

# **External Stairs Structural Calculations**

Calculations for Balcony Structure

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

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Main Beam					10	$\frac{8}{1}$ 20
Dest & Pasaplata					2	$\frac{1}{1}$ 23
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Conclusion						$\frac{29}{30}$
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Diadiaos diawing					5.	1 51

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	GRAHAM HUGHES DESIGN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
		GH	09/03/2023	GH	09/03/2023		
			Loadi	ngs			
	Load				Type	SLS	
					1,10		
•	Flooring DL				Total	0.5	
	Steel			Γ	Dead	0.5	
•	Floor LL			Im	posed	4	
•	Balustrade DL				Total	0.5	
	Dead			Γ	Dead	0.5	
•	Dead DL				Total	1	
	Dead			Γ	Dead	1	
•	Live LL				Total	1	
	Live			Im	posed	1	
•	Factored LL				Total	1	
	Live			Im	posed	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.

# **Execution Class**

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

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6	Н			Calcs fo	or	С	Calculatio	ns fc	or Balcony	/ Sti	ructure	Page	2
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				011		03/03/	2025		011		05/05/2025		
				<u>Com</u>	pone	nt Loads							
Part		Loadir	NG			וח			I		11	Total loads	on each part
Fait	DL/LL	TYPE	sc	URCE			-					DL	LL
Tread	DL	UDL	Flo	ooring	0	.5 x 0.3 = 0	).15 kN/n	n				0.15 kN/m	
	DL	PL		Live		3kN at 0	.425m						
Stringer	וח	וחוו	Flo	oring	0 5	5 v 0 425 =	0 21 kN/	m				0.76 kN/m	1 7 kN/m
Stringer	11		F	loor	0.5	7 0.425 -	0.21 KN		4 x 0	<u>1</u> 21	5 = 1 7 kN/m	0.70 kityin	1.7 KR7111
	DI		Balı	ustrade	0	.5 x 1.1 = 0	).55 kN/n	า	4 / 0.	.72.	5 - 1.7 KN/11		
		002	Dur	ustruue	0	.5 / 1.1 0	0.55 kN/m						
Landing 2	DL	UDL	Flo	ooring	C	).5 x 0.4 = (	0.2 kN/m					0.2 kN/m	1.6 kN/m
	LL	UDL	F	loor					4 x (	0.4	= 1.6 kN/m		
	DL	PL	0	Dead		1.81kN	at 0m						
	LL	PL	I	Live					3.	.23	kN at 0m		
	DL	PL	0	Dead		1.81kN at	t 0.85m						
	LL	PL	I	Live					3.2	3kľ	N at 0.85m		
	DL	PL	0	Dead		1.81kN at 0.95m							
	LL	PL	I	Live				3.2	3k1	N at 0.95m			
	DL	PL	0	Dead		1.81kN a	t 1.8m						
	LL	PL		Live					3.2	23k	N at 1.8m		
Landing 1	וח	וחוו	Pali	ustrado	0	E v 1 1 – 0							
Lanung I				Dood	0	1 91LN	.55 KN/11	'				0.55 KN/11	
				Live		1.01KIN	atom		3	22	kN at Om		
		PL DI		Dead		1 81kN at	+ 0 85m		3.23kN at 0m		KIN dt UIII		
		PL DI		Live		1.01KN at	0.05111		3.2	241	V at 0.85m		
	LL			LIVC					5.2				
Landing 1	DL	UDL	Flo	ooring	C	).5 x 0.6 = (	0.3 kN/m					0.3 kN/m	2.4 kN/m
	LL	UDL	F	loor					4 x (	0.6	= 2.4 kN/m		
Landing 1.2	DL	UDL	Flo	ooring	C	).5 x 0.6 = (	0.3 kN/m					0.85 kN/m	2.4 kN/m
	LL	UDL		loor					4 x (	0.6	= 2.4 kN/m		
	DL	UDL	Ball	ustrade	0	.5 x 1.1 = 0	).55 kN/n	า					
Landing 0.8	DL	UDL	Flo	ooring	C	).5 x 0.4 = (	0.2 kN/m					0.75 kN/m	1.6 kN/m
	LL	UDL	F	loor					4 x (	0.4	= 1.6 kN/m		
	DL	UDL	Balı	ustrade	0	.5 x 1.1 = 0	).55 kN/n	n					
		l	I						I				

		Projec	t		Exte	rnal Stairs		Job No.	G0738-1	
6	Н		Calcs fo	or	Calculatio	ns fo	or Balcony	Structure	Page	3
GRAHAM	HUGHES D	ESIGN	Calcs b	у	Calcs date	Ch	ecked by	Checked date	Approved by	Approved date
			GH		09/03/2023		GH	09/03/2023		
			Com	nponer	it Loads					
									Total loads	on oach part
Part		Loadir	ng		DL			LL	(UDL	only)
	DL/LL	TYPE	SOURCE						DL	LL
			·							2.6144
Main Beam			Flooring	0.	5 x 0.9 = 0.5 kN/m	1	4 v (	9 = 3.6  kN/m	1.1 kN/m	3.6 kN/m
	DI		Balustrade	0	$5 \times 11 = 0.6 \text{ kN/m}$					
	DL	PL	Dead	0.	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	C	0.5 x 1 = 0.5 kN/m				0.5 kN/m	4 kN/m
	LL	UDL	Floor				4 x 1 = 4 kN/m		-	
		-								

				Р	roject		External Stair	'S	Job No.	G0738-1
F			Ī	Са	alcs for	Calculatio	ns for Balcon	y Structure	Page	4
GRAHA	n HUGHES	DESIGN	-	C	alcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
					GH	09/03/2023	GH	09/03/2023	11 2	
				5	<u>Stair Tread</u>	- Steel				
В	н	Α	d		Ad	BH <sup>3</sup> /12	Ad <sup>2</sup>	$I_{\rm b} = BH^3/12 + Ad^2$	-	
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		l	10941			182343		
Y <sub>b</sub> =			ΣAd	/ Σι	4 ( 075					
Y <sub>b</sub> =			109	41/	8/5					
Υ <sub>b</sub> =			131	nm						
<b>ΣΔ x V</b> .	<sup>2</sup> _		875	v 1	2 504²					
ΣΑ χ Υμ	<sup>2</sup> =		136	806	2.504					
2,7,7,10			100		.201					
I =			I <sub>b</sub> - (	(ΣΑ :	x Y <sub>b</sub> ²)					
=			182	343	- 136806	.264				
=			455	37 r	nm⁴					
I =			5 cn	n⁴						
E =			205	000	N/mm <sup>2</sup>					
EI =			1.02	25E-	+10					
Z <sub>b</sub> =			I / Y <sub>t</sub>	b						
Z <sub>b</sub> =			4553	37 /	13					
Z <sub>b</sub> =			3503	3 mr	n³					
Z <sub>b</sub> =			4 cm	1 <sup>3</sup>						
Materia	=		Stee	el - S	275	2				
Yield Sti	rength =				275 N/mm	<u> </u>				
Mom. C	ap =		1.1 k	kNm	I					
Dead Load (kN/m) = 0.15 x										
Live Load (kN) = 3.00 x										
Snan (m) = 0.85										
Moment (kNm) = 0.97										
Deflecti	on (mm)	=	4.22	2						
	()			Se	ection is ad	equate				

