

PROJECT: Park House - IOS

REF: 07020E

DATE: September 2025

CALCULATIONS

CALCULATIONS	PROJECT: Park House - IOS	PROJECT No: 07020E
		DATE: Sept 25
		CALCULATIONS BY: MDH

STRUCTURAL SUMMARY

The project consist of some remodelling of the existing care home into residential flats, the building is of traditional masonry build over two storey's with proprietary roof trusses over. The first floor is of beam and block with the ground floor being an assumed ground bearing slab.

None of the works are considered to be abnormal or outside of the usual scope of a suitably qualified contractor.

Before walls are removed then opening up at each junction to confirm assumptions is required and any anomalies reported.

STRUCTURAL STABILITY

Not affected by proposals

ASSUMPTIONS

A competent contractor will be engaged

DESIGN REFERENCES

Eurocode 1: Actions on structures (EN 1991)
Eurocode 2: Design of concrete structures (EN 1992)
Eurocode 3: Design of steel structures (EN 1993)
Eurocode 5: Design of timber structures (EN 1995)
Eurocode 6: Design of masonry structures (EN 1996)

Calculation

Contract

Park House - 105

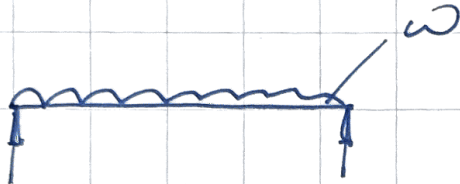
Sheet

1

By

MDH

Designing L.O.1



2200

$$\begin{array}{rcl}
 w: & 5.5 \times \frac{5.0}{2} & 14.0 \\
 & 3.0 & 7.5 \\
 & 20 \times 0.1 \times 1.0 & 2.0 \\
 & & \hline
 & & 16.0 \quad 7.5
 \end{array}$$



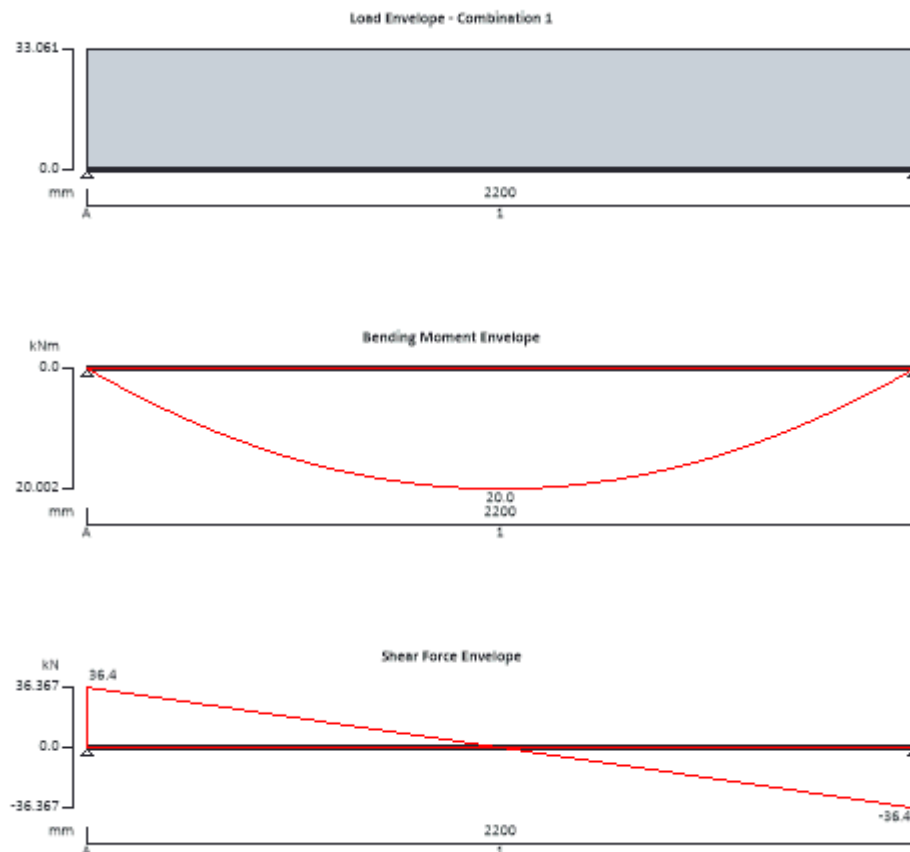
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Project				Job no.	
Park House - IOS				07020E	
Calcs for				Start page no./Revision	
L0.1				2	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MDH	10/09/2025				

