

STRUCTURAL CALCULATIONS



THE PROPERTY

918 Manchester Road,
Bury.
BL9 8DW

PREPARED FOR:

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REPORT REFERENCE:

6523NW-CAL-001-0

SURVEY DATE:

25th November 2021

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STRUCTURAL CALCULATIONS ON *918 Manchester Road, Bury. BL9 8DW*

Prepared by: Mo Makki BSc, MSc

Signed:



Date: 9th February 2022

Checked by: Daniel Pearson BSc (Hons), GMICE

Signed:



Date: 9th February 2022

Approved by: Joe Trotter MEng, CEng, MStructE

Signed:



Date: 9th February 2022

Issue	Revision	Revised by	Revised Date

The appointment of Structural Surveys Ltd shall be governed by, and construed in all respects, in accordance with the laws of England & Wales and each party submits to the exclusive jurisdiction of the Court of England and Wales.

Prepared for: Alex Martin

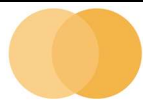
Address of Property: 918 Manchester Road, Bury. BL9 8DW

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**STRUCTURAL
SURVEYS**

Innovation Centre, Sci-Tech Daresbury

Keckwick Lane, Daresbury

Warrington, WA4 4FS

Project

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Scope of Design

This report presents the calculations for structural alterations at 918 Manchester Road in Bury BL9 8DW. Our remit in this report is to provide calculations for the structural elements as followings:

Element ID	Structural Form	Material	Loading
Beam 1	Beam – simply supported	Steel	Flat roof
Beam2	Beam – simply supported	Steel	Existing beam

Note:

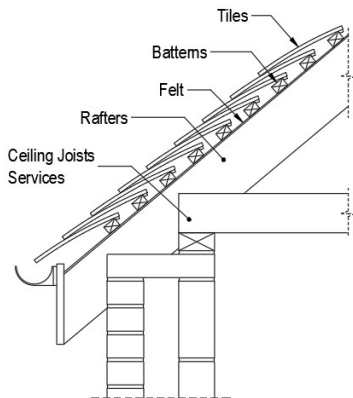
- Structural Surveys Ltd has been employed to design the structural members above. We assume the works will be carried out in accordance with Building Regulations Approved Document Part 'A'.
- The following calculations are based on information recorded during a visual survey, should these differ from what is shown in the design sketches, Structural Survey Ltd are to be informed. We may need to assess the assumptions again.

SKETCHES

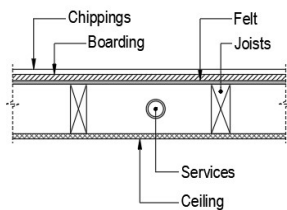
The full details can be found in the structural drawings.

Loading Data

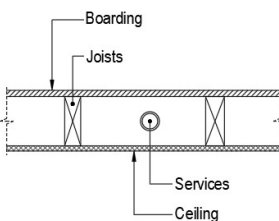
The following load cases will be considered for the design of all structural elements.



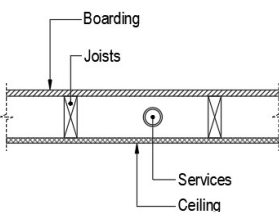
Timber Pitched Roof		Load kN/m ²
Dead Load	Tiles	0.75
	Battens	0.05
	Felt	0.05
	Rafters	0.20
	Ceiling/Services	0.15
On Slope	Total	1.20
On Plan 15deg		1.25
On Plan 30deg		1.39
Imposed Load	Maintenance	0.75



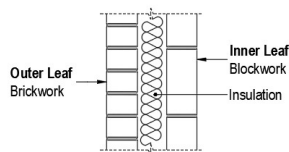
Timber Flat Roof		Load kN/m ²
Dead Load	Chippings	0.20
	Boarding	0.15
	Felt	0.10
	Joists	0.20
	Ceiling/Services	0.15
	Total	0.80
Imposed Load	Maintenance	0.60



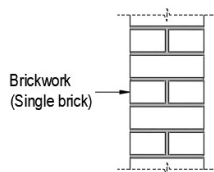
Timber Floor		Load kN/m ²
Dead Load	Boarding	0.15
	Joists	0.30
	Ceiling/Services	0.15
	Total	0.60
Imposed Load	Domestic	1.50



