15th January 2021
Job No: 8962MCN

Structural Calculations

Internal Alterations,
Marshlands,
Cley Next-the-sea,
Holt, NR25 7RZ.

Rev. B – 02/02/21 – Drawings No. 110 (P4) & 301 (P3) amended (page 49 & 51)
Rev. A – 15/01/21 – Drawing No. 110 amended as revision P3 (page 49)

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Introduction

The following calculations are for proposed internal alterations to an existing two storey timber frame dwelling in Holt, Norfolk.

Structural checks have been carried out for the superstructure of this building.

The existing roof is formed by I-joists (designed by others) and fully sarked. The existing 1st floor is formed by I-joists (designed by suppliers), with 50mm screed on top. The existing external stud walls are 38x140 C16 with timber cladding. Same studs sizes for internal loadbearing walls.

Racking assessment has been carried out to check the stability of the building to approve removal of few internal walls. Some of these walls were loadbearing, so new beams have been designed to support the roof joists. A steel post has been introduced to transfer the roof loads to the existing masonry wall at ground floor. Existing staircase trimmers have been checked for additional loads of a hydraulic glass balustrade; loading figures for this, have been provided by the suppliers.

All calculations should be read in conjunction with the structural drawings.

Stability

Stability for the building is achieved by sheathing of timber frame walls in both directions.

Assumptions

Codes to be used include but not limited to:

BS:6399 : Loading for Buildings

Part 1 : Dead & live loads

Part 2 : Wind loads

BS:5268-2 : Structural use of timber

BS:5950-1 : Structural use of steelwork

Loads

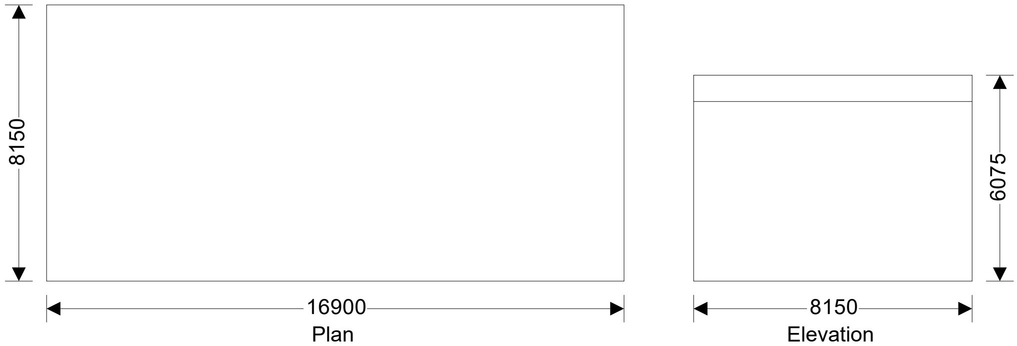
Flat roof	Dead	1.00 kN/m^2
	Live	1.50 kN/m^2
Floor	Dead	1.50 kN/m^2
	Live	1.50 kN/m^2
Stud walls	Dead	0.50 kN/m^2 (+0.75 kN/m^2 timber cladding where applicable)

WIND LOADING (BS6399)

WIND LOADING (BS6399)

In accordance with BS6399

Tedds calculation version 3.0.17



Building data

Type of roof	Flat
Length of building	L = **16900** mm
Width of building	W = **8150** mm
Height to eaves	H = **5300** mm
Eaves type	Parapet
Height of parapet	h_p = **775** mm
Reference height	H_r = **6075** mm

Dynamic classification

Building type factor (Table 1)	K_b = **1.0**
Dynamic augmentation factor (1.6.1)	$C_r = [K_b \times (H_r / (0.1\ m))^{0.75}] / (800 \times \log(H_r / (0.1\ m))) =$ **0.02**

Site wind speed

Location	Holt
Basic wind speed (Figure 6 BS6399:Pt 2)	V_b = **25.0** m/s
Site altitude	Δ_S = **35** m
Upwind distance from sea to site	d_{sea} = **1** km
Direction factor	S_d =**1.00**
Seasonal factor	S_s = **1.00**
Probability factor	S_p = **1.00**
Critical gap between buildings	g = **5000** mm
Topography not significant	
Altitude factor	$S_a = 1 + 0.001 \times \Delta_S / 1m =$ **1.03**
Site wind speed	$V_s = V_b \times S_a \times S_d \times S_s \times S_p =$ **25.9** m/s
Terrain category	Country

Displacement height (sheltering effect excluded)H_d = 0mm

The velocity pressure for the windward face of the building with a 0 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

The velocity pressure for the windward face of the building with a 90 degree wind is to be considered as 1 part as the height h is less than b (cl.2.2.3.2)

Dynamic pressure - windward wall - Wind 0 deg and roof

Reference height (at which q is sought)	H_{ref} = **6075**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **6075**mm
Fetch factor (Table 22)	S_c = **1.020**
Turbulence factor (Table 22)	S_t = **0.187**
Gust peak factor	g_t = **3.44**
Terrain and building factor	$S_b = S_c \times (1 + (g_t \times S_t) + S_h)$ = **1.68**
Effective wind speed	$V_e = V_s \times S_b$ = **43.4** m/s
Dynamic pressure	$q_s = 0.613\ kg/m^3 \times V_e^2$ = **1.154** kN/m^2

Dynamic pressure - windward wall - Wind 90 deg and roof

Reference height (at which q is sought)	H_{ref} = **6075**mm
Effective height	$H_e = \max(H_{ref} - H_d, 0.4 \times H_{ref})$ = **6075**mm
Fetch factor (Table 22)	S_c = **1.020**
Turbulence factor (Table 22)	S_t = **0.187**
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Effective wind speed	$V_e = V_s \times S_b$ = **43.4** m/s
Dynamic pressure	$q_s = 0.613\ kg/m^3 \times V_e^2$ = **1.154** kN/m^2

Size effect factors

Diagonal dimension for gablewall	a_{eg} = **9.7** m	
External size effect factor gablewall	C_{aeg} = **0.959**	
Diagonal dimension for side wall	a_{es} = **17.7** m	
External size effect factor side wall	C_{aes} = **0.921**	
Diagonal dimension for roof	a_{er} = **18.8** m	
External size effect factor roof	C_{aer} = **0.918**	
Room/storey volume for internal size effect factor		V_i = **0.125** m^3
Diagonal dimension for internal size effect factors		$a_i = 10 \times (V_i)^{1/3}$ = **5.000** m
Internal size effect factor	C_{ai} = **1.000**	

Pressures and forces

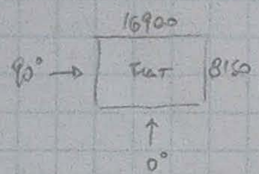
Net pressure	$p = q_s \times c_{pe} \times C_{ae} - q_s \times c_{pi} \times C_{ai}$
Net force	$F_w = p \times A_{ref}$

DESIGN LOADS

FLAT ROOF	FLOOR (SCREED)	WALLS
ⓖ 1.00 kN/m²	ⓖ 1.50 kN/m²	ⓖ 0.50 kN/m² (+0.75 kN/m² CLADDING)
ⓠ 1.50 kN/m²	ⓠ 1.50 kN/m²	

WIND LOAD

h = 35 m   d = 1 km V_b = 25 m/s   'COUNTRY'



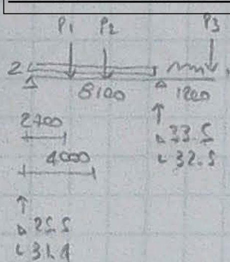
He = 5300 mm
PARAPET H = 775 mm

CHECK 1st FLOOR ONLY

- FRONT TO REAR   50.87 kN
- SIDE TO SIDE   22.78 kN

BEAM B1 (OPTION 2)

SUPERSEDED



NOTE: POINT LOADS TAKEN FROM ORIGINAL DESIGN

P3	P2	P1
ⓖ 7.06 kN	ⓖ 11.17 kN	ⓖ 8.16 kN
ⓠ 3.29 kN	ⓠ 15.08 kN	ⓠ 11.0 kN

ⓖ 1 × 0.3 = 0.3 kN/m   ⓖ 1 × (0.3 + 4.95/2) = 2.8 kN/m

ⓠ 1.5 × 0.3 = 0.45 kN/m   ⓠ 1.5 × (0.3 + 4.95/2) = 4.2 kN/m

REPLACED WITH STEEL UB 457 × 152 × 82
or
2/90 × 450 + 20 × 450 STEEL
M12 BOLTS @400c/c

(23)

Roof load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.73	1.15	0.918	-2.06	7.38	-15.19
B (-ve)	-1.22	1.15	0.918	-1.52	13.15	-19.98
C (-ve)	-0.70	1.15	0.918	-0.97	82.13	-79.81
D (-ve)	-0.20	1.15	0.918	-0.44	35.07	-15.51

Total vertical net force $F_{w,v}$ = **-130.49** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 1 - Wind 0, c_{pi} 0.20, $-c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.49	1.15	0.959	-1.88	14.76	-27.79
B	-0.86	1.15	0.959	-1.19	34.75	-41.24
w	0.81	1.15	0.921	0.62	102.67	64.16
l	-0.50	1.15	0.921	-0.76	89.57	-68.26

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-68.3** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **64.2** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **114.3** kN

Roof load case 2 - Wind 0, c_{pi} -0.3, $+c_{pe}$

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (+ve)	-1.73	1.15	0.918	-1.48	7.38	-10.93
B (+ve)	-1.22	1.15	0.918	-0.94	13.15	-12.40
C (+ve)	-0.70	1.15	0.918	-0.39	82.13	-32.44
D (+ve)	0.20	1.15	0.918	0.56	35.07	19.56

Total vertical net force $F_{w,v}$ = **-36.20** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 2 - Wind 0, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.49	1.15	0.959	-1.31	14.76	-19.28
B	-0.86	1.15	0.959	-0.61	34.75	-21.20
w	0.81	1.15	0.921	1.20	102.67	123.38
l	-0.50	1.15	0.921	-0.19	89.57	-16.59

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-16.6** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **123.4** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **120.8** kN

Roof load case 3 - Wind 90, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (-ve)	-1.73	1.15	0.918	-2.06	3.32	-6.83
B (-ve)	-1.22	1.15	0.918	-1.52	3.32	-5.05
C (-ve)	-0.70	1.15	0.918	-0.97	26.57	-25.82
D (-ve)	-0.20	1.15	0.918	-0.44	104.52	-46.24

Total vertical net force $F_{w,v}$ = **-83.94** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 3 - Wind 90, c_{pi} 0.20, -c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.53	1.15	0.921	-1.86	9.90	-18.40
B	-0.88	1.15	0.921	-1.16	39.61	-46.06
C	-0.81	1.15	0.921	-1.09	53.16	-57.97
w	0.67	1.15	0.959	0.51	49.51	25.13
l	-0.50	1.15	0.959	-0.78	43.20	-33.85

Overall loading

Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-33.8** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **25.1** kN

Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **50.9** kN

Roof load case 4 - Wind 90, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A (+ve)	-1.73	1.15	0.918	-1.48	3.32	-4.92
B (+ve)	-1.22	1.15	0.918	-0.94	3.32	-3.13
C (+ve)	-0.70	1.15	0.918	-0.39	26.57	-10.49
D (+ve)	0.20	1.15	0.918	0.56	104.52	58.30

Total vertical net force $F_{w,v}$ = **39.76** kN

Total horizontal net force $F_{w,h}$ = **0.00** kN

Walls load case 4 - Wind 90, c_{pi} -0.3, +c_{pe}

Zone	Ext pressure coefficient, c_{pe}	Dynamic pressure, q_s (kN/m^2)	External size factor, C_{ae}	Net Pressure, p (kN/m^2)	Area, A_{ref} (m^2)	Net force, F_w (kN)
A	-1.53	1.15	0.921	-1.28	9.90	-12.69
B	-0.88	1.15	0.921	-0.59	39.61	-23.22
C	-0.81	1.15	0.921	-0.51	53.16	-27.31
w	0.67	1.15	0.959	1.08	49.51	53.68
l	-0.50	1.15	0.959	-0.21	43.20	-8.93

