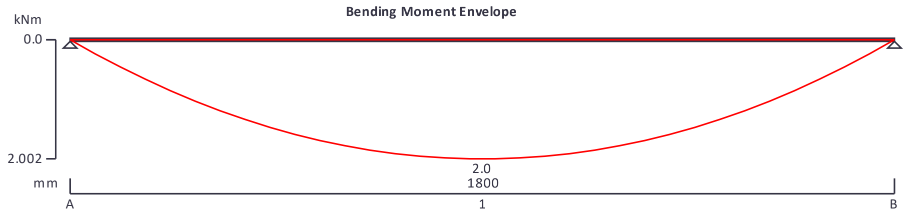
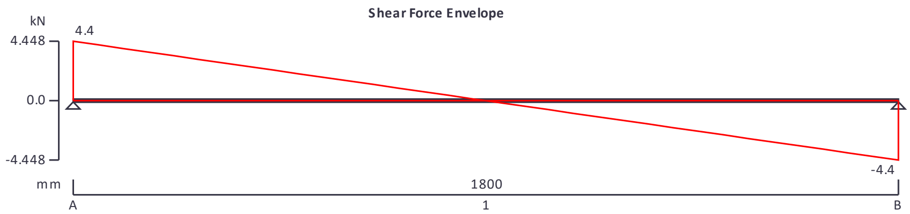
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam * 1
	Permanent full UDL 0.85 kN/m
	Variable full UDL 2.4 kN/m

Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
		Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Project				Job no.	
External Sairs				G0738-1	
Calcs for Landing Beam - 1.2m				Start page no./Revision 13	
Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date

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Analysis results

Maximum moment	M_{max} = **2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **4.4** kN	V_{min} = **-4.4** kN
Deflection	δ_{max} = **0.8** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **4.4** kN	R_{A_min} = **4.4** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.9** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **2.2** kN	
Maximum reaction at support B	R_{B_max} = **4.4** kN	R_{B_min} = **4.4** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.9** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **2.2** kN	

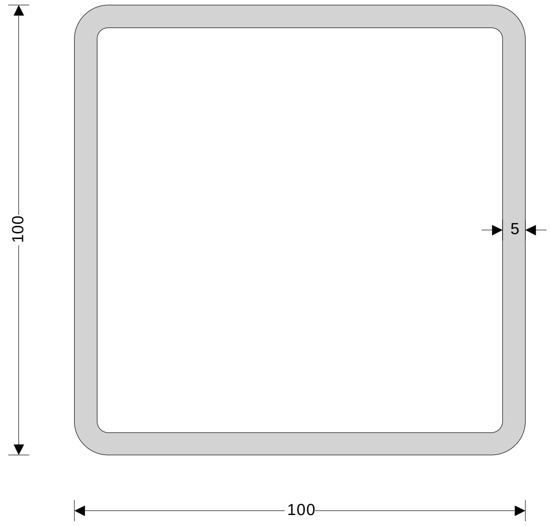
Section details

Section type **SHS 100x100x5.0 (Tata Steel Celsius)**

Steel grade **S235H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = **5.0** mm
Nominal yield strength	f_y = **235** N/mm^2
Nominal ultimate tensile strength	f_u = **360** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.200**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{1.00}$

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = h - 3 * t = \mathbf{85}$ mm

$c / t = 17.0 * \varepsilon <= 72 * \varepsilon$ — Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section — $c = b - 3 * t = \mathbf{85}$ mm

$c / t = 17.0 * \varepsilon <= 33 * \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 * t = \mathbf{90}$ mm

Shear area factor — $\eta = \mathbf{1.000}$

$h_w / t < 72 * \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{4.4}$ kN

Shear area - cl 6.2.6(3) — $A_v = A * h / (b + h) = \mathbf{937}$ mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0} = \mathbf{127.1}$ kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{2}$ kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0} = \mathbf{15.6}$ kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 = \mathbf{5}$ mm

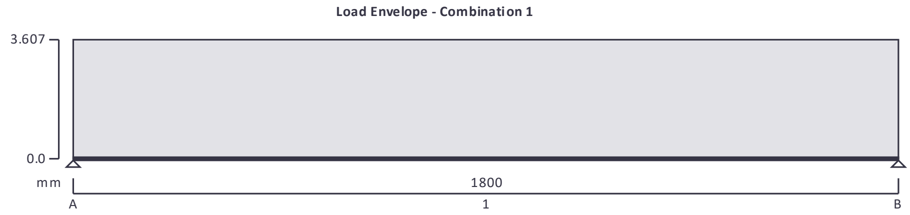
Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.791}$ mm

