JOB TITLE:		JOB NUMBER / FILE:	CALCULATION NUMBER:	Form
24	HEATH DRIVE	162637		
CALCULATION:	1	CALCULATION BY:	DATE: CHECKED BY:	
6-12	LOADING	HD	04/12/18	
CALCULAT	TONS:			
REF			OUTPUT	
	LOADING G-12			
	TIMBERFLOOR. 2M SPAN.			
	$k(2m \times 1.5 k)/m^2$	2.04 KN/M		
	FIRST FLOOR STUD WALL	-AssemED		
	ALLCOAD "A DA DA DA DA			
	U. DONNYM X J. LIV ZI. 10	XN//		
	COOR CLOOR			
	AKOUNU			
	TIMBER FLOOR SPAN 2.5	m		
	N/ 2 CAXI OD KN/M2 ="	7 6 5 KN	Im	
$\langle \rangle$	IC 2. SMX IS KOV/Mª =	3.75K	N/m	
	WALL LOAD			
	DL= 2, 54 KU/MR × 3.3/	m=8.4K	Nm	
	IOTAC LOAD!			
	DI-14 9KN/m	+ 8.2		
	4-6-8 KNIM			
D				
3/2/19				
24	CEM GF 2SOTHL RC	DCAB		
	PrSCAR Z V 781 = 78	Ally of		
	1 × 1,8 = 1,8 4	anton ce		
	TIMBER 1×1,05 = ,05 4	Ma pe		
	1-1,5 = 1,5 1	when he		
	8-	85 DL+ 3L	antha 1	
	Ree tephs			
		52 A=>;	20BUC 71.	

FORM Structural Design Ltd.

	Project	Job no. 162637				
Form Structural Design 77 St John Street	Calcs for BEAM G-12				Start page no./Revision 1	
EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date

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



Load combinations

Support B

Support A

Permanent \times 1.35 Imposed \times 1.50 Permanent \times 1.35 Imposed \times 1.50 Permanent \times 1.35 Imposed \times 1.50

🐺 Tekla					Job no. 162637	
Tedds Form Structural Design					Start nage no /Revision	
77 St John Street	BEAM G-12			2		
London	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
EC1M 4NN	HD	15/01/2019	ROM	-		
Analysis results						
Maximum moment		M _{max} = 83.8	kNm	M _{min} = 0	0 kNm	
Maximum shear		V _{max} = 72.9	kN	V _{min} = -	7 2.9 kN	
Deflection		δ _{max} = 8.3 n	nm	δ _{min} = 0	mm	
Maximum reaction at support A		R _{A_max} = 72	.9 kN	R _{A_min} =	= 72.9 kN	
Unfactored permanent load read	tion at support A	A R _{A_Permanent}	= 36.1 kN			
Unfactored imposed load reaction	n at support A	$R_{A_{Imposed}} =$	16.1 KN	-	70.0 (.))	
Maximum reaction at support B	tion of ourport F	$R_{B_{max}} = 72$.9 KIN - 26 4 KNI	R _{B_min} =	= 72.9 KIN	
Unfactored permanent load read	alon al support E	KB_Permanent	= 30.1 KIN 16 1 KN			
	παι δυμμυτι Β	NB_Imposed =	IQ.I KIN			
Section details						
Section type		UC 203x20	3x71 (BS4-1)			
Steel grade	naduate of stru	S355				
EN 10025-2:2004 - Hot folled p	roducts of stru	t = max(t, t) – 17 3 mm			
Nominal vield strength		f = 345 N/r	$m_{\rm w}^2 = 17.3$ mm ²			
Nominal ultimate tensile strength	ı	f ₀ = 470 N/r	$f_{\rm v} = 470 \text{N/mm}^2$			
Modulus of elasticity		F = 210000	N/mm ²			
modulue of oldelloky	Ť	210000				
-	17.3					
	▲					
	- x x x		- 10			
	×					
-	★ 〒					
		206.4-		→		
Partial factors - Section 6.1						
Resistance of cross-sections		vius = 1 00				
Resistance of members to instal	sility	γmu - 1.00 γmu - 1.00				
Resistance of tensilo mombara t	Resistance of members to instability		$\gamma_{M1} = 1.00$			
		γm2 – 1.10				
Lateral restraint		0	latanel	1 - 1 - · · · · · · · · · · · · · · · ·		
		Span 1 has	ialeral restrair	it at supports only	у	
Effective length factors						
Effective length factor in major a	xis	K _y = 1.000				
Effective length factor in minor a	xis	K _z = 1.000				
Effective length factor for torsion	l	K _{LT.A} = 1.00	0			
		K _{LT.B} = 1.00	U			

🚛 Tekla	Project	Project Job no.				
Tedds		24 HEA1	TH DRIVE		16	2637
Form Structural Design	Calcs for	Calcs for			Start page no./Revision	
London						3
EC1M 4NN	HD	15/01/2019	ROM	Checked date	Approved by	Approved date
Classification of cross sect	ons - Section 5.	.5				
		ε = √[235 N	I/mm ² / f _y] = 0.83	}		
Internal compression parts	subject to bend	ing - Table 5.2 (s	sheet 1 of 3)			
Width of section		c = d = 160	. 8 mm			
		c / t _w = 19.5	5 × ε <= 72 × ε	Class 1		
Outstand flanges - Table 5.2	2 (sheet 2 of 3)					
Width of section		c = (b - t _w -	2 × r) / 2 = 88 m	ım		
		c / t _f = 6.2 >	3 × θ => 3	Class 1		
					Sec	tion is class 1
Check shear - Section 6.2.6						
Height of web		$h_w = h - 2 \times$	t _f = 181.2 mm			
Shear area factor		η = 1.000				
		h _w / t _w < 72	×ε/η			
			:	Shear buckling	resistance c	an be ignored
Design shear force		V _{Ed} = max(a	abs(V _{max}), abs(V	′ _{min})) = 72.9 kN		
Shear area - cl 6.2.6(3)		A _v = max(A	- 2 × b × t _f + (t _w	+ 2 \times r) \times t _f , η \times	h _w × t _w) = 242	27 mm ²
Design shear resistance - cl 6	.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$_{d} = A_{v} \times (f_{y} / \sqrt{[3]})$) / γ _{M0} = 483.5 kN	l	
		PAS	SS - Design she	ar resistance ex	ceeds desig	n shear force
Check bending moment ma	jor (y-y) axis - S	ection 6.2.5				
Design bending moment		M _{Ed} = max(abs(M _{s1_max}), ab	s(M _{s1_min})) = 83.8	kNm	
Design bending resistance mo	oment - eq 6.13	$M_{c,Rd} = M_{pl,R}$	$_{\rm Rd} = W_{\rm pl.y} \times f_y / \gamma_{\rm M}$	₀ = 275.6 kNm		
Slenderness ratio for latera	torsional buck	ling				
Correction factor - Table 6.6		k _c = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (I _z	/ l _y)] = 0.817			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 ×	(1 + v)] = 8076	9 N/mm²		
Unrestrained length		$L = 1.0 \times L_s$	₃₁ = 4600 mm			
Elastic critical buckling mome	nt	M _{cr} = C ₁ × τ 652.8 kNm	$\tau^2 \times E \times I_z / (L^2 \times I_z)$	g) × $\sqrt{[I_w / I_z + L^2]}$	\times G \times I _t / (π^2	$\times E \times I_z)] =$
Slenderness ratio for lateral to	orsional buckling	$\overline{\lambda}_{IT} = \sqrt{W_r}$	$f_{\rm v} \times f_{\rm v} / M_{\rm cr}) = 0.0$	65		
Limiting slenderness ratio	5	$\overline{\lambda}_{LT.0} = 0.4$, <u>,</u>			
Ŭ		,_	$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - Lat$	eral torsional b	uckling cann	ot be ignored
Design resistance for buckl	ing - Section 6 3	2 2 1			U	C C
Buckling curve - Table 6 5	ing - Section 0.0	b				
Imperfection factor - Table 6.3	3	αιτ = 0.34				
Correction factor for rolled sec	ctions	β = 0.75				
LTB reduction determination f	actor	$\phi_{LT} = 0.5 \times$	[1 + α _{LT} × (λ _{LT} -	$\overline{\lambda}_{LT,0}$) + $\beta \times \overline{\lambda}_{LT}^2$] = 0.701	
LTB reduction factor - eg 6.57		$\gamma_{LT} = min(1)$	/ [φ _{LT} + √(φ _{LT} ² - [$3 \times \overline{\lambda}_{LT}^2$], 1, 1 / 2	- λ _{LT} ²) = 0.894	
Modification factor		f = min(1 - 1)	$0.5 \times (1 - k_c) \times [1]$	$-2 \times (\overline{\lambda}_{LT} - 0.8)^{2}$	²], 1) = 0.971	
Modified LTB reduction factor	- eg 6.58	γ _{LT.mod} = mi	n(γ _{LT} / f. 1) = 0.9	20	1, ,	
Design buckling resistance m	, oment - eq 6.55	$M_{b,Rd} = \chi_{LT,r}$	$_{\rm mod} \times W_{\rm pl.v} \times f_{\rm v} / \gamma_{\rm v}$	_{M1} = 253.7 kNm		
	PASS	S - Design buckli	ng resistance r	noment exceed	s design ber	nding moment
		-			-	-

	Project 24 HEATH DRIVE				Job no. 162637	
Form Structural Design 77 St John Street	Calcs for BEAM G-12			Start page no./Revision 4		
EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date

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

Limiting deflection

Maximum deflection span 1

 $\delta_{lim} = min(10 \text{ mm}, L_{s1} / 250) = 10 \text{ mm}$

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{8.271} mm$

	Project	Job no. 162637				
Form Structural Design 77 St John Street	Calcs for BEAM G-14				Start page no./Revision 1	
EC1M 4NN	Calcs by CEM	Calcs date 15/02/2019	Checked by	Checked date	Approved by	Approved date

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



🐙 Tekla					Job no. 162637	
Tedds					102037	
Form Structural Design 77 St John Street	Calcs for BEAM G-14				Start page no./Revision	
London	Color by	Colos data	Chacked by	Chacked data	Approved by	Approved data
EC1M 4NN	CEM	15/02/2019	Checked by	Checked date	Approved by	Approved date
Analysis results						
Maximum moment		M _{max} = 185	. 1 kNm	M _{min} =	0 kNm	
Maximum shear		V _{max} = 164.	5 kN	V _{min} =	-164.5 kN	
Deflection		δ _{max} = 9.2 r	nm	$\delta_{\min} = 0$) mm	
Maximum reaction at support A		R _{A max} = 16	4.5 kN	R _{A min} :	= 164.5 kN	
Unfactored permanent load read	ction at support /	A R _{A Permanent}	= 68.1 kN	_		
Unfactored imposed load reaction	on at support A	R _{A_Imposed} =	48.4 kN			
Maximum reaction at support B		R _{B_max} = 16	4.5 kN	R _{B_min} :	= 164.5 kN	
Unfactored permanent load read	ction at support I	B RB_Permanent	= 68.1 kN			
Unfactored imposed load reaction	on at support B	R _{B_Imposed} =	48.4 kN			
Section details						
Section type		UC 254x25	64x89 (BS4-1)			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled p	products of stru	ictural steels				
Nominal thickness of element		t = max(t _f , 1	w) = 17.3 mm			
Nominal yield strength		f _y = 345 N/i	mm²			
Nominal ultimate tensile strengt	n	$f_u = 470 \text{ N/mm}^2$				
Modulus of elasticity	ς.	E = 210000	N/mm ²			
	+17					
	Т					
	, , , , , , , , , , , , , , , , , , ,					
	- 260.	→	10.3			
	r.					
	<u>↓</u>					
	, I			_		
		200.0-	_	1		
Partial factors - Section 6.1						
Resistance of cross-sections						
Resistance of members to insta	bility	$\gamma_{\rm M4} = 1.00$	$\gamma_{M0} = 1.00$			
Resistance of tensile members	to fracture	$\gamma_{M0} = 1.00$				
		7M2 — 1.1V				
Lateral restraint		Shan 1 has	lateral restrain	at at supports and	V	
		opan i nas	ateral restrail	nt at supports on	У	
Effective length factors						
Effective length factor in major a	ixis	K _y = 1.000				
Effective length factor in minor a	IXIS	K _z = 1.000				
Effective length factor for torsion	ו	K _{LT.A} = 1.00	0			
		К _{LT.В} = 1.00	IU .			