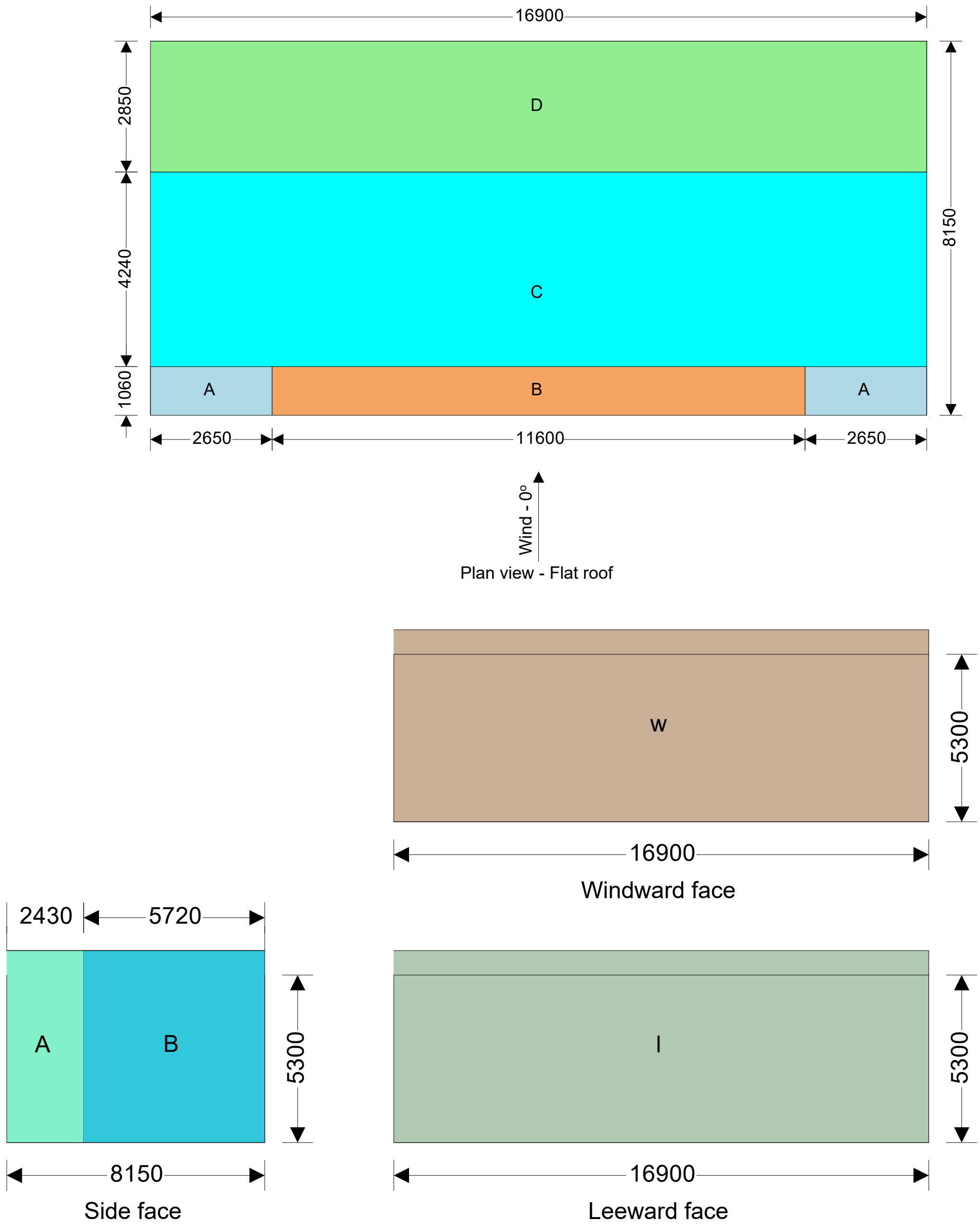
Overall loading

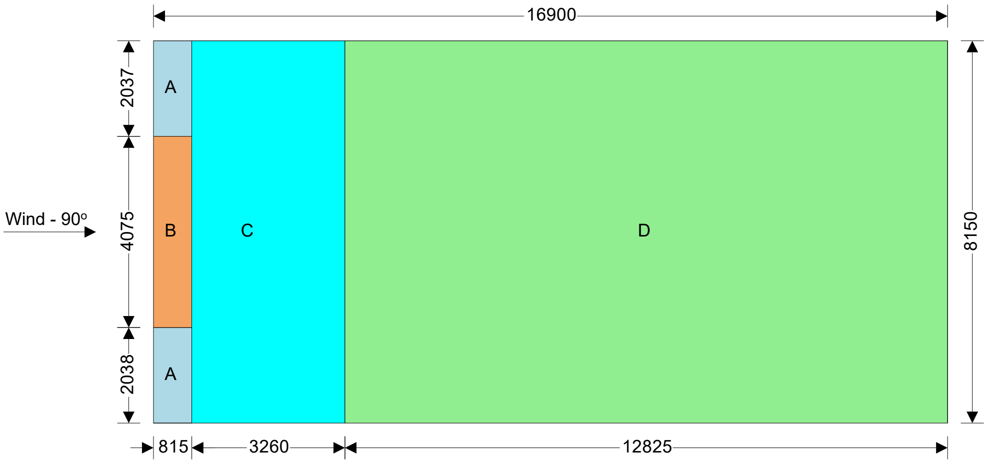
Equiv leeward net force for overall section $F_l = F_{w,wl}$ = **-8.9** kN

Net windward force for overall section $F_w = F_{w,ww}$ = **53.7** kN

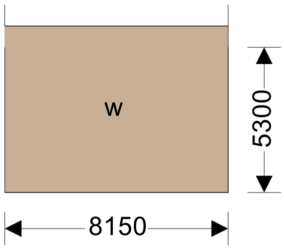
Overall loading overall section $F_{w,w} = 0.85 \times (1 + C_r) \times (F_w - F_l + F_{w,h})$ = **54.0** kN

Project	Sheet no./rev.
8962MCN - Marshlands	6 /
Wind load	By/Date MR / Jul '20

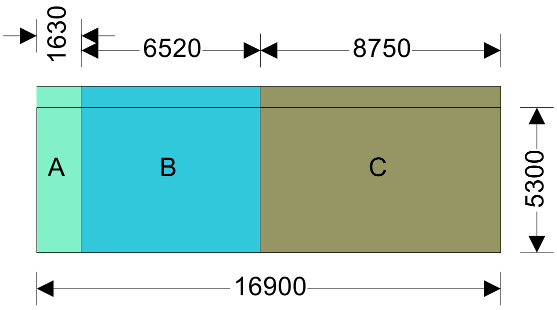




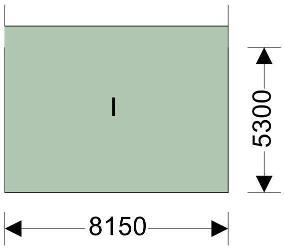
Plan view - Flat roof



Windward face



Side face



Leeward face

Project	
Marshlands	8962MCN
Title Racking Loads, First Floor - revB	**By/Date** 15/01/2021

Check Racking for Front to Back Direction

Racking (To BS 5268 Part 6.1)

Effective racking force = = 50.87 kN

Plasterboard to be fixed at 300mm 3.5Ø plasterboard screws 38mm long @ 300mm c/c

Racking resistance for sheathing 9mm OSB = 1.51 kN

Racking resistance for OSB (9mm) on OSB = 0.76 kN

Racking resistance for p'board (12.5mm) on OSB = 0.12 kN

Racking resistance for plasterboard (12.5mm) = 0.40 kN

Racking resistance for p'board (12.5mm) on p'board = 0.20 kN

Nail Diameter 2.7mm K_{101}= 0.90 Board thickness 9mm K_{103}= 1.00

Wall Ref	Wall Type	Wall Length	Basic Racking Materials		Dead Load	K_m	K_w				Racking Resistance
						k_{102}	k_{104}	k_{105}	k_{106}	k_{107}	
(-)		(m)	1	2	(kN/m)	(-)	(-)	(-)	(-)	(-)	(kN)
Utility	Ext	3.00	1.51	0.12	2.30	1.000	1.00	1.093	1.00	1.18	6.96
Master bed	Ext	4.10	1.51	0.12	2.30	1.000	1.00	1.239	1.00	1.16	10.58
Snug	Ext	4.00	1.51	0.12	2.35	1.000	1.00	1.227	1.00	1.17	10.27
WC	Ext	2.75	1.51	0.12	2.35	1.000	1.00	1.056	1.00	1.19	6.22
WC / Wardrobe	Int	1.70	0.40	0.20	0.00	1.000	1.00	0.708	1.00	1.00	0.79
Wardrobe	Wind	1.55	1.51	0.76	1.00	1.429	1.00	0.646	1.00	1.11	3.94
Master / Living	Wind	3.40	1.51	0.76	0.00	1.429	1.00	1.149	1.00	1.00	13.93
	Int	0.00	0.40	0.20	0.00	1.000	1.00	0.000	1.00	1.00	0.00
	Int	0.00	0.40	0.20	0.00	1.000	1.00	0.000	1.00	1.00	0.00
	Int	0.00	0.40	0.20	0.00	1.000	1.00	0.000	1.00	1.00	0.00

Contribution from plasterboard 3.00 kN

Total racking resistance = 52.70 kN

Total Racking Force = 50.87 kN

Total racking resistance from OSB = 49.70 kN

Total Contribution from Plasterboard = 3.00 kN

Contribution from OSB relative to racking force = 98%

Therefore, contribution from plasterboard = 2% <33% okay

Project	
Marshlands	8962MCN
Title Racking Loads, First Floor - revB	**By/Date** 15/01/2021

Check Racking for Side to Side Direction

Racking (To BS 5268 Part 6.1)

Effective racking force = = **22.78 kN**

Plasterboard to be fixed at 300mm 3.5Ø plasterboard screws 38mm long @ 300mm c/c

Racking resistance for sheathing 9mm OSB = 1.51 kN
Racking resistance for OSB (9mm) on OSB = 0.76 kN
Racking resistance for p'board (12.5mm) on OSB = 0.12 kN
Racking resistance for plasterboard (12.5mm) = 0.40 kN
Racking resistance for p'board (12.5mm) on p'board = 0.20 kN

Nail Diameter 2.7mm K_{101}= 0.90 Board thickness 9mm K_{103}= 1.00

Wall Ref	Wall Type	Wall Length	Basic Racking Materials		Dead Load	K_m	K_w				Racking Resistance
						k_{102}	k_{104}	k_{105}	k_{106}	k_{107}	
(-)		(m)	1	2	(kN/m)	(-)	(-)	(-)	(-)	(-)	(kN)
Master bed	Ext	0.85	1.51	0.12	0.30	1.000	1.00	0.354	1.00	1.04	0.56
Master bed	Ext	1.40	1.51	0.12	0.30	1.000	1.00	0.583	1.00	1.03	1.51
Dining	Ext	2.20	1.51	0.12	0.30	1.000	1.00	0.917	1.00	1.03	3.72
Snug	Ext	1.10	1.51	0.12	0.30	1.000	1.00	0.458	1.00	1.04	0.94
WC	Ext	1.15	1.51	0.12	0.30	1.000	1.00	0.479	1.00	1.04	1.02
Entrance	Ext	1.15	1.51	0.12	0.30	1.000	1.00	0.479	1.00	1.04	1.02
Entrance	Ext	1.20	1.51	0.12	0.30	1.000	1.00	0.500	1.00	1.04	1.12
Kitchen	Ext	2.00	1.51	0.12	0.30	1.000	1.00	0.833	1.00	1.03	3.08
WC	Ext	2.50	1.51	0.12	0.30	1.000	1.00	1.016	1.00	1.03	4.68
Master / WC	Int	3.30	0.40	0.20	0.00	1.000	1.00	1.136	1.00	1.00	2.47
Wardrobe	Int	1.60	0.40	0.20	0.00	1.000	1.00	0.667	1.00	1.00	0.70
Wardrobe	Int	0.60	0.40	0.20	0.00	1.000	1.00	0.250	1.00	1.00	0.10
Wardrobe	Int	1.15	0.40	0.20	0.00	1.000	1.00	0.479	1.00	1.00	0.36
Wardrobe	Int	1.05	0.40	0.20	0.00	1.000	1.00	0.438	1.00	1.00	0.30
Snug	Wind	1.70	1.51	0.12	0.00	1.000	1.00	0.708	1.00	1.00	2.16
	Int	0.00	0.40	0.20	0.00	1.000	1.00	0.000	1.00	1.00	0.00
	Int	0.00	0.40	0.20	0.00	1.000	1.00	0.000	1.00	1.00	0.00
	Int	0.00	0.40	0.20	0.00	1.000	1.00	0.000	1.00	1.00	0.00

Contribution from plasterboard 4.91 kN

Total racking resistance = **23.77 kN**
Total Racking Force = ***22.78 kN***
Total racking resistance from OSB = **18.86 kN**
Total Contribution from Plasterboard = **4.91 kN**

Contribution from OSB relative to racking force = 83%
Therefore, contribution from plasterboard = 17% <33% okay

STRUCTURAL SOLUTIONS

Project 8962MCH - MARSHLANDS

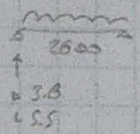
Sheet no./rev. 2

By/Date MR / NOV '20

B1 (OPTION 1)

Ⓢ 1 × (0.3 + 4.95/2) = 2.8 kN/m

Ⓘ 1.5 × (0.3 + 4.95/2) = 4.2 kN/m

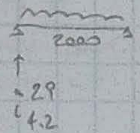


115 × 270 GL24

B2 (OPTION 1)

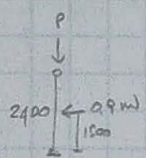
Ⓢ 2.8 kN/m

Ⓘ 4.2 kN/m



3/47 × 195 C24

POST FOR B1 (OPT. 1) & EXTG. BEAM



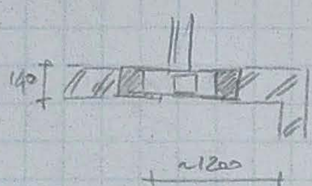
B1 9.3 kN

EXTG. BEAM (FROM ORIGINAL DESIGN) 34.8 + 19.2 = 54 kN

SHS 90 × 90 × 5.0

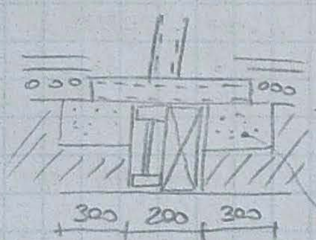
SUPPORT FOR POST (OPT. 1) JAN '21

ASSUMED 140mm WIDE MASONRY WALL BELOW → BLOCKWORK CAPACITY ASSUMED 7N

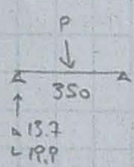


	B1	TRIMMER	EXISTING BEAM	FLOOR TRIMMER	
P Ⓢ	3.8 +	8.6 +	14 +	1.4 =	27.4 kN
Ⓘ	5.5 +	11 +	20.6 +	2.7 =	39.8 kN

• SPREADER BEAM DESIGN



PADSTONE 300 × 140 × 215



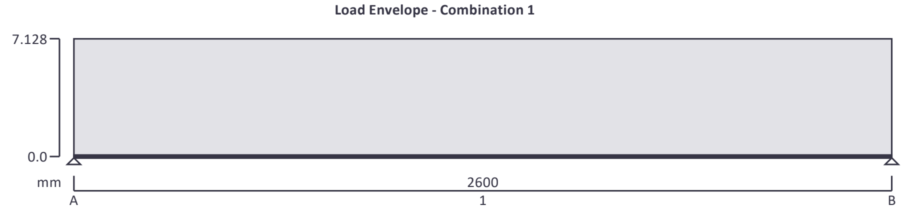
RHS 100 × 50 × 8.0 LAID FLAT

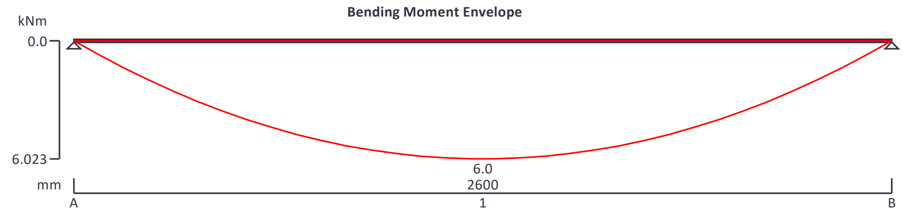
Project 8962MCN - Marshlands	Sheet no./rev. 1 /
Beam B1 (option 1) – revA	By/Date MR / Dec ‘20