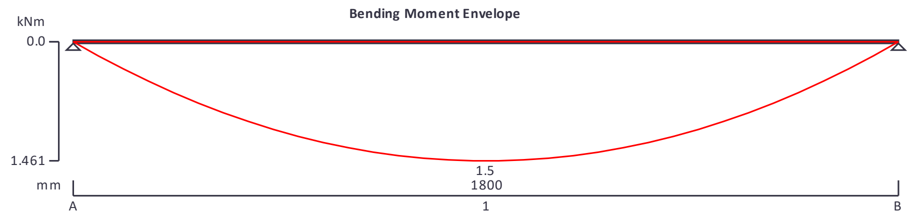
PASS - Maximum deflection does not exceed deflection limit

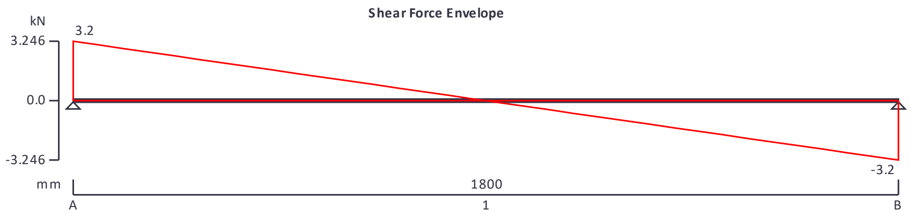
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam * 1	
	Permanent full UDL 0.75 kN/m	
	Variable full UDL 1.6 kN/m	

Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
		Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

Maximum moment	M_{max} = **1.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **3.2** kN	V_{min} = **-3.2** kN
Deflection	δ_{max} = **0.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **3.2** kN	R_{A_min} = **3.2** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.8** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **1.4** kN	
Maximum reaction at support B	R_{B_max} = **3.2** kN	R_{B_min} = **3.2** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.8** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **1.4** kN	

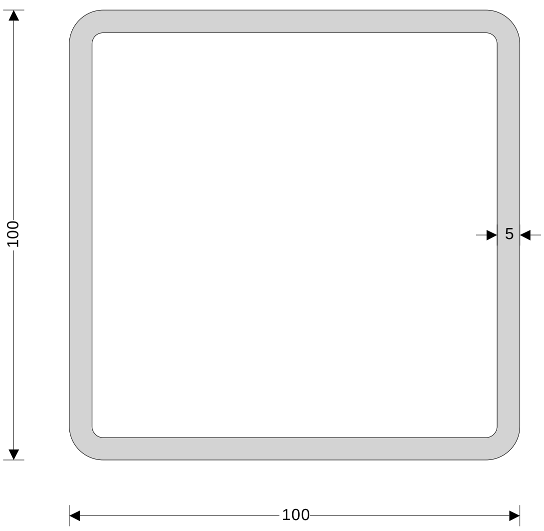
Section details

Section type **SHS 100x100x5.0 (Tata Steel Celsius)**

Steel grade **S235H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = **5.0** mm
Nominal yield strength	f_y = **235** N/mm^2
Nominal ultimate tensile strength	f_u = **360** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.200**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235\ N/mm^2 / f_y]} =$ **1.00**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = h - 3 * t =$ **85** mm

$c / t = 17.0 * \varepsilon <= 72 * \varepsilon$ Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section $c = b - 3 * t =$ **85** mm

$c / t = 17.0 * \varepsilon <= 33 * \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 * t =$ **90** mm

Shear area factor $\eta =$ **1.000**

$h_w / t < 72 * \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = max(abs(V_{max}), abs(V_{min})) =$ **3.2** kN

Shear area - cl 6.2.6(3) $A_v = A * h / (b + h) =$ **937** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **127.1** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **1.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0} =$ **15.6** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **5** mm

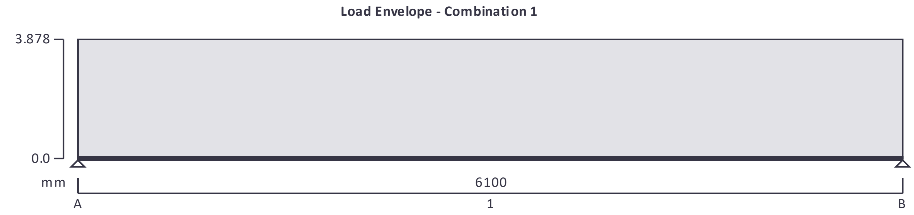
Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **0.581** mm

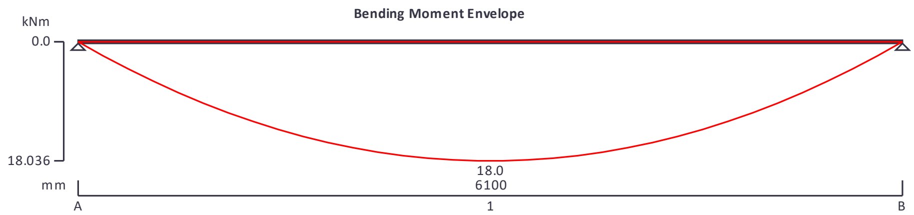
PASS - Maximum deflection does not exceed deflection limit

