Appendix A

Haus Structures Consulting Structural Engineers	Project 92	Albert Road Ho	Job no 25039			
	Calcs for	B1A - Beam			Start page no/Rev 1 A	vision
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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



Haus	Project Job no					
Structures	92	Albert Road Ho	ney Surrey IXI		2000	
Consulting Structural Engineers	Calcs for	B1A - Beam			2 A	EVISION
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		Support B		Permanent	× 1.35	
				Variable × 1	1.50	
Analysis results						
Maximum moment		M _{max} = 9.5 kNn	٦	$M_{min} = 0 \text{ kN}$	m	
Maximum shear		Vmax = 14 KN		Vmin = -14 K	N	
Deflection		δ _{max} = 1.9 mm		δmin = 0 mm) 	
Maximum reaction at support A	_	$R_{A_{max}} = 14 \text{ kN}$		RA_min = 14	kN	
Unfactored permanent load reaction	n at support A	RA_Permanent = 10	0.4 kN			
Maximum reaction at support B		RB_max = 14 kN		RB_min = 14	kN	
Unfactored permanent load reaction	n at support B	RB_Permanent = 10	0.4 kN			
Section details						
Section type		UB 178x102x1	9			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled proc	ducts of structu					
Nominal thickness of element		t = max(tf, tw) =	, 7.9 mm			
Nominal yield strength		$f_y = 355 \text{ N/mm}^2$	-			
Modulus of electicity		$I_{\rm u} = 470 {\rm N/IIIII}^{4}$	- mm ²			
Modulus of elasticity	7.	L = 210000 N/				
		+				
Partial factors - Section 6.1		4.00				
Resistance of cross-sections		γmo = 1.00				
Resistance of members to instabilit	У	γm1 = 1.00				
Resistance of tensile members to fi	racture	үм2 = 1.10				
Lateral restraint		Snan 1 has let	aral restraint a	t sunnorte only		
		opan i nas iau	erai restidilit d			
Effective length factors		K _ 1 000				
Effective length factor in major axis		$K_{-} = 1.000$				
Effective length factor for torsion		$K_{1TA} = 1.000$				
		$K_{\rm ITB} = 1.000$				

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Structures		SZ AIDEIT KUAU I	Ioney Surrey Ki		200.					
Consulting Structural Engineers	Calcs for	B1A - Bear	n		3 A					
Email: info@haus-structures.co.uk Website: www.haus-structures.co.uk	Calc by AS	Calcs Date 7/1/2025	Checked by DG	Checked date July/2025	Approved by DG	Approved date July/2025				
Classification of cross sections	- Section 5 5									
		ε = √[235 N/ι	mm ² / f _y] = 0.81							
Internal compression parts subj	ect to bending	g - Table 5.2 (sh	eet 1 of 3)							
Width of section	Width of section									
		c / t _w = 37.6	× ε <= 72 × ε	Class 1						
Outstand flanges - Table 5.2 (she	eet 2 of 3)									
Width of section		c = (b - t _w - 2	× r) / 2 = 40.6 n	nm						
		$c / t_f = 6.3 \times 10^{-1}$	ε <= 9 × ε	Class 1						
					Sec	tion is class 1				
Check shear - Section 6.2.6										
Height of web		$h_w = h - 2 \times t$	f = 162 mm							
Shear area factor		η = 1.000								
		h _w / t _w < 72 ×	$h_w / t_w < 72 \times \epsilon / \eta$							
				Shear buckling	resistance c	an be ignored				
Shear area - cl 6 2 6(3)	$V_{Ed} = \max(A)$	0S(Vmax), 8DS(Vm 、2 、 b 、 te + (t +	r(n) = 14 KIN	√ t) – 085 mr	\mathbf{n}^2					
Design shear resistance - $cl = 6.2.6(3)$		$= \Delta \dots \times (f_{11} / \sqrt{31})$	/ 2 × 1) × u, I × IIw	× tw) – 303 mi	1					
	<u>~</u>)	Vc,Rd – Vpi,Rd P	ASS - Design s	hear resistance e	exceeds desig	yn shear force				
Check bending moment major ()	y-y) axis - Sect	tion 6.2.5								
Design bending moment		M _{Ed} = max(a	bs(Ms1_max), abs	(Ms1_min)) = 9.5 kN	m					
Design bending resistance momer	it - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 60.8 \text{ kNm}$								
Slenderness ratio for lateral tors	ional buckling	9								
Correction factor - Table 6.6		kc = 0.94								
		$C_1 = 1 / k_c^2 =$	1.132							
Curvature factor		g = √[1 - (Iz /	l _y)] = 0.948							
Poissons ratio		v = 0.3								
Shear modulus		$G=E/[2\times$	(1 + v)] = 80769	N/mm ²						
Unrestrained length		$L = 1.0 \times L_{s1}$	= 2700 mm							
Elastic critical buckling moment		$M_{cr} = C_1 \times \pi^2$ kNm	$\times E \times I_z / (L^2 \times Q)$	$(J) \times \sqrt{[I_w / I_z + L^2 \times J_w]}$	$G \times I_t / (\pi^2 \times E)$	× lz)] = 59.4				
Slenderness ratio for lateral torsion	nal buckling	$\overline{\lambda}$ LT = $\sqrt{(W_{pl.})}$	y × fy / Mcr) = 1.0	12						
Limiting slenderness ratio	0	$\overline{\lambda}$ LT,0 = 0.4	,							
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - L$	ateral torsional l	ouckling canr	not be ignored				
Design resistance for buckling -	Section 6.3.2.	1								
Buckling curve - Table 6.5		b								
Imperfection factor - Table 6.3		αlt = 0.34								
Correction factor for rolled sections	6	$\beta = 0.75$. –							
LTB reduction determination factor		φ∟т = 0.5 × [1	$φ_{LT} = 0.5 \times [1 + α_{LT} \times (λ_{LT} - λ_{LT,0}) + \beta \times λ_{LT}^2] = 0.988$							
LIB reduction factor - eq 6.57		χ∟⊤ = min(1 /	[φLT + √(φLT ² - β	× λlt ²)], 1, 1 / λlt	²) = 0.692					
Modification factor	0 50	t = min(1 - 0)	5 × (1 - kc)× [1 -	2 × (λιτ - 0.8) ²],	1) = 0.973					
Modified LTB reduction factor - eq	6.58	$\chi_{LT,mod} = min$	(χιτ / t, 1) = 0.71	2						