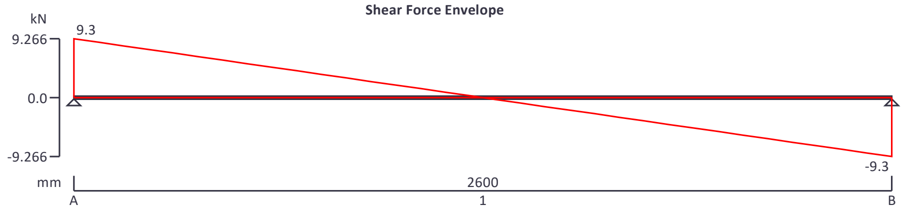
TIMBER BEAM ANALYSIS & DESIGN (BS5268)

GLULAM BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam * 1

Dead full UDL 2.800 kN/m

Imposed full UDL 4.200 kN/m

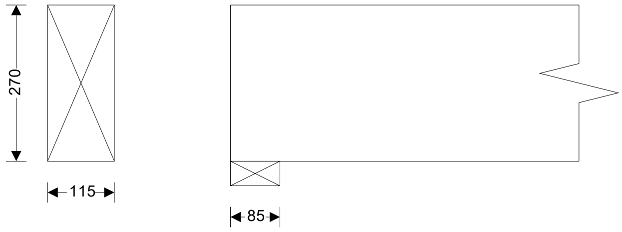
Load combinations

Load combination 1	Support A	Dead * 1.00
		Imposed * 1.00
	Span 1	Dead * 1.00
		Imposed * 1.00
	Support B	Dead * 1.00
		Imposed * 1.00

Analysis results

Maximum moment M_{max} = **6.023** kNm M_{min} = **0.000** kNm

Design moment	M = max(abs(M_{max}),abs(M_{min})) = **6.023** kNm
Maximum shear	F_{max} = **9.266** kN F_{min} = **-9.266** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **9.266** kN
Total load on beam	W_{tot} = **18.533** kN
Reactions at support A	R_{A_max} = **9.266** kN R_{A_min} = **9.266** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **3.806** kN
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **5.460** kN
Reactions at support B	R_{B_max} = **9.266** kN R_{B_min} = **9.266** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **3.806** kN
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **5.460** kN



Glulam section details

Breadth of section	b = **115** mm
Depth of section	h = **270** mm
Number of sections in member	N = **1**
Overall breadth of member	$b_b = N \times b$ = **115** mm
Number of laminations	N_{lam} = **5**
Alignment of laminations	**Horizontal**
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Medium term**
Length of span	L_{s1} = **2600** mm
Length of bearing	L_b = **85** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **31050** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **1397250** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **595125** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **188628750** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **34219688** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **77.9** mm
	$i_y = \sqrt{(I_y / A)}$ = **33.2** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.25**

Bearing stress - Table 18 K_4 = **1.00**

Total depth of member - cl.2.10.6 $K_7 = (300\ mm / h)^{0.11}$ = **1.01**

Bending parallel to grain - Table 24 K_{15} = **1.34**

Tension parallel to grain - Table 24 K_{16} = **1.34**

Compression parallel to grain - Table 24 K_{17} = **1.04**

Compression perpendicular to grain - Table 24 K_{18} = **1.55**

Shear parallel to grain - Table 24 K_{19} = **2.34**

Modulus of elasticity - Table 24 K_{20} = **1.07**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19 **3.00**

Actual depth-to-breadth ratio $h / (N * b)$ = **2.35**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} * K_3 * K_4 * K_{18}$ = **4.650** N/mm²

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N * b * L_b)$ = **0.948** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.204**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m * K_3 * K_7 * K_{15}$ = **12.709** N/mm²

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **4.311** N/mm²

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.339**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau * K_3 * K_{19}$ = **2.077** N/mm²

Applied shear stress $\tau_a = 3 * F / (2 * A)$ = **0.448** N/mm²

τ_a / τ_{adm} = **0.216**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{mean} * K_{20}$ = **11556** N/mm²

Permissible deflection $\delta_{adm} = \min(0.551\ in, 0.003 * L_{s1})$ = **7.800** mm

Bending deflection δ_{b_s1} = **1.946** mm

Shear deflection δ_{v_s1} = **0.322** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **2.268** mm

δ_a / δ_{adm} = **0.291**

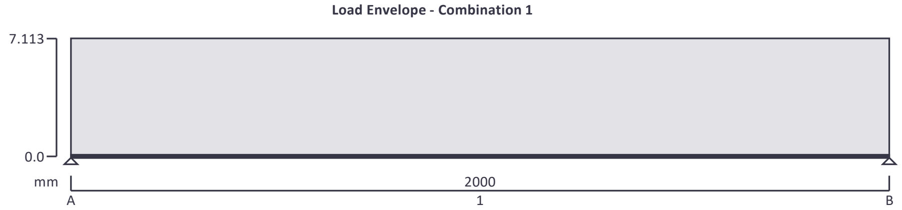
PASS - Total deflection is less than permissible deflection

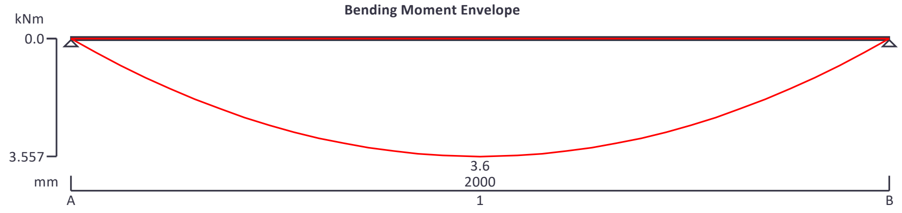
Project 8962MCN - Marshlands	Sheet no./rev. 1 /
Beam B2 (option 1) – revA	By/Date MR / Dec '20

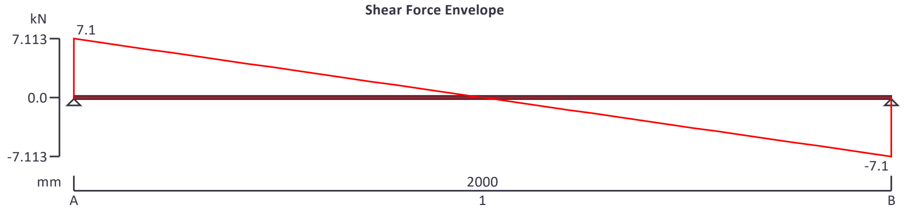
TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam * 1
Dead full UDL 2.800 kN/m
Imposed full UDL 4.200 kN/m

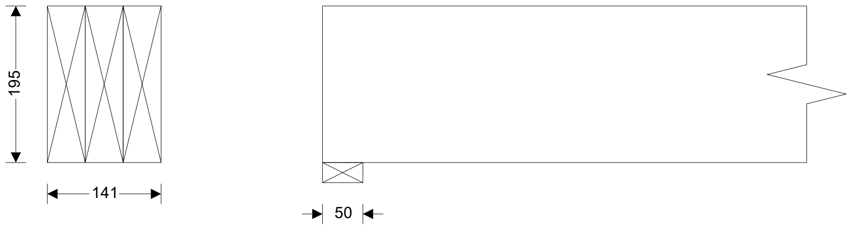
Load combinations

Load combination 1	Support A	Dead * 1.00
		Imposed * 1.00
	Span 1	Dead * 1.00
		Imposed * 1.00
	Support B	Dead * 1.00
		Imposed * 1.00

Analysis results

Maximum moment	M_{max} = **3.557** kNm	M_{min} = **0.000** kNm

Design moment	$M = max(abs(M_{max}),abs(M_{min}))$ = **3.557** kNm	
Maximum shear	F_{max} = **7.113** kN	F_{min} = **-7.113** kN
Design shear	$F = max(abs(F_{max}),abs(F_{min}))$ = **7.113** kN	
Total load on beam	W_{tot} = **14.226** kN	
Reactions at support A	R_{A_max} = **7.113** kN	R_{A_min} = **7.113** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **2.913** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **4.200** kN	
Reactions at support B	R_{B_max} = **7.113** kN	R_{B_min} = **7.113** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **2.913** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **4.200** kN	



Timber section details

Breadth of sections	b = **47** mm
Depth of sections	h = **195** mm
Number of sections in member	N = **3**
Overall breadth of member	$b_b = N \times b$ = **141** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Medium term**
Length of span	L_{s1} = **2000** mm
Length of bearing	L_b = **50** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **27495** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **893588** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **646133** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **87124781** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **45552341** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **56.3** mm
	$i_y = \sqrt{(I_y / A)}$ = **40.7** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.25**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	$K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.05**

Load sharing - cl.2.9 K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19 **3.00**

Actual depth-to-breadth ratio h / (N * b) = **1.38**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} * K_3 * K_4 * K_8$ = **3.000** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N * b * L_b)$ = **1.009** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.336**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m * K_3 * K_7 * K_8$ = **9.830** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **3.980** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.405**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau * K_3 * K_8$ = **0.888** N/mm^2

Applied shear stress $\tau_a = 3 * F / (2 * A)$ = **0.388** N/mm^2

τ_a / τ_{adm} = **0.437**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min}$ = **7200** N/mm^2

Permissible deflection δ_{adm} = min(0.551 in, 0.003 * L_{s1}) = **6.000** mm

Bending deflection δ_{b_s1} = **2.362** mm

Shear deflection δ_{v_s1} = **0.345** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **2.707** mm

δ_a / δ_{adm} = **0.451**

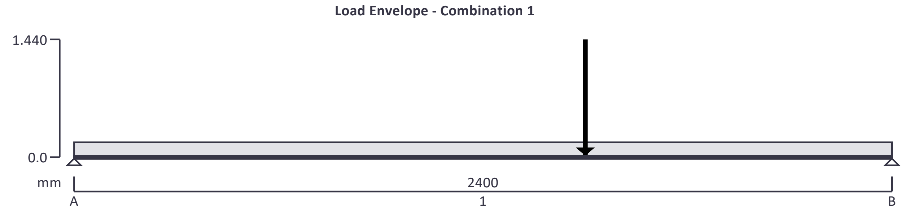
PASS - Total deflection is less than permissible deflection

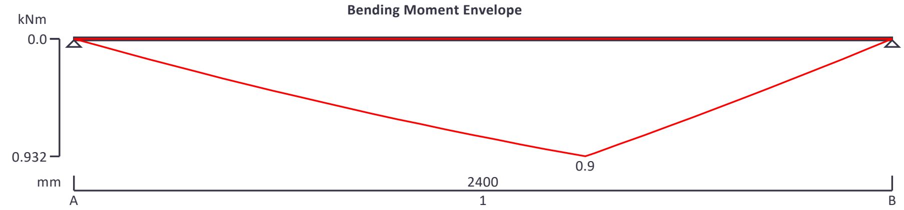
STEEL BEAM ANALYSIS & DESIGN (BS5950)

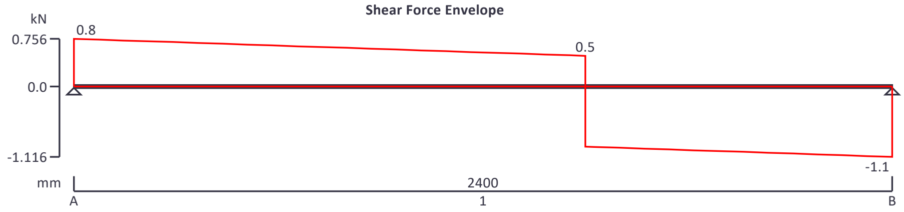
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

Support A Vertically restrained

Rotationally free

Support B Vertically restrained

Rotationally free

Applied loading

Beam loads Dead self weight of beam * 1

Imposed point load 0.9 kN at 1500 mm

Load combinations

Load combination 1 Support A Dead * 1.40

Imposed * 1.60

Dead * 1.40

Imposed * 1.60

Support B Dead * 1.40

Imposed * 1.60

Analysis results

Maximum moment M_{max} = **0.9** kNm M_{min} = **0** kNm

Maximum shear V_{max} = **0.8** kN V_{min} = **-1.1** kN

Deflection δ_{max} = **0.7** mm δ_{min} = **0** mm

Maximum reaction at support A R_{A_max} = **0.8** kN R_{A_min} = **0.8** kN

Unfactored dead load reaction at support A R_{A_Dead} = **0.2** kN

Unfactored imposed load reaction at support A $R_{A\_Imposed}$ = **0.3** kN

Maximum reaction at support B R_{B_max} = **1.1** kN R_{B_min} = **1.1** kN

Unfactored dead load reaction at support B R_{B_Dead} = **0.2** kN

Unfactored imposed load reaction at support B $R_{B\_Imposed}$ = **0.6** kN

Section details

Section type **SHS 90x90x5.0 (Tata Steel Celsius)**

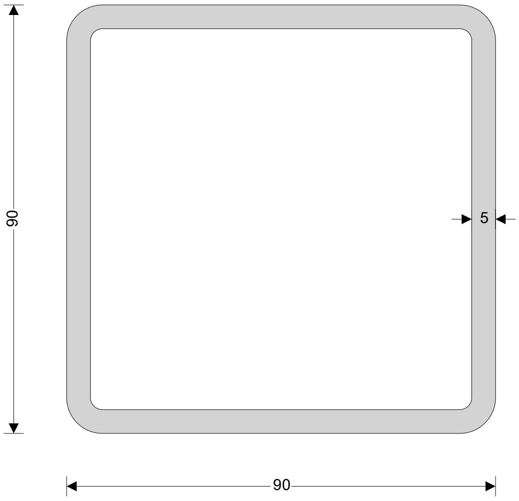
Steel grade **S355**

From table 9: Design strength p_y

Thickness of element t = **5.0** mm

Design strength p_y = **355** N/mm^2

Modulus of elasticity E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis K_x = **2.00**

Effective length factor in minor axis K_y = **2.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.20** + 2 * D