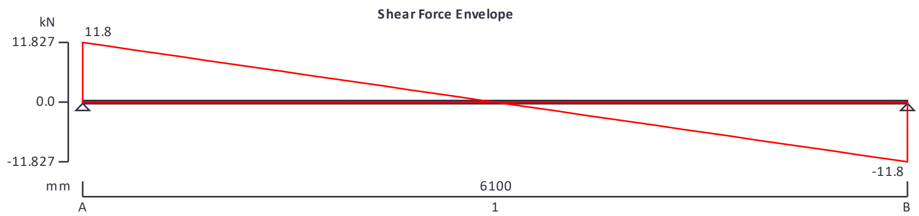
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam * 1
	Permanent full UDL 0.75 kN/m
	Variable full UDL 1.6 kN/m

Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
		Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

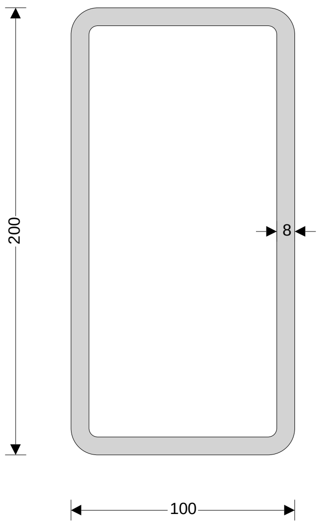
Maximum moment	M_{max} = **18** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **11.8** kN	V_{min} = **-11.8** kN
Deflection	δ_{max} = **10.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **11.8** kN	R_{A_min} = **11.8** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **3.3** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **4.9** kN	
Maximum reaction at support B	R_{B_max} = **11.8** kN	R_{B_min} = **11.8** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **3.3** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **4.9** kN	

Section details

Section type	**RHS 200x100x8.0 (Tata Steel Celsius)**
Steel grade	**S235H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = **8.0** mm
Nominal yield strength	f_y = **235** N/mm^2
Nominal ultimate tensile strength	f_u = **360** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.200**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]}$ = **1.00**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section — $c = h - 3 * t$ = **176** mm

$c / t = 22.0 * \varepsilon <= 72 * \varepsilon$ — Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section — $c = b - 3 * t$ = **76** mm

$c / t = 9.5 * \varepsilon <= 33 * \varepsilon$ — Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web — $h_w = h - 2 * t$ = **184** mm

Shear area factor — η = **1.000**

$h_w / t < 72 * \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force — $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **11.8** kN

Shear area - cl 6.2.6(3) — $A_v = A * h / (b + h)$ = **2984** mm^2

Design shear resistance - cl 6.2.6(2) — $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **404.8** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment — $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **18** kNm

Design bending resistance moment - eq 6.13 — $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0}$ = **66.3** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360$ = **16.9** mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **10.357** mm

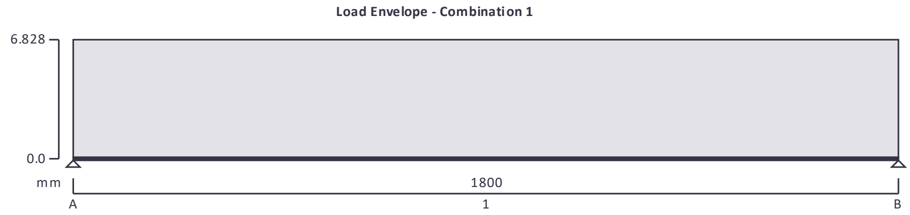
PASS - Maximum deflection does not exceed deflection limit

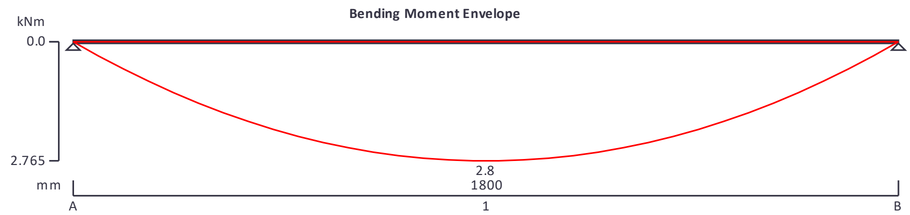
	Project				Job no.	
	External Sairs				G0738-1	
	Calcs for				Start page no./Revision	
	Landing Beam - 1 flight				21	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	GH	09/03/2023	GH	09/03/2023		

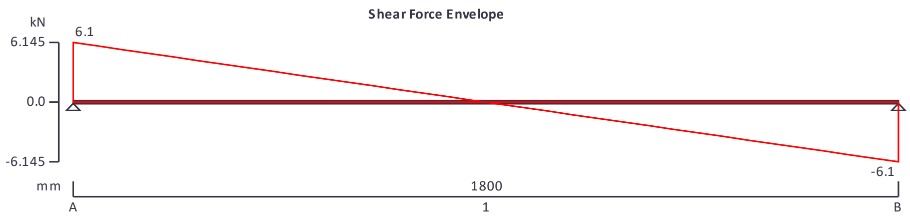
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.14







Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam * 1
	Permanent full UDL 0.5 kN/m
	Variable full UDL 4 kN/m

Load combinations

Load combination 1	Support A	Permanent * 1.35
		Variable * 1.50
		Permanent * 1.35
		Variable * 1.50
	Support B	Permanent * 1.35
		Variable * 1.50

Analysis results

Maximum moment	M_{max} = **2.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **6.1** kN	V_{min} = **-6.1** kN
Deflection	δ_{max} = **2.2** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **6.1** kN	R_{A_min} = **6.1** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **0.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.6** kN	
Maximum reaction at support B	R_{B_max} = **6.1** kN	R_{B_min} = **6.1** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **0.6** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **3.6** kN	

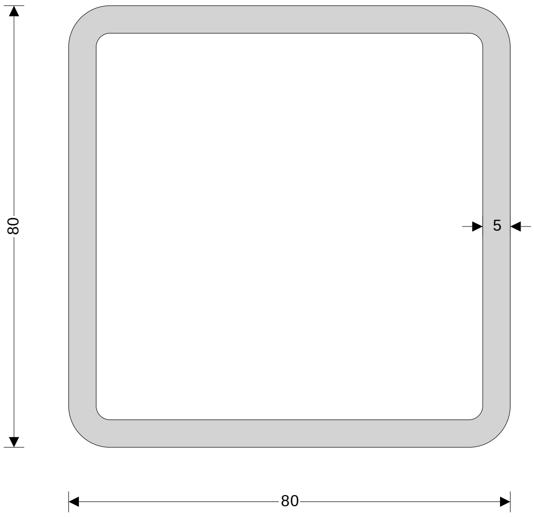
Section details

Section type **SHS 80x80x5.0 (Tata Steel Celsius)**

Steel grade **S235H**

EN 10210-1:2006 - Hot finished structural hollow sections of non-alloy and fine grain steels

Nominal thickness of element	t = **5.0** mm
Nominal yield strength	f_y = **235** N/mm^2
Nominal ultimate tensile strength	f_u = **360** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections	γ_{M0} = **1.00**
Resistance of members to instability	γ_{M1} = **1.00**
Resistance of tensile members to fracture	γ_{M2} = **1.10**

Lateral restraint

Span 1 has full lateral restraint

Effective length factors

Effective length factor in major axis	K_y = **1.000**
Effective length factor in minor axis	K_z = **1.000**
Effective length factor for torsion	$K_{LT.A}$ = **1.200**
	$K_{LT.B}$ = **1.000**

Classification of cross sections - Section 5.5

$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} =$ **1.00**

Internal compression parts subject to bending - Table 5.2 (sheet 1 of 3)

Width of section $c = h - 3 * t =$ **65** mm

$c / t = 13.0 * \varepsilon <= 72 * \varepsilon$ Class 1

Internal compression parts subject to compression only - Table 5.2 (sheet 1 of 3)

Width of section $c = b - 3 * t =$ **65** mm

$c / t = 13.0 * \varepsilon <= 33 * \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 * t =$ **70** mm

Shear area factor $\eta =$ **1.000**

$h_w / t < 72 * \varepsilon / \eta$

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) =$ **6.1** kN

Shear area - cl 6.2.6(3) $A_v = A * h / (b + h) =$ **737** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v * (f_y / \sqrt{[3]}) / \gamma_{M0} =$ **99.9** kN

PASS - Design shear resistance exceeds design shear force

Check bending moment major (y-y) axis - Section 6.2.5

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **2.8** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} * f_y / \gamma_{M0} =$ **9.7** kNm

PASS - Design bending resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **5** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **2.198** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL COLUMN AND BASE PLATE DESIGN (EN1993)