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H Structures	92	Albert Road Ho	25039			
	Calcs for		Start page no/Revision			
		B1A - Beam	4 A			
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Design buckling resistance moment - eq 6.55

 $M_{\text{b,Rd}} = \chi_{\text{LT,mod}} \times W_{\text{pl.y}} \times f_{\text{y}} \, / \, \gamma_{\text{M1}} = \textbf{43.3} \; kNm$

PASS - Design buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads

Limiting deflection

 $\delta_{\text{lim}} = \min(10 \text{ mm}, \text{L}_{s1} / 500) = 5.4 \text{ mm}$

Maximum deflection span 1

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 1.868 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

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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



Haus	Project Job no 92 Albert Road Horley Surrey RH6 7HZ 25039					
Consulting Structural Engineers	Calcs for	B7 - Beam			Start page no/Ro 6 A	evision
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Analysis results		Support B		Variable × Permanent Variable ×	1.50 t × 1.35 1.50	
Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load reaction at support A Unfactored variable load reaction at support A Maximum reaction at support B Unfactored permanent load reaction at support B		$M_{max} = 23.2 \text{ kNm}$ $M_{min} = 0 \text{ kNm}$ $V_{max} = 10.4 \text{ kN}$ $V_{min} = -21.3 \text{ kN}$ $\delta_{max} = 3.5 \text{ mm}$ $\delta_{min} = 0 \text{ mm}$ $R_{A_max} = 10.4 \text{ kN}$ $R_{A_min} = 10.4 \text{ kN}$ $R_{A_presentent} = 4.8 \text{ kN}$ $R_{A_presentent} = 2.7 \text{ kN}$ $R_{B_max} = 21.3 \text{ kN}$ $R_{B_min} = 21.3 \text{ kN}$				
Unfactored variable load reaction a Section details Section type Steel grade EN 10025-2:2004 - Hot rolled pro	at support B ducts of structu	R _{B_Variable} = 5.5 UB 203x102x2 S355 Iral steels	kN 23 (British Ste	eel Section Range	e 2022 (BS4-1)))
Nominal thickness of element Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity	t = max(t _f , t _w) = 9.3 mm f _y = 355 N/mm ² f _u = 470 N/mm ² E = 210000 N/mm ²					
	↓ <u>↓</u> ★ <u>∓</u>	+101.8	+			
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instabili Resistance of tensile members to t	ty racture	γmo = 1.00 γm1 = 1.00 γm2 = 1.10				
Eater an resulation	Span 1 has lateral restraint at supports only					
Effective length factor in major axis	i	Ky = 1.000				