🐙 Tekla	Project	ject Job no.						
Tedds								
Form Structural Design 77 St John Street	Calcs for	BEAN	BEAM G-14			evision 3		
London EC1M 4NN	Calcs by CEM	Calcs date 15/02/2019	Checked by	Checked date	Approved by	Approved date		
Olassifisation of success costin	na Castian E E							
Classification of cross sectio	ns - Section 5.5	ε = √[235 Ν	J/mm ² / f _v] = 0.83	8				
Internal compression parts su	ubject to bonding	a - Tablo 5 2 (e	shoot 1 of 3)	-				
Width of section	abject to benan	c = d = 200	3 mm					
		c / t _w = 23 6	δ×ε<=72×ε	Class 1				
Outstand flanges Table 5.2 (chect 2 of 2)	0, tw 20.						
Width of soction	sheet 2 of 3)	c = (b + t)	$2 \times r$ / 2 - 110	? mm				
Width of Section		$c = (D - I_w - C - C - C - C - C - C - C - C - C - $	$2 \times 1)/2 = 110.$					
		$C / I_{\rm f} = 7.7$	× E >= 9 × E	Class I	Sect	ion is class 1		
					Jech			
Check shear - Section 6.2.6								
Height of web		$h_w = h - 2 \times$: t _f = 225.7 mm					
Shear area factor		η = 1.000						
		$h_w / t_w < 72$	×ε/η	<u></u>				
Design shear force		$V_{-1} = mox($	aba()() aba()(Snear buckling	resistance ca	n be ignored		
Shoar area of 6.2.6(2)		$V_{Ed} = max(A)$	$abs(v_{max}), abs(v_{max})$	min)) = 104.3 KIN	h v t) = 208	I mm ²		
	6(2)	$A_v = \max(P)$	$V_{a} = V_{a} = A_{a} \times (f_{a} / \sqrt{3}) / \sqrt{a} = 613.6 \text{ kN}$					
Design shear resistance - ci 0.2		V c,Rd – V pI,R	a – Av × (ly / v[3] SS - Desian she	$\gamma \gamma M0 = 013.0 \text{ km}$	Vreeds desiru	n shear force		
Charle handing many and main		otion C D E			looodo doolgi			
Design bending moment majo	r (y-y) axis - Se	$M_{-1} = \max($	(abe(M)) ab	c(M)) - 185	1 kNm			
Design bending moment	nent - eg 6 13	$M_{Ed} = M_{ed}$	$abs(W_{s1}_{max}), ab$	$s(Ws_1_mn)) = 103.$				
		ivic,Ra — ivipi,i	Ra — VVpi.y ~ Ty / /IV					
Signderness ratio for lateral to	orsional buckli	ng						
		$K_c = 0.94$	- 1 132					
Curvature factor		$c_1 = 1 / R_c$	(1.132)					
Poissons ratio		y = 03						
Shear modulus		f = 0.5	((1 + y)) = 8076	9 N/mm ²				
Unrestrained length		$U = 10 \times 1$	(1 + 0) = 0070	5 10/11/11				
Elastic critical buckling moment		$E = 1.0 \times E$	s1 - 4300 mm	a) $\times \sqrt{[1] (1 + 1)^2}$	$\sim G \sim L / (\pi^2 \sim$	F ~ 1)] =		
Elastic childar buckling moment		1227 5 kNr	n ~ L ~ 127 (L ~	9) × 1[iw / iz · L				
Slenderness ratio for lateral tors	sional buckling	$\overline{\lambda}_{i,T} = \sqrt{W_i}$	$M_{\rm obv} \times f_{\rm v} / M_{\rm or}) = 0.$	587				
Limiting slenderness ratio	lonal buoking	$\overline{\lambda}_{1T0} = 0.4$						
		70L1,0 011	$\overline{\lambda}_{1,T} > \overline{\lambda}_{1,T,0} - Lat$	eral torsional bi	uckling canne	ot be ianored		
Design resistance for bucklin	a Section 6.2							
Buckling curve. Table 6.5	g - Section 6.3.	2. 1						
Imperfection factor - Table 6.3		0 01 = - 0.34						
Correction factor for rolled secti	one	αL1 = 0.54						
LTB reduction determination fac	stor	β = 0.75	$[1 + \alpha_{1} + \gamma (\overline{\lambda}_{1} + \overline{\lambda}_{2})]$	$\overline{\lambda}_{1}$ = $a + \beta \times \overline{\lambda}_{1}$ = 2^{2}	1 = 0 661			
LTB reduction factor - eq.6.57		$\psi_{LT} = 0.5 \times$	$1 + \alpha_{L} + \sqrt{(\alpha_{L} + 2)}$	$\overline{\lambda} = \overline{\lambda} = 2$ 1 1 1 / $\overline{2}$	$\overline{A}_{1} = 0.001$			
Modification factor		$f = \min(1)$	[,] [Ψ∟⊺ ' '(ΨL⊺ -) 0.5 _× (1 ₋ μ) _∨ [1	- 2 × () 0 8	²] 1) = 0.923			
Modified LTR reduction factor	eg 6 58		$0.0 \wedge (1 - N_c) \times [1]$	- 2 ~ (n _{L1} - 0.0)], 1) - 0.973			
Design buckling resistance mor	rent = eq 6.55	$\chi_{L1,mod} - 111$	$m(\chi_{L} 1, 1, 1) = 0.3$	m = 400.7 kNm				
Design buckling resistance mon		Nib,Rd - XLT,i Design buckli	moa ~ vvpl.y × ly/γ	$m_1 = +00.7 \text{ Kiniff}$	s dasian han	dina momort		
	PA00		ng resistance h		s design bell	any moment		

	Project 24HEATH DRIVE				Job no. 162637	
Form Structural Design 77 St John Street	Calcs for BEAM G-14			Start page no./Revision 4		
EC1M 4NN	Calcs by CEM	Calcs date 15/02/2019	Checked by	Checked date	Approved by	Approved date

Consider deflection due to permanent and imposed loads

Limiting deflection

Maximum deflection span 1

 δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

 $\delta = \text{max}(\text{abs}(\delta_{\text{max}}), \text{ abs}(\delta_{\text{min}})) = \textbf{9.226} \text{ mm}$

ов тит. <u>е</u>		JOB NUMBER / FILE:	CALCULATION NUMBER:	Eorm
24 NEATH DRIVE		162637		,
CALCULATION:		CALCULATION BY:	DATE: CHECKED BY:	
G-14 LOAD TO	IKE DOWN	HD	07/12/18	
CALCULATIONS:				
REFLOAS			OUTPUT	
D-C				
K001	LOAD			
GK =	40.2 KN			
LOAD	S SPREAD OVER 4.6M	OF WA	دد.	
=> 0	L = 71.1 kN/4.6 m L = 40.2 kN/4.6 m	= 15,5 KM	J/m Im	
TWO	FLOORS OF TIMBER			
	1.02KN/m × 8.54/2 1.5KN/m × 8.5M/2	= 4.3 KN/ = 6.4 KN	m	
×2 DL = LL =	8.6KN/m 12.8KN/m			
Two	FLOORS OF BRICK	NORK WALL	7.SM	
2.54 \$	(N/m= × 7.5M= 1	9.1KU/m		
ONE	FLOOR OF STED WALL			
0.558	$cN/m^2 \times 3m = 1.7$	KN/m		
TOT	AC COASE			
DL =	= 29.4 KN/m = 21.5 KN/m			
CEM -	2 G-14 BEAM			
A CONTRACT	4400 R 29,4 pc+Z1.	Sec		
· SEE	TEDDS 2540289			
Minor Vinor Sna	y = 176.9 Wh K K K K K K K K K K K K K K K K K K	2A = RE = 1.60	.91 (66,6pl+47.3 ll) La	V

FORM Structural Design Ltd.

		ONEOOL AHON ONEET
JOB TITLE:	JOB NUMBER / FILE: CALCULATION NUMBER:	For
24 NEATH DRIVE	162637	
CALCULATION	CALCULATION BY: DATE:	CHECKED BY:
BEAM 6-15, 6-24, 6-25	HD	Rom
LOADING.		
CALCULATIONS:		
REF		OUTPUT
DCAN CIE		
SUMPER PROVID FLOO	RHMDER	
supportes another i an		
DL . 1.02 KN/m2 × 4	2m/2 = 2.1 KN/m	ι
110000000000000000000000000000000000000	2 - 2 - 2 - 2 - 2 - 2 - 2 - 1 = 2 - 2 - 1 = 2 - 2 - 2 - 2 - 2 - 2 - 2 - 2 - 2 - 2	2
LO I. SKN/MEX +.	211/12 - 0 2 K N/1	<u>n</u>
BEAM 6-24		
SUPPORTS GROUND F	COOR AND	
login The ck DALL.		
WALL LOAD		
BURNARY 2.54 KN/m2	X3M=7.6KN/~	
FLOOR COAD	1 and [m]/m	
DC I OP KNIM-XI	M F LOZ KNII	
LC = 1.5 KIN / ACT X1		
TOTAL - DL 6 8 6	KN/m	
10%1.5	KNIM	
BEAM G-25		
05475-05-5-		
SUPPORTS GROUND FLOO	P SLAB AT THE	
FRONT OF THE PROPE	RTY.	
2004HICK SLAB.	12 - (-1)/-	
DC 6. GKN/MT X 9	n = 10-2 KN/M	
LE. I.SKU (MEX	2m = SKN/1-	

FORM Structural Design Ltd.

	Project	Job no. 162637				
Form Structural Design 77 St John Street London EC1M 4NN	Calcs for BEAM G-15				Start page no./Revision 1	
	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date

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



Support B

Support A

Applied loading

Beam loads

Load combinations Load combination 1

Rotationally free Vertically restrained Rotationally free

Permanent self weight of beam $\times 1$ Permanent full UDL 2.1 kN/m Imposed full UDL 3.2 kN/m

Support A

Support B

Permanent × 1.35 Imposed \times 1.50 $\text{Permanent} \times 1.35$ Imposed \times 1.50 $\text{Permanent} \times 1.35$ Imposed \times 1.50

TEDDS calculation version 3.0.13

Tekla	Project	Job no. 162637				
Form Structural Design	Calcs for	Calcs for				Revision
77 St John Street		BEAN	N G-15		2	
London EC1M 4NN	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date
Analysis results						-
Maximum moment		M _{max} = 10.2	2 kNm	M _{min} =	0 kNm	
Maximum shear		V _{max} = 12.7	' kN	V _{min} = ·	-12.7 kN	
Deflection		δ _{max} = 2.9 r	nm	δ _{min} = () mm	
Maximum reaction at support A		R _{A_max} = 12	. 7 kN	R _{A_min} :	= 12.7 kN	
Unfactored permanent load rea	ction at support A	RA_Permanent	= 3.7 kN			
Unfactored imposed load reacti	on at support A	R _{A_Imposed} =	5.1 kN			
Maximum reaction at support B		R _{B_max} = 12	2. 7 kN	R _{B_min} =	= 12.7 kN	
Unfactored permanent load rea	ction at support B	B RB_Permanent	= 3.7 kN			
Unfactored imposed load reacti	on at support B	$R_{B_{Imposed}} =$	5.1 kN			
Section details						
Section type		UC 152x15	52x23 (BS4-1)			
Steel grade		S355	, , , , , , , , , , , , , , , , , , ,			
EN 10025-2:2004 - Hot rolled	products of stru	ctural steels				
Nominal thickness of element		t = max(t _f , f	t _w) = 6.8 mm			
Nominal yield strength		f _y = 355 N/I	mm²			
Nominal ultimate tensile streng	th	f _u = 470 N/mm ²				
Modulus of elasticity		E = 210000) N/mm ²			
	8. 9⊥					
	\uparrow $\frac{\bullet}{\bullet}$ \square					
	152.4	-> 4	-5.8			
	◀────			→		
Partial factors - Section 6.1						
Resistance of cross sections		vius – 1 00				
Resistance of mombars to inst	ability	γ _{M0} - 1.00				
Posistance of tancile members	to fracture	γ _{M1} – 1.00				
Acordance of tensile members		γ _{M2} – 1.10				
Lateral restraint		a				
		Span 1 has	s lateral restrai	nt at supports onl	у	
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 torsio	n	K _{LT.A} = 1.00	00			
		K _{LT.B} = 1.00	00			

🐙 Tekla	Project	24 HEA	Job no. 162637					
Form Structural Design	Calcs for	21112,(1			Start page no /Revision			
77 St John Street	Cales IO	BEAM G-15				3		
London	Calcs by	Calcs date	Checked by	Approved by Approved da				
EC1M 4NN	HD	13/02/2019	ROM					
				-				
Classification of cross section	ons - Section 5	.5						
		ε = √[235 Ν	l/mm ² / f _y] = 0.8 ′	1				
Internal compression parts s	subject to bend	ing - Table 5.2 (s	sheet 1 of 3)					
Width of section		c = d = 123	3 .6 mm					
		c / t _w = 26.2	$2 \times \varepsilon \le 72 \times \varepsilon$	Class 1				
Outstand flanges - Table 5.2	(sheet 2 of 3)							
Width of section		c = (b - t _w -	2 × r) / 2 = 65.6	mm				
		c / t _f = 11.9	$\times \epsilon \le 14 \times \epsilon$	Class 3				
					Sec	tion is class 3		
Check shear - Section 6.2.6								
Height of web		h _w = h - 2 ×	t _f = 138.8 mm					
Shear area factor		n = 1.000						
		, h _w / t _w < 72	× ε / η					
			•	Shear buckling	resistance ca	an be ignored		
Design shear force		V _{Ed} = max(abs(V _{max}), abs(\	/ _{min})) = 12.7 kN		-		
Shear area - cl 6.2.6(3)		A _v = max(A	$-2 \times b \times t_{f}$ + (t _w	+ 2 × r) × t _f , η ×	h _w × t _w) = 997	mm²		
Design shear resistance - cl 6.	Design shear resistance - cl 6.2.6(2)			$V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0} = 204.4 \text{ kN}$				
		PAS	SS - Design she	ear resistance ex	xceeds desig	In shear force		
Check bending moment maj	or (y-y) axis - S	ection 6.2.5						
Design bending moment		M _{Ed} = max(abs(M _{s1_max}), ab	s(M _{s1_min})) = 10.2	kNm			
Design bending resistance mo	ment - eq 6.14	$M_{c,Rd} = M_{el,l}$	$_{\rm Rd}$ = W _{el.y} × f _y / $\gamma_{\rm N}$	₄₀ = 58.2 kNm				
Slenderness ratio for lateral	torsional buck	ling						
Correction factor - Table 6.6		k _c = 0.94						
		$C_1 = 1 / k_c^2$	= 1.132					
Curvature factor		g = √[1 - (I₂	/ l _y)] = 0.825					
Poissons ratio		v = 0.3						
Shear modulus		G = E / [2 >	(1 + v)] = 8076	9 N/mm ²				
Unrestrained length		$L = 1.0 \times L_s$	₅1 = 3200 mm					
Elastic critical buckling momen	nt	$M_{cr} = C_1 \times \pi$	$\tau^2 \times E \times I_z / (L^2 \times I_z)$	$(g) \times \sqrt{[I_w / I_z + L^2]}$	\times G \times I _t / (π^2	$\times E \times I_z)$] =		
		110.7 kNm						
Slenderness ratio for lateral to	rsional buckling	$\lambda_{LT} = \sqrt{W_{e}}$	$_{\rm el.y} \times f_{\rm y} / M_{\rm cr}) = 0.$	725				
Limiting slenderness ratio		$\lambda_{LT,0} = 0.4$						
			$\lambda_{LT} > \lambda_{LT,0} - Lat$	teral torsional b	uckling cann	ot be ignored		
Design resistance for buckli	ng - Section 6.3	3.2.1						
Buckling curve - Table 6.5		b						
Imperfection factor - Table 6.3		α _{LT} = 0.34						
Correction factor for rolled sec	tions	β = 0.75	_					
LTB reduction determination fa	actor	$\phi_{LT} = 0.5 \times$	$[1 + \alpha_{LT} \times (\lambda_{LT} \cdot$	- $\lambda_{LT,0}$) + $\beta \times \lambda_{LT^2}$	[[]] = 0.753			
LTB reduction factor - eq 6.57		χ _{LT} = min(1	/ [φ _{LT} + √(φ _{LT} ² -	β × λ _{LT} ²)], 1, 1 / 2	λ _{LT} ²) = 0.857			
Modification factor		f = min(1 -	0.5 × (1 - k _c)× [1	- 2 × (λ _{LT} - 0.8)	²], 1) = 0.970			
Modified LTB reduction factor	- eq 6.58	χ _{LT,mod} = mi	n(χ _{LT} / f, 1) = 0.8	383				
Design buckling resistance mo	ment - eq 6.55	$M_{b,Rd} = \chi_{LT,r}$	$_{ m mod} imes W_{ m el.y} imes f_{ m y} / \gamma$	_{M1} = 51.4 kNm				
	PASS	5 - Design buckli	ng resistance i	moment exceed	s design ben	iding moment		

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Consider deflection due to permanent and imposed loads

Limiting deflection

Maximum deflection span 1

 δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{2.874} mm$

JOB TITLE:	JOB NUMBER / FILE:	CALCULATION NUM	Form	
24 Heath Drive	162637			
CALCULATION:	CALCULATION BY:	DATE:	CHECKED BY:	
BRAMG-16 LOADING	dH	16/01	/19 Rom	