Tedds calculation version 1.0.03

STEEL COLUMN DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

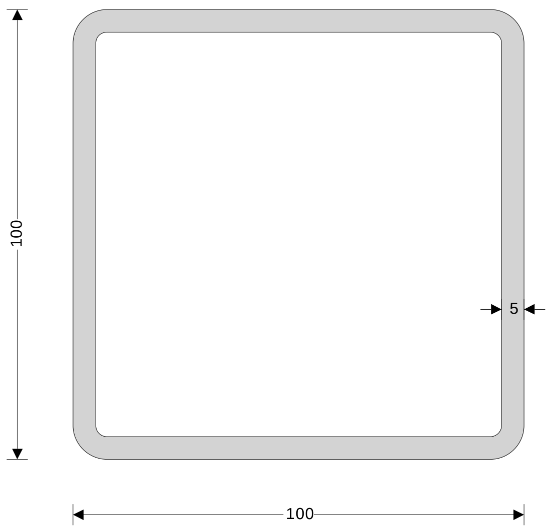
Tedds calculation version 1.1.06

Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1**

Resistance of members to instability γ_{M1} = **1**

Resistance of cross-sections in tension to fracture γ_{M2} = **1.1**



SHS 100x100x5.0 (Tata Steel Celsius)
Section depth, h, 100 mm
Section breadth, b, 100 mm
Mass of section, Mass, 14.7 kg/m
Section thickness, t, 5 mm
Area of section, A, 1873 mm^2
Radius of gyration about y-axis, i_y, 38.623 mm
Radius of gyration about z-axis, i_z, 38.623 mm
Elastic section modulus about y-axis, $W_{el.y}$, 55886 mm^3
Elastic section modulus about z-axis, $W_{el.z}$, 55886 mm^3
Plastic section modulus about y-axis, $W_{pl.y}$, 66358 mm^3
Plastic section modulus about z-axis, $W_{pl.z}$, 66358 mm^3
Second moment of area about y-axis, I_y, 2794323 mm^4
Second moment of area about z-axis, I_z, 2794323 mm^4

Column details

Column section **SHS 100x100x5.0**

Steel grade **User defined**

Yield strength f_y = **355** N/mm^2

Ultimate strength f_u = **470** N/mm^2

Modulus of elasticity E = **210** kN/mm^2

Poisson's ratio ν = **0.3**

Shear modulus $G = E / [2 \times (1 + \nu)]$ = **80.8** kN/mm^2

Column geometry

System length for buckling - Major axis L_y = **4500** mm

System length for buckling - Minor axis L_z = **4500** mm

The column is not part of a sway frame in the direction of the minor axis

The column is not part of a sway frame in the direction of the major axis

Column loading

Axial load N_{Ed} = **15** kN (Compression)

Major axis moment at end 1 - Bottom $M_{y,Ed1}$ = **0.0** kNm

Major axis moment at end 2 - Top $M_{y,Ed2}$ = **0.0** kNm

Minor axis moment at end 1 - Bottom $M_{z,Ed1}$ = **0.0** kNm

Minor axis moment at end 2 - Top $M_{z,Ed2}$ = **0.0** kNm

Major axis shear force $V_{y,Ed}$ = **0** kN

Minor axis shear force $V_{z,Ed}$ = **0** kN

Buckling length for flexural buckling - Major axis

End restraint factor K_y = **1.000**

Buckling length $L_{cr\_y} = L_y \times K_y$ = **4500** mm

Buckling length for flexural buckling - Minor axis

End restraint factor K_z = **1.000**

Buckling length $L_{cr\_z} = L_z \times K_z$ = **4500** mm

Web section classification (Table 5.2)

Coefficient depending on f_y $\varepsilon = \sqrt{(235\ N/mm^2 / f_y)}$ = **0.814**

Depth between fillets $c_w = h - 3 \times t$ = **85.0** mm

Ratio of c/t $ratio_w = c_w / t$ = **17.00**

Length of web taken by axial load $l_w = \min(N_{Ed} / (2 \times f_y \times t), c_w)$ =**4.2** mm

For class 1 & 2 proportion in compression $\alpha = (c_w/2 + l_w/2) / c_w$ = **0.525**

Limit for class 1 web $Limit_{1w} = (396 * \varepsilon) / (13 * \alpha - 1)$ = **55.33**

The web is class 1

Flange section classification (Table 5.2)

Depth between fillets $c_f = b - 3 \times t$ = **85.0** mm

Ratio of c/t $ratio_f = c_f / t$ = **17.00**

Limit for class 1 flange $Limit_{1f} = 33 \times \varepsilon$ = **26.85**

Limit for class 2 flange $Limit_{2f} = 38 \times \varepsilon$ = **30.92**

Limit for class 3 flange $Limit_{3f} = 42 \times \varepsilon$ = **34.17**

The flange is class 1

Overall section classification

The section is class 1

Resistance of cross section (cl. 6.2)

Compression (cl. 6.2.4)

Design force N_{Ed} = **15** kN

Design resistance $N_{c,Rd} = N_{pl,Rd} = A \times f_y / \gamma_{M0}$ = **665** kN

$N_{Ed} / N_{c,Rd}$ = **0.023**

PASS - The compression design resistance exceeds the design force

Buckling resistance (cl. 6.3)

Yield strength for buckling resistance f_y = **355** N/mm²

Flexural buckling - Major axis

Elastic critical buckling force $N_{cr,y} = \pi^2 \times E \times I_y / L_{cr\_y}^2$ = **286** kN

Non-dimensional slenderness $\bar{\lambda}_y = \sqrt{(A \times f_y / N_{cr,y})}$ = **1.525**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) α_y = **0.21**

Parameter Φ $\Phi_y = 0.5 \times [1 + \alpha_y \times (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2]$ = **1.802**

Reduction factor $\chi_y = \min(1.0, 1 / [\Phi_y + \sqrt{(\Phi_y^2 - \bar{\lambda}_y^2)}])$ = **0.362**