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H Structures	92	2 Albert Road Ho	rley Surrey RH6	5 7HZ	2503	9			
Consulting Structural Engineers	Calcs for	B7 - Beam			Start page no/Re 7 A	vision			
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Website: www.haus-structures.co.uk	AS	7/1/2025	DG	July/2025	DG	July/2025			
Effective length factor in minor axis		Kz = 1.000							
Effective length factor for torsion		Klt.a = 1.000							
		Klt.b = 1.000							
Classification of cross sections -	Section 5.5								
		ε = √[235 N/mr	m² / fy] = 0.81						
Internal compression parts subje	ct to bending -	Table 5.2 (she	et 1 of 3)						
Width of section		c = d = 169.4 r	nm						
		$c / t_w = 38.6 \times s$	ε <= 12 × ε	Class 1					
Outstand flanges - Table 5.2 (she	et 2 of 3)								
Width of section		$c = (b - t_w - 2 \times $	r) / 2 = 40.6 mm	1					
		$c / t_f = 5.4 \times \varepsilon <$	3 × θ = 3	Class 1	Cool	ion io ologo 1			
					Sect	ION IS CIASS I			
Check shear - Section 6.2.6		L L O I	404.0						
		n = 1,000							
Snear area factor		$\eta = 1.000$	1						
		$\Pi w / t w < T Z \times \varepsilon$	л ц	Shear buckling	resistance ca	an he ignored			
Design shear force		V _{Ed} = max(abs	(Vmax), abs(Vmin)) = 21.3 kN		in be ignored			
Shear area - cl 6.2.6(3)		$A_v = max(A - 2)$	\times b \times t _f + (t _w + 2	΄ ː×r)×t _f , η×hw>	< tw) = 1238 mi	m²			
Design shear resistance - cl 6.2.6(2	2)	$V_{c,Rd} = V_{pl,Rd} =$	$A_v \times (f_y / \sqrt{[3]}) / \gamma$	мо = 253.7 kN					
		PAS	SS - Design she	ear resistance e	xceeds desig	n shear force			
Check bending moment major (y	-y) axis - Sectio	on 6.2.5							
Design bending moment		$M_{Ed} = max(abs(M_{s1_max}), abs(M_{s1_min})) = 23.2 \text{ kNm}$							
Design bending resistance momen	t - eq 6.13	$M_{c,Rd} = M_{pl,Rd} =$	W pl.y $ imes$ fy / үмо =	83.1 kNm					
Slenderness ratio for lateral tors	onal buckling								
Correction factor - Table 6.6		kc = 0.94							
		$C_1 = 1 / k_c^2 = 1$.132						
Curvature factor		$g = \sqrt{[1 - (I_z / I_y)]}$)] = 0.96						
Poissons ratio		v = 0.3		(
Shear modulus		$G = E / [2 \times (1)]$	(+ v) = 80769 N	/mm²					
Elastic critical buckling moment		$L = 1.0 \times L_{s1} = 3400 \text{ mm}$							
		kNm	$E \times Iz / (L^- \times y)$	× ([iw / iz + L- × (3 × π7 (<i>π</i> − × ⊑ .	× 12)] = 36.7			
Slenderness ratio for lateral torsion	al buckling	$\overline{\lambda}_{LT} = \sqrt{W_{pl.y}} \times$	t fy / Mcr) = 1.19						
Limiting slenderness ratio	0	$\overline{\lambda}$ LT,0 = 0.4							
			λ ιτ > λ ιτ,ο - Lat	eral torsional b	uckling cann	ot be ignored			
Design resistance for buckling -	Section 6.3.2.1								
Buckling curve - Table 6.5		b							
Imperfection factor - Table 6.3		αlt = 0.34							
Correction factor for rolled sections	1	$\beta = 0.75$							
LTB reduction determination factor		φιτ = 0.5 × [1 +	α LT × ($\overline{\lambda}$ LT - $\overline{\lambda}$ L	$T,0) + \beta \times \overline{\lambda} LT^2] =$	1.166				
LTB reduction factor - eq 6.57		χ∟τ = min(1 / [¢	lt + √(φlt² - β ×	$\overline{\lambda}$ LT ²)], 1, 1 / $\overline{\lambda}$ LT ²	²) = 0.585				