CALCULATIONS:	
GRO	UND FLOOR
TIME	$\frac{1000}{100} \times \frac{100}{100} \times \frac{100}{100} \times \frac{100}{100} \times \frac{1000}{100} \times \frac{1000}{100} \times \frac{1000}{1000} \times 100$
_STAL	RS LOAD
	$= 1.02 \text{ kW/m^2} \times 3 \text{ m}/2 = 1.6 \text{ kN/m}$ = 1.5 KN/m ² × 3 m/2 = 2.2 KN/m
Ist STO DL= LL=	$\frac{FLOOR}{PR}$ $= 1.62 \text{ AV/m}^2 \times 3\text{ M/2} = 2.2 \text{ AV/m}$ $= 1.5 \text{ MV/m}^2 \times 3\text{ M/2} = 2.2 \text{ AV/m}$
Time DL = LL =	SER FLOOR, IM SPAN COWS, DERED. $= 1.02 \text{ KN/m}^2 \times 1 \text{ M} = 1.02 \text{ KN/m}$ $= 1.5 \text{ KN/m}^2 \times 1 \text{ M} = 2.2 \text{ KN/m}$
2ND TIM DL = LL =	FLOOR BER FLOOR SPANS SM $\frac{1}{N}$ FOR PARTITIONS = 2.02 KN/m ² × Sm/2 = S.1 KN/m = 1.5 KN/m ² × SM/2 = 3.8 KN/m
WA PL:	$\frac{11 \text{ LOAD}}{2.54 \text{ KN/m}^2 \times 6.6 \text{ m}} = 16.8 \text{ KN/m}$
Roc	FLOAD MROOF DESUGN CAECOLATIONS.
poi	NT LOADS. DL = $SB.2 KN$ LL = $28.9 KN$
anz	$\begin{array}{c} h \\ h $
THE: (4.	SE LONOS ARE PREAD ACTOS THE WALL.
⇒2	DL = (58.2 + 37.3)/6 = 16 KN/m LL = (28.9 + 28.2)/6 = 10 KN/m
De	$\frac{40 \text{ LOADS}}{100 \text{ LOAD}} = \frac{43 \text{ W/m}}{38 \text{ W/m}}$

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



Support B

 $\begin{array}{l} Imposed \times 1.50 \\ Permanent \times 1.35 \\ Imposed \times 1.50 \\ Permanent \times 1.35 \end{array}$

Imposed \times 1.50

Tekla Tekla	Project	24 HEAT	Job no. 162637					
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London EC1M 4NN	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date			
Analysis results Maximum moment		M _{max} = 228	. 1 kNm	M _{min} =	0 kNm	-		
Maximum shear		V _{max} = 186 .	2 kN	V _{min} = ·	- 186.2 kN			
Deflection		δ _{max} = 8.7 n	nm	$\delta_{\min} = 0$) mm			
Maximum reaction at support A		R _{A max} = 18	6.2 kN	Ra min =	= 186.2 kN			
Unfactored permanent load rea	ction at support A	A R _{A Permanent}	= 108.5 kN	_				
Unfactored imposed load reacti	on at support A	R _{A Imposed} =	26.5 kN					
Maximum reaction at support B		R _{B max} = 18	6.2 kN	R _{B min} =	= 186.2 kN			
Unfactored permanent load rea	ction at support E	B RB Permanent	= 108.5 kN	_				
Unfactored imposed load reacti	on at support B	R _B Imposed =	26.5 kN					
Section details								
Section type		LIC 254v25	4132					
Steel grade		S355						
EN 10025-2:2004 - Hot rolled	products of stru	ctural steels						
Nominal thickness of element		$t = max(t_f, t)$	(w) = 25.3 mm					
Nominal vield strength		f _v = 345 N/r	nm²					
Nominal ultimate tensile strengt	h	fu = 470 N/I	$f_{\rm H} = 470 \text{N/mm}^2$					
Modulus of elasticity		E = 210000	$F = 210000 \text{ N/mm}^2$					
5	¥							
	25.3 ← 276.3		-15.3					
	<u>·</u> <u>↑</u> <u>·</u>							
	4	261.3-		→				
Partial factors - Section 6.1								
Resistance of cross-sections		γ _{M0} = 1.00						
Resistance of members to insta	bility	γ _{M1} = 1.00						
Resistance of tensile members	to fracture	γ _{M2} = 1.10						
Lateral restraint								
		Span 1 has	alateral restrair	nt at supports onl	у			
Effective length factors		·						
Effective length factor in major	axis	K., = 1 000						
Effective length factor in minor	axis	$K_{-} = 1.000$						
Effective length factor for torsio	n	K _{1TA} = 1 ΩΩ	0					
	•	KITR = 1.00	0					
			-					

🐙 Tekla	Project		Job no.			
Tedds		24 HEAT	HDRIVE		102037	
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London EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date
		10/01/2010				
Classification of cross section	ns - Section 5.5	;				
		ε = √[235 Ν	l/mm² / f _y] = 0.8 3	}		
Internal compression parts su	ıbject to bendiı	ng - Table 5.2 (s	sheet 1 of 3)			
Width of section		c = d = 200	.3 mm			
		c / t _w = 15.9	θ×ε<=72×ε	Class 1		
Outstand flanges - Table 5.2 (sheet 2 of 3)					
Width of section		c = (b - t _w -	$2 \times r$) / 2 = 110 .	3 mm		
		$c / t_f = 5.3$	<ε<= 9 × ε	Class 1		
		0,1 0.0 /		01000 1	Sect	ion is class 1
					0000	
Check shear - Section 6.2.6		L L O	t 005 7			
Height of web		$n_w = n - 2 \times$	t _f = 225.7 mm			
Shear area factor		η = 1.000				
		h _w / t _w < 72	×ε/η			
		,		Shear buckling	resistance ca	an be ignored
Design shear force		$V_{Ed} = max(a)$	abs(V _{max}), abs(V	(min)) = 186.2 kN		
Shear area - cl 6.2.6(3)	Shear area - cl 6.2.6(3)			+ 2 × r) × t _f , η ×	h _w × t _w) = 462	1 mm²
Design shear resistance - cl 6.2	.6(2)	$V_{c,Rd} = V_{pl,R}$	$d = A_v \times (f_y / \sqrt{3}]$) / γ _{M0} = 920.5 kN		
		PAS	SS - Design she	ar resistance ex	ceeds desig	n shear force
Check bending moment majo	r (y-y) axis - Se	ction 6.2.5				
Design bending moment		M _{Ed} = max(abs(M _{s1_max}), ab	$s(M_{s1_{min}})) = 228.$	1 kNm	
Design bending resistance mor	ent - eq 6.13	$M_{c,Rd} = M_{pl,R}$	$_{\rm Rd} = W_{\rm pl.y} \times f_y / \gamma_N$	₀ = 644.9 kNm		
Slenderness ratio for lateral to	orsional buckli	ng				
Correction factor - Table 6.6		k _c = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (l _z	/ I _y)] = 0.816			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 ×	(1 + v)] = 8076	9 N/mm²		
Unrestrained length		L = 1.0 × L _s	₃₁ = 4900 mm			
Elastic critical buckling moment		$M_{cr} = C_1 \times \pi$	$\tau^2 imes E imes I_z$ / ($L^2 imes$	g) × $\sqrt{[I_w / I_z + L^2]}$	\times G \times I _t / (π^2 >	$(E \times I_z)$] =
		2121.2 kNr	n			
Slenderness ratio for lateral tors	ional buckling	$\overline{\lambda}_{LT} = \sqrt{W_{F}}$	$d_{\text{bl.y}} \times f_y / M_{\text{cr}}) = 0.$	551		
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$				
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - Lat$	eral torsional bi	uckling cann	ot be ignored
Design resistance for bucklin	g - Section 6.3.	2.1				
Buckling curve - Table 6.5	-	b				
Imperfection factor - Table 6.3		α _{LT} = 0.34				
Correction factor for rolled section	ons	β = 0.75				
LTB reduction determination fac	tor	$\phi_{LT} = 0.5 \times$	[1 + α _{LT} × (λ _{LT} -	$\overline{\lambda}_{LT,0}$) + $\beta \times \overline{\lambda}_{LT^2}$] = 0.640	
LTB reduction factor - eq 6.57		$\chi_{LT} = \min(1)$	- / [φιτ + √(φιτ² - [$3 \times \overline{\lambda}_{LT^2}$], 1, 1 / $\overline{\lambda}_{LT^2}$	- λ _{LT} ²) = 0.939	
Modification factor		f = min(1 - 1)	0.5 × (1 - k _c)× [1	$-2 \times (\overline{\lambda}_{LT} - 0.8)^2$	²], 1) = 0.974	
Modified LTB reduction factor -	eg 6.58	γ _{LT.mod} = mi	$n(\gamma_{LT} / f. 1) = 0.9$	64	- ,	
Design buckling resistance mon	nent - ea 6.55	$M_{\rm b Rd} = \gamma_{\rm LT}$	$_{\rm mod} \times W_{\rm nlv} \times f_{\rm v} / v$	_{M1} = 621.6 kNm		
	PASS	- Desian buckli	na resistance r	noment exceed	s desian hen	dina moment
			J			

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Check vertical	deflection	- Section 7.2.1
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Consider deflection due to permanent and imposed loads

Limiting deflection

Maximum deflection span 1

 δ_{lim} = min(10 mm, L_{s1} / 250) = **10** mm

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{8.741} mm$

JOB TITLE	JOB NUMBER / FILE:	CALCULATION NUMBER:	Form
24 MEATTINITIVE,		DATE: OUR DUE DU	
6-17, 6-22, 6-23, 6-18		DATE: CHECKED BY:	the second second
	<u>Inc</u>		
REF		OUTPUT	
UT-IF LOHD			
SPAN = 4.6 m			
LOADING			
TIMBER FLOOR			
PL: 1.02 KN/Mt X 2M	3.06	KN/m	
	ISKICK WA		
pl. o. m x 2m	X 20KV/M3.	$\pm 4 \text{ kN/m}$	
TOTAL! DL=7.0	6 KN/M		
24 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1 1		0	
SSO deep × 300 wid	re ke ban	tores .	
		•	
G-22 & G-23 SPAN	14.6M		
LOADING			
MASONARY WALL.			
DC: 3.3mx 0.1m;	× 20KN/m3.	= 6.6 kW/m	
TIMBER FLOOR. I.S.	M SPAN X	2 (29/0015)	
DL: 1.02 KN/ME X	SM = 1.5	3KN/m	
Total Di-6 (1) allow		a TINI/M	
16 = 2.25 KN/	MX2 = 4.	SKN/M	
350 deep x 300 w	ide RE DYN	Ford	
C- 10 / -4			
UTIN LOAD			
G22 & G23 REACTIONS			
GK = 30.4 IV			
XM2 10.2 KN	a li		
BRICK WOME PIERS AT GITT	MER END - HE	IGHIES. BM	
01 0.1m × 3.3mx 20HU/m3	= 6,6 KU/m		

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



Applied loading Beam loads

Load combinations Load combination 1

Rotationally free

Permanent self weight of beam $\times 1$ Permanent full UDL 7.1 kN/m Imposed full UDL 4.5 kN/m

Support A

Support B

 $Permanent \times 1.35$ Imposed \times 1.50 $\text{Permanent} \times 1.35$ Imposed \times 1.50 $Permanent \times 1.35$ Imposed \times 1.50

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Tedds					2037	
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Analysis results						
Maximum momont		M - 44 9	kNm	М. –		
Maximum shear		Wimax - 44.0	N	V	30 KNI	
		V max - 33 K		v min — ·		
Maximum reaction at support A		$D_{max} - 7.3$		O _{min} – C	- 20 //N	
Linfactored permanent load rea	ction at support /	$\mathbf{R}_{A}_{max} - 33$	- 17 1 LNI	►A_min ·	- 39 KIN	
Unfactored imposed load read	on of support A	A RA_Permanent	- 17.4 KIN			
Maximum reaction at support P	on at support A			р.	- 20 1/1	
Linfactored permanent load rea	otion of support F	$\mathbf{R}_{\mathrm{B}_{\mathrm{max}}} = 33$	- 17 1 LNI	™ B_min •	- 39 KIN	
Unfactored permanent load rea	ction at support E	D RB_Permanent	- 17.4 KIN			
Official imposed load reacti	on at support B	RB_Imposed -	10.4 KIN			
Section details						
Section type		UC 203x20	3x46 (BS4-1)			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled	products of stru	ictural steels	\ 44 0			
Nominal thickness of element		$t = max(t_f, t_f)$	w) = 11.0 mm			
Nominal yield strength		$f_y = 355 N/r$	nm- 2			
Nominal ultimate tensile strengt	n	$I_u = 470 \text{ N/mm}^2$				
Modulus of elasticity		E = 210000	N/mm²			
	<u>▲</u> <u>∓</u> <u></u>					
			-7.2			
	◀	203.6-		→		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to insta Resistance of tensile members	bility to fracture	γ _{M0} = 1.00 γ _{M1} = 1.00 γ _{M2} = 1.10				
Lateral restraint		Snan 1 had	ateral restrair	nt at sunnorts on	v	
Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for torsio	axis axis n	Span 1 nas K _y = 1.000 K _z = 1.000 K _{LT.A} = 1.00 K _{LT.B} = 1.00	00	ιι αι supports on	у	