Design buckling resistance $N_{b,y,Rd} = \chi_y \times A \times f_y / \gamma_{M1}$ = **240.8** kN

$N_{Ed} / N_{b,y,Rd}$ = **0.062**

PASS - The flexural buckling resistance exceeds the design axial load

Flexural buckling - Minor axis

Elastic critical buckling force $N_{cr,z} = \pi^2 \times E \times I_z / L_{cr\_z}^2$ = **286** kN

Non-dimensional slenderness $\bar{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})}$ = **1.525**

Buckling curve (Table 6.2) **a**

Imperfection factor (Table 6.1) α_z = **0.21**

Parameter Φ $\Phi_z = 0.5 \times [1 + \alpha_z \times (\bar{\lambda}_z - 0.2) + \bar{\lambda}_z^2]$ = **1.802**

Reduction factor $\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \bar{\lambda}_z^2)}])$ = **0.362**

Design buckling resistance $N_{b,z,Rd} = \chi_z \times A \times f_y / \gamma_{M1}$ = **240.8** kN

$N_{Ed} / N_{b,z,Rd}$ = **0.062**

PASS - The flexural buckling resistance exceeds the design axial load

Minimum buckling resistance

Minimum buckling resistance $N_{b,Rd} = \min(N_{b,y,Rd}, N_{b,z,Rd})$ = **240.8** kN

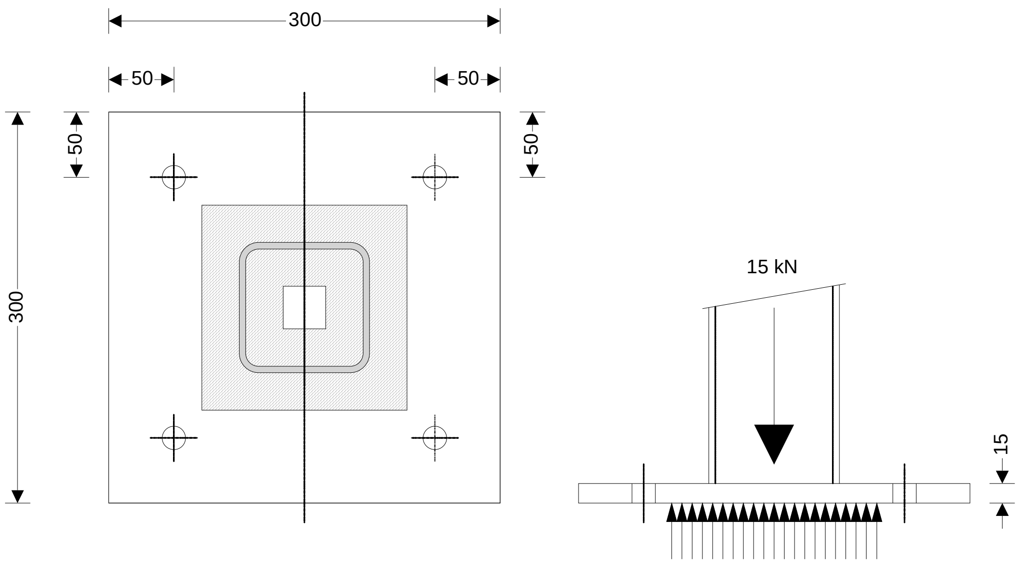
$N_{Ed} / N_{b,Rd}$ = **0.062**

PASS - The axial load buckling resistance exceeds the design axial load

COLUMN BASE PLATE DESIGN

In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009, and EN1993-1-8:2005 incorporating Corrigenda December 2005, September 2006 and July 2009, and the UK national annex

Tedds calculation version 2.0.09



Design forces

Design axial force (compression) N_{Ed} = **15** kN

Design shear force V_{Ed} = **0** kN

Design moment M_{Ed} = **0** kNm

Column Details

Column section **SHS 100x100x5.0**

Depth D = **100** mm

Width B = **100** mm

Thickness $t = \mathbf{5}$ mm

Base plate details

Length $h_p = \mathbf{300}$ mm

Width $b_p = \mathbf{300}$ mm

Thickness $t_p = \mathbf{15}$ mm

Column eccentricity in x-axis $e_{bpx} = \mathbf{0}$ mm

Anchor details

Number of anchors LHS $n_1 = \mathbf{2}$

Edge distance in x-axis $e_{x1} = \mathbf{50}$ mm

Edge distance in y-axis $e_{y1} = \mathbf{50}$ mm

Number of anchors RHS $n_2 = \mathbf{2}$

Edge distance in x-axis $e_{x2} = \mathbf{50}$ mm

Edge distance in the y-axis $e_{y2} = \mathbf{50}$ mm

Anchor diameter $d_{a,b} = \mathbf{16}$ mm

Foundation details

Foundation depth $t_{fnd} = \mathbf{500}$ mm

Concrete details

Concrete strength class **C25/30**

Characteristic compressive cylinder strength $f_{ck} = \mathbf{25}$ N/mm^2

Characteristic compressive cube strength $f_{ck,cube} = \mathbf{30}$ N/mm^2

Partial factor for concrete $\gamma_c = \mathbf{1.50}$

Compressive strength coefficient $\alpha_{cc} = \mathbf{0.85}$

Design compressive concrete strength $f_{cd} = \alpha_{cc} \times (f_{ck} / \gamma_c) = \mathbf{14.17}$ N/mm^2