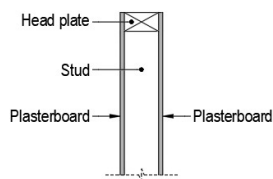
Loft Floor		Load kN/m ²
Dead Load	Boarding	0.05
	Joists	0.10
	Ceiling/Services	0.15
	Total	0.30
Imposed Load	Light Storage	0.25


Masonry Cavity Wall Load kN/m²

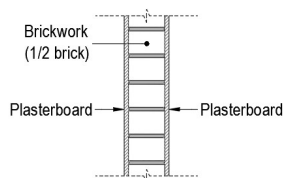
Dead Load	Inner leaf	2.00
	Outerleaf	2.80
	Insulation	0.10
Total		4.90


Masonry Solid Wall (single brick) Load kN/m²

Dead Load	Solid Wall	4.50
Total		4.50


Timber Stud Wall Load kN/m²

Dead Load	Stud Wall	0.50
Total		0.50


Masonry Partition Wall Load kN/m²

Dead Load	Masonry Wall	2.00
	Plaster	0.20
Total		2.20

Wind Load on Wall Load kN/m²

Imposed Load	Wind	0.70
Total		0.70

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Loading

Calculations are prepared in accordance with the following Standards:

- Weight of building materials BS 648
- Loading BS 6399, BS EN 1991
- Structural concrete BS 8110, BS EN 1992
- Structural steel BS 5950, BS EN 1993
- Structural timber BS 5268, BS EN 1995

Line loads for the beam were then determined using the load cases above. The line loads are presented in the below table:

Beam	Clear Span (m)	No. of Beams	No. of Storeys Loading	Effective Floor Width (m)	Effective Roof Width (m)	Wall Height (m)	Line Loads (kN/m)				
							Floor		Wall	Roof	
							Perm	Imp	Perm	Perm	Imp
1	3.20	1	1	-	1.50	0.3	-	-	0.66	1.20	1.13
Extg	3.10	1	1	1.70	2.50	3.0	1.02	4.25	14.70	3.00	1.88
2	2.40	1	1	-	-	0.3	-	-	0.66	-	-

Design Principles and Assumptions

1. It is assumed that we are performing "Designer" duties only in accordance with CDM 2015.
2. This document should be read in conjunction with our structural drawings and risk assessment.
3. Beam lengths should not be taken from this calculation. The lengths shown are for the clear span i.e. distance between supports. Any discrepancies should be brought to our attention at the earliest opportunity.
4. Contractors and Fabricators are to check required beam lengths on site before ordering/fabrication.
5. It is the Contractors responsibility to provide adequate temporary propping during construction.
6. We have assumed a safe ground bearing capacity of 150kN/m². Any concerns over the condition of the ground should be brought to our attention at the earliest convenience.
7. It is recommended that the Contractor excavates trial pits to determine the existing foundations.
8. Where possible, steel fabrication and welding should be completed off-site in a quality controlled environment.
9. All bolts to be grade 8.8 unless noted otherwise.
10. All steel to be grade S275 JR unless noted otherwise.
11. All steel to be given minimum 30 minutes fire protection unless noted otherwise.
12. All below ground steelwork must be concrete encased with minimum 100mm thickness.
13. Calculations to be checked by Local Authority before work commences. Client to ensure all of contractors' works on site to comply with and meet Approval of the relevant British Standards and the Local Authority including Building Control and Planning Departments.