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F				Ca	alcs for	Calculatio	ns for Balcon	y Structure	Page	5
GRAHA	m HUGHES	DESIGN		С	alcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
					GH	09/03/2023	GH	09/03/2023		
					Short Stri	nger				
В	Н	Α	c	ł	Ad	BH <sup>3</sup> /12	Ad <sup>2</sup>	$I_b = BH^3/12 + Ad^2$	Sectio	on Weight
10	220	2200	11	10	242000	8873334	26620000	35493334	()	‹N/m)
									4	0.17
						-			-	
		2200	J		242000			35493334	4	
<b>v</b> . =			ΣΔd	1/5	Δ					
Y <sub>b</sub> =			242	2000	) / 2200					
Y <sub>b</sub> =			110	)mm	י,י ו					
ΣA x Y <sub>b</sub>	<sup>2</sup> =		220	)0 x	110²					
ΣA x Y <sub>b</sub>	<sup>2</sup> =		266	26620000						
i =			l <sub>b</sub> -	(ΣΑ	x Y <sub>b</sub> <sup>2</sup> )					
I =			354	1933	334 -					
=			887	7333	84 mm⁴					
=			888	3 cm	4					
Z <sub>h</sub> =			I / Y	, h						
Z <sub>b</sub> =			887	3334	4/110					
Z <sub>b</sub> =			806	67 n	nm³					
Z <sub>b</sub> =			81 c	cm³						
Materia	l = .		Stee	el - S	275	2				
Yield St	rength =		22.2		275 N/mm	2				
viom. C	ap =		22.2	2/5 8	kinm					
Dead Lo	oad (kN/r	m) =	0.76	5						
Live Loa	ıd (kN/m	ı) =	1.70	)						
Factore	d UDL (k	Nm) =	3.58	3						
Span (m	1) =		3.80	) -						
IVIOMEN	τ (KNM)	=	6.45	2 7						
Denecti	on (mm)	=	5.67	Se	ection is ad	equate				
Reactio	n - DL (k	:N) =	1.77	7 7		Cynaic				
Reactio	n - LL (kl	, N) =	3.23	3						
Reactio	n - ULT(	(kN) =	7.23	3						

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		Externa	G0738-1			
GHD	Calcs for	Landing Bea	Start page no./Revision 6			
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In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex



Support conditions Support A

Support B

Applied loading Beam loads Vertically restrained Rotationally free Vertically restrained Rotationally free

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

F	Project Job no. External Sairs G0738-1								
GHD	Calcs for	Landing Be	am - 2 flights		Start page no./F	Revision 7			
GRAHAM HUGHES DESIGN	Calcs by GH	External Sairs         G0738           Landing Beam - 2 flights         7           Calos date         Checked by         Checked date           09/03/2023         GH         09/03/2023         Approved by         Ap           Support A         Permanent * 1.35         Variable * 1.50         Permanent * 1.35         Variable * 1.50           Support B         Permanent * 1.35         Variable * 1.50         Permanent * 1.35         Variable * 1.50           Mmax = 7.4 kNm         Mmin = 0 kNm         Vmin = -9.3 kN         Smin = 0 kNm         Vmin = -9.3 kN           Mmax = 2.5 mm         Son = 0 mm         Ra_min = 17.2 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN           1A         Ra_permanent = 3.9 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN           1B         Ra_permanent = 3.9 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN           1B         Ra_permanent = 3.9 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN           1B         Ra_permanent = 3.9 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN         Ra_min = 17.2 kN           1B         Ra_permanent = 3.9 kN         Ra_min = 17.2 kN         Saint = 0 min = 17.2 kN         Min = 1.00 min = 10.00         Min = 1.00	Approved date						
Load combinations Load combination 1		Support A		Perman Variable	ent * 1.35 e * 1.50				
		Support B		Perman Variable Perman Variable	ent * 1.35 e * 1.50 ent * 1.35 e * 1.50				
Analysis results Maximum moment Maximum shear		M <sub>max</sub> = <b>7.4</b>   V <sub>max</sub> = <b>9.9</b>	k N	M <sub>min</sub> = 0 V <sub>min</sub> = -9	kNm A 9 kN				
Deflection		δ <b>- 25</b> p		δ <b>-</b> Ο	mm				
Maximum reaction at support A		$D_{max} = 2.3$ II	) レNI	17 2 kN					
Unfactored permanent load reaction Unfactored variable load reaction	Unfactored permanent load reaction at support A Unfactored variable load reaction at support A			NA_min —	17.2 NN				
Maximum reaction at support B	Maximum reaction at support B			R <sub>B_min</sub> =	17.2 kN				
Unfactored permanent load reacti	B R <sub>B_Permanent</sub>	= <b>3.9</b> kN							
Unfactored variable load reaction	at support B	R <sub>B_Variable</sub> =	<b>7.9</b> kN						
Section details Section type Steel grade EN 10210-1:2006 - Hot finished	structural holi	SHS 100x1 S235H	00x5.0 (Tata Si	teel Celsius) I fine grain stee	le				
Nominal thickness of element		t = 5.0 mm	i non anoy and	The grain stee	15				
Nominal vield strength		f <sub>v</sub> = <b>235</b> N/r	nm²						
Nominal ultimate tensile strength		$f_{\rm H} = 360  {\rm N/r}$	nm²						
Modulus of elasticity		E = 210000	N/mm <sup>2</sup>						
100			-> 5	€-					
	4	100							
Partial factors - Section 6.1									
Resistance of cross-sections		γ <sub>M0</sub> = <b>1.00</b>							
Resistance of members to instabi	lity	γм1 = <b>1.00</b>							
Resistance of tensile members to	Resistance of tensile members to fracture			γ <sub>M2</sub> = <b>1.10</b>					
l ateral restraint									
_ato, al i ooti allit		Span 1 has	full lateral restr	aint					

	Project	Extern	al Sairs		Job no.	)738-1
	Calcs for				Start page no /	Revision
GHU		Landing Be	am - 2 flights		Clart page no./	8
CHHHIII HUGHES DESTRIN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date
Effective length factors						
Effective length factor in maj	or axis	K <sub>y</sub> = <b>1.000</b>				
Effective length factor in min	or axis	K <sub>z</sub> = <b>1.000</b>				
Effective length factor for tors	sion	K <sub>LT.A</sub> = <b>1.20</b>	00			
		K <sub>LT.B</sub> = <b>1.00</b>	00			
Classification of cross sec	tions - Section	5.5				
		ε = √[235 Ν	I/mm <sup>2</sup> / f <sub>y</sub> ] = <b>1.0</b>	0		
Internal compression parts	subject to ben	ding - Table 5.2 (	sheet 1 of 3)			
Width of section	-	c = h - 3 * t	= <b>85</b> mm			
		c / t = 17.0	* ε <b>&lt;= 72</b> * ε	Class 1		
Internal compression parts	subject to com	pression only - T	able 5.2 (shee	t 1 of 3)		
Width of section		c = b - 3 * t	= 85 mm			
		c/t = 17.0	* ε <= 33 * ε	Class 1		
					Sec	tion is class
Check shear - Section 6.2.6	5					
Height of web		h <sub>w</sub> = h - 2 *	t = <b>90</b> mm			
Shear area factor		$\eta = 1.000$				
		h <sub>w</sub> / t < 72 *	'ε/η			
				Shear buckling	resistance o	an be ignored
Design shear force		$V_{Ed} = max(a)$	abs(V <sub>max</sub> ), abs('	V <sub>min</sub> )) = <b>9.9</b> kN		
Shear area - cl 6.2.6(3)		$A_v = A * h /$	(b + h) = <b>937</b> n	nm²		
Design shear resistance - cl	6.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$_{d} = A_{v} * (f_{y} / \sqrt{[3]})$	]) / γ <sub>M0</sub> = <b>127.1</b> kM	١	
		PAS	S - Design she	ear resistance e	xceeds desi	gn shear force
Check bending moment ma	ajor (y-y) axis - S	Section 6.2.5				
Design bending moment		$M_{Ed} = max($	abs(M <sub>s1_max</sub> ), at	$bs(M_{s1_min})) = 7.4$	kNm	
Design bending resistance m	oment - eq 6.13	$M_{c,Rd} = M_{pl,l}$	$_{Rd} = W_{pl.y} * f_y / \gamma_{f}$	<sub>M0</sub> = <b>15.6</b> kNm		
	PAS	S - Design bendi	ng resistance i	moment exceed	s design bei	nding momen
Check vertical deflection -	Section 7.2.1					
Consider deflection due to pe	ermanent and va	riable loads				
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	860 = <b>5</b> mm			
Maximum deflection span 1		$\delta = \max(ab)$	$s(\delta_{max})$ , abs( $\delta_{min}$	n)) = <b>2.53</b> mm		
		PAS	S - Maximum d	deflection does	not exceed o	deflection limit