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

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

TEDDS calculation version 3.0.16



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

Permanent full UDL 16 kN/m

Variable full UDL 7.5 kN/m

Load combinations

Load combination 1

Support A

Permanent $\times 1.35$

Variable $\times 1.50$

Permanent $\times 1.35$

Variable $\times 1.50$

Support B

Permanent $\times 1.35$

Variable $\times 1.50$



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Project				Job no.	
Park House - IOS				07020E	
Calcs for				Start page no./Revision	
L0.1				3	
Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
MDH	10/09/2025				

Analysis results

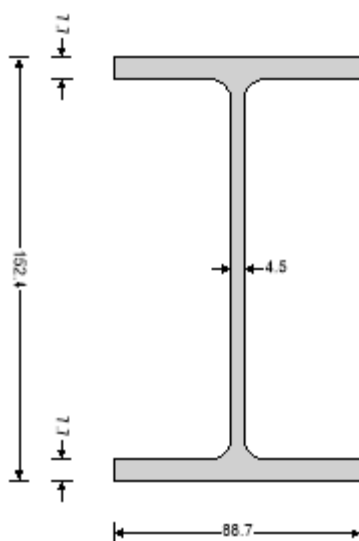
Maximum moment	$M_{max} = 20 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 36.4 \text{ kN}$	$V_{min} = -36.4 \text{ kN}$
Deflection	$\delta_{max} = 4.1 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 36.4 \text{ kN}$	$R_{A_min} = 36.4 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 17.8 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 8.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 36.4 \text{ kN}$	$R_{B_min} = 36.4 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 17.8 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 8.2 \text{ kN}$	

Section details

Section type	UKB 152x89x16 (Tata Steel Advance)
Steel grade	S275

EN 10025-2:2004 - Hot rolled products of structural steels

Nominal thickness of element	$t = \max(t_f, t_w) = 7.7 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$



Partial factors - Section 6.1


Resistance of cross-sections	$\gamma_{M0} = 1.00$
Resistance of members to instability	$\gamma_{M1} = 1.00$
Resistance of tensile members to fracture	$\gamma_{M2} = 1.10$

Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

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

 structure Haus Ltd	Project Park House - IOS				Job no. 07020E	
	Calcs for L0.1				Start page no./Revision 4	
	Calcs by MDH	Calcs date 10/09/2025	Checked by	Checked date	Approved by	Approved date

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = \mathbf{0.92}$$

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

Width of section

$$c = d = \mathbf{121.8 \text{ mm}}$$

$$c / t_w = 29.3 \times \varepsilon \leq 72 \times \varepsilon \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section

$$c = (b - t_w - 2 \times r) / 2 = \mathbf{34.5 \text{ mm}}$$

$$c / t_f = 4.8 \times \varepsilon \leq 9 \times \varepsilon \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web

$$h_w = h - 2 \times t_f = \mathbf{137 \text{ mm}}$$

Shear area factor

$$\eta = \mathbf{1.000}$$

$$h_w / t_w < 72 \times \varepsilon / \eta$$

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{36.4 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{818 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{129.8 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{20 \text{ kNm}}$$

Design bending resistance moment - eq 6.13

$$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = \mathbf{33.9 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{0.94}$$

$$C_1 = 1 / k_c^2 = \mathbf{1.132}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.945}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

$$G = E / [2 \times (1 + \nu)] = \mathbf{80769 \text{ N/mm}^2}$$

Unrestrained length

$$L = 1.0 \times L_{s1} = \mathbf{2200 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{51.9 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = \mathbf{0.808}$$

Limiting slenderness ratio

$$\bar{\lambda}_{LT,0} = \mathbf{0.4}$$

$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$ - Lateral torsional buckling cannot be ignored

Design resistance for buckling - Section 6.3.2.1

Buckling curve - Table 6.5

$$b$$

Imperfection factor - Table 6.3

$$\alpha_{LT} = \mathbf{0.34}$$

Correction factor for rolled sections

$$\beta = \mathbf{0.75}$$

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{0.814}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.813}$$