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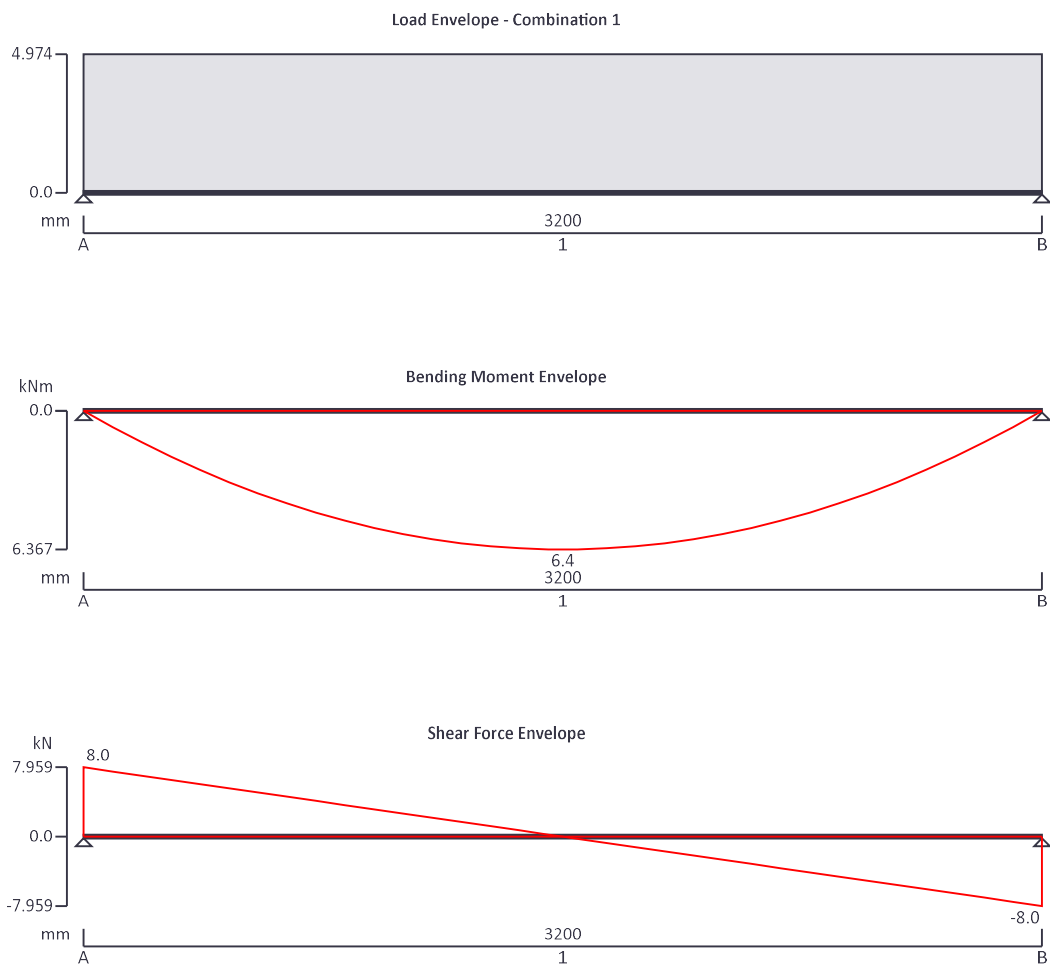
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BEAM 1

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 3.0.14



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

Applied loading

Beam loads

Permanent self weight of beam 1



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Wall - Permanent full UDL 1 kN/m

Flat Roof DL - Permanent full UDL 1.22 kN/m

Flat Roof IL - Variable full UDL 1.15 kN/m

Load combinations

Load combination 1

Support A

Permanent γ 1.35

Variable γ 1.50

Permanent γ 1.35

Variable γ 1.50

Support B

Permanent γ 1.35

Variable γ 1.50

Analysis results

Maximum moment;

$M_{\max} = 6.4$ kNm;

$M_{\min} = 0$ kNm

Maximum shear;

$V_{\max} = 8$ kN;

$V_{\min} = -8$ kN

Deflection;

$\delta_{\max} = 1.7$ mm;

$\delta_{\min} = 0$ mm

Maximum reaction at support A;

$R_{A_{\max}} = 8$ kN;

$R_{A_{\min}} = 8$ kN

Unfactored permanent load reaction at support A; $R_{A_{\text{Permanent}}} = 3.9$ kN

Unfactored variable load reaction at support A; $R_{A_{\text{Variable}}} = 1.8$ kN

Maximum reaction at support B;

$R_{B_{\max}} = 8$ kN;

$R_{B_{\min}} = 8$ kN

Unfactored permanent load reaction at support B; $R_{B_{\text{Permanent}}} = 3.9$ kN

Unfactored variable load reaction at support B; $R_{B_{\text{Variable}}} = 1.8$ kN