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London EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date	
		L.		·			
Classification of cross section	ons - Section 5	.5					
		ε = √[235 Ν	I/mm ² / f _y] = 0.8 ′	1			
Internal compression parts	subject to bend	ing - Table 5.2 (s	sheet 1 of 3)				
Width of section		c = d = 160).8 mm				
		c / t _w = 27.4	4 × ε <= 72 × ε	Class 1			
Outstand flanges - Table 5.2	(sheet 2 of 3)						
Width of section		c = (b - t _w -	2 × r) / 2 = 88 n	nm			
		c / t _f = 9.8 >	< ε <= 10 × ε	Class 2			
					Sec	tion is class 2	
Check shear - Section 6.2.6							
Height of web		$h_w = h - 2 \times$	t _f = 181.2 mm				
Shear area factor		η = 1.000					
		h _w / t _w < 72	×ε/η				
				Shear buckling	resistance c	an be ignored	
Design shear force		V _{Ed} = max(abs(V _{max}), abs(\	/ _{min})) = 39 kN			
Shear area - cl 6.2.6(3)		A _v = max(A	$x - 2 \times b \times t_f + (t_w$, + 2 × r) × t _f , η ×	h _w × t _w) = 169	8 mm ²	
Design shear resistance - cl 6.	Design shear resistance - cl 6.2.6(2)) / γ _{M0} = 347.9 kN	1		
		PAS	SS - Design she	ear resistance ex	xceeds desig	n shear force	
Check bending moment maj	or (y-y) axis - S	ection 6.2.5					
Design bending moment		M _{Ed} = max((abs(M _{s1_max}), ab	os(M _{s1_min})) = 44.8	kNm		
Design bending resistance mo	ment - eq 6.13	$M_{c,Rd} = M_{pl,l}$	$_{\rm Rd}$ = W _{pl.y} × f _y / $\gamma_{\rm N}$	₄₀ = 176.6 kNm			
Slenderness ratio for lateral	torsional buck	ling					
Correction factor - Table 6.6		k _c = 0.94					
		$C_1 = 1 / k_c^2$	= 1.132				
Curvature factor		g = √[1 - (l₂	(/ l _y)] = 0.813				
Poissons ratio		v = 0.3					
Shear modulus		G = E / [2 >	< (1 + v)] = 8076	9 N/mm ²			
Unrestrained length		$L = 1.0 \times L_{s}$	_{s1} = 4600 mm				
Elastic critical buckling momer	it	$M_{cr} = C_1 \times c$	$\pi^2 \times E \times I_z / (L^2 \times I_z)$	$(x g) \times \sqrt{[I_w / I_z + L^2]}$	\times G \times I _t / (π^2	$\times E \times I_z)] =$	
		306.1 kNm					
Slenderness ratio for lateral to	rsional buckling	$\lambda_{LT} = \sqrt{W_{I}}$	$_{\text{pl.y}} \times f_{\text{y}} / M_{\text{cr}}) = 0.$	759			
Limiting slenderness ratio		λ _{LT,0} = 0.4					
			$\lambda_{LT} > \lambda_{LT,0} - Lat$	teral torsional bi	uckling cann	ot be ignored	
Design resistance for buckli	ng - Section 6.3	3.2.1					
Buckling curve - Table 6.5		b					
Imperfection factor - Table 6.3		α _{LT} = 0.34					
Correction factor for rolled sec	tions	$\beta = 0.75$	_				
LTB reduction determination fa	actor	φ _{LT} = 0.5 ×	$[1 + \alpha_{LT} \times (\lambda_{LT})]$	- $\lambda_{LT,0}$) + $\beta \times \lambda_{LT^2}$	²] = 0.777		
LTB reduction factor - eq 6.57		χ _{LT} = min(1	/ [φ _{LT} + √(φ _{LT} ² -	β × λ _{LT} ²)], 1, 1 / 2	λ _{LT} ²) = 0.839		
Modification factor		f = min(1 -	0.5 × (1 - k _c)× [1	$-2 \times (\lambda_{LT} - 0.8)^{2}$	²], 1) = 0.970		
Modified LTB reduction factor	- eq 6.58	χ _{LT,mod} = mi	n(χ _{LT} / f, 1) = 0.8	365			
Design buckling resistance mo	ment - eq 6.55	$M_{b,Rd} = \chi_{LT,t}$	mod × Wpl.y × fy / γ	_{/M1} = 152.7 kNm			
	PASS	5 - Design buckli	ng resistance i	moment exceed	s design ben	iding moment	

	Project	24 HEAT	H DRIVE		Job no. 162637	
Form Structural Design 77 St John Street	Calcs for	BEAN	I G-12		Start page no./Re	evision 4
EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date

Check vertical deflection - Sec	tion 7.2.1				
Consider deflection due to perma	anent and impos	sed loads			
Limiting deflection		$s_{\rm m} = \min(10)$	mm = 1.0250	- 10 mm	
			J IIIII, Ls1 / Z30)	- 10 11111	
Maximum deflection span 1		$\delta = max(abs$	$(\delta_{max}) = abs(\delta_{min})$	= 7 325 mm	
Maximum deneotion span 1					

	Project 24 HEATH DRIVE				Job no. 162637	
Form Structural Design 77 St John Street	Calcs for	Start page no./Re	evision 1			
EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date

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



Support A Support B

Applied loading

Beam loads

Rotationally free Vertically restrained Rotationally free Permanent self weight of beam × 1

Permanent point load 23 kN at 600 mm Imposed point load 10.4 kN at 600 mm Permanent point load 23 kN at 2600 mm Imposed point load 10.4 kN at 2600 mm Permanent partial UDL 6.6 kN/m from 0 mm to 600 mm Permanent partial UDL 6.6 kN/m from 2600 mm to 3200 mm

Load combinations

Load combination 1

 $\begin{array}{l} \text{Permanent} \times 1.35 \\ \text{Imposed} \times 1.50 \end{array}$

🐙 Tekla	Project	24 HEAT			Job no. 162637	
Tedds Form Structural Design	Calao for			Start nage no /Revision		
77 St John Street		BEAN	/I G-18		Start page 10./	2
London	Calcs by	Calcs date Checked by		Checked date	Approved by	Approved date
EC1M 4NN	HD	15/01/2019	ROM	-		
				Permar	ent × 1.35	
				Impose	d × 1.50	
		Support B		Permar	ent \times 1.35	
				Impose	d × 1.50	
Analysis results						
Maximum moment		M _{max} = 30 k	Nm	$M_{min} = 0$) kNm	
Maximum shear		V _{max} = 52.5	kN	$V_{min} = -$	52.5 kN	
Deflection		δ _{max} = 5.9 n	nm	δ _{min} = 0	mm	
Maximum reaction at support A		R _{A_max} = 52	.5 kN	R _{A_min} =	52.5 kN	
Unfactored permanent load read	ction at support A	A R _{A_Permanent}	= 27.3 kN			
Unfactored imposed load reaction	on at support A	RA_Imposed =	10.4 KN	D -	50 5 LAL	
Maximum reaction at support B	ation at augment	$R_{B_{max}} = 52$.5 KN - 27 2 kN	R _{B_min} =	52.5 KN	
Linfactored imposed load reactiv	cion al support E	B RB_Permanent	- 27.3 KN			
	on at support D	T B_Imposed -	10.4 KN			
Section details						
Section type		UB 203X10	I2X23 (B54-1)			
EN 10025-2:2004 - Hot rolled r	products of stru	ctural steels				
Nominal thickness of element		t = max(t _f , t	(w) = 9.3 mm			
Nominal vield strength		f _v = 355 N/r	mm ²			
Nominal ultimate tensile strengt	h	f _u = 470 N/I	mm²			
Modulus of elasticity		E = 210000) N/mm ²			
	- 9.3					
	$\overline{\uparrow}$					
	T					
	203.2		5.4			
	− 9.3					
	Į.	4	>			
			.1			
Partial factors - Section 6.1		- 1 00				
	L 1114 /	$\gamma_{\rm M0}=1.00$				
Resistance of members to insta		γ _{M1} = 1.00				
Resistance of tensile members	to tracture	γ _{M2} = 1.10				
Lateral restraint		_				
		Span 1 has	s lateral restrair	nt at supports only	1	
Effective length factors						
Effective length factor in major a	axis	K _y = 1.000				

🚝 Tekla	Project	Job no.				
Tedds		24 HEA1	'H DRIVE		16	2637
Form Structural Design	Calcs for	s for			Start page no./F	Revision
London		BEAN	/I G-18	1		3
EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date
Effective length factor in minor	axis	K _z = 1.000				
Effective length factor for torsio	n	K _{LT.A} = 1.00	0			
-		K _{LT.B} = 1.00	0			
Classification of cross section	ons - Section 5.5	;				
		ε = √[235 N	/mm ² / f _v] = 0.81			
Internal compression parts s	ubiect to bendi	ng - Table 5.2 (s	sheet 1 of 3)			
Width of section		c = d = 169	.4 mm			
		c / t _w = 38.6	δ×ε <= 72×ε	Class 1		
Outstand flanges - Table 5.2	(sheet 2 of 3)					
Width of section		c = (b - t _w -	2 × r) / 2 = 40 6	mm		
Width of Coolion		$c/t_{f} = 54$	2 × 1) / 2 - 40.0	Class 1		
		07 tj = 0.4 A		01035 1	Seci	ion is class 1
Check check Section 6.2.6						
Height of woh		h - h 2 v	t 1916 mm			
Shear area factor		$n_{\rm w} = 11 - 2 \times 1000$	u – 184.8 mm			
		$\eta = 1.000$	veln			
		11, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1, 1,	^ 67 II	Shear buckling	resistance ca	an he ianored
Design shear force		V _{Ed} = max(a	abs(V _{max}), abs(V _l	_{min})) = 52.5 kN		in be ignored
Shear area - cl 6.2.6(3)	Shear area - cl 6.2.6(3)			+ 2 × r) × t _f , η ×	h _w × t _w) = 123	8 mm²
Design shear resistance - cl 6.2	2.6(2)	$V_{c,Rd} = V_{pl,Rd}$	$d = A_v \times (f_v / \sqrt{[3]})$	/ γ _{M0} = 253.7 kN	1	
		PAS	S - Design she	ar resistance ex	ceeds desig	n shear force
Check bending moment maic	or (v-v) axis - Se	ction 6.2.5				
Design bending moment		M _{Ed} = max(abs(M _{s1_max}), abs	s(M _{s1_min})) = 30 k	Nm	
Design bending resistance mor	ment - eq 6.13	$M_{c,Rd} = M_{pl,F}$	$R_{d} = W_{pl.y} \times f_y / \gamma_{M0}$	o = 83.1 kNm		
Slenderness ratio for lateral t	orsional buckli	ng				
Correction factor - Table 6.6		k _c = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (I _z	/ l _y)] = 0.96			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 ×	(1 + v)] = 8076	9 N/mm ²		
Unrestrained length		$L = 1.0 \times L_s$	₁ = 3200 mm			
Elastic critical buckling moment	t	$M_{cr} = C_1 \times \tau$	$t^2 \times E \times I_z / (L^2 \times$	g) × $\sqrt{[I_w / I_z + L^2]}$	\times G \times It / (π^2 :	×Ε×Ιz)] =
		63.6 kNm				
Slenderness ratio for lateral tor	sional buckling	$\lambda_{LT} = \sqrt{W_{F}}$	$f_{\rm M.y} \times f_{\rm y} / M_{\rm cr}$ = 1.1	143		
Limiting slenderness ratio		$\lambda_{LT,0} = 0.4$				
			λιτ > λιτ,ο - Late	eral torsional b	uckling cann	ot be ignored
Design resistance for bucklir	ng - Section 6.3.	2.1				
Buckling curve - Table 6.5		b				
Imperfection factor - Table 6.3		α _{LT} = 0.34				
Correction factor for rolled sect	ions	β = 0.75				
LTB reduction determination fa	ctor	ϕ_{LT} = 0.5 \times	[1 + αιτ × (λιτ -	$\lambda_{LT,0}$) + $\beta \times \lambda_{LT}^{2}$] = 1.116	
LTB reduction factor - eq 6.57		χ∟⊤ = min(1	/ [φ _{LT} + √(φ _{LT} ² - β	5 × λ _{LT} ²)], 1, 1 / 1	λ _{LT} ²) = 0.613	
Modification factor		f = min(1 - 0	$0.5 \times (1 - k_c) \times [1]$	- 2 × (λ _{LT} - 0.8)	²], 1) = 0.977	
Modified LTB reduction factor -	eq 6.58	$\chi_{LT,mod} = mi$	n(χ _{LT} / f, 1) = 0.6	27		

	Project	24 HEAT	H DRIVE		Job no. 162637	
Form Structural Design 77 St John Street	Calcs for	BEAN	1 G-18		Start page no./R	evision 4
London EC1M 4NN	Calcs by HD	Calcs date 15/01/2019	Checked by ROM	Checked date	Approved by	Approved date
Design buckling resistance m	oment - eq 6.55 PASS	M _{b,Rd} = χ _{LT,n} : - Design buckli	nod × W _{pl.y} × f _y / ng resistance	γ _{M1} = 52.1 kNm moment exceed	ls design ben	ding momen
Check vertical deflection - S	Section 7.2.1	nsed loads				
Limiting deflection		$\delta_{\text{lim}} = \min(1)$	0 mm, L₅₁ / 250	0) = 10 mm		
Maximum deflection span 1		δ = max(ab	s(δ _{max}), abs(δ _m S - Maximum	_{in})) = 5.937 mm	not avcoad de	oflection lim
		PAS		denection does	not exceed de	enection initia

🚝 Tekla	Project				Job no.	
Tedds	24 HEATH DRIVE					
Form Structural Design	Calcs for S				Start page no./Revision	
77 St John Street		BEAM	G-19			1
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	HD	19/12/2018	ROM			

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



Load combination 1

Support B

Support A

 $\begin{array}{l} \text{Permanent} \times 1.35\\ \text{Imposed} \times 1.50\\ \text{Permanent} \times 1.35\\ \text{Imposed} \times 1.50\\ \text{Permanent} \times 1.35\\ \text{Imposed} \times 1.50 \end{array}$

	Project	roject Job no. 24 HEATH DRIVE				
Form Structural Design	Calcs for				Start page no./F	Revision
77 St John Street	-	BEAN	BEAM G-19			2
London EC1M 4NN	Form Structural Design 77 St John Street London EC1M 4NN Calcs for Analysis results Calcs by HD Maximum moment Maximum shear Deflection Maximum reaction at support A Unfactored permanent load reaction at support A Unfactored permanent load reaction at support A Unfactored permanent load reaction at support B Unfactored permanent load reaction at support B Section details Section details Section type Steel grade EN 10025-2:2004 - Hot rolled products of str Nominal thickness of element Nominal ultimate tensile strength Modulus of elasticity Image: Section 6.1 Resistance of cross-sections Resistance of tensile members to fracture Lateral restraint Effective length factors Effective length factor in major axis	Calcs date 19/12/2018	Checked by ROM	Checked date	Approved by	Approved date
Analysis results			I.		1	1
Maximum moment		M _{max} = 42.	i kNm	M _{min} =	0 kNm	
Maximum shear		V _{max} = 31.2	2 kN	$V_{min} = \cdot$	-31.2 kN	
Deflection		δ _{max} = 9.3 r	nm	δmin = () mm	
Maximum reaction at support A		RA_max = 31	. 2 kN	RA_min =	= 31.2 kN	
Unfactored permanent load rea	iction at support A	A RA_Permanent	= 9.6 kN			
Unfactored imposed load react	ion at support A	RA_Imposed =	12.2 kN			
Maximum reaction at support B		R _{B_max} = 31	. 2 kN	R _{B_min} =	= 31.2 kN	
Unfactored permanent load rea	iction at support E	B RB_Permanent	= 9.6 kN			
Unfactored imposed load react	ion at support B	$R_{B_{Imposed}} =$	12.2 kN			
Section details						
Section type		UC 203x20)3x46 (BS4-1)			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled	products of stru	ctural steels				
Nominal thickness of element		t = max(t _f , t	t _w) = 11.0 mm			
Nominal yield strength		fy = 355 N/	mm²			
Nominal ultimate tensile streng	th	f _u = 470 N/	mm²			
Modulus of elasticity		E = 21000	0 N/mm²			
-	$\mathbf{A} \stackrel{\downarrow}{=}$					
	203.2	-> 4	-7.2			
-	▼ ↑					
		203.6-		→		
Partial factors - Section 6.1						
Resistance of cross-sections		γмо = 1.00				
Resistance of members to insta	ability	γ _{M1} = 1.00				
Resistance of tensile members	to fracture	γ _{M2} = 1.10				
Latoral rostraint		·				
		Span 1 has	s lateral restrai	nt at supports on	ly	
Effective length factors					-	
Effective length factor in major	axis	K _v = 1 000				
Effective length factor in minor	axis	K _z = 1.000				
Effective length factor for torsio	'n	KLT.A = 1.00	00			
U		K _{LT.B} = 1.00	00			
Classification of cross section	ns - Section 5 5					
		ε = √[235 Ν	J/mm² / f _y] = 0.8	31		