$K_{LT.B}$ = **1.20** + 2 * D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Web - major axis - Table 12

Depth of section d= D - 3 * t = **75** mm

Stress ratios r1 = min(F_c / (2 * d * t * p_{yw}), 1) = **0.238**

r2 = F_c / (A * p_{yw}) = **0.107**

d / t = 17.0 * ε <= max(64 * ε / (1 + r1), 40 * ε) Class 1 plastic

Flange - major axis - Table 12

Width of section b = B - 3 * t = **75** mm

b / t = 17.0 * ε <= 40 * ε Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force F_v = max(abs(V_{max}), abs(V_{min})) = **1.1** kN

(D - 3 * t) / t < 70 * ε

STR SOL STRUCTURAL SOLUTIONS

Web does not need to be checked for shear buckling

Shear area $A_v = A * D / (D + B) =$ **837** mm²

Design shear resistance $P_v = 0.6 * p_y * A_v =$ **178.2** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) =$ **0.9** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange $\beta_{2f} = \min(32 * \varepsilon, 62 * \varepsilon - 0.5 * d / t) =$ **28.165**

Limiting value for class 3 semi-compact flange $\beta_{3f} = 40 * \varepsilon =$ **35.206**

Limiting value for class 2 compact web $\beta_{2w} = \max(80 * \varepsilon / (1 + r1), 40 * \varepsilon) =$ **56.887**

Limiting value for class 3 semi-compact web $\beta_{3w} = \max(120 * \varepsilon / (1 + 2 * r2), 40 * \varepsilon) =$ **87.061**

Effective plastic modulus - cl.3.5.6.3

$S_{eff} = \min(Z + (S - Z) * \min([(\beta_{3w} / (d / t) - 1) / (\beta_{3w} / \beta_{2w} - 1)], [(\beta_{3f} / (b / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S) =$ **52992** mm³

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y * S_{eff}, 1.2 * p_y * Z) =$ **18.8** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.2 * L_{s1} + 2 * D =$ **3060** mm

Slenderness ratio $\lambda = L_E / r_{yy} =$ **88.598**

Equivalent slenderness - Annex B.2.6.1

Torsion constant $J =$ **3155046** mm⁴

$\gamma_b = (1 - I_{yy} / I_{xx}) \times (1 - J / (2.6 \times I_{xx})) =$ **0.000**

$\phi_b = [S_{xx}^2 \times \gamma_b / (A \times J)]^{0.5} =$ **0.000**

Ratio - cl.4.3.6.9 $\beta_W = S_{eff} / S_{xx} =$ **1.000**

Equivalent slenderness $\lambda_{LT} = 2.25 \times \sqrt{[\phi_b \times \lambda \times \beta_W]} =$ **0.000**

Limiting slenderness - Annex B.2.2 $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} =$ **30.198**

$\lambda_{LT} < \lambda_{L0}$ - No allowance need be made for

lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength $p_b = p_y =$ **355** N/mm^2

Buckling resistance moment $M_b = p_b * S_{eff} =$ **18.8** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force $F_c =$ **63.3** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ex} = L_{s1} * K_x =$ **4800** mm

Slenderness ratio - cl.4.7.2 $\lambda_x = L_{Ex} / r_{xx} =$ **138.977**

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} =$ **15.099**

Strut curve - Table 23 a

Robertson constant $\alpha_x =$ **2.0**

Perry factor $\eta_x = \alpha_x * (\lambda_x - \lambda_0) / 1000 =$ **0.248**

Euler stress $p_{Ex} = \pi^2 * E / \lambda_x^2 =$ **104.8** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) * p_{Ex}) / 2 =$ **242.9** N/mm^2

Compressive strength - Annex C.1 $p_{cx} = p_{Ex} * p_y / (\phi_x + (\phi_x^2 - p_{Ex} * p_y)^{0.5}) =$ **95.2** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cx} = A * p_{cx} =$ **159.4** kN

PASS - Compression resistance exceeds design compression force

Effective length for minor (y-y) axis buckling - Section 4.7.3

Effective length for buckling $L_{Ey} = L_{s1\_seg1} * K_y =$ **4800** mm

Slenderness ratio - cl.4.7.2 $\lambda_y = L_{Ey} / r_{yy} =$ **138.977**

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **15.099**

Strut curve - Table 23 a

Robertson constant α_y = **2.0**

Perry factor $\eta_y = \alpha_y * (\lambda_y - \lambda_0) / 1000$ = **0.248**

Euler stress $p_{Ey} = \pi^2 * E / \lambda_y^2$ = **104.8** N/mm²

$\phi_y = (p_y + (\eta_y + 1) * p_{Ey}) / 2$ = **242.9** N/mm²

Compressive strength - Annex C.1 $p_{cy} = p_{Ey} * p_y / (\phi_y + (\phi_y^2 - p_{Ey} * p_y)^{0.5})$ = **95.2** N/mm²

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cy} = A * p_{cy}$ = **159.4** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2 $F_c / (A * p_y) + M / M_c$ = **0.156**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Moment at quarter point of segment M_2 = **0.4** kNm

Moment at centre-line of segment M_3 = **0.8** kNm

Moment at three quarter point of segment M_4 = **0.6** kNm

Maximum moment in centre half of segment M_{24} = **0.9** kNm

Maximum moment in segment M_{abs} = **0.9** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs}$ = **0.9** kNm

Equiv. uniform moment factor for flexural buckling $m_x = \max(0.2 + (0.1 * M_2 + 0.6 * M_3 + 0.1 * M_4) / M_{abs}, 0.8 * M_{24} / M_{abs})$ = **0.815**

Buckling resistance checks - cl.4.8.3.3.3 $F_c / P_{cx} + m_x * M / M_c * (1 + 0.5 * F_c / P_{cx})$ = **0.446**

$F_c / P_{cy} + 0.5 * m_{LT} * M_{LT} / M_c$ = **0.422**

PASS - Member buckling

resistance checks are satisfied

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **6.667** mm

Maximum deflection span 1 $\delta = max(abs(\delta_{max}), abs(\delta_{min})) =$ **0.719** mm

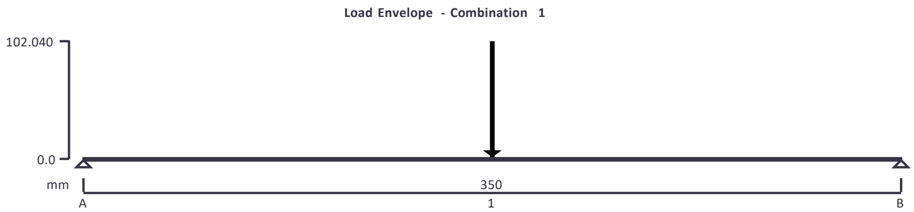
PASS - Maximum deflection does not exceed deflection limit

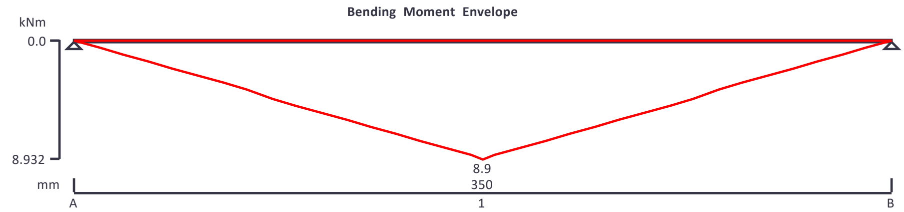
STEEL BEAM ANALYSIS & DESIGN (BS5950)

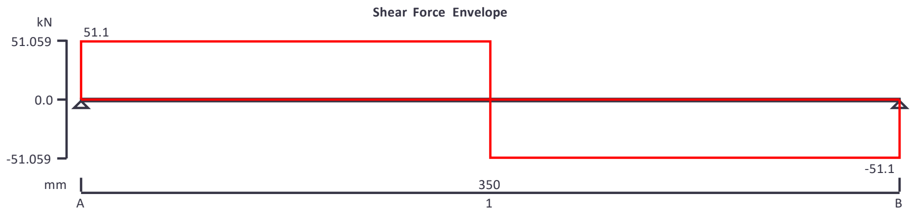
STEEL BEAM ANALYSIS & DESIGN (BS5950)

In accordance with BS5950-1:2000 incorporating Corrigendum No.1

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam * 1
	P - Dead point load 27.4 kN at 175 mm
	P - Imposed point load 39.8 kN at 175 mm

Load combinations

Load combination 1	Support A	Dead * 1.40
		Imposed * 1.60
		Dead * 1.40

		Imposed * 1.60
	Support B	Dead * 1.40
		Imposed * 1.60

Analysis results

Maximum moment	M_{max} = **8.9** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **51.1** kN	V_{min} = **-51.1** kN
Deflection	δ_{max} = **0.4** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **51.1** kN	R_{A_min} = **51.1** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **13.7** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **19.9** kN	
Maximum reaction at support B	R_{B_max} = **51.1** kN	R_{B_min} = **51.1** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **13.7** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **19.9** kN	

Section details

Section type **RHS 100x50x8.0 (Tata Steel Celsius)**

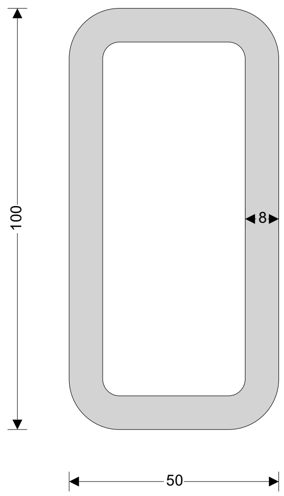
Steel grade **S355**

From table 9: Design strength p_y

Thickness of element t = **8.0** mm

Design strength p_y = **355** N/mm^2

Modulus of elasticity E = **205000** N/mm^2



Lateral restraint

Span 1 has lateral restraint at supports only

Effective length factors

Effective length factor in major axis K_x = **1.00**

Effective length factor in minor axis K_y = **1.00**

Effective length factor for lateral-torsional buckling $K_{LT.A}$ = **1.00** + 2 * D

$K_{LT.B}$ = **1.00** + 2 * D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.88**

Web - minor axis - Table 12

Depth of section — d= B - 3 * t = **26** mm

d / t = 3.7 * ε <= 64 * ε — Class 1 plastic

Flange - minor axis - Table 12

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

b / t = 10.8 * ε <= min(28 * ε, 80 * ε - d / t)Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — F_v = max(abs(V_{max}), abs(V_{min})) = **51.1** kN

(D - 3 * t) / t < 70 * ε

Web does not need to be checked for shear buckling

Shear area — $A_v = A_x$ = **692** mm^2

Design shear resistance — $P_v = 0.6 * p_y * A_v$ = **147.3** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — M = max(abs(M_{s1_max}), abs(M_{s1_min})) = **8.9** kNm

Moment capacity low shear - cl.4.2.5.2 — M_c = min($p_y * S_{yy}$, 1.2 * $p_y * Z_{yy}$) = **12.2** kNm

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360$ = **0.972** mm

Maximum deflection span 1 — δ = max(abs(δ_{max}), abs(δ_{min})) = **0.408** mm

PASS - Maximum deflection does not exceed deflection limit

MASONRY BEARING DESIGN (BS5628)

MASONRY BEARING DESIGN TO BS5628-1:2005

TEDDS calculation version 1.0.07

Masonry details

Masonry type **Aggregate concrete blocks (25% or less formed voids)**

Compressive strength of unit p_{unit} = **7.3** N/mm^2

Mortar designation **iii**

Least horizontal dimension of masonry units l_{unit} = **140** mm

Height of masonry units h_{unit} = **215** mm

Category of masonry units **Category II**

Category of construction control **Normal**

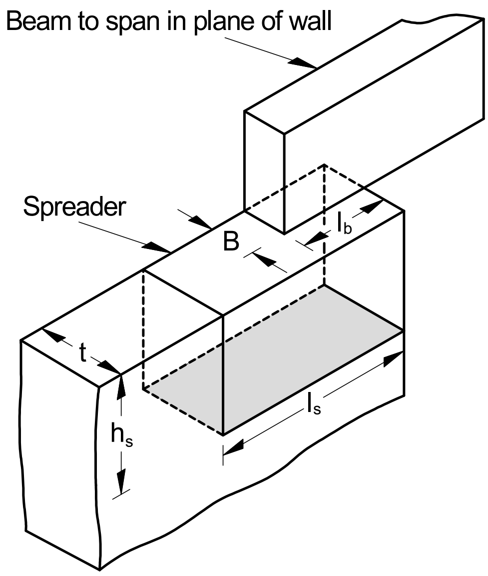
Partial safety factor for material strength γ_m = **3.5**

Thickness of load bearing leaf t = **140** mm

Effective thickness of masonry wall t_{ef} = **140** mm

Height of masonry wall h = **2500** mm

Effective height of masonry wall h_{ef} = **2500** mm



Bearing details

Beam spanning in plane of wall

Width of bearing B = **100** mm

Length of bearing l_b = **160** mm

Compressive strength from Table 2 BS5628:Part 1 - aggregate concrete blocks (25% or less formed voids)

Mortar designation Mortar = **"iii"**

Block compressive strength p_{unit} = **7.3** N/mm^2

Characteristic compressive strength (Table 2c) f_{kc} = **3.20** N/mm^2

Characteristic compressive strength (Table 2d) f_{kd} = **6.40** N/mm^2

Height of solid block h_{unit} = **215.0** mm

Least horizontal dimension l_{unit} = **140.0** mm

Block ratio ratio = h_{unit} / l_{unit} = **1.5**

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength f_k = **5.34** N/mm^2

Loading details

Characteristic concentrated dead load G_k = **14** kN

Characteristic concentrated imposed load Q_k = **20** kN

Design concentrated load F = (G_k × 1.4) + (Q_k × 1.6) = **51.0** kN

Characteristic distributed dead load g_k = **0.0** kN/m

Characteristic distributed imposed load q_k = **0.0** kN/m

Design distributed load f = (g_k × 1.4) + (q_k × 1.6) = **0.0** kN/m

Masonry bearing type

Bearing type **Type 2**

Bearing safety factor γ_{bear} = **1.50**

Check design bearing without a spreader

Design bearing stress f_{ca} = F / (B × l_b) + f / t = **3.189** N/mm^2

Allowable bearing stress f_{cp} = γ_{bear} × f_k / γ_m = **2.288** N/mm^2

FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader

Spreader details

Length of spreader l_s = **300** mm

Depth of spreader h_s = **215** mm

Edge distance s_{edge} = max(0 mm, x_{edge} – (l_s - B) / 2) = **0** mm

Spreader bearing type

Bearing type **Type 3**

Bearing safety factor γ_{bear} = **2.00**

Check design bearing with a spreader

Loading acts eccentrically outside middle third – triangular stress distribution

Offset distance x_{off} = l_b / 2 = **80** mm

Maximum bearing stress f_{ca} = 2 × F / (3 × x_{off} × t) + f / t = **3.037** N/mm^2