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GHD	Calcs for	Landing Be		Start page no./Revision 9		
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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 Permanent \* 1.35 Support A Variable \* 1.50

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		Extern	al Sairs		G0	738-1	
GHD	Calcs for	Landing Be	am - 1 flight		Start page no./R	Revision 10	
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	GH	09/03/2023	GH	09/03/2023	Approved by	Approved date	
						_	
				Perman	ent * 1.35		
		Current D		Variable	ent * 4.25		
		Support B		Perman	ient ^ 1.35		
				variable	9 " 1.50		
Analysis results							
Maximum moment		M <sub>max</sub> = <b>5.1</b>	kNm	M <sub>min</sub> = <b>0</b>	kNm		
Maximum shear		V <sub>max</sub> = 7.7 k	<n n<="" td=""><td>V<sub>min</sub> = -7</td><td><b>′.8</b> kN</td><td></td></n>	V <sub>min</sub> = -7	<b>′.8</b> kN		
Deflection		δ <sub>max</sub> = <b>1.8</b> n	nm	$\delta_{min} = 0$	mm		
Maximum reaction at support A		$R_{A_{max}} = 7.7$	R <sub>A_max</sub> = <b>7.7</b> kN R <sub>A_r</sub>				
Unfactored permanent load read	ction at support /	A RA_Permanent	= <b>1.6</b> kN				
Unfactored variable load reaction	n at support A	R <sub>A_Variable</sub> =	<b>3.7</b> kN				
Maximum reaction at support B		R <sub>B_max</sub> = 15	.1 kN	$R_{B_{min}} =$	15.1 kN		
Unfactored permanent load read	ction at support I	B RB_Permanent	= <b>3.3</b> kN				
Unfactored variable load reaction	n at support B	R <sub>B_Variable</sub> =	7.1 kN				
Section details							
Section type	SHS 100x1	00x5.0 (Tata St	eel Celsius)				
Steel grade	S235H						
EN 10210-1:2006 - Hot finishe	d structural hol	low sections o	f non-alloy and	fine grain stee	ls		
Nominal thickness of element		t = <b>5.0</b> mm					
Nominal yield strength		f <sub>y</sub> = <b>235</b> N/r	nm²				
Nominal ultimate tensile strengt	h	$f_u = 360 \text{ N/r}$	nm²				
Modulus of elasticity		E = 210000	N/mm²				
, ,	3 2 	100	→ 5 ·	€-			
Doution footone . Or other 0.4							
Resistance of cross-sections		$\gamma_{M0} = 1 00$					
Resistance of members to insta	bility	$\gamma M0 = 1.00$					
Resistance of tensile members	$\gamma_{M2} = 1.00$						
Lateral restraint		1112 - 1110					
	Span 1 has	full lateral restra	aint				
Effective length factors		1					
Effective length factor in major a	axis	K <sub>v</sub> = 1.000					
Effective length factor in minor a	axis	K <sub>z</sub> = <b>1.000</b>					

	Project				Job no.	
		Extern	al Sairs		G07	38-1
	Calcs for				Start page no./Re	evision
GUD		Landing Be	am - 1 flight		1	1
GRAHAM HUGHES DESIGN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	GH	09/03/2023	GH	09/03/2023		
Effective length factor for torsion	٦	$K_{LT.A} = 1.20$	0			
		KLT.B = 1.00	iu ii			
Classification of cross section	ns - Section 5.5	5				
		ε = √[235 N	$[/mm^2 / f_y] = 1.00$	)		
Internal compression parts su	ibject to bendi	ng - Table 5.2 (s	sheet 1 of 3)			
Width of section		c = h - 3 * t	= <b>85</b> mm			
		c / t = 17.0	* ε <b>&lt;= 72</b> * ε	Class 1		
Internal compression parts su	bject to comp	ression only - T	able 5.2 (sheet	1 of 3)		
Width of section		c = b - 3 * t	= <b>85</b> mm			
		c / t = 17.0	* ε <b>&lt;= 33</b> * ε	Class 1		
					Secti	on is class 1
Check shear - Section 6.2.6						
Height of web		h <sub>w</sub> = h - 2 *	t = <b>90</b> mm			
Shear area factor		η = <b>1.000</b>				
		h <sub>w</sub> / t < 72 *	ε/η			
			9	Shear buckling	resistance ca	n be ignored
Design shear force		$V_{Ed} = max(a)$	abs(V <sub>max</sub> ), abs(V	′ <sub>min</sub> )) = <b>7.8</b> kN		
Shear area - cl 6.2.6(3)		$A_v = A * h /$	(b + h) = <b>937</b> m	m²		
Design shear resistance - cl 6.2	.6(2)	$V_{c,Rd} = V_{pl,R}$	$_{d} = A_{v} * (f_{y} / \sqrt{[3]})$	/ γ <sub>M0</sub> = <b>127.1</b> kN	1	
		PAS	S - Design she	ar resistance ex	ceeds desigr	n shear force
Check bending moment majo	r (y-y) axis - Se	ction 6.2.5				
Design bending moment		M <sub>Ed</sub> = max(	abs(M <sub>s1_max</sub> ), ab	$s(M_{s1_{min}})) = 5.1$	kNm	
Design bending resistance mon	nent - eq 6.13	$M_{c,Rd} = M_{pl,F}$	$R_{d} = W_{pl.y} * f_y / \gamma_M$	<sub>0</sub> = <b>15.6</b> kNm		
	PASS	- Design bendii	ng resistance n	noment exceed	s design benc	ding moment
Check vertical deflection - Se	ction 7.2.1					
Consider deflection due to perm	anent and varia	able loads				
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	60 = <b>5</b> mm			
Maximum deflection span 1		$\delta$ = max(ab	s( $\delta_{max}$ ), abs( $\delta_{min}$ )	)) = <b>1.771</b> mm		
		PAS	S - Maximum de	eflection does r	not exceed de	flection limit

	Project				Job no.	
	External Sairs				G0738-1	
GHD	Calcs for	Landing Be	eam - 1.2m		Start page no./Revision 12	
GRAHAM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date

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



Support conditions Support A

Support B

Applied loading Beam loads

Load combinations Load combination 1 Vertically restrained Rotationally free Vertically restrained Rotationally free

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

Support A

Support B

Permanent \* 1.35 Variable \* 1.50 Permanent \* 1.35 Variable \* 1.50 Permanent \* 1.35 Variable \* 1.50