Steel details

Base plate steel grade **S275**

Base plate nominal yield strength $f_{yp\_plt} = \mathbf{275}$ N/mm^2

Base plate nominal ultimate tensile strength $f_{u\_plt} = \mathbf{410}$ N/mm^2

Column steel grade **User defined**

Column nominal yield strength $f_{yp\_col} = \mathbf{355}$ N/mm^2

Column nominal ultimate tensile strength $f_{u\_col} = \mathbf{470}$ N/mm^2

Partial safety factor cross sections $\gamma_{M0} = \mathbf{1.00}$

Partial safety factor welds $\gamma_{M2} = \mathbf{1.25}$

Foundation bearing strength - EN1992-1-1 Section 6.7

Loaded area $A_{c0} = b_p \times h_p = \mathbf{90000}$ mm^2

Design distribution width $b_{p,dist} = \mathbf{800}$ mm

Design distribution length $h_{p,dist} = \mathbf{800}$ mm

Maximum design distribution area $A_{c1} = b_{p,dist} \times h_{p,dist} = \mathbf{640000}$ mm^2

Concentrated design resistance force $F_{Rdu} = \mathrm{Min}(A_{c0} \times f_{cd} \times \sqrt{(A_{c1} / A_{c0})}, 3 \times f_{cd} \times A_{c0}) = \mathbf{3400}$ kN

Foundation joint material coefficient $\beta_j = \mathbf{0.67}$

Design bearing strength of the joint $f_{jd} = \beta_j \times F_{Rdu} / (b_p \times h_p) = \mathbf{25.19}$ N/mm^2

Base plate compressive resistance

Additional bearing width $c = t_p \times \sqrt{(f_{yp\_plt} / (3 \times f_{jd} \times \gamma_{M0}))} = \mathbf{28.6}$ mm

Effective bearing area $A_{eff} = \mathbf{23649}$ mm^2

Design compressive resistance $N_{c,Rd} = f_{jd} \times A_{eff} = \mathbf{595.6}$ kN

PASS - Design compressive resistance exceeds applied axial load

Project				Job no.	
External Stairs				G0738-1	
Calcs for				Start page no./Revision	
Column & Baseplate				28	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
GH	09/03/2023	GH	09/03/2023		

Fixings

Steel - Steel

The highest reaction at the ends of any of the beams is 3.3 kN (DL) + 4.9 kN (LL) = 11.8 kN (Factored)

A single M16 grade 8.8 bolt has a shear resistance of 58.9 kN, so two such bolt per steel - steel connection will suffice.

6mm fillet welds have a shear strength of 215 N/mm^2 which equates to 0.9 kN/mm, so welded end connections with at least 50mm (45 kN) of weld will suffice.

Wall - Upper Landing Point Loads

The reaction at the end of any of the beams is 3.9 kN (DL) + 7.9 kN (LL) = 11.8 kN (unfactored)

An M12 sleeve anchor will have a shear capacity on dense blockwork of 3 kN, so 4 such bolts at the end of each upper landing beam will suffice.