Tekla	Project	24 HEA ⁻	Job no.					
Form Structural Design	Calcs for				Start page no./R	evision		
77 St John Street		BEAN	1 G-19			3		
London	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date		
EC1M 4NN	HD	19/12/2018	ROM					
Internal compression parts	subject to bend	ding - Table 5.2 (s	sheet 1 of 3)					
Width of section		c = d = 160	. 8 mm					
		c / t _w = 27.4	↓ ×ε <= 72 ×ε	Class 1				
Outstand flanges - Table 5.2	2 (sheet 2 of 3)							
Width of section	(,	c = (b - t _w -	2 × r) / 2 = 88 m	ım				
		c / t _f = 9.8 >	< ε <= 10 × ε	Class 2				
					Sect	ion is class 2		
Check shear - Section 6.2.6								
Height of web		h _w = h - 2 ×	t _f = 181.2 mm					
Shear area factor		η = 1.000						
		h⊮ / t⊮ < 72	×ε/η					
			S	Shear buckling	resistance ca	n be ignored		
Design shear force		V _{Ed} = max(abs(V _{max}), abs(V	(_{min})) = 31.2 kN		-		
Shear area - cl 6.2.6(3)		A _v = max(A	$-2 \times b \times t_f$ + (t _w	+ 2 \times r) \times tf, η \times	hw × tw) = 169 8	B mm ²		
Design shear resistance - cl 6	5.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$d = A_v \times (f_y / \sqrt{[3]})$) / γмо = 347.9 kM	N			
		PAS	S - Design shea	ar resistance ex	ceeds desig	n shear force		
Check bending moment ma	jor (y-y) axis - S	Section 6.2.5						
Design bending moment		M _{Ed} = max(abs(Ms1_max), ab	s(M _{s1_min})) = 42. 1	l kNm			
Design bending resistance me	oment - eq 6.13	$M_{c,Rd} = M_{pl,l}$	$Rd = W_{pl.y} \times f_y / \gamma N$	10 = 176.6 kNm				
Slenderness ratio for lateral	torsional buck	ling						
Correction factor - Table 6.6		k _c = 0.94						
		$C_1 = 1 / k_c^2$	= 1.132					
Curvature factor		g = √[1 - (Iz	/ Iy)] = 0.813					
Poissons ratio		v = 0.3						
Shear modulus		G = E / [2 >	(1 + v)] = 8076	9 N/mm²				
Unrestrained length		$L = 1.0 \times L_{\odot}$	₃1 = 5400 mm					
Elastic critical buckling mome	nt	$M_{cr} = C_1 \times T_2$	$\tau^2 \times E \times I_z / (L^2 \times$	g) × $\sqrt{[I_w / I_z + L^2]}$	$^{2} \times G \times I_{t} / (\pi^{2})$	< E × Iz)] =		
		244.6 kNm						
Slenderness ratio for lateral to	orsional buckling	$\lambda_{LT} = \sqrt{(VV)}$	$p_{I,y} \times f_y / M_{cr} = 0.5$	85				
Limiting slenderness ratio		λ _{LT,0} = 0.4						
		7	llt > ХLT,0 - Late	eral torsional b	uckling canno	ot be ignored		
Design resistance for buckl	ing - Section 6.	3.2.1						
Buckling curve - Table 6.5		b						
Imperfection factor - Table 6.3	3	αιτ = 0.34						
Correction factor for rolled see	ctions	β = 0.75	=					
LTB reduction determination f	actor	φιτ = 0.5 ×	[1 + αιτ × (λιτ -	$\lambda_{\text{LT,0}} + \beta \times \lambda_{\text{LT}}$	²] = 0.847			
LIB reduction factor - eq 6.57		χ∟⊤ = min(1	/ [φ∟т + √(φ∟т² - μ	$3 \times \lambda LT^{2}$], 1, 1 /	λιτ ²) = 0.789			
Modification factor	0.50	f = min(1)	0.5 × (1 - Kc)× [1	-2×(λιτ-0.8)	²], 1) = 0.970			
Noamea LIB reduction factor	- eq 6.58	χLT,mod = mi	$\Pi(\chi_{LT} / T, 1) = 0.8$) 14 				
Design Duckling resistance m	ornent - eq 6.55	$IVIb,Rd = \chi LT,$	mod × VVpl.y × Ty /γ	$m_1 = 143.7 \text{ KNM}$	o docian bo-	dina moment		
	PA53	b - שפוקוו DucKill	ny resistance n	ioment exceed	s design ben	ung moment		
Check vertical deflection - S	Section 7.2.1	nonad lands						
Limiting deflection	manent and m	905eu 10aus 8 min/1	0 mm 1 - 4 / 250	= 10 mm				
			0 mm, Ls1 / 200)					

Tekla Tedds	Project 24 HEATH DRIVE				Job no.	
Form Structural Design 77 St John Street	Calcs for BEAM G-19				Start page no./Revision 4	
London Calc EC1M 4NN	Calcs by HD	Calcs date 19/12/2018	Checked by ROM	Checked date	Approved by	Approved date

Maximum deflection span 1

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 9.294 \text{ mm}$

JOSTITLE	JOB NUMBER / FILE:	CALCULATION NUMBER:	Form
24 MEATH DRIVE	626 37		
CALCULATION:	CALCULATION BY:	DATE: CHECKED BY:	E ALAST
G-20 LOADING	HD	04/12/18	
CALCULATIONS:			
REF GARDING.		OUTPUT	
proceeding of the series		1.2	
NUT LOFF TO MI O INC	S ₹ , [, 53₩	N/M	
$L = 1.6 \times 1.53 \text{ (N/m^2} = 2.4 \text{ (N)}$	m		
SECOND FLOOR LOAD			
TIMBERFLOOR SPAN 2M,	I.OZKN/A	n ²	
PL = 2m × 1.02kN/M2 = 2.	OH KN/M		
$\mu = \pi \times 1.5 \times N/M^2 = 3$	KNM		
FIRST FLOOR LOAT		1/2 2	
I LAISER E COOR SPAN G. IM	1 .02 KI		
$DC = 4 M \times 1 - 02 KN/m^2$	= 4.21	K)/m	
MAGE CORPS			
3.2 m of BRICK WALL, 10	ommthick, 2	.54 KN/m²	
DL= 3.2m × 2.54 KN/M= =	B. F KN/	n	
2.9m of STUD WALL DE = 2	.9m x 0.55kl	1/m² = 1.6	
TOTAL LOADS ACTING UPO	N EXISIN	6	
FIRST FLOOR BEAM!			
D4 = 10.74 kV/m			
SEAM SDAN = 3.5M	OKN		
RK = 2	3.8 KN	- new d	
BEAM 2	EXISTING 1	FIRST HOOR	
4m of FLOOR 2 LEVES	AND DUE	LEVC OF	
$DL = 44 \text{ M} \times 1.02 \text{ KN/m}^3 \times 2 + 1$	0.55×3.2M	1= lokn/m	
BEAM SPAN = 3. M			
SUPPORT REACTIONS GA	= 16 (KN)		
Q1	C= 18.6KN		

FORM Structural Design Ltd.

OI UMATE DENIE	JOB NUMBER / FILE:	CALCULATION NUMBER:	Form
24 MEATH DRIVE			
2001 - ADING CONTINUES	CALCULATION BY:	DATE: CHECKED BY:	San a state of the
The Constants Contract LD.	HD	04/12/18	
ALCULATIONS:			
F-2010ADIADIA CONTINU	20	OUTPUT	
GROOND YLODR LOAD	2.600 5	SPAN.	
2.6m × 1.02 KN/12 = 2.	7KN/m		
2.6M XI.5 KN/Mi = 3.	9 KNM		
GK = 30.0 kN	04.6 M		
QK- 23.8 KN			
BEAM2 LOAD OUT. JM			
QK= 18.6 KN			
G-17 AG-18 ALSO ACT	spow t	sefm	
6-20			
PTLAS STEEL REACTIONS 6K =	17.4 LN		
@K =	16.4 IW		
PTR AS STEE REALTIONS GK	= 273 KN		
ak	- 10.4 M		
G-18/9-7R 6-25	ACSUME	6-25	
+P/44	BIMILAR	TO G-18	
N Stoo 7			
G-18@1224= (17,4p1+10	44) + (27	,3a410,4u)	
6-25@ 4350= 27, 30L+1	0.411		

	Project	Job no. 162637				
Form Structural Design 77 St John Street	Calcs for BEAM G-20				Start page no./Revision 1	
EC1M 4NN	Calcs by HD	Calcs date 05/02/2019	Checked by ROM	Checked date	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

Vertically restrained Rotationally free Vertically restrained Rotationally free

Permanent self weight of beam × 1 Permanent full UDL 2.7 kN/m Permanent point load 30 kN at 1000 mm Permanent point load 60.8 kN at 3600 mm Imposed full UDL 3.9 kN/m Imposed point load 23.8 kN at 1000 mm Imposed point load 39.4 kN at 3600 mm

Load combinations

Load combination 1

```
Support A
```

Permanent \times 1.35 Imposed \times 1.50

🐙 Tekla	Project	24 HEAT			Job no.	2637
Tedds Form Structural Design	Calas far				Start name no //	Devision
77 St John Street	Calcs for	BEAN	/ G-20		Start page no./r	2
London	Calcs by	Calcs date		Checked date	Approved by	
EC1M 4NN	HD	05/02/2019	ROM	Checked date	Approved by	Approved date
				Permar	ent $ imes$ 1.35	
				Impose	$d \times 1.50$	
		Support B		Permar	ent $ imes$ 1.35	
				Impose	d × 1.50	
Analysis results						
Maximum moment		M _{max} = 123	.6 kNm	$M_{min} = 0$	kNm	
Maximum shear		V _{max} = 107.	9 kN	V _{min} = -	156.2 kN	
Deflection		δ _{max} = 9.6 n	nm	δ _{min} = 0	mm	
Maximum reaction at support A		R _{A max} = 10	7.9 kN	R _{A min} =	107.9 kN	
Unfactored permanent load read	ction at support /	A R _{A Permanent}	= 42 kN			
Unfactored imposed load reaction	on at support A	R _{A Imposed} =	34.1 kN			
Maximum reaction at support B		R _{B_max} = 15	6.2 kN	R _{B_min} =	156.2 kN	
Unfactored permanent load read	ction at support I	B R _{B_Permanent}	= 64.4 kN			
Unfactored imposed load reaction	on at support B	R _{B_Imposed} =	46.2 kN			
Section details						
Section type		UC 203x20	3x86 (BS4-1)			
Steel grade		S355	(,			
EN 10025-2:2004 - Hot rolled p	products of stru	ctural steels				
Nominal thickness of element		t = max(t _f , t				
Nominal yield strength		f _y = 345 N/r	mm²			
Nominal ultimate tensile strengt	h	f _u = 470 N/r	mm²			
Modulus of elasticity		E = 210000) N/mm ²			
	50.5					
	Ť					
	-222	→ ◄	-12.7			
	↓			_		
	50.5					
	Ť					
	4	209.1-		→		
Partial factors - Section 6.1						
Resistance of cross-sections		γ _{M0} = 1.00				
Resistance of members to insta	bility	γ _{M1} = 1.00				
Resistance of tensile members	to fracture	γ _{M2} = 1.10				
Lateral restraint						
		Span 1 has	s lateral restrain	nt at supports only	,	
Effective length factors	wie	V - 4 000				
Enecuve length lactor in major a	IXIS	r _y = 1.000				

🐙 Tekla	Project	Job no.				
Tedds		24 HEA1	"H DRIVE		16	2637
Form Structural Design	Calcs for	REAN	1 G-20		Start page no./F	Revision
London	O al a a hui	DEAN	Obersheed hu		A	5
EC1M 4NN	HD	05/02/2019	ROM	Checked date	Approved by	Approved date
Effective length factor in minor	axis	K _z = 1.000				
Effective length factor for torsio	n	K _{LT.A} = 1.00	0			
		K _{LT.B} = 1.00	0			
Classification of cross sectio	ns - Section 5.5	;				
		ε = √[235 N	/mm ² / f _y] = 0.83			
Internal compression parts s	ubiect to bendir	ng - Table 5.2 (s	heet 1 of 3)			
Width of section		c = d = 160	.8 mm			
		c / t _w = 15.3	3 × ε <= 72 × ε	Class 1		
Outstand flanges - Table 5.2	(sheet 2 of 3)					
Width of section	(011001 2 01 0)	c = (b - t _w -	2 × r) / 2 = 88 m	m		
		$c / t_f = 5.2 >$	ε <= 9 × ε	Class 1		
		0, 1 0.2 /		01000 1	Sec	tion is class 1
Chack shear - Section 6.2.6						
Height of web		h = h - 2 ×	t₂ = 181 2 mm			
Shear area factor		n = 1000	u – 101.2 mm			
		h / t < 72	x ɛ / 'n			
		11w / tw • / 2	S	Shear buckling	resistance ca	an be ianored
Design shear force		V _{Ed} = max(a	abs(V _{max}), abs(V _l	_{min})) = 156.2 kN		in be ignored
Shear area - cl 6.2.6(3)		A _v = max(A	$-2 \times b \times t_{f} + (t_{w})$	+ 2 × r) × t _f , η ×	h _w × t _w) = 306	9 mm²
Design shear resistance - cl 6.2	2.6(2)	$V_{c,Rd} = V_{pl,Rd}$	$d = A_v \times (f_y / \sqrt{[3]})$	/ γ _{M0} = 611.3 kN	1	
		PAS	S - Design she	ar resistance ex	ceeds desig	n shear force
Check bending moment majo	or (y-y) axis - Se	ction 6.2.5				
Design bending moment		M _{Ed} = max(abs(M _{s1_max}), abs	s(M _{s1_min})) = 123 .	6 kNm	
Design bending resistance mor	nent - eq 6.13	$M_{c,Rd} = M_{pl,F}$	$R_{d} = W_{pl.y} \times f_y / \gamma_{M0}$	₀ = 337 kNm		
Slenderness ratio for lateral t	orsional buckli	na				
Correction factor - Table 6.6		k _c = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (I _z	/ l _y)] = 0.818			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 ×	(1 + ν)] = 8076	9 N/mm ²		
Unrestrained length		L = 1.0 × L _s	₁ = 4400 mm			
Elastic critical buckling moment	t	$M_{cr} = C_1 \times \tau$	$t^2 \times E \times I_z / (L^2 \times$	g) × $\sqrt{[I_w / I_z + L^2]}$	\times G \times It / (π^2	$(E \times I_z)$] =
		962.6 kNm				
Slenderness ratio for lateral tor	sional buckling	$\overline{\lambda}_{LT} = \sqrt{W_{F}}$	$f_{\rm M,y} \times f_{\rm y} / M_{\rm cr}$ = 0.5	592		
Limiting slenderness ratio		$\overline{\lambda}_{LT,0} = 0.4$				
		2	$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - Late$	eral torsional b	uckling cann	ot be ignored
Design resistance for bucklin	ig - Section 6.3.	2.1				
Buckling curve - Table 6.5		b				
Imperfection factor - Table 6.3		α _{LT} = 0.34				
Correction factor for rolled sect	ions	β = 0.75		_		
LTB reduction determination factor	ctor	ϕ_{LT} = 0.5 \times	[1 + αιτ × (λιτ -	$\lambda_{\text{LT},0}$) + $\beta \times \overline{\lambda}_{\text{LT}}^2$] = 0.664	
LTB reduction factor - eq 6.57		χ _{∟⊤} = min(1	/ [φ _{LT} + √(φ _{LT} ² - β	$B \times \overline{\lambda}_{LT}^2$], 1, 1 / 2	λ _{LT} ²) = 0.921	
Modification factor		f = min(1 - 0	$0.5 \times (1 - k_c) \times [1]$	- 2 × (λ _{LT} - 0.8)	²], 1) = 0.973	
Modified LTB reduction factor -	eq 6.58	$\chi_{LT,mod} = mi$	n(χ _{LT} / f, 1) = 0.9	47		