TEDDS calculation version 3.0.14

	Project	Extern	al Sairs		Job no.	)738-1
	Calcs for				Start page no./I	Revision
GUU		Landing B	eam - 1.2m			13
GRAHAM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved da
Analysis results						
Maximum moment		M <sub>max</sub> = 2 kN	Nm	$M_{min} = 0$	) kNm	
Maximum shear		V <sub>max</sub> = <b>4.4</b>	kN	V <sub>min</sub> = -	<b>4.4</b> kN	
Deflection		$\delta_{max} = 0.8 r$	nm	$\delta_{min} = 0$	mm	
Maximum reaction at support A		$R_{A_{max}} = 4.4$	<b>4</b> kN	R <sub>A_min</sub> =	• <b>4.4</b> kN	
Unfactored permanent load reac	tion at support A	A RA_Permanent	= <b>0.9</b> kN			
Unfactored variable load reaction	n at support A	R <sub>A_Variable</sub> =	2.2 kN			
Maximum reaction at support B		$R_{B_{max}} = 4.4$	<b>4</b> kN	R <sub>B_min</sub> =	• <b>4.4</b> kN	
Unfactored permanent load reac	tion at support E	B R <sub>B_Permanent</sub>	= <b>0.9</b> kN			
Unfactored variable load reaction	n at support B	$R_{B_Variable} =$	<b>2.2</b> kN			
Section details			100×5 0 /Tata 9	Stool Colsius)		
Steel grade		S235H	10085.0 (1818 3	Steel Celsius)		
EN 10210-1:2006 - Hot finished	l structural hol	low sections o	of non-allov an	d fine grain stee	als	
Nominal thickness of element		t = 5.0  mm	in non-anoy an	a fille grain stee	15	
Nominal vield strength		$f_v = 235 N/r$	mm²			
Nominal ultimate tensile strength	n	$f_{\rm H} = 360  {\rm N}/{\rm I}$	mm²			
i toniono on ongu	•		$\mathbf{D}$ N/mm <sup>2</sup>			
Modulus of elasticity		E = 210000				
Modulus of elasticity		E = 21000	<b>5</b> IV/IIIII-	5		
Modulus of elasticity		E = 210000		5		
Modulus of elasticity		E = 210000	→ 8	5		
Modulus of elasticity		E = 210000		5 ◀-		
Modulus of elasticity	pility	E = 210000 100 γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00		5		
Modulus of elasticity	bility o fracture	E = 210000 100 γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10		5 ◀-		
Modulus of elasticity	bility o fracture	E = 210000 100 γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10 Span 1 has	s full lateral rest	s ►		
Modulus of elasticity	bility of fracture	E = 210000 100 γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10 Span 1 has	s full lateral rest	traint		
Modulus of elasticity  Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instate Resistance of tensile members to Lateral restraint  Effective length factors Effective length factors	bility o fracture	E = 210000 100 γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10 Span 1 has K <sub>w</sub> = 1.000	s full lateral rest	s ← ► traint		
Modulus of elasticity	bility o fracture	E = 210000 γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10 Span 1 has K <sub>y</sub> = 1.000 K <sub>z</sub> = 1.000	s full lateral rest	traint		
Modulus of elasticity  Partial factors - Section 6.1 Resistance of cross-sections Resistance of tensile members to Lateral restraint  Effective length factors Effective length factor in major a Effective length factor in major a Effective length factor for torsion	bility o fracture xis xis	E = 210000 $γ_{M0} = 1.00$ $γ_{M1} = 1.00$ $γ_{M2} = 1.10$ Span 1 has $K_y = 1.000$ $K_z = 1.000$ $K_z = 1.000$	s full lateral rest	traint		

		Project	E /			Job no.	700.4
_			Extern	al Sairs		GU	0738-1
6	HD	Calcs for	Landing B	eam - 1.2m		Start page no./	Revision 14
GRAHAN	NHUGHES DESIGN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
		GH	09/03/2023	GH	09/03/2023		
Classific	ation of cross se	ctions - Section 5	5				
Classific			ε = √[235 Ν	$J/mm^2 / f_y] = 1.0$	00		
Internal	compression part	ts subject to bend	ling - Table 5.2 (	sheet 1 of 3)			
Width of	section		c = h - 3 * t	: = <b>85</b> mm			
			c / t = 17.0	* ε <= 72 * ε	Class 1		
Internal	compression part	ts subject to com	pression only - T	able 5.2 (she	et 1 of 3)		
Width of	section		c = b - 3 * t	: = <b>85</b> mm			
			c / t = 17.0	* ε <b>&lt;= 33</b> * ε	Class 1		
						Sec	tion is class 1
Check sł	hear - Section 6.2	2.6					
Height of	web		h <sub>w</sub> = h - 2 *	t = <b>90</b> mm			
Shear are	ea factor		η = <b>1.000</b>				
			h <sub>w</sub> / t < 72 *	*ε/η			
					Shear buckling	resistance c	an be ignored
Design sl	hear force		V <sub>Ed</sub> = max(	abs(V <sub>max</sub> ), abs	$s(V_{min})) = 4.4 \text{ kN}$		
Shear are	ea - cl 6.2.6(3)		$A_v = A * h /$	' (b + h) = <b>937</b>	mm <sup>2</sup>		
Design sl	hear resistance - c	cl 6.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$A_{d} = A_{v} * (f_{y} / \sqrt{3})$	3]) / γ <sub>M0</sub> = <b>127.1</b> kľ	N	
			PAS	s - Design sn	iear resistance e	xceeas aesi	gn snear force
Check be	ending moment n	najor (y-y) axis - S	ection 6.2.5				
Design be	ending moment		$M_{Ed} = max($	(abs(M <sub>s1_max</sub> ), a	$abs(M_{s1_{min}})) = 2 k$	Nm	
Design be	ending resistance	moment - eq 6.13	$M_{c,Rd}=M_{Pl,l}$	$Rd = W_{pl.y} * f_y / r$	γ <sub>M0</sub> = <b>15.6</b> kNm		
		PASS	5 - Design bendi	ng resistance	e moment exceed	is design bei	naing moment
Check ve	ertical deflection	- Section 7.2.1					
Consider	deflection due to p	permanent and var	able loads				
Limiting c	deflection		$\delta_{\text{lim}} = L_{s1} / 3$	360 = <b>5</b> mm			
Maximum	n deflection span 1		$\delta = \max(ab)$	os(ð <sub>max</sub> ), abs(δ <sub>n</sub>	<sub>min</sub> )) = <b>0.791</b> mm	, <b>.</b>	
			PAS	S - Maximum	deflection does	not exceed c	leflection limit

	Project				Job no.	
	External Sairs				G0738-1	
GHD	Calcs for	Landing Be	eam - 0.8m		Start page no./Revision 15	
GRAHAM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date

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



Support conditions Support A

Support B

Applied loading Beam loads

Load combinations Load combination 1 Vertically restrained Rotationally free Vertically restrained Rotationally free

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

Support A

Support B

Permanent \* 1.35 Variable \* 1.50 Permanent \* 1.35 Variable \* 1.50 Permanent \* 1.35 Variable \* 1.50