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Modification factor		f = min(1 - 0.	5 × (1 - kc)× [1 -	2 × (λ̃lt - 0.8) ²],	1) = 0.979		
Modified LTB reduction factor - eq	6.58	$\chi_{LT,mod} = min($	χιτ / f, 1) = 0.5 9)7			
Design buckling resistance mome	nt - eq 6.55	$M_{b,Rd} = \chi_{LT,mod}$	$H imes W_{pl.y} imes f_y / \gamma_M$	1 = 49.6 kNm			
Check vertical deflection - Secti	PAS on 7.2.1	SS - Design buck	ling resistance	e moment excee	ds design be	nding moment	
Consider deflection due to permar	ient and variat						
Limiting deflection		$\delta \lim = \min(14)$	mm, $L_{s1} / 250$) :	= 13.6 mm			
Maximum deflection span 1		$\delta = \max(abs($	δ max), abs(δ min))	= 3.496 mm			

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

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Support B

Support A

Applied loading

Beam loads

Load combinations Load combination 1

Rotationally free Vertically restrained Rotationally free

Permanent self weight of beam $\times 1$ Roof - Permanent full UDL 2.4 kN/m Roof - Variable full UDL 2.25 kN/m

Support A

Support B

Permanent × 1.35 Variable \times 1.50 $Permanent \times 1.35$ Variable \times 1.50 Permanent × 1.35 Variable \times 1.50

TEDDS calculation version 3.0.14

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Structures	Calcs for				Start page no./F	Revision
Consulting Structural Engineers		Ridge Be	am - B6a		1	10 A
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Website: www.haus-structures.co.uk	DG	02/07/2025	DG	July 2025	DG	July 2025
A						
Analysis results		M – 21 k	Nm	M - 0	kNm	
Maximum shear		$V_{max} = 17.2$	kN	Vmin = -1	17.2 kN	
Deflection		δ _{max} = 6.1 m	nm	$\delta_{\min} = 0$	mm	
Maximum reaction at support A		RA max = 17.	2 kN	RA min =	17.2 kN	
Unfactored permanent load read	tion at support A	RA_Permanent =	= 6.6 kN			
Unfactored variable load reaction	n at support A	R_{A} variable =	5.5 kN			
Maximum reaction at support B		RB_max = 17.	2 kN	RB_min =	17.2 kN	
Unfactored permanent load reaction at support B		B RB_Permanent =	= 6.6 kN			
Unfactored variable load reaction	n at support B	RB_Variable =	5.5 kN			
Section details						
Section type		UB 203x13	3x30 (British S	Steel Section Ra	nge 2022 (BS	64-1))
Steel grade		S355				
EN 10025-2:2004 - Hot rolled p	roducts of stru	ctural steels				
Nominal thickness of element		$t = max(t_f, t_f)$	v) = 9.6 mm			
Nominal yield strength		f _y = 355 N/n	nm²			
Nominal ultimate tensile strength	Nominal ultimate tensile strength					
Modulus of elasticity	Q	E = 210000	N/mm ²			
	206.8 9.6.↓		5.4			
Partial factors - Section 6.1						
Resistance of cross-sections		умо = 1.00				
Resistance of members to instal	oility	γм1 = 1.00				
Resistance of tensile members t	o fracture	γm2 = 1.10				
Lateral restraint		Span 1 has	lateral restrain	t at supports only	1	
Effortivo longth factors		epun i nuo				
Effective length factors	vie	K 1 000				
Effective length factor in minor of	nio Vis	rvy = 1.000 K ₂ - 1.000				
Effective length factor for torsion	лю	$K_{1.7.4} = 1.000$	0 + 2 ∨ h			
		KLTD - 1.20	• + 2 × 11 • + 2 ∨ h			
		TALI.B = 1.20	V T Z A II			