Section details

Section type;

UB 178x102x19 (BS4-1)

Steel grade;

S355

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

Nominal thickness of element;

$t = \max(t_f, t_w) = 7.9$ mm

Nominal yield strength;

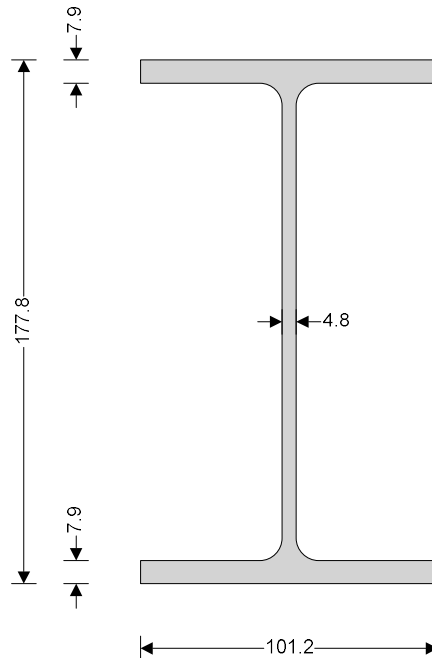
$f_y = 355$ N/mm²

Nominal ultimate tensile strength;

$f_u = 470$ N/mm²

Modulus of elasticity;

$E = 210000$ N/mm²



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; $K_{LT,A} = 1.200; + 2 \sqrt{h}$
 $K_{LT,B} = 1.200; + 2 \sqrt{h}$

Classification of cross sections - Section 5.5

$$\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$$

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

Width of section; $c = d = 146.8 \text{ mm}$

$$c / t_w = 37.6 \sqrt{\varepsilon} \leq 72 \sqrt{\varepsilon}; \quad \text{Class 1}$$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section; $c = (b - t_w - 2 \sqrt{r}) / 2 = 40.6 \text{ mm}$

$$c / t_f = 6.3 \sqrt{\varepsilon} \leq 9 \sqrt{\varepsilon}; \quad \text{Class 1}$$

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Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 \cdot t_f = 162 \text{ mm}$

Shear area factor; $\eta = 1.000$

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

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 8 \text{ kN}$

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 \cdot b \cdot t_f + (t_w + 2 \cdot r) \cdot t_f, \eta \cdot h_w \cdot t_w) = 985 \text{ mm}^2$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v \cdot (f_y / \sqrt{3}) / \gamma_{M0} = 201.9 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 6.4 \text{ kNm}$

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} \cdot f_y / \gamma_{M0} = 60.8 \text{ kNm}$

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; $g = \sqrt{1 - (I_z / I_y)} = 0.948$

Poissons ratio; $\nu = 0.3$

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

Unrestrained length; $L = 1.2 \cdot L_{s1} + 2 \cdot h = 4196 \text{ mm}$

Elastic critical buckling moment; $M_{cr} = C_1 \cdot \pi^2 \cdot E \cdot I_z / (L^2 \cdot g) \cdot \sqrt{[I_w / I_z + L^2 \cdot G \cdot I_t / (\pi^2 \cdot E \cdot I_z)]} = 32.9 \text{ kNm}$

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} \cdot f_y / M_{cr})} = 1.359$

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; $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = 1.356$

LTB reduction factor - eq 6.57; $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = 0.493$

Modification factor; $f = \min(1 - 0.5 \cdot (1 - k_c) \cdot [1 - 2 \cdot (\bar{\lambda}_{LT} - 0.8)^2], 1) = 0.989$

Modified LTB reduction factor - eq 6.58; $\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = 0.498$

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Design buckling resistance moment - eq 6.55; $M_{b,Rd} = \chi_{LT,mod} \cdot W_{pl,y} \cdot f_y / \gamma_{M1} = 30.3 \text{ kNm}$

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; $\delta_{lim} = L_{s1} / 250 = 12.8 \text{ mm}$

Maximum deflection span 1; $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = 1.705 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

; A 2No.178x102x19 UB S355 beams are required.