Modification factor

$$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = \mathbf{0.970}$$


Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.838}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{28.4 \text{ kNm}}$$

PASS - Design buckling resistance moment exceeds design bending moment

 structure Haus Ltd	Project				Job no.	
	Park House - IOS				07020E	
	Calcs for				Start page no./Revision	
	L0.1				5	
	Calcs by	Calcs date	Checked by	Checked date	Approved by	Approved date
	MDH	10/09/2025				

Check vertical deflection - Section 7.2.1

Consider deflection due to permanent and variable loads


Limiting deflection

$$\delta_{lim} = L_{s1} / 360 = \mathbf{6.1 \text{ mm}}$$

Maximum deflection span 1

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{4.119 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

 structure Haus Ltd	Project Park House - IOS				Job no. 07020E	
	Calcs for Lift Infill				Start page no./Revision 6	
	Calcs by MDH	Calcs date 10/09/2025	Checked by	Checked date	Approved by	Approved date

TIMBER JOIST ANALYSIS & DESIGN (EN1995-1-1:2004)

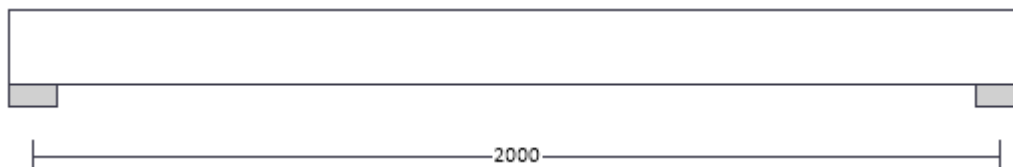
In accordance with EN1995-1-1:2004 + A2:2014 incorporating corrigendum June 2006 and the UK national annex

Tedds calculation version 1.0.09

Joist details

Description 47 x 150 C16 timber joists

Joist spacing $S_{Joist} = 300$ mm



Forces input on Joist

Vertical permanent load on joist $F_{G_Joist} = 1.00$ kN/m²

Vertical imposed load on joist $F_{Q_Joist} = 3.00$ kN/m²

Joist loading details

Distributed loads

Vertical permanent load on joist $p_G = F_{G_Joist} \times S_{Joist} = 0.30$ kN/m

Vertical imposed load on joist $p_Q = F_{Q_Joist} \times S_{Joist} = 0.90$ kN/m

Member results summary	Unit	Capacity	Maximum	Utilisation	Result
Bearing stress	N/mm ²	1.489	0.292	0.196	PASS
Bending stress	N/mm ²	10.831	5.061	0.467	PASS
Shear stress	N/mm ²	2.166	0.567	0.262	PASS
Beam stability check				0.467	PASS
Deflection	mm	8	4.706	0.588	PASS

ANALYSIS

Tedds calculation version 1.0.38

Loading


Self weight included (Permanent x 1)

Load combination factors

Load combination	Permanent	Imposed	Snow	Wind
1.35G + 1.50Q (Strength)	1.35	1.50	0.00	0.00
1.00G + 1.00Q (Service)	1.00	1.00	0.00	0.00

Member Loads

Member	Load case	Load Type	Orientation	Description
Member	Permanent	UDL	GlobalZ	0.3 kN/m at 0 m to 2 m
Member	Imposed	UDL	GlobalZ	0.9 kN/m at 0 m to 2 m

 structure Haus Ltd	Project Park House - IOS				Job no. 07020E	
	Calcs for Lift Infill				Start page no./Revision 7	
	Calcs by MDH	Calcs date 10/09/2025	Checked by	Checked date	Approved by	Approved date

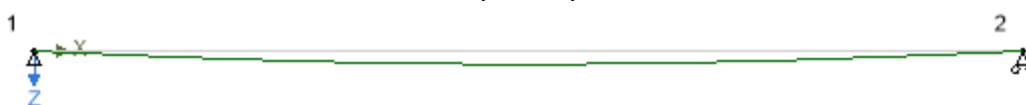
Results

Total deflection

1.35G + 1.50Q (Strength) - Total deflection



1.00G + 1.00Q (Service) - Total deflection



Node deflections

Load combination: 1.35G + 1.50Q (Strength)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.32218	
2	0	0	-0.32218	

Load combination: 1.00G + 1.00Q (Service)