Allowable bearing stress f_{cp} = γ_{bear} × f_k / γ_m = **3.051** N/mm^2

PASS - Allowable bearing stress exceeds design bearing stress

Check design bearing at 0.4 × h below the bearing level

Slenderness ratio h_{ef} / t_{ef} = **17.86**

Eccentricity at top of wall e_x = **20.0** mm

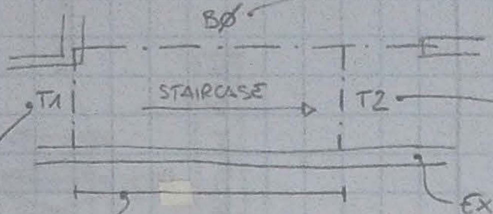
From BS5628:1 Table 7

Capacity reduction factor	β = **0.65**
Length of bearing distributed at 0.4 × h	l_d = **1160** mm
Maximum bearing stress	$f_{ca} = F / (l_d \times t) + f / t$ = **0.314** N/mm^2
Allowable bearing stress	$f_{cp} = \beta \times f_k / \gamma_m$ = **0.995** N/mm^2

PASS - Allowable bearing stress at 0.4 * h below bearing level exceeds design bearing stress

Project	8962MCH - MARSHLANDS	Sheet no./rev. 3
		By/Date MR / DEC '20

CHECK FLOOR BEAM/TRIMMERS FOR HYDRAULIC GLASS BALUSTRADE



WHEN GLAZING IS UP, BØ IS FULLY LOADED BY IT

T2 DOESN'T HAVE ADDITIONAL LOADS FROM GLAZING, ONLY STAIRCASE

EXISTING MASONRY WALL

WHEN GLAZING IS DOWN, HALF OF LOAD IS ONTO WALL & HALF ONTO BØ

T1 IS FULLY LOADED BY GLAZING, NO MATTER IF UP OR DOWN

→ GLASS WEIGHT ~208 Kg

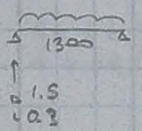
→ METALWORK WEIGHT ~300 Kg

SAY 2.1 + 3 = 5.1 KN

OVERALL DIMENSIONS OF GLAZING 1 x 3.21

→ BLANKET LOAD $\frac{5.1}{1 \times 3.21} = 1.59$ kN/m²

T1



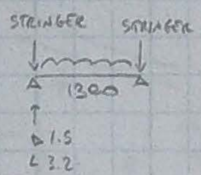
FLOOR GLAZING

(D) 1.5 x 0.3 + 1.59 x 1.1 = 2.2 kN/m

(L) 1.5 x 0.3 = 0.45 kN/m

ASSUMED EXISTING 2/44 x 195 C24

T2



(D) 1.5 x 0.3 = 0.45 kN/m

(L) 1.5 x 0.3 = 0.45 kN/m

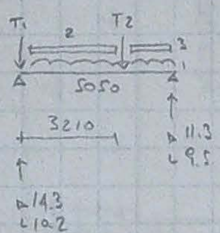
STR

(D) 0.75 x 3.2 x 1/2 = 1.2 kN

(L) 1.5 x 3.2 x 1/2 = 2.4 kN

ASSUMED EXISTING 2/44 x 195 C24

BØ (GLASS UP)



1 (D) 1.5 x 3.8/2 = 2.85 kN/m

(L) 1.5 x 3.8/2 = 2.85 kN/m

GLASS

2 (D) 1.59 x 1 = 1.59 kN/m

3 (D) 1.5 x 1.3/2 = 0.98

(L) 1.5 x 1.3/2 = 0.98

EXISTING 140 x 450 GL24 (S = 5.5 m)

BØ (GLASS DOWN)

(SAME REACTIONS)

1 (D) 2.85 kN/m

(L) 2.85 kN/m

2 (D) 1.59 x 1/2 = 0.8 kN/m

(L) 1.5 x 1/2 = 0.75 kN/m

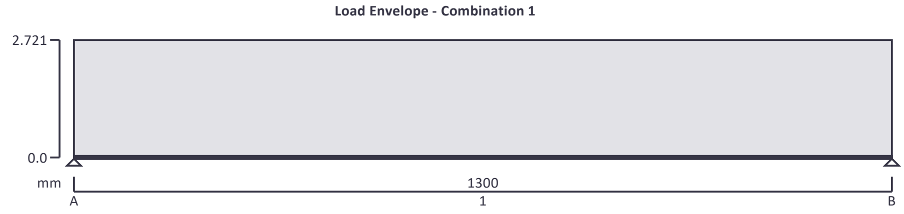
3 (D) 0.98 kN/m

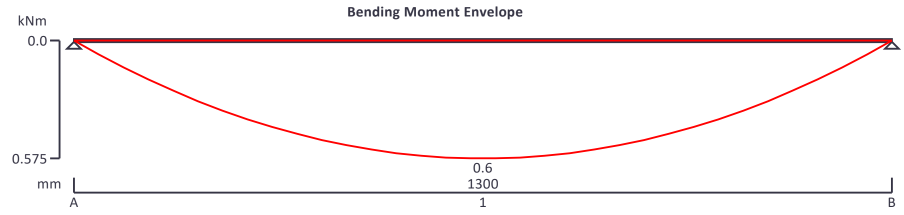
(L) 0.98 kN/m

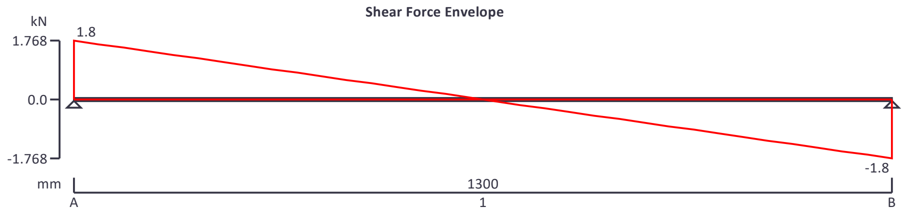
TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

Dead self weight of beam * 1
Dead full UDL 2.200 kN/m
Imposed full UDL 0.450 kN/m

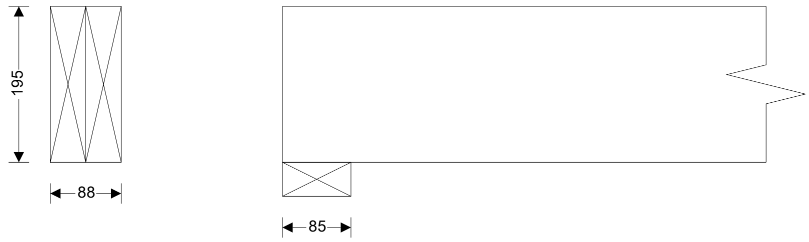
Load combinations

Load combination 1	Support A	Dead * 1.00
		Imposed * 1.00
	Span 1	Dead * 1.00
		Imposed * 1.00
	Support B	Dead * 1.00
		Imposed * 1.00

Analysis results

Maximum moment M_{max} = **0.575** kNm M_{min} = **0.000** kNm

Design moment	$M = \max(\text{abs}(M_{max}),\text{abs}(M_{min}))$ = **0.575** kNm	
Maximum shear	F_{max} = **1.768** kN	F_{min} = **-1.768** kN
Design shear	$F = \max(\text{abs}(F_{max}),\text{abs}(F_{min}))$ = **1.768** kN	
Total load on beam	W_{tot} = **3.537** kN	
Reactions at support A	R_{A_max} = **1.768** kN	R_{A_min} = **1.768** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.476** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **0.293** kN	
Reactions at support B	R_{B_max} = **1.768** kN	R_{B_min} = **1.768** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.476** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **0.292** kN	



Timber section details

Breadth of sections	b = **44** mm
Depth of sections	h = **195** mm
Number of sections in member	N = **2**
Overall breadth of member	$b_b = N \times b$ = **88** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **1300** mm
Length of bearing	L_b = **85** mm

Section properties

Cross sectional area of member	$A = N \times b \times h$ = **17160** mm^2
Section modulus	$Z_x = N \times b \times h^2 / 6$ = **557700** mm^3
	$Z_y = h \times (N \times b)^2 / 6$ = **251680** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12$ = **54375750** mm^4
	$I_y = h \times (N \times b)^3 / 12$ = **11073920** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)}$ = **56.3** mm
	$i_y = \sqrt{(I_y / A)}$ = **25.4** mm

Modification factors

Duration of loading - Table 17	K_3 = **1.00**
Bearing stress - Table 18	K_4 = **1.00**
Total depth of member - cl.2.10.6	$K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.05**

Load sharing - cl.2.9 K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19 **3.00**

Actual depth-to-breadth ratio h / (N * b) = **2.22**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) $\sigma_{c\_adm} = \sigma_{cp1} * K_3 * K_4 * K_8$ = **2.400** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N * b * L_b)$ = **0.236** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.099**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m * K_3 * K_7 * K_8$ = **7.864** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x$ = **1.031** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.131**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau * K_3 * K_8$ = **0.710** N/mm^2

Applied shear stress $\tau_a = 3 * F / (2 * A)$ = **0.155** N/mm^2

τ_a / τ_{adm} = **0.218**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{min}$ = **7200** N/mm^2

Permissible deflection δ_{adm} = min(0.551 in, 0.003 * L_{s1}) = **3.900** mm

Bending deflection δ_{b_s1} = **0.258** mm

Shear deflection δ_{v_s1} = **0.089** mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.348** mm

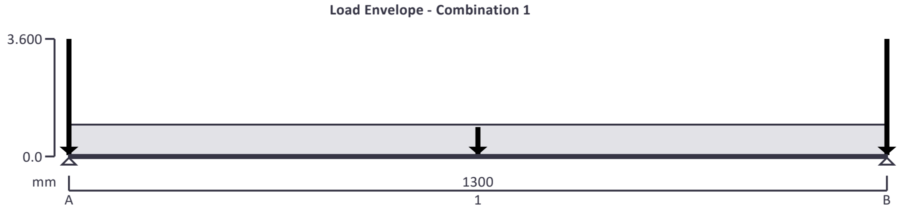
δ_a / δ_{adm} = **0.089**

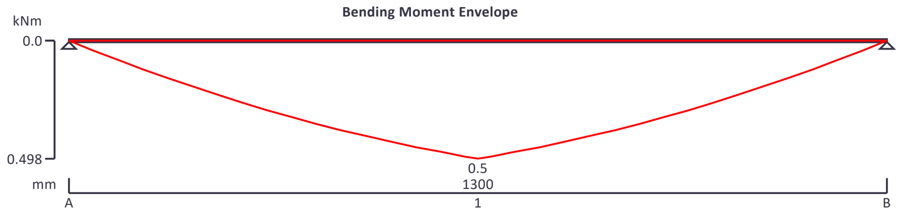
PASS - Total deflection is less than permissible deflection

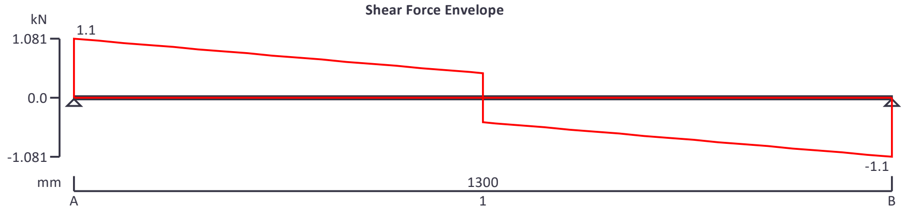
TIMBER BEAM ANALYSIS & DESIGN (BS5268)

TIMBER BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

	Dead self weight of beam * 1
	Dead full UDL 0.450 kN/m
	Imposed full UDL 0.450 kN/m
Stringer	Dead point load 1.200 kN at 0 mm
Stringer	Imposed point load 2.400 kN at 0 mm
Stringer	Dead point load 1.200 kN at 1300 mm
Stringer	Imposed point load 2.400 kN at 1300 mm
	Imposed point load 0.900 kN at 650 mm

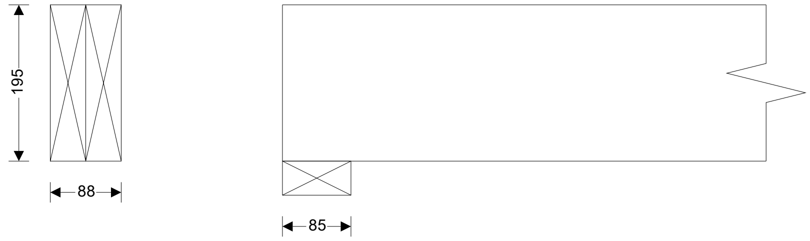
Load combinations

Load combination 1	Support A	Dead * 1.00
		Imposed * 1.00
	Span 1	Dead * 1.00

		Imposed * 1.00
	Support B	Dead * 1.00
		Imposed * 1.00

Analysis results

Maximum moment	M_{max} = **0.498** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **0.498** kNm	
Maximum shear	F_{max} = **1.081** kN	F_{min} = **-1.081** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **1.081** kN	
Total load on beam	W_{tot} = **9.362** kN	
Reactions at support A	R_{A_max} = **4.681** kN	R_{A_min} = **4.681** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **1.538** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **3.143** kN	
Reactions at support B	R_{B_max} = **4.681** kN	R_{B_min} = **4.681** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **1.538** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **3.143** kN	



Timber section details

Breadth of sections	b = **44** mm
Depth of sections	h = **195** mm
Number of sections in member	N = **2**
Overall breadth of member	b_b = N × b = **88** mm
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **1300** mm
Length of bearing	L_b = **85** mm

Section properties

Cross sectional area of member	A = N × b × h = **17160** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **557700** mm^3
	Z_y = h × (N × b)^2 / 6 = **251680** mm^3
Second moment of area	I_x = N × b × h^3 / 12 = **54375750** mm^4
	I_y = h × (N × b)^3 / 12 = **11073920** mm^4
Radius of gyration	i_x = √(I_x / A) = **56.3** mm

$i_y = \sqrt{(I_y / A)}$ = **25.4** mm

Modification factors

Duration of loading - Table 17 — K_3 = **1.00**

Bearing stress - Table 18 — K_4 = **1.00**

Total depth of member - cl.2.10.6 — $K_7 = (300 \text{ mm} / h)^{0.11}$ = **1.05**

Load sharing - cl.2.9 — K_8 = **1.00**

Lateral support - cl.2.10.8

Ends held in position

Permissible depth-to-breadth ratio - Table 19 — **3.00**

Actual depth-to-breadth ratio — h / (N * b) = **2.22**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress (no wane) — $\sigma_{c\_adm} = \sigma_{cp1} * K_3 * K_4 * K_8$ = **2.400** N/mm²

Applied bearing stress — $\sigma_{c\_a} = R_{A\_max} / (N * b * L_b)$ = **0.626** N/mm²

$\sigma_{c\_a} / \sigma_{c\_adm}$ = **0.261**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress — $\sigma_{m\_adm} = \sigma_m * K_3 * K_7 * K_8$ = **7.864** N/mm²

Applied bending stress — $\sigma_{m\_a} = M / Z_x$ = **0.892** N/mm²

$\sigma_{m\_a} / \sigma_{m\_adm}$ = **0.113**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress — $\tau_{adm} = \tau * K_3 * K_8$ = **0.710** N/mm²

Applied shear stress — $\tau_a = 3 * F / (2 * A)$ = **0.094** N/mm²

τ_a / τ_{adm} = **0.133**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection — $E = E_{min}$ = **7200** N/mm²

Permissible deflection — $\delta_{adm} = \min(0.551 \text{ in}, 0.003 * L_{s1})$ = **3.900** mm

Bending deflection — δ_{b_s1} = **0.197** mm

Shear deflection — δ_{v_s1} = **0.077** mm

Total deflection — $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **0.275** mm

δ_a / δ_{adm} = **0.070**

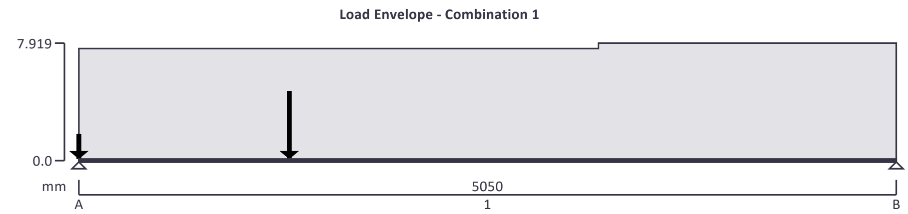
PASS - Total deflection is less than permissible deflection

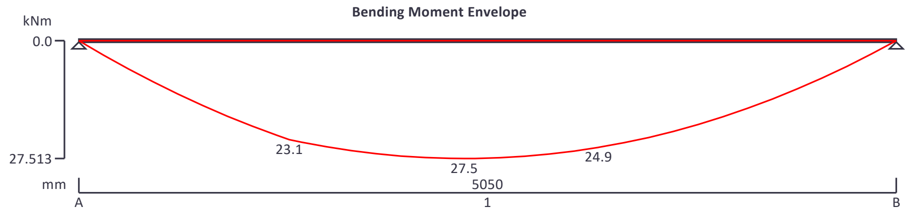
Project 8962MCN - Marshlands	Sheet no./rev. 1 /
Beam B0 (glass up)	By/Date MR / Dec ‘20

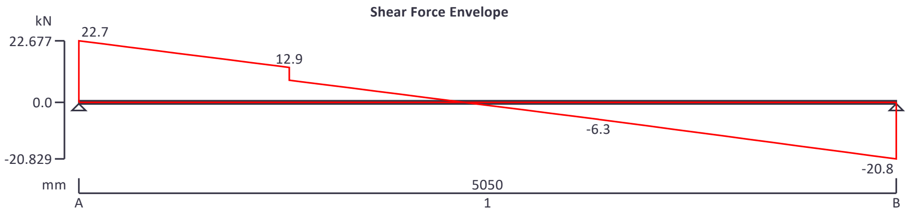
TIMBER BEAM ANALYSIS & DESIGN (BS5268)

GLULAM BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

	Dead self weight of beam * 1
	Dead full UDL 2.850 kN/m
	Imposed full UDL 2.850 kN/m
T1	Dead point load 1.500 kN at 0 mm
T1	Imposed point load 0.300 kN at 0 mm
T2	Dead point load 1.500 kN at 1300 mm
T2	Imposed point load 3.200 kN at 1300 mm
	Dead partial UDL 1.590 kN/m from 0 mm to 3210 mm
	Dead partial UDL 0.980 kN/m from 3210 mm to 5050 mm
	Imposed partial UDL 0.980 kN/m from 3210 mm to 5050 mm

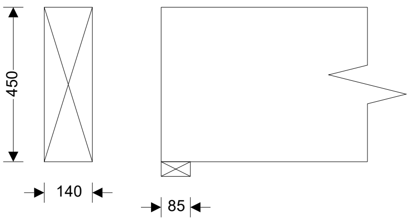
Load combinations

Load combination 1	Support A	Dead * 1.00

		Imposed * 1.00
	Span 1	Dead * 1.00
		Imposed * 1.00
	Support B	Dead * 1.00
		Imposed * 1.00

Analysis results

Maximum moment	M_{max} = **27.513** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **27.513** kNm	
Maximum shear	F_{max} = **22.677** kN	F_{min} = **-20.829** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **22.677** kN	
Total load on beam	W_{tot} = **45.306** kN	
Reactions at support A	R_{A_max} = **24.477** kN	R_{A_min} = **24.477** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **14.276** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **10.201** kN	
Reactions at support B	R_{B_max} = **20.829** kN	R_{B_min} = **20.829** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **11.334** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **9.495** kN	



Glulam section details

Breadth of section	b = **140** mm
Depth of section	h = **450** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **140** mm
Number of laminations	N_{lam} = **10**
Alignment of laminations	**Horizontal**
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **5050** mm
Length of bearing	L_b = **85** mm

Section properties

Cross sectional area of member	A = N × b × h = **63000** mm^2
Section modulus	Z_x = N × b × h^2 / 6 = **4725000** mm^3

	$Z_y = h \times (N \times b)^2 / 6 =$ **1470000** mm^3
Second moment of area	$I_x = N \times b \times h^3 / 12 =$ **1063125000** mm^4
	$I_y = h \times (N \times b)^3 / 12 =$ **102900000** mm^4
Radius of gyration	$i_x = \sqrt{(I_x / A)} =$ **129.9** mm
	$i_y = \sqrt{(I_y / A)} =$ **40.4** mm

Modification factors

Duration of loading - Table 17	$K_3 =$ **1.00**
Bearing stress - Table 18	$K_4 =$ **1.00**
Total depth of member - cl.2.10.6	$K_7 = 0.81 * (h^2 + 92300 mm^2) / (h^2 + 56800 mm^2) =$ **0.92**
Bending parallel to grain - Table 24	$K_{15} =$ **1.42**
Tension parallel to grain - Table 24	$K_{16} =$ **1.42**
Compression parallel to grain - Table 24	$K_{17} =$ **1.04**
Compression perpendicular to grain - Table 24	$K_{18} =$ **1.55**
Shear parallel to grain - Table 24	$K_{19} =$ **2.34**
Modulus of elasticity - Table 24	$K_{20} =$ **1.07**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19	**4.00**
Actual depth-to-breadth ratio	$h / (N * b) =$ **3.21**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress	$\sigma_{c\_adm} = \sigma_{cp1} * K_3 * K_4 * K_{18} =$ **3.720** N/mm^2
Applied bearing stress	$\sigma_{c\_a} = R_{A\_max} / (N * b * L_b) =$ **2.057** N/mm^2
	$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.553**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress	$\sigma_{m\_adm} = \sigma_m * K_3 * K_7 * K_{15} =$ **9.808** N/mm^2
Applied bending stress	$\sigma_{m\_a} = M / Z_x =$ **5.823** N/mm^2
	$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.594**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress	$\tau_{adm} = \tau * K_3 * K_{19} =$ **1.661** N/mm^2
Applied shear stress	$\tau_a = 3 * F / (2 * A) =$ **0.540** N/mm^2
	$\tau_a / \tau_{adm} =$ **0.325**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection $E = E_{mean} * K_{20} = \mathbf{11556}$ N/mm^2

Permissible deflection $\delta_{adm} = \min(0.551 \text{ in}, 0.003 * L_{s1}) = \mathbf{13.995}$ mm

Bending deflection $\delta_{b\_s1} = \mathbf{6.002}$ mm

Shear deflection $\delta_{v\_s1} = \mathbf{0.726}$ mm

Total deflection $\delta_a = \delta_{b\_s1} + \delta_{v\_s1} = \mathbf{6.727}$ mm

$\delta_a / \delta_{adm} = \mathbf{0.481}$

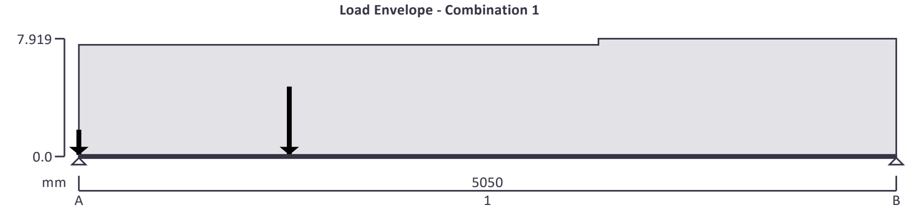
PASS - Total deflection is less than permissible deflection

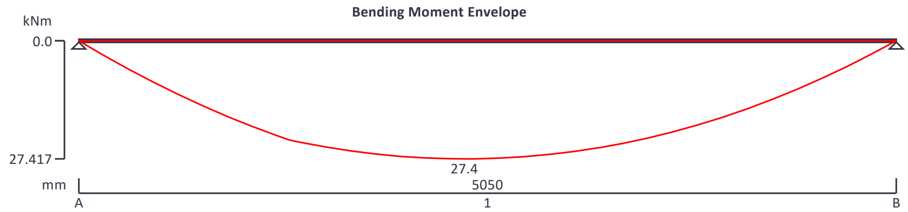
Project 8962MCN - Marshlands	Sheet no./rev. 1 /
Beam B0 (glass down)	By/Date MR / Dec ‘20

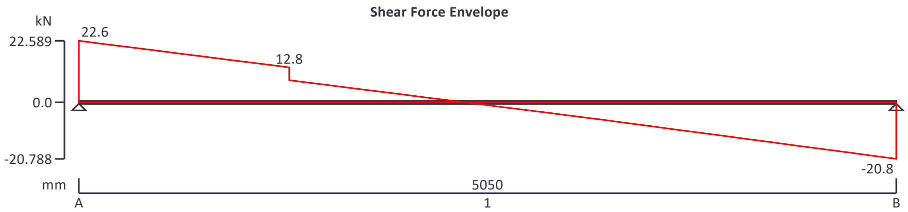
TIMBER BEAM ANALYSIS & DESIGN (BS5268)

GLULAM BEAM ANALYSIS & DESIGN TO BS5268-2:2002

TEDDS calculation version 1.7.02







Applied loading

Beam loads

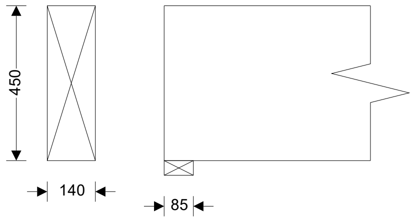
	Dead self weight of beam * 1
	Dead full UDL 2.850 kN/m
	Imposed full UDL 2.850 kN/m
T1	Dead point load 1.500 kN at 0 mm
T1	Imposed point load 0.300 kN at 0 mm
T2	Dead point load 1.500 kN at 1300 mm
T2	Imposed point load 3.200 kN at 1300 mm
	Dead partial UDL 0.800 kN/m from 0 mm to 3210 mm
	Imposed partial UDL 0.750 kN/m from 0 mm to 3210 mm
	Dead partial UDL 0.980 kN/m from 3210 mm to 5050 mm
	Imposed partial UDL 0.980 kN/m from 3210 mm to 5050 mm

Load combinations

Load combination 1	Support A	Dead * 1.00
		Imposed * 1.00
	Span 1	Dead * 1.00
		Imposed * 1.00
	Support B	Dead * 1.00
		Imposed * 1.00

Analysis results

Maximum moment	M_{max} = **27.417** kNm	M_{min} = **0.000** kNm
Design moment	M = max(abs(M_{max}),abs(M_{min})) = **27.417** kNm	
Maximum shear	F_{max} = **22.589** kN	F_{min} = **-20.788** kN
Design shear	F = max(abs(F_{max}),abs(F_{min})) = **22.589** kN	
Total load on beam	W_{tot} = **45.177** kN	
Reactions at support A	R_{A_max} = **24.389** kN	R_{A_min} = **24.389** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **12.546** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **11.843** kN	
Reactions at support B	R_{B_max} = **20.788** kN	R_{B_min} = **20.788** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **10.528** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **10.260** kN	



Glulam section details

Breadth of section	b = **140** mm
Depth of section	h = **450** mm
Number of sections in member	N = **1**
Overall breadth of member	b_b = N × b = **140** mm
Number of laminations	N_{lam} = **10**
Alignment of laminations	**Horizontal**
Timber strength class	**C24**

Member details

Service class of timber	**1**
Load duration	**Long term**
Length of span	L_{s1} = **5050** mm
Length of bearing	L_b = **85** mm

Section properties

Cross sectional area of member $A = N \times b \times h =$ **63000** mm^2

Section modulus $Z_x = N \times b \times h^2 / 6 =$ **4725000** mm^3

$Z_y = h \times (N \times b)^2 / 6 =$ **1470000** mm^3

Second moment of area $I_x = N \times b \times h^3 / 12 =$ **1063125000** mm^4

$I_y = h \times (N \times b)^3 / 12 =$ **102900000** mm^4

Radius of gyration $i_x = \sqrt{(I_x / A)} =$ **129.9** mm

$i_y = \sqrt{(I_y / A)} =$ **40.4** mm

Modification factors

Duration of loading - Table 17 $K_3 =$ **1.00**

Bearing stress - Table 18 $K_4 =$ **1.00**

Total depth of member - cl.2.10.6 $K_7 = 0.81 * (h^2 + 92300 mm^2) / (h^2 + 56800 mm^2) =$ **0.92**

Bending parallel to grain - Table 24 $K_{15} =$ **1.42**

Tension parallel to grain - Table 24 $K_{16} =$ **1.42**

Compression parallel to grain - Table 24 $K_{17} =$ **1.04**

Compression perpendicular to grain - Table 24 $K_{18} =$ **1.55**

Shear parallel to grain - Table 24 $K_{19} =$ **2.34**

Modulus of elasticity - Table 24 $K_{20} =$ **1.07**

Lateral support - cl.2.10.8

Ends held in position and members held in line, as by purlins or tie rods at centres not more than 30 times the breadth of the member

Permissible depth-to-breadth ratio - Table 19 **4.00**

Actual depth-to-breadth ratio $h / (N * b) =$ **3.21**

PASS - Lateral support is adequate

Compression perpendicular to grain

Permissible bearing stress $\sigma_{c\_adm} = \sigma_{cp1} * K_3 * K_4 * K_{18} =$ **3.720** N/mm^2

Applied bearing stress $\sigma_{c\_a} = R_{A\_max} / (N * b * L_b) =$ **2.049** N/mm^2

$\sigma_{c\_a} / \sigma_{c\_adm} =$ **0.551**

PASS - Applied compressive stress is less than permissible compressive stress at bearing

Bending parallel to grain

Permissible bending stress $\sigma_{m\_adm} = \sigma_m * K_3 * K_7 * K_{15} =$ **9.808** N/mm^2

Applied bending stress $\sigma_{m\_a} = M / Z_x =$ **5.803** N/mm^2

$\sigma_{m\_a} / \sigma_{m\_adm} =$ **0.592**

PASS - Applied bending stress is less than permissible bending stress

Shear parallel to grain

Permissible shear stress $\tau_{adm} = \tau * K_3 * K_{19} =$ **1.661** N/mm^2

Applied shear stress $\tau_a = 3 * F / (2 * A) =$ **0.538** N/mm^2

τ_a / τ_{adm} = **0.324**

PASS - Applied shear stress is less than permissible shear stress

Deflection

Modulus of elasticity for deflection — $E = E_{mean} * K_{20}$ = **11556** N/mm^2

Permissible deflection — δ_{adm} = min(0.551 in, 0.003 * L_{s1}) = **13.995** mm

Bending deflection — δ_{b_s1} = **5.982** mm

Shear deflection — δ_{v_s1} = **0.723** mm

Total deflection — $\delta_a = \delta_{b\_s1} + \delta_{v\_s1}$ = **6.706** mm

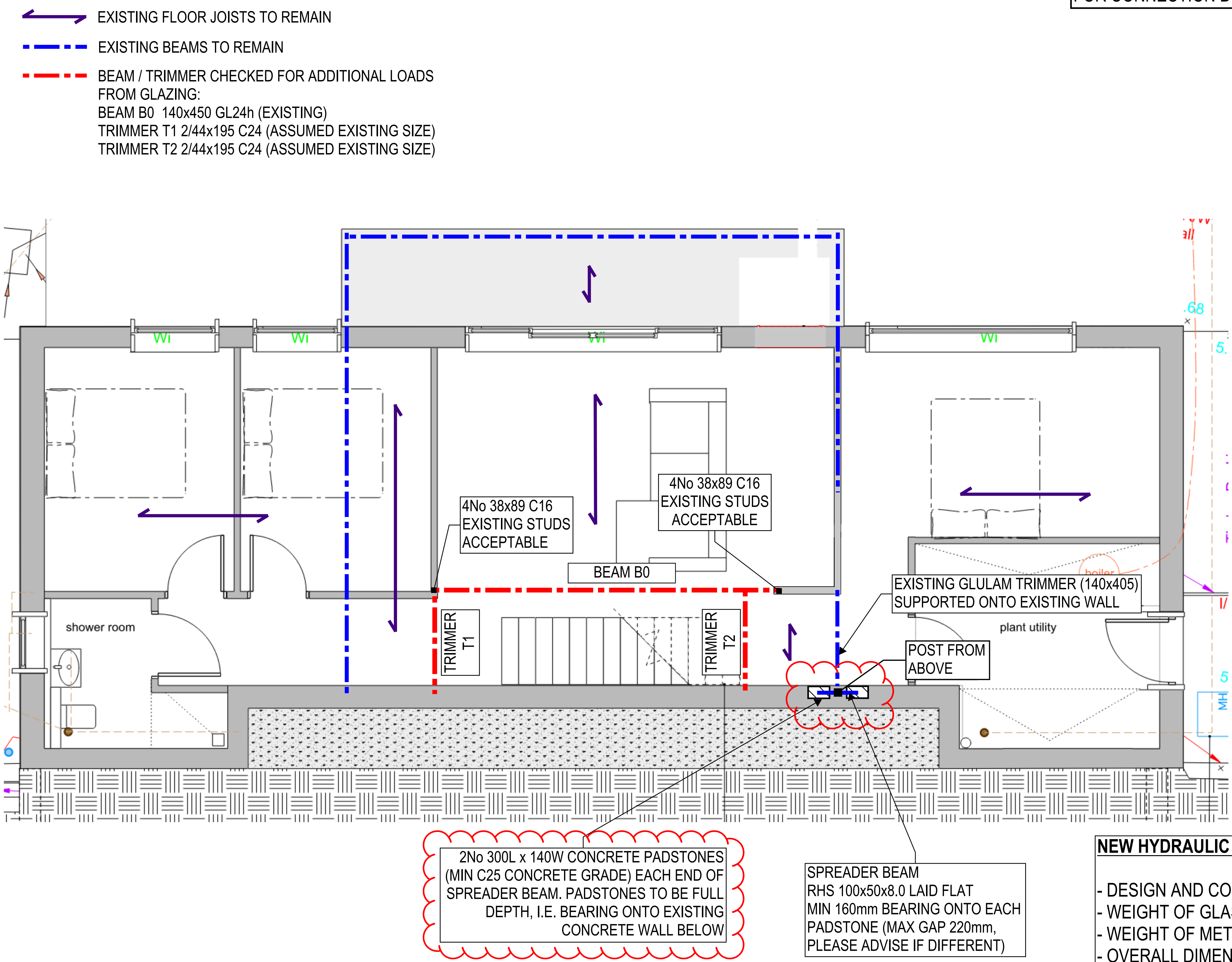
δ_a / δ_{adm} = **0.479**

PASS - Total deflection is less than permissible deflection

NOTE
REFER TO SEPARATE DRGS FOR CONNECTION DETAILS

GENERAL NOTES:

1. Do not scale from this drawings, if in doubt ask.
2. This drawing is to be read in conjunction with Architect's drawings and any other Services or Engineers drawings and specifications. Any discrepancies are to be reported immediately in writing for verification.
3. Refer to Architect's drawings for setting out dimensions.
4. The contractor is responsible for stability of the building whilst the works are in progress.
5. All temporary works are the responsibility of the contractor. The contractor shall establish and maintain safe working practices and shall fully discharge their obligations under Health and Safety regulations.
6. For coordination of all service holes / penetrations through floors, refer to Architect's setting out / layout drawings and details.
7. All proprietary materials to be used in strict accordance with manufacturer's instructions.
8. Plasterboard lining for walls to be min 12.5mm thk Gypsum plasterboard as per BS EN 520 'Type A', screwed at 300mm centres throughout using 3.5mm ø screws minimum 38mm long.
9. All Plywood used for wall sheathing, structural floor decking and structural roof decking to be in accordance with BS EN 636: Plywood. Specifications, and BS EN 13986: Wood-based panels for use in construction. Characteristics, evaluation of conformity and marking.
10. Existing external walls 38 x 140 C16 studs @ 600mm c/c.
11. Internal existing load bearing walls 38 x 89 C16 studs @ 600mm c/c.
12. Allow 15mm clear gap between underside of roof joists and top of all non-loadbearing wall panels. Use Simpson Strong-Tie PWR Partition Wall Restraint brackets (or similar approved) to connect panel heads to each joists. Where partitions run parallel with joists above, use full depth timber noggings @max 600mm c/c fixed between adjacent joists and fix connectors to noggings.
13. Multiple-ply timber joists and timber beams must be securely fixed together using M12 bolts @ max 600mm c/c at center-line of joists, unless noted otherwise.



NEW HYDRAULIC GLASS BALUSTRADE OVER STAIRCASE ASSUMPTIONS:

- DESIGN AND CONNECTIONS BY SUPPLIERS
- WEIGHT OF GLASS ASSUMED MAX 2.1kN
- WEIGHT OF METALWORK AND COMPONENTS ASSUMED MAX 3kN
- OVERALL DIMENSIONS (IN PLAN) 3.21m x 1.0m
- **BEAM B0**:
  ¬ WHEN GLAZING DOWN, ASSUMED GLASS TO SPAN BETWEEN BEAM AND EXTERNAL MASONRY WALL (SUPPORT FOR GLASS ONTO WALL DETAILED BY OTHERS).
  ¬ WHEN GLAZING UP, BEAM SUPPORTS VERTICAL GLASS BALUSTRADE.
- **TRIMMER T1**: SUPPORTS DISAPPEARING GLAZING (BOTH GLASS UP AND DOWN CASE)
- **TRIMMER T2**: NO ADDITIONAL LOADS FROM GLAZING

PLEASE ADVISE IF ARRANGEMENT DIFFERENT FROM THAT STATED.

STRUCTURAL SOLUTIONS

MARSHLANDS, CLEY NEXT-THE SEA, HOLT, NR25 7RZ

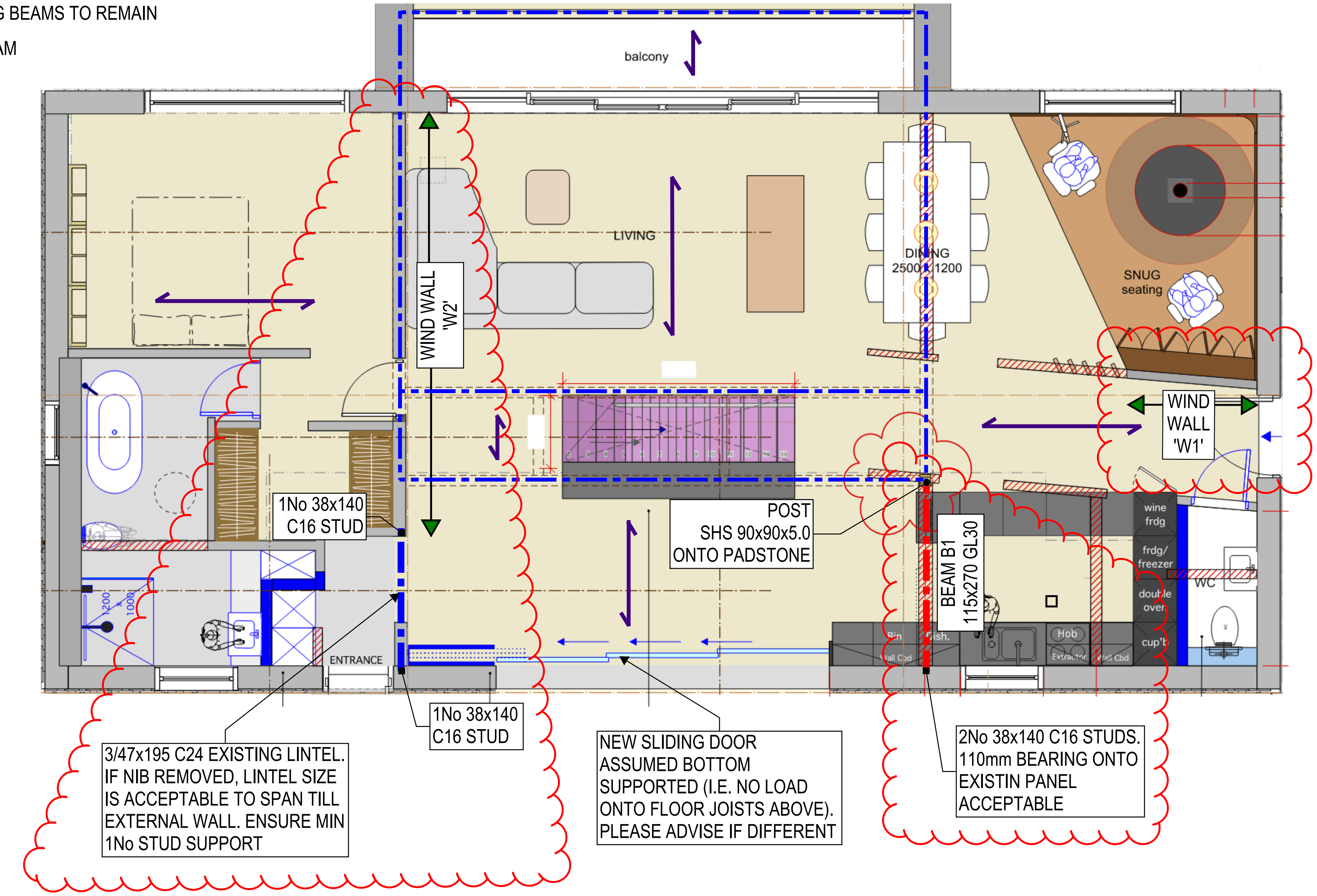
INTERNAL ALTERATIONS - GROUND FLOOR PLAN SHOWING 1st FLOOR STRUCTURE

DRG. No. 110 / REV. P4 - PADSTONES NOTE REVISED - 02/02/21

MR - 01/12/2020

EXISTING ROOF JOISTS TO REMAIN
EXISTING BEAMS TO REMAIN
NEW BEAM

OPTION 1:
RETAIN EXISTING BEAM OVER DINING, NEW STEEL POST INSTEAD OF STUD CLUSTER AND NEW BEAM B1



EXISTING WALLS TO REMAIN
EXISTING WALLS TO BE REMOVED
NEW WALLS

NOTE:
WHEN REMOVING EXISTING STUD WALL IN DINING AND REVEALING STUD CLUSTER OF EXISTING GLULAM BEAM, BUILDER TO ENSURE THAT TEMPORARY BRACING ARE INSTALLED TO SECURE STUDS

NOTE
REFER TO SEPARATE DRGS FOR CONNECTION DETAILS

2No INTERNAL WIND WALLS REQUIRED:

'W1' : 9mm OSB (OR SIMILAR CATEGORY 1 BOARD) SHEATHING ON ONE SIDE AND STANDARD NAILED. STANDARD NAILING: 150mm c/c ON PERIMETER, 300mm c/c INTERNALLY. ALL OSB TO WALL NAILED USING 2.7mm DIA x 50mm LONG GALVANISED WIRE NAILS (OR EQUALLY APPROVED).

'W2' : 9mm OSB (OR SIMILAR CATEGORY 1 BOARD) SHEATHING ON TWO SIDES AND DOUBLE NAILED. DOUBLE NAILING: 75mm c/c ON PERIMETER, 150mm c/c INTERNALLY. ALL OSB TO WALL NAILED USING 2.7mm DIA x 50mm LONG GALVANISED WIRE NAILS (OR EQUALLY APPROVED).

ALL EXTERNAL WALLS ASSUMED SHEATHED ON ONE SIDE AND STANDARD NAILED

GENERAL NOTES:

1. Do not scale from this drawings, if in doubt ask.
2. This drawing is to be read in conjunction with Architect's drawings and any other Services or Engineers drawings and specifications. Any discrepancies are to be reported immediately in writing for verification.
3. Refer to Architect's drawings for setting out dimensions.
4. The contractor is responsible for stability of the building whilst the works are in progress.
5. All temporary works are the responsibility of the contractor. The contractor shall establish and maintain safe working practices and shall fully discharge their obligations under Health and Safety regulations.
6. For coordination of all service holes / penetrations through floors, refer to Architect's setting out / layout drawings and details.
7. All proprietary materials to be used in strict accordance with manufacturer's instructions.
8. Plasterboard lining for walls to be min 12.5mm thk Gypsum plasterboard as per BS EN 520 'Type A', screwed at 300mm centres throughout using 3.5mm ø screws minimum 38mm long.
9. All Plywood used for wall sheathing, structural floor decking and structural roof decking to be in accordance with BS EN 636: Plywood. Specifications, and BS EN 13986: Wood-based panels for use in construction. Characteristics, evaluation of conformity and marking.
10. Existing external walls 38 x 140 C16 studs @ 600mm c/c.
11. Internal existing load bearing walls 38 x 140 C16 studs @ 600mm c/c.
12. Allow 15mm clear gap between underside of roof joists and top of all non-loadbearing wall panels. Use Simpson Strong-Tie PWR Partition Wall Restraint brackets (or similar approved) to connect panel heads to each joists. Where partitions run parallel with joists above, use full depth timber noggings @max 600mm c/c fixed between adjacent joists and fix connectors to noggings.
13. Multiple-ply timber joists and timber beams must be securely fixed together using M12 bolts @ max 600mm c/c at center-line of joists, unless noted otherwise.
14. All concentrated loads from beams and lintels are to be blocked through floors and stud clusters taken down to sole plate.

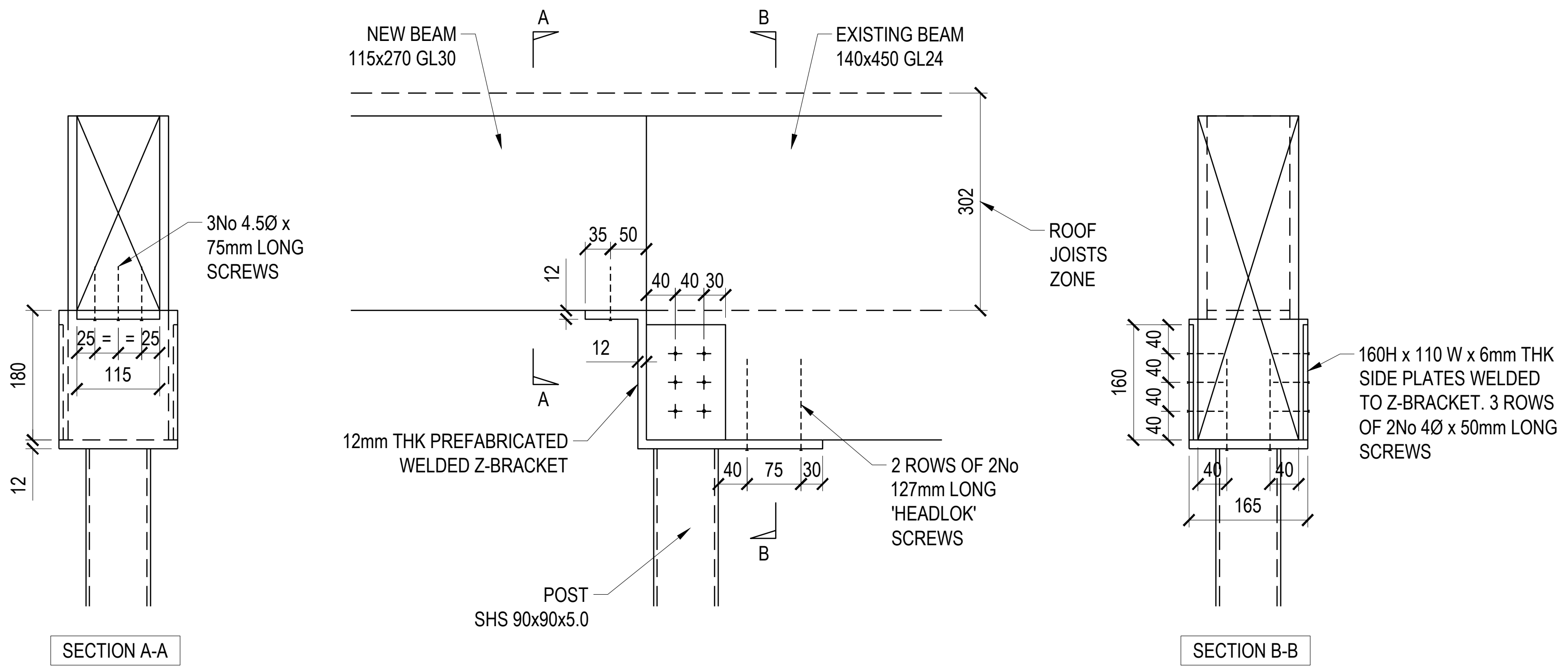
MARSHLANDS, CLEY NEXT-THE SEA, HOLT, NR25 7RZ

INTERNAL ALTERATIONS - 1st FLOOR PLAN SHOWING ROOF STRUCTURE

DRG. No. 120 / REV. P2 - WIND WALLS REVISED TO SUIT NEW INTERNAL LAYOUT, BEAMS REVISED, OPTION 2 OMITTED - 11/12/20

MR - 01/12/2020

STRUCTURAL SOLUTIONS

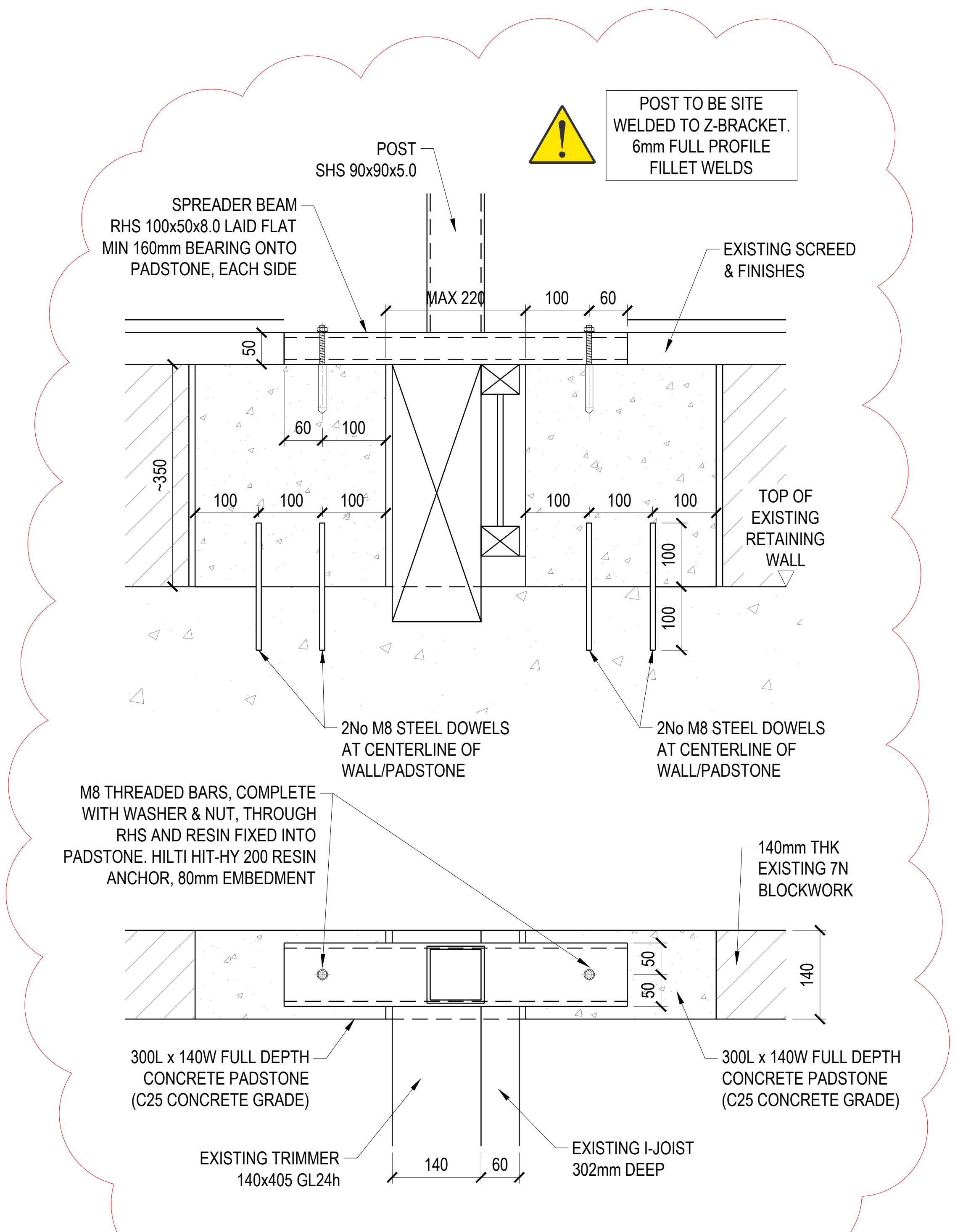


DETAIL D01

GLULAM BEAMS ONTO POST

(1:10)

POST TO BE SITE WELDED TO Z-BRACKET. 6mm FULL PROFILE FILLET WELDS



DETAIL D02

POST BASE DETAIL

(1:10)

GENERAL NOTES:

1. DO NOT SCALE FROM THIS DRAWINGS, IF IN DOUBT ASK.
2. ALL CONNECTION DRAWINGS MUST BE READ IN CONJUNCTION WITH STRUCTURAL SOLUTIONS MARKUP DRAWINGS.
3. REFER TO ARCHITECT'S DRAWINGS FOR ALL SETTING OUT DIMENSIONS FOR FABRICATION PURPOSES
4. ANY DISCREPANCIES TO BE REPORTED TO STRUCTURAL SOLUTIONS IMMEDIATELY.
5. ALL STEELWORK TO BE EXECUTION CLASS EXC2 IN ACCORDANCE WITH BS EN 1090-1.
6. ALL WELDS TO BE 6mm FULL PROFILE FILLET WELDS, UNLESS NOTED OTHERWISE.
7. ALL STEEL SECTIONS / FLATS TO MINIMUM GRADE S275 IN ACCORDANCE WITH BS EN 10025.
8. ALL HOLLOW SECTIONS TO BE MINIMUM GRADE S355 IN ACCORDANCE WITH BS EN 10210.
9. EXTERNAL STEELWORK OR STEELWORK NOTED AS GALVANISED TO BE HOT DIP GALVANISED TO BS EN ISO 1461.
10. ALL STEEL SURFACES TO BE BLAST CLEANED TO MIN SA 2 1/2 AND PAINTED WITH 2 COATS OF HIGH BUILD ZINC PHOSPHATE PRIMER, 75 MICRONS THK DRY FILM.
11. ALL PROPRIETARY FIXINGS MUST BE INSTALLED IN ACCORDANCE WITH MANUFACTURER'S GUIDELINES.
12. RESIN ANCHORS INTO CONCRETE PLINTHS TO BE HILTI HIT-V-F 8.8 (OR SIMILAR APPROVED), UNLESS NOTED OTHERWISE.
13. BOLTS FOR TIMBER CONNECTIONS TO BE MIN GRADE 4.6. USE ENLARGED WASHERS BETWEEN BOLT HEAD/NUT & TIMBER TO SUIT BOLT DIAMETER (D): WASHER DIAMETER = 4xD & WASHER THICKNESS = D/3, UNLESS NOTED OTHERWISE.

REV	DATE	AMENDMENT		
P3	21/01/21	FULL DEPTH PADSTONES ADDED IN D02	MR	MO
P2	15/01/21	AMENDMENTS AS BUBBLED	MR	MO
P1	11/12/20	FIRST ISSUE	MR	MO

DRAWING STATUS: PRELIMINARY

PROJECT: MARSHLANDS, CLEY NEXT-THE-SEA, HOLT, NR25 7RZ

TITLE: CONNECTION DETAILS - 1

SCALE	DATE	SHEET SIZE
1:10	11/12/20	A3

PROJECT NO.	DRAWING NO.			REV.
8962MCN	301	MR	MO	P3

STRUCTURAL SOLUTIONS
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0117 924 5014