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WALL BEARING/ UNDER BEAM1 (LEFT SIDE OF BEAM1 INTERNAL & EXTERNAL)

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.12

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	15.9 kN	42.5 kN	0.95 N/mm ²	2.13 N/mm ²	0.449	Pass

Masonry panel details

Panel length;	L = 4000 mm
Panel height;	h = 2500 mm
Thickness of load bearing leaf;	t = 100 mm
Effective height;	h _{ef} = 2500 mm
Effective thickness;	t _{ef} = 126 mm

Masonry material details

Unit type;	Aggregate concrete - Group 1
Compressive strength of masonry unit;	f _c = 5.2 N/mm ²
Height of unit;	h _u = 215 mm
Width of unit;	w _u = 100 mm
Conditioning factor;	k = 1.0
- Conditioning to the air dry condition in accordance with cl.7.3.2	
Shape factor - Table A.1;	d _{sf} = 1.38
Mean compressive strength of masonry unit;	f _b = f _c ¹ k ¹ d _{sf} = 7.18 N/mm ²
Specific weight of units;	γ = 18 kN/m ³
Mortar type;	M6 - General Purpose
Compressive strength of mortar;	f _m = 6.0 N/mm ²
Compressive strength factor - Tbl. NA 4;	K = 0.75
Characteristic compressive strength - eq. 3.1;	f _k = K ¹ f _b ^{0.7} ¹ f _m ^{0.3} = 5.10 N/mm ²
Short term secant modulus of elasticity factor;	K _E = 1000
Modulus of elasticity - cl.3.7.2;	E _w = K _E ¹ f _k = 5101 N/mm ²

Design compressive strength of masonry

Category of manufacturing control;	Category II
------------------------------------	-------------



Class of execution control;

Class 2

Partial factor for compressive strength;

$\gamma_M = 3.00$

Cross-sectional area of wall;

$A = L \cdot t = 0.40 \text{ m}^2$;

Design compressive strength of masonry;

$f_d = f_k / \gamma_M = 1.70 \text{ N/mm}^2$

Partial safety factors for design loads

Partial safety factor for permanent load;

$\gamma_{FG} = 1.35$

Partial safety factor for variable load;

$\gamma_{FQ} = 1.50$

Superimposed vertical loading details

Permanent UDL at top of wall;

$g_k = 0.00 \text{ kN/m}$

Variable UDL at top of wall;

$q_k = 0.00 \text{ kN/m}$

Eccentricity of permanent UDL load;

$e_{gu} = 0 \text{ mm}$

Eccentricity of variable UDL load;

$e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit;

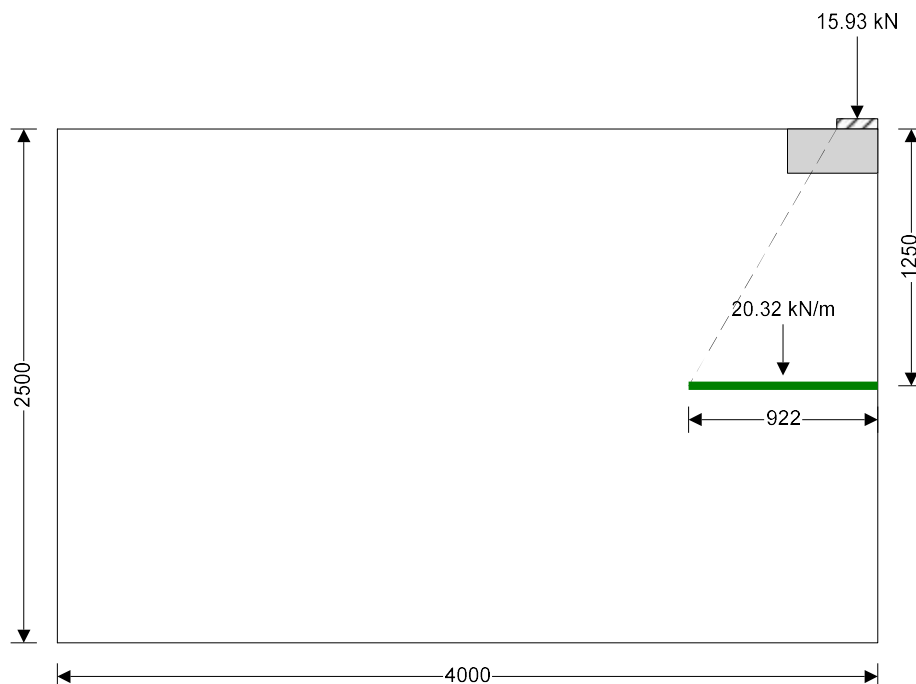
$\lambda_{lim} = 27$

Slenderness ratio;