Form Structural Design 77 Si Joins Structural Debide Calce for HD BEAM G-20 Start page no.Revealor 4 Calce Start HD Calce Start US/02/2013 Calce Start ROM Calce Start ROM Approved to Mapping Calce Start ROM A		Project	24 HEAT	H DRIVE		Job no. 162637	
T'S Lains Steed EEAM G-20 4 Colors by HD Colors by G-05/02/2019 Checked by ROM Checked date Approved Approved Design buckling resistance moment - eq.6.55 More 2 Munet MSS - Design buckling resistance moment exceeds design bending mo Check vortical deflection - Section 7.2.1 Consider deflection due to permanent and imposed loads Limiting deflection Sm = min(10.04 mm, L_1 / 250) = 10 mm Maximum deflection span 1 S = max(abs(Smu), abs(Smu)) = 9.566 mm PASS - Maximum deflection does not exceed deflection	Form Structural Design	Calcs for	Start page no./Revision				
Enn 4N Cale by HD Cale date ROM Checked by ROM Checked by ROM Approved Part of PASS - Design buckling resistance moment exceeds design bending mo DASS - Design buckling resistance moment exceeds design bending mo Check vertical deflection - Section 7.2.1 Maxie 2 (x_1, x_2, x_3, x_4, x_1, y_3) = 319 kNm Consider deflection due to permanent and imposed loads Sime 1 min(10.04 mm, L_1 / 250) = 10 mm Maximum deflection span 1 S = max(abs(5mm), abs(5mm)) = 9.566 mm Maximum deflection span 1 S = max(abs(5mm), abs(5mm), abs(5mm)) = 9.566 mm PASS - Maximum deflection does not exceed deflection	77 St John Street		BEAN	1 G-20			4
Design buckling resistance moment - eq 6.55 M _B M ₀ = 7,1,max × W _{A2} × f, / γ _{M1} = 319 kNm PASS - Design buckling resistance moment exceeds design bending mo Check vertical deflection - Section 7.2.1 Consider deflection us to permanent and imposed loads Limiting deflection $\delta_m = \min(10.04 \text{ mm}, L_1 / 250) = 10 \text{ mm}$ Maximum deflection span 1 $\delta_m = \max(abs(\delta_{mm}), abs(\delta_m)) = 9.566 \text{ mm}$ PASS - Maximum deflection does not exceed deflection	EC1M 4NN	Calcs by HD	Calcs date 05/02/2019	Checked by ROM	Checked date	Approved by	Approved date
PASS - Jessing huckning resistance moment exceeds design behaving mo Check vertical deflection - Section 7.21 Consider deflection Semi = min(10.04 mm, L_1 / 250) = 10 mm Maximum deflection span 1 S = max(abs(Sma), abs(Sma)) = 9.566 mm PASS - Maximum deflection does not exceed deflection	Design buckling resistance mon	nent - eq 6.55	$M_{b,Rd} = \chi_{LT,n}$	$_{\rm nod} imes W_{\rm pl.y} imes f_{\rm y}$ / γ	_{γм1} = 319 kNm		
Consider deflection Semmannet and imposed loads Limiting deflection Sem = min(10.04 mm, Ls./250) = 10 mm Maximum deflection span 1 Sem = max(abs(Sem.)) = 9.566 mm PASS - Maximum deflection does not exceed deflection	Chaok vertical deflection - Ca	PASS	- Design buckli	ng resistance	moment exceed	s design bend	ling momen
Limiting deflection δ _{im} = min(10.04 mm, L ₁₁ /250) = 10 mm Maximum deflection span 1 δ = max(abs(δ _{im}), abs(δ _{im})) = 9.566 mm PASS - Maximum deflection does not exceed deflection	Consider deflection due to perm	ction 7.2.1 anent and impos	sed loads				
Maximum deflection span 1 ⁶ = max(abs(δmm)) = 9.566 mm) PASS - Maximum deflection does not exceed deflection	Limiting deflection		δ _{lim} = min(1	0.04 mm, L _{s1} / 2	250) = 10 mm		
PASS - Maximum deflection does not exceed deflection	Maximum deflection span 1		δ = max(ab	s(δ_{max}), abs(δ_{min}	n)) = 9.566 mm		
			PAS	S - Maximum d	deflection does i	not exceed de	flection limit

	Project	Job no. 162637				
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EC1M 4NN	Calcs by ROM	Calcs date 15/02/2019	Checked by	Checked date	Approved by	Approved date

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



Load combinations

Support B

Support A

Imposed point load 48.4 kN at 900 mm

Permanent \times 1.35 Imposed \times 1.50 Permanent \times 1.35 Imposed \times 1.50 Permanent \times 1.35 Imposed \times 1.50

🐙 Tekla	Project Job no.					Job no.	
Tedds		24 HEA	IHDRIVE		10	2037	
Form Structural Design 77 St John Street	Calcs for	Start pa BEAM G-21			Start page no./I	Revision 2	
London EC1M 4NN	Cales forStart page no.Cales by ROMCales date 15/02/2019Checked byChecked dateApproved byMmax = 37.2 kNm Vmax = 41.8 kN $M_{max} = 0.1 \text{ mm}$ $M_{max} = 0.1 \text{ mm}$ $M_{max} = 0.1 \text{ mm}$ $M_{max} = 0.1 \text{ mm}$ $M_{max} = 0.1 \text{ mm}$ $M_{min} = 0 \text{ mm}$ RA_max = 41.8 kN RA_max = 41.8 kN RA_max = 41.8 kN 	Approved date					
Analysis results Maximum moment Maximum shear Deflection Maximum reaction at support A		M _{max} = 37.2 V _{max} = 41.8 δ _{max} = 0.1 r R _{A max} = 41	2 kNm 5 kN nm .8 kN	M _{min} = V _{min} = · δ _{min} = (Βα	0 kNm -124.1 kN) mm = 41.8 kN		
Unfactored permanent load react Unfactored imposed load reaction Maximum reaction at support B Unfactored permanent load reaction Unfactored imposed load reaction	ction at support A on at support A ction at support B on at support B	RA_Permanent RA_Imposed = RB_max = 12 RB_Permanent RB_Imposed =	= 17.5 kN 12.1 kN 4.1 kN = 51.6 kN 36.3 kN	R _{B_min} ·	= 124.1 kN		
Section details Section type Steel grade EN 10025-2:2004 - Hot rolled p	products of stru	UC 254x25 S355 ctural steels	54x89 (BS4-1)				
Nominal thickness of element Nominal yield strength Nominal ultimate tensile strength Modulus of elasticity		t = max(t _f , t f _y = 345 N/t f _u = 470 N/t E = 210000	iw) = 17.3 mm mm ² mm ²) N/mm ²				
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to instal Resistance of tensile members t	bility to fracture	$\gamma_{M0} = 1.00$ $\gamma_{M1} = 1.00$ $\gamma_{M2} = 1.10$					
Laterai restraint		Span 1 has	s lateral restrair	nt at supports onl	у		
Effective length factors Effective length factor in major a Effective length factor in minor a Effective length factor for torsior	ixis ixis 1	K _y = 1.000 K _z = 1.000 K _{LT.A} = 1.00 K _{LT.B} = 1.00)0)0				

🚝 Tekla	Project		Job no.						
Tedds		24 HEAT	TH DRIVE		16	2637			
Form Structural Design	Calcs for		10.04		Start page no./R	evision			
77 St John Street London		BEAN	/I G-21			3			
EC1M 4NN	Calcs by ROM	Calcs date 15/02/2019	Checked by	Checked date	Approved by	Approved date			
Classification of cross secti	ons - Section 5.	5							
		ε = √[235 Ν	l/mm ² / f _y] = 0.8	33					
Internal compression parts	subject to bend	ing - Table 5.2 (s	sheet 1 of 3)						
Width of section		c = d = 200	0.3 mm						
		c / t _w = 23.6	δ×ε <= 72×ε	Class 1					
Outstand flanges - Table 5.2	(sheet 2 of 3)								
Width of section		c = (b - t _w -	2 × r) / 2 = 110).3 mm					
		c / t _f = 7.7 >	< ε <= 9 × ε	Class 1					
					Sect	ion is class 1			
Check shear - Section 6.2.6									
Height of web		$h_w = h - 2 \times$	t _f = 225.7 mm						
Shear area factor		η = 1.000							
		h _w / t _w < 72	×ε/η						
				Shear buckling	resistance ca	an be ignored			
Design shear force		V _{Ed} = max($V_{Ed} = max(abs(V_{max}), abs(V_{min})) = 124.1 \text{ kN}$						
Shear area - cl 6.2.6(3)		A _v = max(A	$-2 \times b \times t_{f} + (t)$	t_w + 2 × r) × t _f , η ×	$h_w \times t_w$) = 308	1 mm ²			
Design shear resistance - cl 6	.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$_{\rm d}$ = A _v × (f _y / $\sqrt{3}$	3]) / γмо = 613.6 kN	N				
		PAS	SS - Design sh	ear resistance e	xceeds desig	n shear force			
Check bending moment maj	jor (y-y) axis - S	ection 6.2.5							
Design bending moment		M _{Ed} = max(abs(M _{s1_max}), a	bs(M _{s1_min})) = 37.2	2 kNm				
Design bending resistance mo	oment - eq 6.13	$M_{c,Rd} = M_{pl,l}$	$_{\rm Rd}$ = W _{pl.y} × t _y / γ	_{′M0} = 422.2 kNm					
Slenderness ratio for lateral	torsional buckl	ing							
Correction factor - Table 6.6		$k_c = 0.94$	4 4 9 9						
Curveture factor		$C_1 = 1 / K_c^2$	= 1.132						
		g = v[1 - (1z	/ ly)] – U.O I Z						
Poissons ratio		$\nabla = 0.3$	· (1 · · ·)] - 907	$co N/mm^2$					
		G = E/[Z]	- 1200 mm	69 1\//11111					
Electic critical buckling memory	at	$L = 1.0 \times L_{\odot}$		$(\mathbf{a}) < \sqrt{1}$	$2 \times C \times 1 / (-2)$	(EVI)) -			
Elastic childar buckling momen	п	12302 5 kN	l × ⊑ × lz / (L Im	× y) × v[Iw / Iz + L	× G × It / (n >	< = × I _z)] =			
Slenderness ratio for lateral to	rsional buckling	$\overline{\lambda}_{1T} = \sqrt{W}$	$M_{\rm ex} \times f_{\rm v} / M_{\rm ex} = 0$.185					
l imiting slenderness ratio		$\overline{\lambda}_{1T0} = 0.4$	J.y · · · y / ····ci /						
			$\overline{\lambda_{IT}} < \overline{\lambda_{ITO}}$	- Lateral torsion	al bucklina ca	an be ianored			
	PASS	S - Design bendi	ng resistance	moment exceed	ls design ben	ding moment			
Check vertical deflection - S	ection 7.2.1	U	•		Ū	U			
Consider deflection due to per	manent and imp	osed loads							
Limiting deflection		$\delta_{\text{lim}} = L_{s1} / 2$	250 = 4.8 mm						
Maximum deflection span 1		δ = max(ab	s(δ _{max}), abs(δ _{mi}	n)) = 0.099 mm					
		PAS	S - Maximum	deflection does	not exceed d	eflection limit			

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Tedds						
Form Structural Design	Calcs for				Start page no./Revision	
77 St John Street		BEAM G-22 AND G-23				1
EC1M 4NN	Calcs by HD	Calcs date 16/01/2019	Checked by ROM	Checked date	Approved by	Approved date
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Form Structural Design	Calao far	21112/0	Brate		Start name no //	Devision
77 St John Street	Calcs for	BEAM G-2	2 AND G-23		Start page no./i	2
EC1M 4NN	Calcs by HD	Calcs date 16/01/2019	Checked by ROM	Checked date	Approved by	Approved date
Analysis results						
Maximum moment		M _{max} = 54. 1	l kNm	M _{min} =	0 kNm	
Maximum shear		V _{max} = 47 k	N	$V_{min} = $	-47 kN	
Deflection		δ _{max} = 8.9 n	nm	δ _{min} = () mm	
Maximum reaction at support <i>i</i>	4	RA_max = 47	kN	R _{A_min} :	= 47 kN	
Unfactored permanent load read	action at support A	RA_Permanent	= 23.3 kN			
Unfactored imposed load reac	tion at support A	R _{A_Imposed} =	10.4 kN			
Maximum reaction at support I	3	R _{B_max} = 47	kN	R _{B_min} :	= 47 kN	
Unfactored permanent load rea	action at support E	B RB_Permanent	= 23.3 kN			
Unfactored imposed load reac	tion at support B	$R_{B_{Imposed}} =$	10.4 kN			
Section details						
Section type		UC 203x20	3x46 (BS4-1)			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled	products of stru	ctural steels				
Nominal thickness of element		t = max(t _f , t) = 11.0 mm			
Nominal yield strength		f _y = 355 N/r	mm²			
Nominal ultimate tensile streng	jth	f _u = 470 N/mm ²				
Modulus of elasticity		E = 210000 N/mm ²				
	<u> </u>					
	503.2	203.6-	7.2			
Portial factors - Castier 0.4						
Resistance of cross-sections		VM0 = 1 00				
Resistance of members to inst	Resistance of cross-sections					
Resistance of tensile members			$\gamma_{M1} = 1.00$			
		7M2 - 1.10				
Lateral restraint		Span 1 has	s lateral restrair	nt at supports onl	У	
Effective length factors						
Effective length factor in major	axis	K _v = 1.000				
Effective length factor in minor	axis	K ₇ = 1.000				
Effective length factor for torsid	on	KITA = 1.00	0			
		Кіт.в = 1.00	0			