TEDDS calculation version 3.0.14

	Project	Externa	al Sairs		Job no.	)738-1
	Calcs for				Start page no./	Revision
GRU		Landing Be	am - 0.8m			16
GRAHRM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date
Analysis results						
Maximum moment		M <sub>max</sub> = <b>1.5</b> k	Nm	$M_{min} = 0$	kNm	
Maximum shear		V <sub>max</sub> = <b>3.2</b> k	N	V <sub>min</sub> = -:	<b>3.2</b> kN	
Deflection		δ <sub>max</sub> <b>= 0.6</b> m	m	$\delta_{min} = 0$	mm	
Maximum reaction at support A		R <sub>A_max</sub> = <b>3.2</b>	kN	$R_{A_{min}} =$	<b>3.2</b> kN	
Unfactored permanent load rea	iction at support /	A RA_Permanent =	= <b>0.8</b> kN			
Unfactored variable load reaction	on at support A	$R_{A_Variable} = 1$	I <b>.4</b> kN	5	0.01.11	
Maximum reaction at support B	ation at augment l	RB_max = <b>3.2</b>		KB_min =	3.2 KIN	
Unlactored permanent load rea	iction at support i	B RB_Permanent =	= <b>U.8</b> KIN			
Section details	on at support b	RB_Variable =	1.4 KIN			
Section type		SHS 100x1	00x5.0 (Tata \$	Steel Celsius)		
Steel grade		S235H				
EN 10210-1:2006 - Hot finishe	ed structural hol	low sections of	non-alloy an	d fine grain stee	ls	
Nominal thickness of element		t = <b>5.0</b> mm	0			
Nominal yield strength		f <sub>y</sub> = <b>235</b> N/m	1m <sup>2</sup>			
Nominal ultimate tensile streng	th	$f_u = 360 \text{ N/m}$	1m²			
-						
			-	5 -		
-		100	-	5 •-		
Partial factors - Section 6.1		100	-	5 ←-		
Partial factors - Section 6.1 Resistance of cross-sections		100 γ <sub>M0</sub> = <b>1.00</b>	-	▶		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta	ability			5 ←-		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members	ability to fracture	γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10		5 ←		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint	ability to fracture			5 ←		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint	ability to fracture		full lateral res	5 ← ►		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors	ability to fracture		full lateral res	s ← traint		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors Effective length factor in major	ability to fracture		full lateral res	s ← traint		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors Effective length factor in major Effective length factor in major	ability to fracture	γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10 γ <sub>M2</sub> = 1.10 Span 1 has K <sub>y</sub> = 1.000 K <sub>z</sub> = 1.000	full lateral res	5 ← traint		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor in minor	ability to fracture axis axis axis n	γ <sub>M0</sub> = 1.00 γ <sub>M1</sub> = 1.00 γ <sub>M2</sub> = 1.10 γ <sub>M2</sub> = 1.10 Span 1 has K <sub>y</sub> = 1.000 K <sub>z</sub> = 1.000 K <sub>LTA</sub> = 1.200	full lateral res	traint		

	Project	E /			Job no.	700.4
		Extern	al Sairs		G	0738-1
GHD	Calcs for	Landing B	eam - 0.8m		Start page no./	Revision 17
GRAHAM HUGHES DESIGN	Calcs by	Calcs date		Checked date	Approved by	Approved date
	GH	09/03/2023	GH	09/03/2023	, approved by	Approved date
		•	•		•	
Classification of cross see	ctions - Section 5	5.5 s − √[235 N	l/mm² / f.1 – <b>1</b>	00		
Internal compression part	s subject to ben	، 200 با 200 م / 2 5 2 ماله - Table 5	sheet 1 of 3)			
Width of section		c = h - 3 * f	t = 85 mm			
		c / t = 17.0	* ε <= 72 * ε	Class 1		
Internal compression part	s subject to com	pression only - 1	able 5.2 (she	et 1 of 3)		
Width of section		c = b - 3 * t	t = <b>85</b> mm			
		c / t = 17.0	* ε <b>&lt;= 33</b> * ε	Class 1		
					Sec	tion is class 1
Check shear - Section 6.2	.6					
Height of web		h <sub>w</sub> = h - 2 *	t = <b>90</b> mm			
Shear area factor		η = <b>1.000</b>				
		h <sub>w</sub> / t < 72 *	*ε/η	04		
Docian shoar force		$V_{r} = mov($	abc(1/) abc		resistance d	an be ignored
Shear area - cl 6.2.6(3)		$A_{\rm v} = A * h /$	(b + h) = 937	$mm^2$ mm <sup>2</sup>		
Design shear resistance - c	l 6.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$A_{d} = A_{v} * (f_{v} / \sqrt{s})$	3]) / γ <sub>M0</sub> = <b>127.1</b> kl	Ν	
Ũ		PAS	SS - Design sh	hear resistance e	xceeds desi	gn shear force
Check bending moment m	najor (y-y) axis - S	Section 6.2.5				
Design bending moment		$M_{Ed} = max($	(abs(M <sub>s1_max</sub> ), a	abs(M <sub>s1_min</sub> )) = <b>1.5</b>	kNm	
Design bending resistance	moment - eq 6.13	$M_{c,Rd} = M_{pl,}$	$Rd = W_{pl.y} * f_y / f_y$	γ <sub>M0</sub> = <b>15.6</b> kNm		
	PAS	S - Design bendi	ng resistance	e moment exceed	ls design bei	nding moment
Check vertical deflection -	Section 7.2.1					
Consider deflection due to p	permanent and va	riable loads				
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	360 = 5  mm	N 0 504		
iviaximum deflection span 1		ð = max(ab	os(ð <sub>max</sub> ), abs(ðr	min)) = 0.581  mm	not avaacd	aflaction limit
		PAS	s - waximum	dellection does	not exceed t	ienection limit

	Project				Job no.	
	External Sairs				G0738-1	
GHD	Calcs for Main Beam				Start page no./Revision 18	
GRAHAM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date

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



Support conditions Support A

Support B

Applied loading Beam loads

Load combinations

Vertically restrained Rotationally free Vertically restrained Rotationally free

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

Support A

Support B

Permanent \* 1.35 Variable \* 1.50 Permanent \* 1.35 Variable \* 1.50 Permanent \* 1.35 Variable \* 1.50

TEDDS calculation version 3.0.14

	Project	<b>F</b>		Job no.		
		Extern	ai Sairs		GO	1/ 38-1
GHD	Calcs for	Main	Beam		Start page no./F	Revision 19
GRAHAM HUGHES DESIGN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	GH	09/03/2023	GH	09/03/2023		
Analysis results						
Maximum moment		M <sub>max</sub> = <b>18</b> k	Nm	$M_{min} = 0$	) kNm	
Maximum shear		V <sub>max</sub> = <b>11.8</b>	kN	V <sub>min</sub> = -*	11.8 kN	
Deflection		$\delta_{max} = 10.4$	mm	$\delta_{min} = 0$	mm	
Maximum reaction at support A		R <sub>A max</sub> = 11	<b>.8</b> kN	R <sub>A min</sub> =	11.8 kN	
Unfactored permanent load read	ction at support	A RA_Permanent	= <b>3.3</b> kN	_		
Unfactored variable load reaction	n at support A	R <sub>A_Variable</sub> =	<b>4.9</b> kN			
Maximum reaction at support B		R <sub>B_max</sub> = 11	<b>.8</b> kN	R <sub>B_min</sub> =	11.8 kN	
Unfactored permanent load read	ction at support l	B R <sub>B_Permanent</sub>	= <b>3.3</b> kN			
Unfactored variable load reaction	n at support B	R <sub>B_Variable</sub> =	<b>4.9</b> kN			
Section details						
Section type		RHS 200x1	00x8.0 (Tata S	Steel Celsius)		
Steel grade		S235H				
EN 10210-1:2006 - Hot finishe	d structural hol	low sections o	f non-alloy an	d fine grain stee	ls	
Nominal thickness of element		t = <b>8.0</b> mm				
Nominal yield strength		$f_y = 235 \text{ N/r}$	nm²			
Nominal ultimate tensile strengt	h	f <sub>u</sub> = <b>360</b> N/r	mm²			
Modulus of elasticity		E = <b>210000</b>	) N/mm²			
	200	100	→ 8 ←			
Partial factors - Section 6.1		1.00				
Resistance of cross-sections		γ <sub>M0</sub> = <b>1.00</b>				
Resistance of members to insta	bility	γ <sub>M1</sub> = <b>1.00</b>				
Resistance of tensile members	to fracture	γ <sub>M2</sub> = <b>1.10</b>				
Lateral restraint						
		Span 1 has	s full lateral rest	raint		
Effective length factors						
Effective length factor in major a	axis	K <sub>y</sub> = <b>1.000</b>				
Effective length factor in minor a	axis	K <sub>z</sub> = <b>1.000</b>				
Effective length factor for torsion	า	K <sub>LT.A</sub> = <b>1.20</b>	0			
		K <sub>LT.B</sub> = <b>1.00</b>	0			