$\lambda = h_{ef} / t_{ef} = 19.8$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - Beam 1 Load



Permanent concentrated load;

$G_{kc1} = 7.80 \text{ kN}$

Variable concentrated load;

$Q_{kc1} = 3.60 \text{ kN}$

Eccentricity of concentrated load; $e_{c1} = 0$ mm
Length of concentrated load; $L_{c1} = 200$ mm
Width of concentrated load; $w_{c1} = 100$ mm
Height of concentrated load; $h_{c1} = 2500$ mm
Distance of load to right vertical edge; $r_{11} = 0$ mm
Distance of load to nearest vertical edge; $a_{11} = 0$ mm

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check; $e_{c1} \leq t / 4$

PASS - Eccentricity of load is less than $t/4$

Area of bearing; $A_{b1} = L_{c1} \times w_{c1} = 20000$ mm²
Effective length of bearing at mid-height; $l_{efm1} = L_{c1} + h_{c1} / 2 \times \tan(30) + r_{11} = 922$ mm
Effective bearing area; $A_{ef1} = l_{efm1} \times t = 92169$ mm²
Bearing area ratio check; $A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = 0.22$
Initial enhancement factor; $\beta_{init1} = \text{Max}((1 + 0.3 \times a_{11} / h_{c1}) \times (1.5 - 1.1 \times A_{ratio1}), 1.0) = 1.26$

Maximum enhancement factor; $\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \times h_{c1}), 1.5) = 1.25$

Enhancement factor for concentrated loads; $\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = 1.25$

Design value of the concentrated load; $N_{Edc1} = G_{kc1} \times \gamma_{fG} + Q_{kc1} \times \gamma_{fQ} = 15.93$ kN

Design value concentrated load resistance; $N_{Rdc1} = \beta_1 \times A_{b1} \times f_d = 42.51$ kN

PASS - Design resistance exceeds applied concentrated load

Design of spreader beam

Type of spreader;

Concrete padstone

Type of bearing onto spreader;

Point load

Location of load from RHS of spreader;

$P_{11} = 100$ mm

Length of spreader;

$L_{sp1} = 440$ mm

Height of spreader;

$h_{sp1} = 215$ mm

Width of spreader;

$w_{sp1} = 100$ mm

Eccentricity of load on spreader;

$e_{sp1} = 0$ mm

Modulus of elasticity;

$E_{sp1} = 29962$ N/mm²

Second moment of area;

$I_{sp1} = 1/12 \times w_{sp1} \times h_{sp1}^3 = 82819792$ mm⁴

Modulus of the wall;

$k_0 = E_w / h = 2.04$ N/mm²/mm

Winkler's constant;

$K_{c1} = k_0 \times w_{sp1} = 204.03$ N/mm/mm

Characteristic of the system;

$\alpha_1 = (K_{c1} / (4 \times E_{sp1} \times I_{sp1}))^{1/4} = 0.00213$ mm⁻¹

Classification of spreader;

$\alpha L_1 = \alpha_1 \times L_{sp1} = 0.94$; **Medium**

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Krilov's functions for the spreader length;

$$B_{\alpha L1} = 1/2 \cdot (\cosh(\alpha L_1) \cdot \sin(180 \cdot \alpha L_1 / \pi) + \sinh(\alpha L_1) \cdot \cos(180 \cdot \alpha L_1 / \pi)) = \mathbf{0.91}$$

$$C_{\alpha L1} = 1/2 \cdot (\sinh(\alpha L_1) \cdot \sin(180 \cdot \alpha L_1 / \pi) = \mathbf{0.44}$$

$$D_{\alpha L1} = 1/4 \cdot (\cosh(\alpha L_1) \cdot \sin(180 \cdot \alpha L_1 / \pi) - \sinh(\alpha L_1) \cdot \cos(180 \cdot \alpha L_1 / \pi)) = \mathbf{0.14}$$