Tekla	Project	24 HEA ⁻	TH DRIVE		Job no.	
Form Structural Design	Calcs for				Start page no./F	Revision
77 St John Street		BEAM G-2	2 AND G-23		1.0	3
London EC1M 4NN	Calcs by HD	Calcs date 16/01/2019	Checked by ROM	Checked date	Approved by	Approved date
Classification of cross section	ons - Section 5	.5 ε = √[235 Ν	$J/mm^2 / f_{\rm s} = 0.8$	1		
		ε = ν[255 f		•		
Midth of soction	subject to bend	a - d - 160	Sheet 1 of 3)			
		c - u = 100	1.0 1 × c <= 72 × c	Class 1		
		$C / t_{W} - 2 / .$	+ ^ & <= 12 ^ &			
Outstand flanges - Table 5.2	(sheet 2 of 3)					
width of section		$c = (p - t_w - t_w)$	2 × r) / 2 = 88 n	nm Olasa 0		
		$C / t_f = 9.8$	3 × U1 => 3 ×	Class 2	See	lian ia alaan 7
					Sect	ION IS CIASS Z
Check shear - Section 6.2.6						
Height of web		h _w = h - 2 >	< t _f = 181.2 mm			
Shear area factor		η = 1.000				
		h _w / t _w < 72	. × ε / η			
) (Shear buckling	resistance ca	an be ignored
Design snear force		$V_{Ed} = max($	$abs(V_{max}), abs(V_{max})$	(min) = 47 KN	h	0
Shear area - ci 6.2.6(3)	2 (2)	$A_v = \max(F)$	$A - Z \times D \times l_f + (l_v)$	$v + 2 \times r \times t_f, \eta \times $	$n_w \times l_w) = 169$	ð mm-
Design shear resistance - cro.	2.0(2)	Vc,Rd - Vpl,F	$\mathbf{A} = \mathbf{A}_{\mathbf{v}} \times (\mathbf{I}_{\mathbf{y}} / \mathbf{v}_{\mathbf{i}})$]) / γmo - 347.9 Ki	N Nacada dasia	n choor force
			55 - Design site	ear resistance e	kceeus uesig	II Shear Torce
Check bending moment maj	or (y-y) axis - S	Section 6.2.5 $M = mox$		(M) = EAA	l kNm	
Design bending moment	mont og 6 12		$(aDS(IVI_{s1}_{max}), all = 10/100 \text{ yf} / 1000$	$(VI_{s1}_{min}) = 34.1$	I KINITI	
Design bending resistance mo	ment - eq 0.13	IVIc,Rd – IVIpI,	$Rd - VV pl.y \times Ty / \gamma f$	M0 – 170.0 KINIII		
Slenderness ratio for lateral	torsional buck	ling				
Correction factor - Table 6.6		$K_c = 0.94$	- 4 422			
Cupyature factor		$C_1 = 1 / K_c$	- 1.132 /1)] - 0.813			
		y = 03	2 / Ty)] – 0.013			
Shear modulus		V = 0.3	$(1 \pm y) = 8076$	9 N/mm ²		
		G = E / [2]	(1 + 0) = 8070	9 11/11111		
Elastic critical buckling momor	x t	$L = 1.0 \times L$	$\pi^2 \times E \times L / (L^2)$	(a) x 1/1 / 1 + 12	$2 \times C \times L/(\pi^2)$	
	it.	306 1 kNm		(y) < ([iw / iz + L	× G × It / (<i>n</i>)	×∟×ız/] −
Slenderness ratio for lateral to	rsional buckling	$\overline{\lambda}_{1T} = \sqrt{W}$	$f_{\rm rlv} \times f_{\rm v} / M_{\rm er}) = 0$	759		
Limiting slenderness ratio	loional buoking	$\overline{\lambda}_{LT0} = 0.4$				
			$\overline{\lambda}_{1,1} > \overline{\lambda}_{1,1,0} - Ia$	teral torsional b	uckling cann	ot be ianored
					uoning ounn	et ze ignered
Buckling curve Table 6.5	ng - Section 6.	5.2.1 b				
Imperfection factor - Table 6.3		0 au - 0 3 4				
Correction factor for rolled sec	tions	β = 0 75				
I TB reduction determination f	actor	β = 0.75 ω τ = 0.5 ×	$[1 + \alpha_{1T} \times (\overline{\lambda}_{1T})]$	$-\overline{\lambda}$ $+\beta \times \overline{\lambda}$	² 1 = 0 777	
LTB reduction factor - eq.6.57		$\psi_{LT} = 0.0 \times$	$1 + \alpha \epsilon + \sqrt{\alpha r^2}$	$\beta \propto \overline{\lambda} - 2$ 1 1 1 /	$\overline{\lambda}_{1} - 2$ = 0.839	
Modification factor		$f = \min(1 - 1)$	∩ 5 ∨ (1 - k)∨ [1	-2×(<u>)</u> , -, -, 0, 8)	$(21 \ 1) = 0.000$	
Modified LTR reduction factor	- ea 6 58	r – mm(r - vr-,	$\sin(\gamma_{1T} / f_{1}) = 0$	865	, ., – 0.010	
Design huckling resistance mo	oment - ea 6 55		$\mathbb{W}_{\mathcal{K}^{\perp}}$ $\mathcal{W}_{\mathbb{W}^{\perp}} \sim \mathbf{f}_{\mathbb{W}} / \mathbf{f}_{\mathbb{W}}$	/M1 = 152 7 kNm		
	ρ <u>Δ</u> ς	S - Desian huckl	ing resistance	moment exceed	s desian hen	ding moment
	1400					

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Check vertical deflection - Section 7.2.1	
Consider deflection due to permanent and	imposed loads
Limiting deflection	δ_{lim} = min(10 mm, L _{s1} / 250) = 10 mm
Maximum deflection span 1	$\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 8.905 \text{ mm}$
	PASS - Maximum deflection does not exceed deflection limit

	Project	24 HEAT	H DRIVE		Job no. 162637	
Form Structural Design 77 St John Street	Calcs for BEAM G-24				Start page no./Revision 1	
EC1M 4NN	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date

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



Applied loading

Beam loads

Load combinations Load combination 1

Rotationally free

Permanent self weight of beam $\times 1$ Permanent full UDL 8.6 kN/m Imposed full UDL 1.5 kN/m

Support A

Support B

Permanent × 1.35 Imposed \times 1.50 $\text{Permanent} \times 1.35$ Imposed \times 1.50 $\text{Permanent} \times 1.35$ Imposed \times 1.50

🐙 Tekla	Project	24 HEA	Job no. 162637			
Iedds Form Structural Design	Calas far	21112/			Start page pg //	Devision
77 St John Street	Calcs for	BEA	M G-24		Start page no./i	2
London EC1M 4NN	Calcs by HD	Calcs date 13/02/2019	Checked by ROM	Checked date	Approved by	Approved date
Analysis results						
Maximum moment		M _{max} = 8.6	kNm	M _{min} =	0 kNm	
Maximum shear		V _{max} = 15.7	' kN	V _{min} = ·	-15.7 kN	
Deflection		δ _{max} = 0.9 r	nm	$\delta_{\min} = 0$) mm	
Maximum reaction at support A	4	RA may = 15		RA min =	= 15.7 kN	
Unfactored permanent load rea	action at support	A RA Permanent	= 9.8 kN			
Unfactored imposed load reac	tion at support A	RA Imposed =	1.7 kN			
Maximum reaction at support I	3	R _{B max} = 15	5.7 kN	R _{B min} =	= 15.7 kN	
Unfactored permanent load rea	action at support	B RB Permanent	= 9.8 kN	5		
Unfactored imposed load reac	tion at support B	R _{B Imposed} =	1.7 kN			
Section details						
Section type		UC 152v1	2v20 (BSA 1)			
Steel grade		00 152X 13	52X30 (B34-1)			
EN 10025-2:2004 - Hot rolled	products of str	uctural stools				
Nominal thickness of element		$t = max(t_c)$) = 9 4 mm			
Nominal vield strength		$f_{\rm v} = 355 {\rm N}/{\rm s}$	mm^2			
Nominal ultimate tensile strend	ıth	$f_{\rm u} = 470 \text{N/mm}^2$				
Modulus of elasticity	jui	F = 21000	$E = 210000 \text{ N/mm}^2$			
inequiae of elacienty	Ţ					
			-6.5			
	' ∢			→		
Partial factors - Section 6.1 Resistance of cross-sections Resistance of members to inst Resistance of tensile members	ability s to fracture	γ _{M0} = 1.00 γ _{M1} = 1.00 γ _{M2} = 1.10				
Lateral restraint						
		Span 1 has	s lateral restrai	nt at supports onl	у	
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 torsid	on	K _{LT.A} = 1.0 0	00			
-		K _{LT.B} = 1.0	00			

🐙 Tekla	Project				Job no.	
Tedds		24 HEA	TH DRIVE		16	2637
Form Structural Design	Calcs for	REAL	A G-24		Start page no./R	Revision
London	Calaa hy		Chacked by	Chasked data	Approved by	Approved date
EC1M 4NN	HD	13/02/2019	ROM	Checked date	Approved by	Approved date
Classification of areas asati	one Section E	5				
	ons - Section 5.	ε = √[235 Ν	J/mm ² / f _y] = 0.8 °	1		
Internal compression parts	subiect to bendi	ng - Table 5.2 (s	sheet 1 of 3)			
Width of section	···· , ·····	c = d = 123	3.6 mm			
		c / t _w = 23.4	4 × ε <= 72 × ε	Class 1		
Outstand flanges - Table 5.2	(sheet 2 of 3)					
Width of section	(0.000 - 0.0)	c = (b - t _w -	2 × r) / 2 = 65.6	mm		
		c / t _f = 8.6 :	×ε <= 9 ×ε	Class 1		
				_	Sect	tion is class 1
Check shear - Section 6.2.6						
Height of web		h = h - 2 >	c t∉ = 138 .8 mm			
Shear area factor		n = 1.000				
		h _w / t _w < 72	x ɛ / 'n			
				Shear buckling	resistance ca	an be ianored
Design shear force		V _{Ed} = max(abs(V _{max}), abs(\	/ _{min})) = 15.7 kN		J
Shear area - cl 6.2.6(3)		A _v = max(A	$x - 2 \times b \times t_f + (t_w)$	$v + 2 \times r) \times t_{f}, \eta \times t_{f}$	h _w × t _w) = 115	6 mm ²
Design shear resistance - cl 6.	2.6(2)	$V_{c,Rd} = V_{pl,R}$	$A_{d} = A_v \times (f_y / \sqrt{3})$]) / γ _{M0} = 236.9 kN	1	
		PAS	SS - Design she	ear resistance ex	xceeds desig	n shear force
Check bending moment maj	or (y-y) axis - Se	ection 6.2.5				
Design bending moment		M _{Ed} = max	(abs(M _{s1_max}), ab	os(M _{s1_min})) = 8.6 I	kNm	
Design bending resistance mo	ment - eq 6.13	$M_{c,Rd} = M_{pl,}$	$_{\rm Rd}$ = W _{pl.y} × f _y / $\gamma_{\rm N}$	_{M0} = 87.9 kNm		
Slenderness ratio for lateral	torsional buckli	ing				
Correction factor - Table 6.6		k _c = 0.94				
		$C_1 = 1 / k_c^2$	= 1.132			
Curvature factor		g = √[1 - (I₂	z / l _y)] = 0.824			
Poissons ratio		v = 0.3				
Shear modulus		G = E / [2 >	< (1 + v)] = 8076	39 N/mm²		
Unrestrained length		L = 1.0 × L	_{s1} = 2200 mm			
Elastic critical buckling momer	nt	$M_{cr} = C_1 \times c_1$	$\pi^2 \times E \times I_z / (L^2 \times$	$(g) \times \sqrt{[I_w / I_z + L^2]}$	\times G \times I _t / (π^2 >	$(E \times I_z)$] =
		313.2 kNm				
Slenderness ratio for lateral to	rsional buckling	$\lambda_{LT} = \sqrt{W}$	$_{\text{pl.y}} \times f_{\text{y}} / M_{\text{cr}}) = 0$.53		
Limiting slenderness ratio		λ _{LT,0} = 0.4				
			$\overline{\lambda}_{LT} > \overline{\lambda}_{LT,0} - Lat$	teral torsional b	uckling cann	ot be ignored
Design resistance for buckli	ng - Section 6.3	.2.1				
Buckling curve - Table 6.5		b				
Imperfection factor - Table 6.3		α _{LT} = 0.34				
Correction factor for rolled sec	tions	β = 0.75	_			
LTB reduction determination fa	actor	$\phi_{LT} = 0.5 \times$	$[1 + \alpha_{LT} \times (\lambda_{LT})]$	- $\lambda_{LT,0}$) + $\beta \times \lambda_{LT}^2$	[[]] = 0.627	
LTB reduction factor - eq 6.57		χ _{∟⊤} = min(1	/ [ϕ_{LT} + √(ϕ_{LT}^2 -	$\beta \times \lambda_{LT^2}$], 1, 1 / 2	λ _{LT} ²) = 0.948	
Modification factor		f = min(1 -	0.5 × (1 - k _c)× [1	$-2 \times (\lambda_{LT} - 0.8)^{-1}$	²], 1) = 0.974	
Modified LTB reduction factor	- eq 6.58	χ _{LT,mod} = mi	n(χ _{LT} / f, 1) = 0.	973		
Design buckling resistance mo	oment - eq 6.55	$M_{b,Rd} = \chi_{LT,i}$	$_{ m mod} imes W_{ m pl.y} imes f_{ m y}$ / γ	_{/M1} = 85.5 kNm		
	PASS	- Design buckli	ing resistance	moment exceed	s design ben	ding moment

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Check vertical	deflection - Section 7.2.1
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Consider deflection due to permanent and imposed loads

Limiting deflection

Maximum deflection span 1

 $\delta_{\text{lim}} = \min(10 \text{ mm}, L_{s1} / 250) = 8.8 \text{ mm}$

 $\delta = \max(abs(\delta_{max}), abs(\delta_{min})) = 0.864 \text{ mm}$

	Project	24 HEAT	H DRIVE		Job no. 162637	
Form Structural Design 77 St John Street	Calcs for BEAM G-25				Start page no./Revision 1	
EC1M 4NN	Calcs by HD	Calcs date 13/03/2019	Checked by ROM	Checked date	Approved by	Approved date