	Project	Extern	al Sairs		Job no.	)738-1
	Calcs for	Exton			Start page no //	Revision
GHU		Main	Beam		Clair page no./	20
GRAHAM HUGHES DESIGN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	ОП	09/03/2023	GH	09/03/2023		
Classification of cross see	ctions - Section	5.5				
		ε = √[235 Ν	$1/mm^2 / f_y] = 1$ .	.00		
Internal compression part	s subject to ben	ding - Table 5.2 (	sheet 1 of 3)			
Width of section		c = h - 3 * t	= <b>176</b> mm			
		c / t = 22.0	* ε <= 72 * ε	Class 1		
Internal compression part	s subject to com	pression only - T	able 5.2 (she	et 1 of 3)		
Width of section		c = b - 3 * t	= <b>76</b> mm			
		c / t = 9.5 *	ε <= 33 * ε	Class 1		
					Sec	tion is class 1
Check shear - Section 6.2.	.6					
Height of web		h <sub>w</sub> = h - 2 *	t = <b>184</b> mm			
Shear area factor		η = <b>1.000</b>				
		h <sub>w</sub> / t < 72 *	'ε/η			
				Shear buckling	resistance c	an be ignored
Design shear force		V <sub>Ed</sub> = max(	abs(V <sub>max</sub> ), abs	$S(V_{min})) = 11.8 \text{ kN}$		
Shear area - cl 6.2.6(3)		$A_v = A * h /$	(b + h) = 298	4 mm²		
Design shear resistance - cl	16.2.6(2)	$V_{c,Rd} = V_{pl,R}$	d = Av * (fy / √[	3]) / γ <sub>M0</sub> = <b>404.8</b> kl	Ν	
		PAS	S - Design si	near resistance e	xceeas aesię	gn snear force
Check bending moment m	najor (y-y) axis - S	Section 6.2.5				
Design bending moment		$M_{Ed} = max($	abs(M <sub>s1_max</sub> ), a	$abs(M_{s1_min})) = 18$	kNm	
Design bending resistance	moment - eq 6.13	$M_{c,Rd} = M_{pl,l}$	$Rd = W_{pl.y} * f_y /$	γ <sub>M0</sub> = <b>66.3</b> kNm		
	PAS	S - Design bendi	ng resistance	e moment exceed	ls design bei	nding moment
Check vertical deflection -	Section 7.2.1					
Consider deflection due to p	permanent and va	riable loads				
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	860 = <b>16.9</b> mm	n		
Maximum deflection span 1		$\delta = \max(ab)$	$s(\delta_{max})$ , $abs(\delta_{r})$	<sub>min</sub> )) = <b>10.357</b> mm		
		PAS	S - Maximum	deflection does	not exceed a	leflection limit

	Project				Job no.	
		Externa	G0738-1			
GHD	Calcs for	lcs for Landing Beam - 1 flight				vision 21
GRAHAM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved date

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





	Project	Extern	al Sairs		Job no. G(	)738-1
CUD	Calcs for				Start page no./	Revision
GUD		Landing Be	eam - 1 flight			22
GRAHAM HUGHES DESIGN	Calcs by GH	Calcs date 09/03/2023	Checked by GH	Checked date 09/03/2023	Approved by	Approved da
Analysis results						
Maximum moment		$M_{max} = 2.8$	kNm	$M_{min} = 0$	) kNm	
Maximum shear		V <sub>max</sub> = 6.1	kN	V <sub>min</sub> = -	6.1 kN	
Deflection		$\delta_{max} = 2.2 r$	nm	$\delta_{min} = 0$	mm	
Maximum reaction at support A	N N	$R_{A_{max}} = 6.$	1 kN	R <sub>A_min</sub> =	6.1 kN	
Unfactored permanent load rea	action at support A	A RA_Permanent	= <b>0.6</b> kN			
Unfactored variable load reaction	on at support A	$R_{A_Variable} =$	3.6 kN	_		
Maximum reaction at support B	3 	$R_{B_{max}} = 6.7$	1 kN	R <sub>B_min</sub> =	6.1 kN	
Unfactored permanent load rea	action at support E	B RB_Permanent	= 0.6 KN			
Unfactored variable load reaction	on at support B	$R_{B_{Variable}} =$	3.6 KN			
Section details		SHS 80x80	)x5 0 (Tata Ste	el Celsius)		
Steel grade		S235H		er Gersius)		
EN 10210-1:2006 - Hot finishe	ed structural holl	low sections of	of non-allov an	d fine grain stee	ls	
Nominal thickness of element		t = <b>5.0</b> mm	, non anoy an	a into grain otos		
Nominal vield strength		f <sub>v</sub> = <b>235</b> N/r	mm²			
		f - 360 N/	mm <sup>2</sup>			
Nominal ultimate tensile streng	th	III = 300 IV				
Nominal ultimate tensile streng Modulus of elasticity	th	E = 210000	) N/mm <sup>2</sup>			
Nominal ultimate tensile streng Modulus of elasticity	th	E = 210000	) N/mm <sup>2</sup>	•		
Nominal ultimate tensile streng Modulus of elasticity	th	E = 210000	D N/mm <sup>2</sup>	•		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1	th	E = 210000	D N/mm <sup>2</sup>	•		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections	th	E = 210000 E = 210000 80 γμο = 1.00	D N/mm <sup>2</sup>	•		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta	ability	Tu = 300 T0/1         E = 210000         80         γM0 = 1.00         γM1 = 1.00	D N/mm <sup>2</sup>	•		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members	ability to fracture	γM0 = 1.00         γM1 = 1.00         γM2 = 1.10	) N/mm <sup>2</sup>	•		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint	ability to fracture	80         γM0 = 1.00         γM1 = 1.00         γM2 = 1.10         Span 1 has	5 full lateral res	traint		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint	ability to fracture	R = 300 ft/r         E = 210000         β         γM0 = 1.00         γM1 = 1.00         γM2 = 1.10         Span 1 has	5 full lateral res	traint		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors Effective length factor in major	axis	80         γM0 = 1.00         γM1 = 1.00         γM2 = 1.10         Span 1 has         Ky = 1.000	5 full lateral res	traint		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors Effective length factor in major Effective length factor in major	th ability to fracture axis axis	γM0 = 1.00         γM0 = 1.00         γM1 = 1.00         γM2 = 1.10         Span 1 has         Ky = 1.000         K <sub>2</sub> = 1.000	D N/mm <sup>2</sup>	traint		
Nominal ultimate tensile streng Modulus of elasticity Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members Lateral restraint Effective length factors Effective length factor in major Effective length factor in minor Effective length factor in minor	ability to fracture	80         γM0 = 1.00         γM1 = 1.00         γM2 = 1.10         Span 1 has         Ky = 1.000         Kz = 1.000         KLTA = 1.20	5 full lateral res	traint		