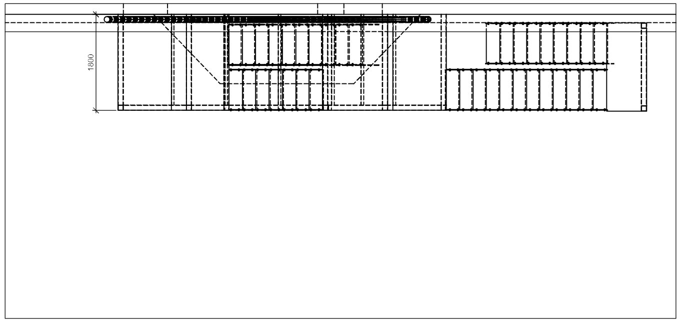
Wall - Main Landing Point Loads

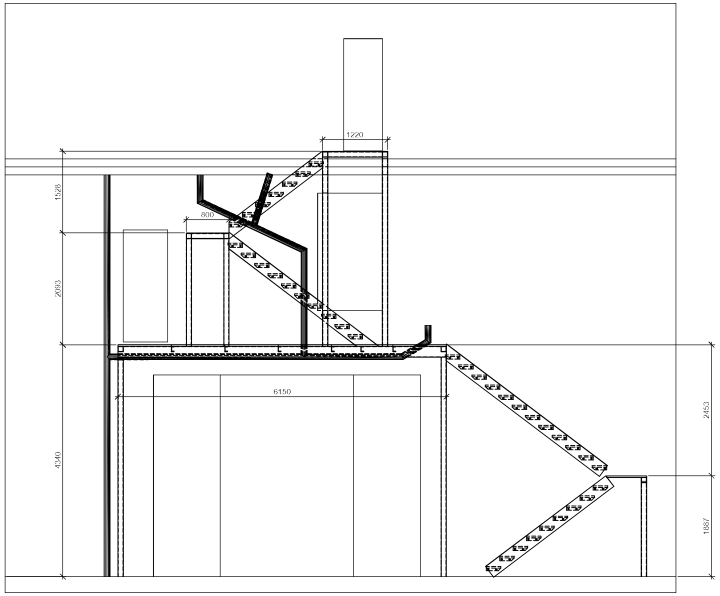
The reaction along the wall s is 0.6 kN (DL) + 3.6 kN (LL) = 4.2 kN (unfactored)

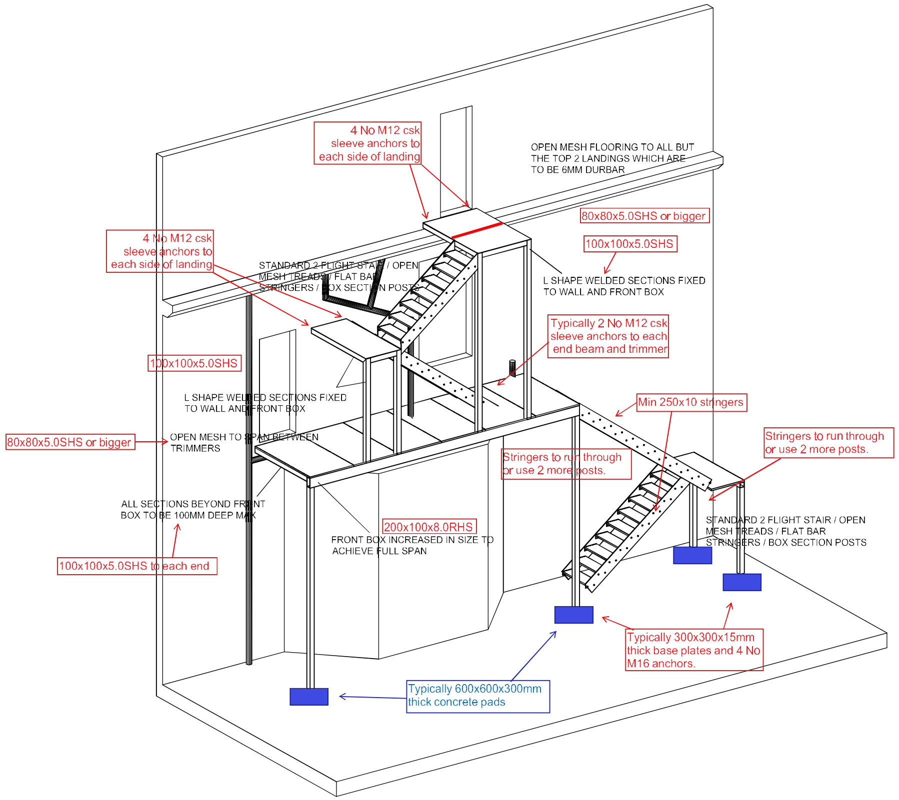
An M12 sleeve anchor will have a shear capacity on dense blockwork of 3 kN, so 2 such bolts at the end of the trimmers will suffice.

Conclusions

The structural framework and fixings shown on the mark-up Bradfabs Drawing will be adequate.







QDOC022 REV 1

Template Size: A1

PROJECTION:

General Notes:

- All dimensions in millimeters.
- General working tollarance +/- 5mm.
- All holes Ø18 unless noted otherwise.
- All welds to be 6mm fillet weld or full pen butt weld unless noted otherwise.
- All galvanised closed sections to be vented.
- All open sections, plates & flats grade S275 unless noted otherwise.
- All hollow sections grade S355 unless noted otherwise.

FABRICATION.......................
FOLDING.......................
CUTTING.......................
PUNCH/DRILLING.......................

WELDER NO.	WELD TYPE	LOCATION

BRADFABS LIMITED
FINAL INSPECTION
PASSED BY:.......................
DATE:.......................

CONSUMABLES (BATCH NO.) :

BRADFABS LTD
DESIGN FABRICATE INSTALL
Telephone: 01274 400 401 | Email: sales@bradfabs.co.uk

Index	Author	Date	Description

CE EXECUTION CLASS: | FINISH:
DRAWN BY: | DATE: | SCALE:
CONTRACT:
DESCRIPTION:
JOB No. | DRG No. [G] - | Rev.