Krilov's functions at the point load;

$$A_{\alpha P11} = \cosh(\alpha_1 \cdot P_{11}) \cdot \cos(180 \cdot \alpha_1 \cdot P_{11} / \pi) = \mathbf{1.00}$$

$$B_{\alpha P11} = 1/2 \cdot (\cosh(\alpha_1 \cdot P_{11}) \cdot \sin(180 \cdot \alpha_1 \cdot P_{11} / \pi) + \sinh(\alpha_1 \cdot P_{11}) \cdot \cos(180 \cdot \alpha_1 \cdot P_{11} / \pi)) = \mathbf{0.21}$$

Using method of initial conditions

Initial moment of LH edge;

$$M_{01} = \mathbf{0 \text{ kNm}}$$

Initial shear of LH edge;

$$V_{01} = \mathbf{0 \text{ kN}}$$

Which gives;

$$(4 \cdot \alpha_1^2 \cdot C_{\alpha L1} \cdot \delta_{01} + 4 \cdot \alpha_1 \cdot D_{\alpha L1} \cdot \Phi_{01}) \cdot E_{sp1} \cdot I_{sp1} - B_{\alpha P11} / \alpha_1 \cdot N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

and;

$$(4 \cdot \alpha_1^3 \cdot B_{\alpha L1} \cdot \delta_{01} + 4 \cdot \alpha_1^2 \cdot C_{\alpha L1} \cdot \Phi_{01}) \cdot E_{sp1} \cdot I_{sp1} - A_{\alpha P11} \cdot N_{Edc1} = \mathbf{0.00 \text{ kN}}$$

Therefore,

Initial deflection of LH edge;

$$\delta_{01} = \mathbf{-0.11276 \text{ mm}}$$

Initial rotation of LH edge;

$$\Phi_{01} = \mathbf{0.001317}$$

Location of maximum deflection;

$$x_{def1} = \mathbf{440 \text{ mm}}$$

Krilov's functions at the spreader length;

$$A_{\alpha xdef1} = \cosh(\alpha_1 \cdot x_{def1}) \cdot \cos(180 \cdot \alpha_1 \cdot x_{def1} / \pi) = \mathbf{0.87}$$

$$B_{\alpha xdef1} = 1/2 \cdot (\cosh(\alpha_1 \cdot x_{def1}) \cdot \sin(180 \cdot \alpha_1 \cdot x_{def1} / \pi) + \sinh(\alpha_1 \cdot x_{def1}) \cdot \cos(180 \cdot \alpha_1 \cdot x_{def1} / \pi)) = \mathbf{0.91}$$

Distance of point load right of loaction;

$$p_{1def1} = \mathbf{100 \text{ mm}}$$

Krilov's functions at the spreader length;

$$D_{\alpha p1def1} = 1/4 \cdot (\cosh(\alpha_1 \cdot p_{1def1}) \cdot \sin(180 \cdot \alpha_1 \cdot p_{1def1} / \pi) - \sinh(\alpha_1 \cdot p_{1def1}) \cdot \cos(180 \cdot \alpha_1 \cdot p_{1def1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load;

$$\delta'_{11} = D_{\alpha p1def1} / \alpha_1^3 \cdot N_{Edc1} / (I_{sp1} \cdot E_{sp1}) = \mathbf{0.001 \text{ mm}}$$

Maximum deflection;

$$\delta_{max1} = A_{\alpha xdef1} \cdot \delta_{01} + B_{\alpha xdef1} \cdot \Phi_{01} / \alpha_1 + \delta'_{11} = \mathbf{0.467 \text{ mm}}$$

Location of maximum moment;

$$x_{M1} = \mathbf{340 \text{ mm}}$$

Krilov's functions at the spreader length;

$$C_{\alpha xM1} = 1/2 \cdot (\sinh(\alpha_1 \cdot x_{M1}) \cdot \sin(180 \cdot \alpha_1 \cdot x_{M1} / \pi) = \mathbf{0.26}$$

$$D_{\alpha xM1} = 1/4 \cdot (\cosh(\alpha_1 \cdot x_{M1}) \cdot \sin(180 \cdot \alpha_1 \cdot x_{M1} / \pi) - \sinh(\alpha_1 \cdot x_{M1}) \cdot \cos(180 \cdot \alpha_1 \cdot x_{M1} / \pi)) = \mathbf{0.06}$$