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



Load combination 1

Support B

Imposed \times 1.50 $\text{Permanent} \times 1.35$ Imposed \times 1.50 $\text{Permanent} \times 1.35$

Imposed \times 1.50

🐙 Tekla	Project	24 HEA		Job no. 162637		
Tedds		24 1127				2037
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London EC1M 4NN	Calcs by HD	Calcs date 13/03/2019	Checked by ROM	Checked date	Approved by	Approved date
Analysis results						
Maximum moment		M _{max} = 25.8	3 kNm	M _{min} =	0 kNm	
Maximum shear		V _{max} = 34.4	k N	$V_{min} = 0$	-34.4 kN	
Deflection		δ _{max} = 1.8 r	nm	δ _{min} = () mm	
Maximum reaction at support A		R _{A_max} = 34	. 4 kN	R _{A_min} :	= 34.4 kN	
Unfactored permanent load read	ction at support A	A RA_Permanent	= 20.5 kN			
Unfactored imposed load reaction	on at support A	R _{A_Imposed} =	4.5 kN			
Maximum reaction at support B		R _{B_max} = 34	. 4 kN	R _{B_min} :	= 34.4 kN	
Unfactored permanent load read	ction at support E	B RB_Permanent	= 20.5 kN			
Unfactored imposed load reaction	on at support B	R _{B_Imposed} =	4.5 kN			
Section details						
Section type		UC 203x20)3x46 (BS4-1)			
Steel grade		S355				
EN 10025-2:2004 - Hot rolled	products of stru	ctural steels				
Nominal thickness of element		t = max(t _f , t	t _w) = 11.0 mm			
Nominal yield strength		f _y = 355 N/	mm²			
Nominal ultimate tensile strengt	h	f _u = 470 N/mm ²				
Modulus of elasticity		E = 21000	0 N/mm²			
-	<u>↓</u> <u></u>					
			-7.2			
-	 ↓ ↓	203.6				
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	s lateral restrair	nt at supports on	у	
Effective length factors						
Effective length factor in major a	axis	K _y = 1.000				
Effective length factor in minor a	axis	K _z = 1.000				
Effective length factor for torsion	n	K _{LT.A} = 1.00	00			
		K _{LT.B} = 1.00	00			

🐙 Tekla	Project	Project 24 HEATH DRIVE				Job no. 162637	
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London EC1M 4NN	Calcs by HD	Calcs date 13/03/2019	Checked by ROM	Checked date	Approved by	Approved date	
Classification of cross secti	ons - Section 5.	5					
		ε = √[235 Ν	I/mm² / f _y] = 0.81				
Internal compression parts	subject to bendi	ng - Table 5.2 (s	sheet 1 of 3)				
Width of section		c = d = 160	0.8 mm				
		$c / t_w = 27.4$	4 × ε <= 72 × ε	Class 1			
Outstand flanges - Table 5.2	(sheet 2 of 3)						
Width of section		c = (b - t _w -	2 × r) / 2 = 88 m	ım			
		c / t _f = 9.8 >	< ε <= 10 × ε	Class 2			
					Sec	tion is class 2	
Check shear - Section 6.2.6							
Height of web		h _w = h - 2 ×	: t _f = 181.2 mm				
Shear area factor		η = 1.000					
		h _w / t _w < 72	×ε/η				
				Shear buckling	resistance c	an be ignored	
Design shear force		V _{Ed} = max(abs(V _{max}), abs(V	(_{min})) = 34.4 kN		•	
Shear area - cl 6.2.6(3)	(-)	$A_v = \max(A)$	$A_{v} = \max(A - 2 \times b \times t_{f} + (t_{w} + 2 \times r) \times t_{f}, \eta \times h_{w} \times t_{w}) = 1698 \text{ mm}^{2}$				
Design shear resistance - cl 6	.2.6(2)	$V_{c,Rd} = V_{pl,R}$	$A_{d} = A_{v} \times (f_{y} / \sqrt{3})$) / γ _{M0} = 347.9 kN			
		PAS	ss - Design sne	ar resistance ex	ceeas aesig	n snear force	
Check bending moment maj	or (y-y) axis - Se	ection 6.2.5		- (14)) 05 0	L.N.L.		
Design bending moment		$M_{Ed} = max($	abs(M _{s1_max}), ab	$S(M_{s1}_{min})) = 25.8$	KNM		
Design bending resistance mo	oment - eq 6.13	$M_{c,Rd} = M_{pl,l}$	$_{\rm Rd} = VV_{\rm pl.y} \times T_y / \gamma_N$	₁₀ = 176.6 KNM			
Slenderness ratio for lateral	torsional buckli	ng					
Correction factor - Table 6.6		$K_c = 0.94$	- 4 422				
Curvature factor		$C_1 = 1 / K_c^2$	= 1.132				
		g = v[1 - (i₂	(iy)] – 0.013				
Shear modulus		V = 0.3	((1 + y)) = 8076	9 N/mm ²			
		G = L/[Z/	(1, 0) = 3000 mm	3 N/11111			
Elastic critical buckling momen	at	$L = 1.0 \times L_{1}$	$\pi^2 \times E \times L / (L^2 \times E)$	a) $\times \sqrt{1} + 1^2$	$\times G \times L / (\pi^2)$		
	it.	592 5 kNm	it × L × Iz / (L ×	9) × ([iw / iz ' L	× 0 × It / (//	× L × Iz/J =	
Slenderness ratio for lateral to	rsional buckling	$\overline{\lambda}_{1T} = \sqrt{W_1}$	$f_{\rm M} \times f_{\rm V} / M_{\rm er}$ = 0.	546			
Limiting slenderness ratio		$\overline{\lambda}_{1T0} = 0.4$	51.y ** 1y , 10.01) •				
			$\overline{\lambda}_{1,T} > \overline{\lambda}_{1,T,0} - Lat$	eral torsional bi	ucklina cann	ot be ianored	
Decian registeres for buckli	ng Section 6 2	2.4					
Buckling curve - Table 6.5	ing - Section 6.5.	. 2. 1					
Imperfection factor - Table 6.3	4	αιτ = 0.34					
Correction factor for rolled sec	tions	β = 0.75					
I TB reduction determination fa	actor	φιτ = 0.5 ×	$[1 + \alpha_{LT} \times (\overline{\lambda}_{LT} + \alpha_{LT} + \alpha_{LT} + \alpha_{LT} \times (\overline{\lambda}_{LT} + \alpha_{LT} + \alpha_{LT} + \alpha_{LT} \times (\overline{\lambda}_{LT} + \alpha$	$\overline{\lambda}_{1,T,0}$ + $\beta \times \overline{\lambda}_{1,T^2}$] = 0.637		
LTB reduction factor - eq 6.57		$\gamma_{1T} = \min(1)$	/ [φι τ + √(φι τ ² - [$3 \times \overline{\lambda}_{1T}^{2}$]. 1. 1 / 2	λ _{ι τ} ²) = 0.941		
Modification factor		f = min(1 - 1)	$0.5 \times (1 - k_c) \times [1]$	$-2 \times (\overline{\lambda}_{1T} - 0.8)^2$	²]. 1) = 0.974		
Modified LTB reduction factor	- eg 6.58	γ _{IT mod} = mi	n(γ _{LT} / f. 1) = 0 .9	(112) 010)	.,		
Design buckling resistance mo	oment - ea 6.55	$M_{\rm b,Rd} = \gamma_{\rm IT}$	$_{\rm mod} \times W_{\rm pl.v} \times f_{\rm v} / v$	_{M1} = 170.6 kNm			
	PASS	- Design buckli	ing resistance r	noment exceed	s design ben	ding moment	
		J			0		

	Project 24 HEATH DRIVE				Job no. 162637	
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Check vertical deflection - Section 7.2.1	
Consider deflection due to permanent and imposed loads	

Limiting deflection

Maximum deflection span 1

 $\delta_{\text{lim}} = \min(10 \text{ mm}, L_{s1} / 250) = 10 \text{ mm}$

 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) = \textbf{1.831} mm$

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Vertically restrained Rotationally free

Support conditions Support A

Support B

Applied loading

Beam loads

Load combinations

Load combination 1

Vertically restrained Rotationally free Permanent self weight of beam × 1

G-7 - Permanent point load 21 kN at 2800 mm CHIMNEY BREAST - Permanent partial UDL 50.22 kN/m from 1800 mm to 3600 mm

Support A Permanent × 1.35 Variable × 1.50 Permanent × 1.35 Variable × 1.50 Support B Permanent × 1.35

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		Variable \times 1.50
Analysis results		
Maximum moment	M _{max} = 188.3 kNm	M _{min} = 0 kNm
Maximum moment span 1 segment 1	M _{s1_seg1_max} = 188.3 kNm	M _{s1_seg1_min} = 0 kNm
Maximum moment span 1 segment 2	M _{s1_seg2_max} = 188.3 kNm	M _{s1_seg2_min} = 0 kNm
Maximum shear	V _{max} = 81.3 kN	V _{min} = -77 kN
Maximum shear span 1 segment 1	V _{s1_seg1_max} = 81.3 kN	V _{s1_seg1_min} = -18.8 kN
Maximum shear span 1 segment 2	$V_{s1_seg2_max} = 0 \text{ kN}$	V _{s1_seg2_min} = -77 kN
Deflection segment 3	δ _{max} = 11 mm	δ _{min} = 0 mm
Maximum reaction at support A	R _{A_max} = 81.3 kN	R _{A_min} = 81.3 kN
Unfactored permanent load reaction at support A	R _{A_Permanent} = 60.3 kN	
Maximum reaction at support B	R _{B_max} = 77 kN	R _{B_min} = 77 kN
Unfactored permanent load reaction at support B	R _{B_Permanent} = 57 kN	
Section details		
Section type	UC 254x254x107 (BS4-1)	
Steel grade	S355	
EN 10025-2:2004 - Hot rolled products of struct	ural steels	
Nominal thickness of element	t = max(t _f , t _w) = 20.5 mm	
Nominal yield strength	f _y = 345 N/mm ²	
Nominal ultimate tensile strength	f _u = 470 N/mm ²	
Modulus of elasticity	E = 210000 N/mm ²	
20.5		
★ ★		
→		
6.7 —		
- 26		
50 .5		
<u>↓</u>		
* *		
 4	258.8	
Partial factors - Section 6.1		
Resistance of cross-sections	γ _{M0} = 1.00	
Resistance of members to instability	γ _{M1} = 1.00	

γ_{M2} = **1.10**

K_y = 1.000 K_z = 1.000

K_{LT.A} = **1.000**

Lateral restraint

Span 1 has lateral restraint at supports plus midspan

Effective length factors				
Effective length factor in major axis				

Effective length factor in minor axis Effective length factor for torsion

Resistance of tensile members to fracture

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EC1M 4NN	Calcs by ROM	Calcs date 06/02/2019	Checked by	Checked date	Approved by	Approved date
		- 1 00	0			
		$K_{LT.B} = 1.00$	0			
Classification of cross section	ns - Section 5.5					
		ε = √[235 N	/mm ² / f _y] = 0.83			
Internal compression parts su	ubject to bendir	ng - Table 5.2 (s	heet 1 of 3)			
Width of section		c = d = 200	.3 mm			
		c / t _w = 19.0	×ε <= 72 ×ε	Class 1		
Outstand flanges - Table 5.2 (sheet 2 of 3)					
Width of section		c = (b - t _w -	2 × r) / 2 = 110.3	3 mm		
		c / t _f = 6.5 ×	ε <= 9 × ε	Class 1		
					Secti	ion is class 1
Check shear - Section 6.2.6						
Height of web		h = h - 2 ×	t₌ 225 7 mm			
Shear area factor		n = 1.000				
		h = 1.000	vsln			
		11. 1	, i i s	Shear buckling	resistance ca	n he ianored
Design shear force		$V_{Ed} = max(a)$	abs(V _{max}) abs(V _r	min()) = 81.3 kN	esistance ca	n be ignored
Shear area - $cl 6 2 6(3)$		$A_{\rm v} = \max(A)$	$-2 \times b \times t_f + (t_w)$	+ 2 × r) × t∉ n ×	h × t) = 381 1	l mm²
Design shear resistance - cl 6 2	6(2)		$= \Delta_{u} \times (f_{u} / \sqrt{3})$	$/_{2M0} = 759 \text{ kN}$		
	.0(2)	ν c, κα – ν ρι, κα PAS	S - Desian she	ar resistance ex	ceeds desia	n shear force
Check bending moment at sp	an 1 soamont 1	maior (v-v) avi	s - Section 6 2 /	5	J	
Design bending moment	an i segment i	M _{rd} = max($abs(M_{o1}, agg1, max)$	abs(Mai and min)	i) = 188 3 kNm	1
Design bending resistance mor	nent - eg 6 13	$M_{e,Rd} = M_{pl,R}$	$d = W_{\text{ply}} \times f_{y} / \gamma_{\text{Max}}$	a = 512.1 kNm		
Correction factor Table 6.6	orsional ducklir	1g k = 0.91				
		$C_1 = 1 / k_0^2$	= 1 525			
Curvature factor		a = √[1 - (l-	/] = 0 813			
Poissons ratio		y = 03				
Shear modulus		$G = E / [2] \times$	(1 + y) = 80769	N/mm^2		
		U = 10 × 1	(1 + v) = 2800 mm			
Elastic critical buckling moment		$L = 1.0 \times L_s$	$r_2^2 \times \mathbf{E} \times \mathbf{I} / (\mathbf{I}^2 \times \mathbf{E})$	$a > \sqrt{1} + \frac{1}{2}$	$\sim C \times L/(\pi^2)$	
Elastic childar buckling moment		4556 4 kNn		9) × v[iw / iz + L	× G × It / (n ×	∟×ız/] −
Slenderness ratio for lateral tors	ional buckling	$\overline{\lambda}_{1,\tau} = \sqrt{0}$	' √ f / M) = 0 3	335		
Limiting slondornoss ratio	sonal buckling	$\overline{\lambda}_{L1} = \sqrt{\sqrt{v_p}}$	$1.y \times 1y / 101cr) = 0.2$	55		
Limiting sienderness ratio		λ _{LT,0} – 0.4	$\overline{2}$ $\overline{2}$	l ataral tarajana	huakling og	n ha ianarad
	DASS	Docian bondi	$\lambda LT < \lambda LT, 0 - I$	Lateral torsiona	n Ducking ca	ding moment
	FA33	- Design benan	ng resistance n	ioment exceed	s design bend	
Check vertical deflection - Se	ction 7.2.1					
Consider deflection due to perm	ianent and varial		F0 - 20 4			
		$\delta_{\text{lim}} = L_{s1} / 2$	50 = 22.4 mm			
Maximum deflection span 1		δ = max(ab	$s(\delta_{max}), abs(\delta_{min})$) = 11.008 mm		
	PASS - Maximum deflection does not exceed deflection limit					