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Website: www.haus-structures.co.uk	DG	02/07/2025	DG	July 2025	DG	July 2025		
Classification of cross costion	Section E E							
Classification of cross section	15 - Section 5.5	ε = √[235 N	l/mm² / fy] = 0.81					
Internal compression parts su	bject to bendir	ng - Table 5.2 (s	sheet 1 of 3)					
Width of section		c = d = 172	.4 mm					
		c / t _w = 33.1	$\times \epsilon \le 72 \times \epsilon$	Class 1				
Outstand flanges - Table 5.2 (sheet 2 of 3)							
Width of section		c = (b - t _w -	2 × r) / 2 = 56.2	mm				
		c / tf = 7.2 >	3 × θ => 3	Class 1				
					Secti	on is class 1		
Check shear - Section 6.2.6								
Height of web		h _w = h - 2 ×	t _f = 187.6 mm					
Shear area factor		η = 1.000						
		h _w / t _w < 72	×ε/η					
			S	Shear buckling r	resistance ca	n be ignored		
Design shear force	$V_{Ed} = max(a)$	abs(V _{max}), abs(V	(min)) = 17.2 kN					
Shear area - cl 6.2.6(3)	$A_v = \max(A_v)$	$-2 \times b \times t_f + (t_w)$	+ 2 × r) × t _f , η × l	hw × tw) = 145ξ	\$ mm ²			
Design shear resistance - cl 6.2	$V_{c,Rd} = V_{pl,R}$	$d = A_v \times (f_y / \sqrt{[3]})$) / γmo = 298.7 kN	l Iocada daciar	obser force			
.		PAS	3 - Design snea	al resistance ex	ceeus desigi	I SHEAF TOLCE		
Check bending moment major	r (y-y) axis - Se	$M_{\rm T} = max($	abs(Marana) ab	c(M.()) - 21 k	Nm			
Design bending resistance more	ent = eq 6 13		$abs(Ws1_max), ab$	5(IVIS1_min)) = 21 k no - 111 6 kNm				
		IVIC,Rd — IVIPI,F	ka — νν ρι.y × Ty / γιν					
Sienderness ratio for lateral to	orsional buckli	ng k = 0.04						
		$R_c = 0.94$ $C_1 = 1 / k_c^2$	= 1 132					
Curvature factor		$a = \sqrt{1 - (1 - 1)}$	/ l _v)] = 0.931					
Poissons ratio		v = 0.3	, ,,,] 0.001					
Shear modulus		G = E / [2 ×	((1 + y)) = 8076	9 N/mm²				
Unrestrained length		$L = 1.2 \times L_s$	s1 + 2 × h = 6294	mm				
Elastic critical buckling moment		$M_{cr} = C_1 \times \tau$	$\tau^2 \times E \times I_z / (L^2 \times$	q) × $\sqrt{[l_w/l_z + L^2]}$	\times G \times It / (π^2 \times	$(E \times I_z)] =$		
Ŭ		55.3 kNm	, ,	0, 1	, , , , , , , , , , , , , , , , , , ,			
Slenderness ratio for lateral tors	ional buckling	$\overline{\lambda}_{LT} = \sqrt{W_{F}}$	$d_{y} \times f_y / M_{cr}) = 1.4$	421				
Limiting slenderness ratio		$\overline{\lambda}$ LT,0 = 0.4						
		Ā	LT > λ LT,0 - Late	eral torsional bu	uckling canno	ot be ignored		
Design resistance for buckling	g - Section 6.3.	2.1						
Buckling curve - Table 6.5		b						
Imperfection factor - Table 6.3		αlt = 0.34						
Correction factor for rolled section	ons	$\beta = 0.75$						
LTB reduction determination fac	tor	ϕ LT = 0.5 \times	[1 + αιτ × (λιτ -	$\overline{\lambda}$ LT,0) + $\beta \times \overline{\lambda}$ LT ²] = 1.431			
LTB reduction factor - eq 6.57		χ∟⊤ = min(1	/ [φιτ + √(φιτ ² - β	3 × λ̃lt²)], 1, 1 /	λιτ²) = 0.463			
Modification factor		f = min(1 - 0	0.5 × (1 - k _c)× [1	- 2 × (λ̃lt - 0.8)	²], 1) = 0.993			
Modified LTB reduction factor -	eq 6.58	χlt,mod = mi	n(χ∟⊤ / f, 1) = 0.4	66				
Design buckling resistance mor	nent - eq 6.55	$M_{b,Rd} = \chi_{LT,r}$	$mod imes W$ pl.y $ imes$ fy / γ	м1 = 52 kNm				
	PASS -	Design bucklir	ng resistance m	noment exceeds	s design benc	ling moment		

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Check vertica	deflection -	Section	7.2.1
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Consider deflection due to permanent and variable loads

Limiting deflection

Maximum deflection span 1

 $\delta {\sf lim} = L_{s1} \ / \ 500 = \textbf{9.8} \ mm$

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{6.103} \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit