

Project MARUBBI'S CAFE, 10 BANK STREET, WREXHAM				Job Ref. BMS217	
Section CALCULATIONS				Sheet no./rev. 1	
Calc. by JC	Date 07/10/2022	Chk'd by DJD	Date 07/10/2022	App'd by BC	Date 07/10/2022

Project: **MARUBBI'S CAFE,
10 BANK STREET,
WREXHAM
LL11 1AH.**

Client: **MR HARI GOULD**

Brief: **DESIGN NEW BEAMS TO ALTERATIONS.
& SPECIFY PADSTONES.
THESE CALCULATIONS ARE FOR BUILDING REGULATION
APPROVAL.
OUR BRIEF DOES NOT INCLUDE FOR ANY DRAWINGS.**

**British
Standards:** **BS648 - Weights
BS6399 - Loadings
EN1993 - Steelwork
BS5628 - Masonry**

References: **SITE VISIT**

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Loadings

Exg roof: -

TILES	=	0.50
F + B	=	0.10
RAFTERS	=	0.05
PURLINS	=	0.05
TRUSSES	=	<u>0.05</u>
		0.75/cos 30°
	=	0.87
CEILING	=	0.11
CEILING SUPER	=	0.25
INSULATION	=	0.02
ROOF SUPER	=	<u>0.75</u>
		<u>2.00 kN/m²</u>

2nd & 1st floor (Timber): -

BOARDS	=	0.15
JOISTS	=	0.15
CEILING	=	0.13
INSULATION	=	0.02
SUPER	=	1.50
PARTITIONS	=	<u>0.25</u>
		<u>2.30 kN/m²</u>

walls: -

4½" BRICK + PLASTER	=	<u>2.88 kN/m²</u>
STUD PARTITION	=	<u>0.50 kN/m²</u>
102 BRICK + PLASTER	=	<u>2.15 kN/m²</u>
100 BLOCK + PLASTER	=	<u>1.20 kN/m²</u>
9" BRICK + PLASTER	=	<u>5.20 kN/m²</u>
13" BRICK + PLASTER	=	<u>7.50 kN/m²</u>
NEW CAVITY WALL	=	<u>3.60 kN/m²</u>

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Member Sizes (Grade S355JR)

CE Marking – Consequences Class 2, Execution Class EXC2

ALL SPANS, DIMENSIONS, HEIGHTS, LEVELS etc. TO BE CONFIRMED ON SITE BY CONTRACTOR/FABRICATOR PRIOR TO ANY FABRICATION

Beam B1 over Kitchen

2No 152 × 89 × 16 kg/m UB's Grade S355

Fixed together with M12 bolts and spacers @ 500mm crs

Combined Padstones

225mm x 225mm x 150mm deep each end

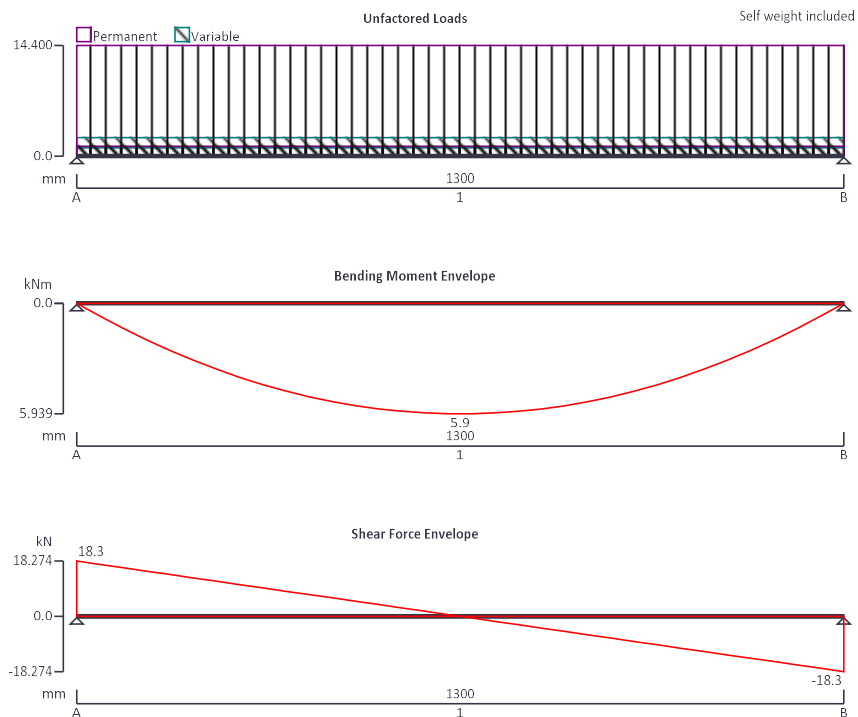
DO NOT USE BEAM SPANS FOR FABRICATION PURPOSES

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Steel beam analysis & design (EN1993-1-1:2005) - Beam B1 over Kitchen (INNER - Worst Case)

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

TEDDS calculation version 3.0.14



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam $\times 1$

1.1m x ROOF - Permanent full UDL 1.1 kN/m

1.1m x ROOF - Variable full UDL 1.1 kN/m

1.6m x 1st FLOOR - Permanent full UDL 1.28 kN/m

1.6m x 1st FLOOR - Variable full UDL 2.4 kN/m

5m x WALL - Permanent full UDL 14.4 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 $\times 1.50$

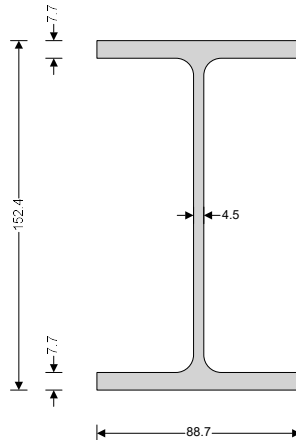
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Analysis results

Maximum moment	$M_{max} = 5.9 \text{ kNm}$	$M_{min} = 0 \text{ kNm}$
Maximum shear	$V_{max} = 18.3 \text{ kN}$	$V_{min} = -18.3 \text{ kN}$
Deflection	$\delta_{max} = 0.4 \text{ mm}$	$\delta_{min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 18.3 \text{ kN}$	$R_{A_min} = 18.3 \text{ kN}$
Unfactored permanent load reaction at support A	$R_{A_Permanent} = 11 \text{ kN}$	
Unfactored variable load reaction at support A	$R_{A_Variable} = 2.3 \text{ kN}$	
Maximum reaction at support B	$R_{B_max} = 18.3 \text{ kN}$	$R_{B_min} = 18.3 \text{ kN}$
Unfactored permanent load reaction at support B	$R_{B_Permanent} = 11 \text{ kN}$	
Unfactored variable load reaction at support B	$R_{B_Variable} = 2.3 \text{ kN}$	

Section details

Section type	UB 152x89x16 (BS4-1)
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	$K_{LT,A} = 1.000$
	$K_{LT,B} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section	$c = d = 121.8 \text{ mm}$
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	$c / t_w = 29.3 \times \varepsilon \leq 72 \times \varepsilon$	Class 1
Outstand flanges - Table 5.2 (sheet 2 of 3)		
Width of section	$c = (b - t_w - 2 \times r) / 2 = 34.5 \text{ mm}$	
	$c / t_f = 4.8 \times \varepsilon \leq 9 \times \varepsilon$	Class 1
		Section is class 1
Check shear - Section 6.2.6		
Height of web	$h_w = h - 2 \times t_f = 137 \text{ mm}$	
Shear area factor	$\eta = 1.000$	
	$h_w / t_w < 72 \times \varepsilon / \eta$	
		Shear buckling resistance can be ignored
Design shear force	$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 18.3 \text{ kN}$	
Shear area - cl 6.2.6(3)	$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = 818 \text{ mm}^2$	
Design shear resistance - cl 6.2.6(2)	$V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = 129.8 \text{ kN}$	
		PASS - Design shear resistance exceeds design shear force
Check bending moment major (y-y) axis - Section 6.2.5		
Design bending moment	$M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 5.9 \text{ kNm}$	
Design bending resistance moment - eq 6.13	$M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \times f_y / \gamma_{M0} = 33.9 \text{ kNm}$	
Slenderness ratio for lateral torsional buckling		
Correction factor - Table 6.6	$k_c = 0.94$	
	$C_1 = 1 / k_c^2 = 1.132$	
Curvature factor	$g = \sqrt{[1 - (I_z / I_y)]} = 0.945$	
Poissons ratio	$\nu = 0.3$	
Shear modulus	$G = E / [2 \times (1 + \nu)] = 80769 \text{ N/mm}^2$	
Unrestrained length	$L = 1.0 \times L_{s1} = 1300 \text{ mm}$	
Elastic critical buckling moment	$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = 116.8 \text{ kNm}$	
Slenderness ratio for lateral torsional buckling	$\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \times f_y / M_{cr})} = 0.539$	
Limiting slenderness ratio	$\bar{\lambda}_{LT,0} = 0.4$	
	$\bar{\lambda}_{LT} > \bar{\lambda}_{LT,0}$	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	$\alpha_{LT} = 0.34$	
Correction factor for rolled sections	$\beta = 0.75$	
LTB reduction determination factor	$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 0.632$	
LTB reduction factor - eq 6.57	$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.944$	
Modification factor	$f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.974$	
Modified LTB reduction factor - eq 6.58	$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.969$	
Design buckling resistance moment - eq 6.55	$M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = 32.8 \text{ kNm}$	
		PASS - Design buckling resistance moment exceeds design bending moment
Check vertical deflection - Section 7.2.1		
Consider deflection due to permanent and variable loads		
Limiting deflection	$\delta_{lim} = L_{s1} / 360 = 3.6 \text{ mm}$	
Maximum deflection span 1	$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = 0.434 \text{ mm}$	
		PASS - Maximum deflection does not exceed deflection limit

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ASSUMPTIONS AND NOTES TO CONTRACTOR

- 1) Assumptions made in the design are to be confirmed on site by the Principal Contractor and Crawford & Co to be informed immediately if found to be different.

- 2) All existing dimensions, wall thicknesses, heights, sizes of existing beams, joist spans etc to be checked on site prior to any fabrication. Any discrepancies to be reported immediately to the Client/Architect and ourselves.

- 3) The Principal Contractor is to allow for exposing existing beams on site so that precise connection details to new beams can be determined.

- 4) The Principal Contractor is to visit the Property and familiarise themselves with the existing structure. Any areas of concern are to be brought to the attention of the Client/Architect and ourselves.

- 5) The Principal Contractor is responsible for all temporary propping during construction and installation of new beams. Any movement as a result of inadequate propping must be repaired at the Contractors own cost.

- 6) New steelwork to be Grade S355/JR CE Marking – Consequences Class 2, Execution Class EXC2 and to be thoroughly cleaned and wire brushed prior to painting with two coats of high build zinc phosphate primer to a minimum dry film thickness of 75 microns per coat unless noted otherwise.

- 7) New steelwork hidden in cavity's to be galvanised or painted with two coats of black Bitumen.

- 8) All welds to be 6mm Continuous Fillet Weld unless noted otherwise.

- 9) All bolts to be Grade 8.8. Minimum 2No M20 bolts per connection unless noted otherwise.

- 10) These calculations to be read in conjunction with the Architects drawings and The Health and Safety Plan.

- 10) Concrete for padstones/base, to be grade GEN 3 minimum.

- 12) Lintols to be installed in accordance with manufacturers instructions.

- 13) Nothing in the contract confers nor purports to confer on any third party any benefit nor any right to enforce any terms of this Contract pursuant to the Contracts (Right of Third Parties) Act 1999.