Node	Deflection		Rotation (°)	Co-ordinate system
	X (mm)	Z (mm)		
1	0	0	0.22059	
2	0	0	-0.22059	

Total base reactions

Load case/combination	Force	
	FX (kN)	FZ (kN)
1.35G + 1.50Q (Strength)	0	3.6
1.00G + 1.00Q (Service)	0	2.4

Element end forces

Load combination: 1.35G + 1.50Q (Strength)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2	1	0	-1.8	0
		2	0	-1.8	0

Load combination: 1.00G + 1.00Q (Service)

Element	Length (m)	Nodes Start/End	Axial force (kN)	Shear force (kN)	Moment (kNm)
1	2	1	0	-1.2	0
		2	0	-1.2	0



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Project

Park House - IOS

Job no.

07020E

Calcs for

Lift Infill

Start page no./Revision

8

Calcs by

MDH

Calcs date

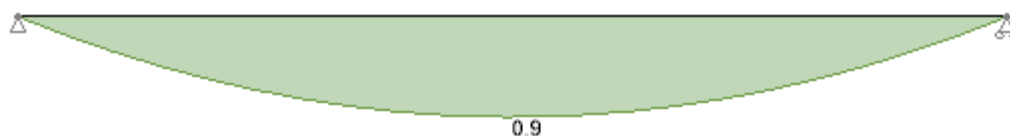
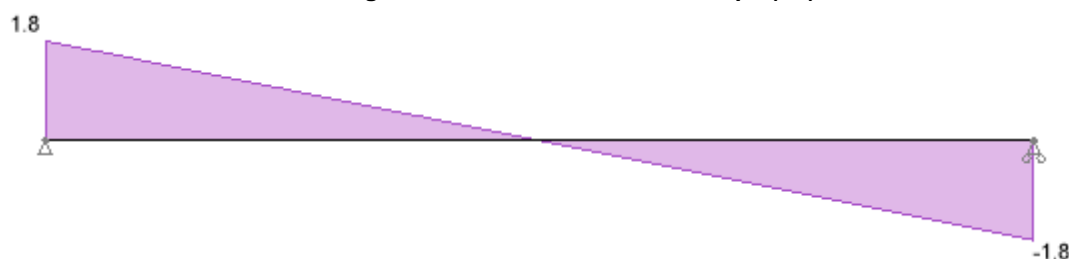
10/09/2025

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Forces**Strength combinations - Moment envelope (kNm)****Strength combinations - Shear envelope (kN)****Member results****Envelope - Strength combinations**

Member	Position (m)	Shear force (kN)		Moment (kNm)	
Member	0	1.8 (max abs)		0 (min)	
	1	0		0.9 (max)	
	2	-1.8		0 (min)	

Tedds calculation version 2.2.25

Member - Span 1**Partial factor for material properties and resistances**Partial factor for material properties - Table 2.3 $\gamma_M = 1.300$ **Member details**

Load duration - cl.2.3.1.2

Medium-term

Service class - cl.2.3.1.3

2

Timber section details

Number of timber sections in member

N = 1

Breadth of sections


b = 47 mm

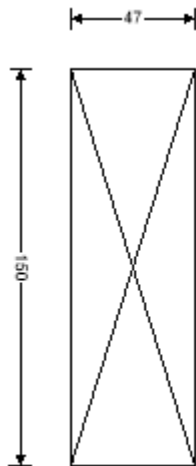
Depth of sections

h = 150 mm

Timber strength class - EN 338:2016 Table 1

C16

 structure Haus Ltd	Project Park House - IOS				Job no. 07020E	
	Calcs for Lift Infill				Start page no./Revision 9	
	Calcs by MDH	Calcs date 10/09/2025	Checked by	Checked date	Approved by	Approved date



47x150 timber section

Cross-sectional area, A , 7050 mm²

Section modulus, $W_{y,z}$, 176250 mm³

Section modulus, $W_{x,y}$, 55225 mm³

Second moment of area, $I_{y,z}$, 13218750 mm⁴

Second moment of area, $I_{x,y}$, 1297787 mm⁴

Radius of gyration, i_y , 43.3 mm

Radius of gyration, i_x , 13.6 mm

Timber strength class C16

Characteristic bending strength, $f_{t,k}$, 16 N/mm²

Characteristic shear strength, $f_{v,k}$, 3.2 N/mm²

Characteristic compression strength parallel to grain, $f_{c,0,k}$, 17 N/mm²

Characteristic compression strength perpendicular to grain, $f_{c,90,k}$, 2.2 N/mm²