	Project	Extern	al Sairs		Job no.	738-1
	Color for	Extern			Start page pg /E	
GHU	Calcs for	Landing Be	am - 1 flight		Start page no./P	23
GRAHAM HUGHES DESIGN	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	GH	09/03/2023	GH	09/03/2023		
 Classification of cross soci	ions - Section 5	5				
	ions - Section 5.	ε = √[235 N	/mm <sup>2</sup> / f <sub>y</sub> ] = <b>1.(</b>	00		
Internal compression parts	subject to bend	ing - Table 5.2 (s	sheet 1 of 3)			
Width of section		c = h - 3 * t	= <b>65</b> mm			
		c / t = 13.0	* ε <= 72 * ε	Class 1		
Internal compression parts	subject to comp	pression only - T	able 5.2 (shee	et 1 of 3)		
Width of section		c = b - 3 * t	= <b>65</b> mm			
		c / t = 13.0	* ε <b>&lt;= 33</b> * ε	Class 1		
					Sec	tion is class 1
Check shear - Section 6.2.6						
Height of web		h <sub>w</sub> = h - 2 *	t = <b>70</b> mm			
Shear area factor		η = <b>1.000</b>				
		h <sub>w</sub> / t < 72 *	ε/η			
				Shear buckling	resistance c	an be ignored
Design shear force		$V_{Ed} = max(a)$	abs(V <sub>max</sub> ), abs	(V <sub>min</sub> )) = <b>6.1</b> kN		
Shear area - cl 6.2.6(3)		$A_v = A * h /$	(b + h) = <b>737</b> i	mm²		
Design shear resistance - cl 6	5.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$d = A_v * (f_y / \sqrt{3})$	3]) / γ <sub>M0</sub> = <b>99.9</b> kN		
		PAS	S - Design sh	ear resistance e	xceeds desig	In shear force
Check bending moment ma	jor (y-y) axis - S	ection 6.2.5				
Design bending moment		M <sub>Ed</sub> = max(	abs(M <sub>s1_max</sub> ), a	$abs(M_{s1_{min}})) = 2.8$	kNm	
Design bending resistance mo	oment - eq 6.13	$M_{c,Rd} = M_{pl,F}$	$Rd = W_{pl.y} * f_y / \gamma$	<sub>(M0</sub> = <b>9.7</b> kNm		
	PASS	S - Design bendii	ng resistance	moment exceed	s design ben	ding moment
Check vertical deflection - S	Section 7.2.1					
Consider deflection due to per	rmanent and vari	able loads				
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 3$	60 = <b>5</b> mm			
Maximum deflection span 1		$\delta = \max(ab)$	s( $\delta_{max}$ ), abs( $\delta_{max}$	<sub>nin</sub> )) = <b>2.198</b> mm		
		PAS	S - Maximum	deflection does	not exceed d	eflection limit

GRAHAM HUGHES DESIGN	Project	Externa	Job no. G0738-1			
	Calcs for	Column &	Start page no./Revision 24			
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		•		•		•

### STEEL COLUMN AND BASE PLATE DESIGN (EN1993)

Tedds calculation version 1.0.03

#### STEEL COLUMN DESIGN

e UK national

culation version 1.1.06

In accordance with EN1993-1-1:2005 incorporati annex	ing Corrigenda February 2006 and April 2009 and th
	Tedds calo
Partial factors - Section 6.1	
Resistance of cross-sections	γмо = <b>1</b>
Resistance of members to instability	γ <sub>M1</sub> = <b>1</b>
Resistance of cross-sections in tension to fracture	γ <sub>M2</sub> = <b>1.1</b>
	<ul> <li>SHS 100x100x5.0 (Tata Steel Celsius)</li> <li>Section depth, h, 100 mm</li> <li>Section breadth, b, 100 mm</li> <li>Mass of section, Mass, 14.7 kg/m</li> <li>Section thickness, t, 5 mm</li> <li>Area of section, A, 1873 mm<sup>2</sup></li> <li>Radius of gyration about y-axis, i<sub>y</sub>, 38.623 mm</li> <li>Elastic section modulus about y-axis, W<sub>el.y</sub>, 55866 mm<sup>3</sup></li> <li>Elastic section modulus about y-axis, W<sub>gl.y</sub>, 66358 mm<sup>3</sup></li> <li>Plastic section modulus about y-axis, I<sub>y</sub>, 2794323 mm<sup>4</sup></li> <li>Second moment of area about z-axis, I<sub>z</sub>, 2794323 mm<sup>4</sup></li> </ul>
Column details	
Column section	SHS 100x100x5.0
Steel grade	User defined
Yield strength	fy <b>= 355</b> N/mm <sup>2</sup>
Ultimate strength	$f_u = 470 \text{ N/mm}^2$
Modulus of elasticity	E = <b>210</b> kN/mm <sup>2</sup>
Poisson's ratio	v = <b>0.3</b>
Shear modulus	G = E / [2 × (1 + v)] = <b>80.8</b> kN/mm <sup>2</sup>
<b>Column geometry</b> System length for buckling - Major axis System length for buckling - Minor axis The column is not part of a sway frame in the direct The column is not part of a sway frame in the direct	$L_y = 4500 \text{ mm}$ $L_z = 4500 \text{ mm}$ tion of the minor axis tion of the major axis
Column loading	
Axial load	N <sub>Ed</sub> = <b>15</b> kN (Compression)
Major axis moment at end 1 - Bottom	M <sub>y,Ed1</sub> = <b>0.0</b> kNm
Major axis moment at end 2 - Top	M <sub>y,Ed2</sub> = <b>0.0</b> kNm
Minor axis moment at end 1 - Bottom	M <sub>z,Ed1</sub> = <b>0.0</b> kNm

	Project	Extorp	ol Staire		Job no.	729 1
		Externa			60	730-1
GHD	Calcs for	Column &	Baseplate		Start page no./F	Revision 25
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Minor axis moment at end 2 - 1	ор	M <sub>z,Ed2</sub> = <b>0.0</b>	kNm			
Major axis shear force		V <sub>y,Ed</sub> = <b>0</b> kN				
Minor axis shear force		V <sub>z,Ed</sub> = <b>0</b> kN				
Buckling length for flexural b	ouckling - Majo	or axis				
End restraint factor		K <sub>y</sub> = <b>1.000</b>				
Buckling length		$L_{cr\_y} = L_y \times I$	K <sub>y</sub> = <b>4500</b> mm			
Buckling length for flexural b	ouckling - Mino	or axis				
End restraint factor		K <sub>z</sub> = <b>1.000</b>				
Buckling length		$L_{cr_z} = L_z \times I$	Kz = <b>4500</b> mm			
Web section classification (T	able 5.2)					
Coefficient depending on fy	-	ε = √(235 N	/mm <sup>2</sup> / f <sub>y</sub> ) = <b>0.8</b>	314		
Depth between fillets		$c_w = h - 3 \times$	t = <b>85.0</b> mm			
Ratio of c/t		ratio <sub>w</sub> = c <sub>w</sub> /	t = <b>17.00</b>			
Length of web taken by axial lo	ad	$I_w = min(N_{Eo})$	$_{\rm i}$ / (2 × f <sub>y</sub> × t), c	<sub>w</sub> ) = <b>4.2</b> mm		
For class 1 & 2 proportion in co	ompression	$\alpha = (c_w/2 +$	w/2) / c <sub>w</sub> = <b>0.52</b>	25		
Limit for class 1 web		$Limit_{1w} = (3)$	96 * ε) / (13 * α	a - 1) = <b>55.33</b>		
					The	web is class 1
Flange section classification	(Table 5.2)					
Depth between fillets		$c_f = b - 3 \times 1$	: = <b>85.0</b> mm			
Ratio of c/t		$ratio_f = c_f / t$	= 17.00			
Limit for class 1 flange		Limit <sub>1f</sub> = 33	× ε <b>= 26.85</b>			
Limit for class 2 flange		Limit <sub>2f</sub> = 38	×ε <b>= 30.92</b>			
Limit for class 3 flange		Limit <sub>3f</sub> = 42	×ε <b>= 34.17</b>			
					The fla	nge is class 1
Overall section classification	1				The sec	tion is class 1
Resistance of cross section	(cl. 6.2)					
Compression (cl. 6.2.4)						
Design force		N <sub>Ed</sub> = <b>15</b> kN	l			
Design resistance		$N_{c,Rd} = N_{pl,Rd}$	$d = A \times f_y / \gamma_{MO} =$	= <b>665</b> kN		
		$N_{Ed} / N_{c,Rd} =$	0.023			
		PASS - The co	ompression de	esign resistance	exceeds the	e design force
Buckling resistance (cl. 6.3)						
Yield strength for buckling resist	stance	f <sub>y</sub> = <b>355</b> N/n	nm²			
Flexural buckling - Major axi	5					
Elastic critical buckling force		$N_{cr,y} = \pi^2 \times I$	$\Xi \times I_y / L_{cr_y^2} = 2$	286 kN		
Non-dimensional slenderness		$\lambda_y = \sqrt{A \times A}$	f <sub>y</sub> / N <sub>cr,y</sub> ) = <b>1.52</b>	25		
Buckling curve (Table 6.2)		a				
Imperiection factor (Table 6.1)		$\alpha_y = 0.21$	1		<b>•</b>	
		$\Psi_{\rm y} = 0.5 \times [$	1 + αy × ( λy - (	$(0.2) + \lambda y^2 = 1.80$	<u>ک</u>	
Reduction factor		$\chi_y = \min(1.0)$	λ, Ι / [Ψy + ∿(Φ λ λ λ f - / ···	$y^{-} - \lambda y^{-})]) = 0.362$		
Design Duckling resistance		$\mathbf{N}$ b,y,Rd = $\chi$ y >	κ Α × Iy / γM1 =	24U.0 KIN		