Characteristic tension strength parallel to grain, $f_{t,k}$, 8.5 N/mm²

Mean modulus of elasticity, E_{mean} , 8000 N/mm²

Fifth percentile modulus of elasticity, $E_{0.05}$, 6400 N/mm²

Shear modulus of elasticity, G_{mean} , 500 N/mm²

Characteristic density, ρ_k , 310 kg/m³

Mean density, ρ_{mean} , 370 kg/m³

Span details

Bearing length

$L_b = 100$ mm

Consider Combination 1 - 1.35G + 1.50Q (Strength)

Modification factors

Duration of load and moisture content - Table 3.1 $k_{mod} = 0.8$

Deformation factor - Table 3.2 $k_{def} = 0.8$

Bending stress re-distribution factor - cl.6.1.6(2) $k_m = 0.7$

Crack factor for shear resistance - cl.6.1.7(2) $k_{cr} = 0.67$

System strength factor - cl.6.6 $k_{sys} = 1.1$

Check design at start of span

Check compression perpendicular to the grain - cl.6.1.5

Design perpendicular compression - major axis $F_{c,y,90,d} = 1.784$ kN

Effective contact length $L_{b,ef} = L_b + \min(L_b, 30 \text{ mm}) = 130$ mm

Design perpendicular compressive stress - exp.6.4 $\sigma_{c,y,90,d} = F_{c,y,90,d} / (b \times L_{b,ef}) = 0.292$ N/mm²

Design perpendicular compressive strength $f_{c,y,90,d} = k_{mod} \times k_{sys} \times f_{c,90,k} / \gamma_M = 1.489$ N/mm²

$\sigma_{c,y,90,d} / (k_{c,90} \times f_{c,y,90,d}) = 0.196$

PASS - Design perpendicular compression strength exceeds design perpendicular compression stress

Check shear force - Section 6.1.7

Design shear force $F_{y,d} = 1.784$ kN

Design shear stress - exp.6.60 $\tau_{y,d} = 1.5 \times F_{y,d} / (k_{cr} \times b \times h) = 0.567$ N/mm²

Design shear strength $f_{v,y,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = 2.166$ N/mm²

$\tau_{y,d} / f_{v,y,d} = 0.262$

PASS - Design shear strength exceeds design shear stress

Check design 1000 mm along span


Check bending moment - Section 6.1.6

Design bending moment $M_{y,d} = 0.892$ kNm

Design bending stress $\sigma_{m,y,d} = M_{y,d} / W_y = 5.061$ N/mm²

Design bending strength $f_{m,y,d} = k_{mod} \times k_{sys} \times f_{m,k} / \gamma_M = 10.831$ N/mm²

$\sigma_{m,y,d} / f_{m,y,d} = 0.467$

 structure Haus Ltd	Project Park House - IOS				Job no. 07020E	
	Calcs for Lift Infill				Start page no./Revision 10	
	Calcs by MDH	Calcs date 10/09/2025	Checked by	Checked date	Approved by	Approved date

PASS - Design bending strength exceeds design bending stress

Check beams subjected to either bending or combined bending and compression - cl.6.3.3

Lateral buckling factor - exp.6.34

$k_{crit} = 1.000$

Beam stability check - exp.6.33

$\sigma_{m,y,d} / (k_{crit} \times f_{m,y,d}) = 0.467$

PASS - Beam stability is acceptable

Consider Combination 2 - 1.00G + 1.00Q (Service)

Check design 1000 mm along span

Check y-y axis deflection - Section 7.2

Instantaneous deflection

$\delta_y = 2.6 \text{ mm}$

Quasi-permanent variable load factor

$\psi_2 = 0.3$

Final deflection with creep

$\delta_{y,Final} = \delta_y \times (1 + k_{def}) = 4.7 \text{ mm}$

Allowable deflection

$\delta_{y,Allowable} = L_{m1,s1} / 250 = 8 \text{ mm}$

$\delta_{y,Final} / \delta_{y,Allowable} = 0.588$

PASS - Allowable deflection exceeds final deflection