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 $N_{Ed} \ / \ N_{b,y,Rd} = \textbf{0.062}$  <code>PASS - The flexural buckling resistance exceeds the design axial load</code>

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

#### Flexural buckling - Minor axis

Elastic critical buckling force	$N_{cr,z} = \pi^2 \times E \times I_z / L_{cr_z^2} = 286 \text{ kN}$
Non-dimensional slenderness	$\overline{\lambda}_z = \sqrt{(A \times f_y / N_{cr,z})} = 1.525$
Buckling curve (Table 6.2)	а
Imperfection factor (Table 6.1)	α <sub>z</sub> = <b>0.21</b>
Parameter $\Phi$	$\Phi_z = 0.5 \times [1 + \alpha_z \times (\overline{\lambda}_z - 0.2) + \overline{\lambda}_z^2] = 1.802$
Reduction factor	$\chi_z = \min(1.0, 1 / [\Phi_z + \sqrt{(\Phi_z^2 - \overline{\lambda}_z^2)}]) = 0.362$
Design buckling resistance	$N_{b,z,Rd} = \chi_z \times A \times f_y$ / $\gamma_{M1} = 240.8$ kN
	N <sub>Ed</sub> / N <sub>b,z,Rd</sub> = <b>0.062</b>
	PASS - The flexural buckling resistance exceeds the design axial load
Minimum buckling resistance	
Minimum buckling resistance	N <sub>b,Rd</sub> = min(N <sub>b,y,Rd</sub> , N <sub>b,z,Rd</sub> ) = <b>240.8</b> kN

#### 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

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



	Project External Stairs				Job no. G0738-1					
GHD	Calcs for	Column &	Basenlate		Start page no./F	Revision 27				
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	GH	09/03/2023	GH	09/03/2023						
Thickness		t = <b>5</b> mm								
Base plate details										
Length		h <sub>p</sub> = <b>300</b> m	m							
Width		b <sub>p</sub> = <b>300</b> m	m							
Thickness	t <sub>p</sub> = <b>15</b> mm									
Column eccentricity in x-axis		e <sub>bpx</sub> = <b>0</b> mm	ו							
Anchor details										
Number of anchors LHS		n1 = <b>2</b>								
Edge distance in x-axis		e <sub>x1</sub> = <b>50</b> mr	n							
Edge distance in y-axis		e <sub>y1</sub> = <b>50</b> mr	n							
Number of anchors RHS		n <sub>2</sub> = <b>2</b>								
Edge distance in x-axis		e <sub>x2</sub> = <b>50</b> mr	n							
Edge distance in the y-axis		e <sub>y2</sub> = <b>50</b> mr	n							
Anchor diameter		d <sub>a,b</sub> = <b>16</b> m	m							
Foundation details										
Foundation depth		$t_{fnd} = 500 \text{ m}$	ım							
Concrete details										
Concrete strength class		C25/30								
Characteristic compressive cylir	nder strength	f <sub>ck</sub> = <b>25</b> N/n	nm²							
Characteristic compressive cube	e strength	f <sub>ck,cube</sub> = <b>30</b>	N/mm <sup>2</sup>							
Partial factor for concrete	$\gamma_{\rm c} = 1.50$									
Compressive strength coefficier	Compressive strength coefficient			$\alpha_{cc} = 0.85$						
Design compressive concrete st	$f_{cd} = \alpha_{cc} \times (1)$	<sub>ck</sub> / γ <sub>c</sub> ) = <b>14.17</b>	′ N/mm²							
Steel details										
Base plate steel grade		S275								
Base plate nominal yield strengt	:h	f <sub>yp_plt</sub> = <b>275</b>	N/mm <sup>2</sup>							
Base plate nominal ultimate ten	sile strength	f <sub>u_plt</sub> = <b>410</b> I	N/mm²							
Column steel grade		User defin	ed							
Column nominal yield strength		f <sub>yp_col</sub> = <b>355</b>	N/mm <sup>2</sup>							
Column nominal ultimate tensile	strength	$f_{u\_col} = 470$	N/mm²							
Partial safety factor cross section	ns	γмо = <b>1.00</b>								
Partial safety factor welds		γ <sub>M2</sub> = <b>1.25</b>								
Foundation bearing strength -	EN1992-1-1	Section 6.7								
Loaded area		$A_{c0} = b_p \times h$	<sub>p</sub> = <b>90000</b> mm	2						
Design distribution width		b <sub>p,dist</sub> = <b>800</b>	mm							
Design distribution length		h <sub>p,dist</sub> = <b>800</b>	mm							
Maximum design distribution are	ea	$A_{c1} = b_{p,dist}$	× h <sub>p,dist</sub> = <b>6400</b>	<b>00</b> mm²						
Concentrated design resistance	force	$F_{Rdu} = Min(A)$	$A_{c0} \times f_{cd} \times \sqrt{A_{c0}}$	$_{c1}$ / A <sub>c0</sub> ), 3 × f <sub>cd</sub> × A	<sub>co</sub> ) = <b>3400</b> kN					
Foundation joint material coeffic	ient	$\beta_{j} = 0.67$								
Design bearing strength of the ju	pint	$f_{jd} = \beta_j \times F_{Rd}$	$d_u / (b_p \times h_p) = 1$	<b>25.19</b> N/mm²						
Base plate compressive resis	tance									
Additional bearing width		$c = t_p \times \sqrt{f_w}$	$_{\rm 0}$ plt / (3 $\times$ fid $\times$ )	(M0)) = <b>28.6</b> mm						
Effective bearing area		$A_{eff} = 23649$	<b>9</b> mm <sup>2</sup>	-,,						
Design compressive resistance		$N_{c,Rd} = f_{id} \times$	A <sub>eff</sub> = <b>595.6</b> kl	N						
		PASS - De	sign compres	sive resistance	exceeds app	lied axial load				
			- •							

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	External Stairs				G0738-1	
GHD	Calcs for	Start page no./Revision				
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GHD	Calcs for	Calculatio	ons for Balcony	Page	29	
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## **Fixings**

<u>Steel - Steel</u>

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<sup>2</sup> 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.

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## **Conclusions**

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