Distance of point load right of loaction;

$$p_{1M1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length;

$$B_{\alpha p1M1} = 1/2 \cdot (\cosh(\alpha_1 \cdot p_{1M1}) \cdot \sin(180 \cdot \alpha_1 \cdot p_{1M1} / \pi) + \sinh(\alpha_1 \cdot p_{1M1}) \cdot \cos(180 \cdot \alpha_1 \cdot p_{1M1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load;

$$M'_1 = -B_{\alpha p1M1} / \alpha_1 \cdot N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

Maximum moment;

$$M_{Edsp1} = (4 \cdot \alpha_1^2 \cdot C_{\alpha xM1} \cdot \delta_{01} + 4 \cdot \alpha_1 \cdot D_{\alpha xM1} \cdot \Phi_{01}) \cdot (I_{sp1} \cdot E_{sp1}) + M'_1 = \mathbf{0.43 \text{ kNm}}$$

Location of maximum shear;

$$x_{V1} = \mathbf{340 \text{ mm}}$$

Krilov's functions at the spreader length;

$$B_{\alpha xV1} = 1/2 \cdot (\cosh(\alpha_1 \cdot x_{V1}) \cdot \sin(180 \cdot \alpha_1 \cdot x_{V1} / \pi) + \sinh(\alpha_1 \cdot x_{V1}) \cdot \cos(180 \cdot \alpha_1 \cdot x_{V1} / \pi)) = \mathbf{0.72}$$

$$C_{\alpha xV1} = 1/2 \cdot \sinh(\alpha_1 \cdot x_{V1}) \cdot \sin(180 \cdot \alpha_1 \cdot x_{V1} / \pi) = \mathbf{0.26}$$

Distance of point load right of loaction;

$$p_{1V1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length;

$$A_{\alpha p1V1} = \cosh(\alpha_1 \cdot p_{1V1}) \cdot \cos(180 \cdot \alpha_1 \cdot p_{1V1} / \pi) = \mathbf{1.00}$$

Particular integral due to load;

$$V'_1 = -A_{\alpha p1V1} \cdot N_{Edc1} = \mathbf{-15.93 \text{ kN}}$$

Shear at concentrated point load;

$$V_1 = (4 \cdot \alpha_1^3 \cdot B_{\alpha xV1} \cdot \delta_{01} + 4 \cdot \alpha_1^2 \cdot C_{\alpha xV1} \cdot \Phi_{01}) \cdot (I_{sp1} \cdot E_{sp1}) + V'_1 = \mathbf{-8.20 \text{ kN}}$$

Maximum shear;

$$V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = \mathbf{8.20 \text{ kN}}$$

Maximum allowable stress under spreader;

$$\sigma_{Rdsp1} = \beta_1 \cdot f_d = \mathbf{2.13 \text{ N/mm}^2}$$

Maximum reaction;

$$N_{Edsp1} = K_{c1} \cdot \delta_{max1} = \mathbf{95.35 \text{ kN/m}}$$

Design stress;

$$\sigma_{Edsp1} = N_{Edsp1} / w_{sp1} = \mathbf{0.95 \text{ N/mm}^2}$$

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section 6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$$e_{gmu1} = e_{gu} \cdot h_{c1} / (2 \cdot h) = \mathbf{0.0 \text{ mm}}$$

Eccentricity of variable UDL at mid-height below concentrated load

$$e_{qmu1} = e_{qu} \cdot h_{c1} / (2 \cdot h) = \mathbf{0.0 \text{ mm}}$$

Eccentricity of concentrated load at mid-height;

$$e_{mc1} = e_{c1} / 2 = \mathbf{0.0 \text{ mm}}$$

Initial eccentricity - cl.5.5.1.1(4);

$$e_{init} = h_{ef} / 450 = \mathbf{5.6 \text{ mm}}$$

Concentrated load at mid-height as UDL;

$$N_{mc1} = N_{Edc1} / l_{efm1} = \mathbf{17.28 \text{ kN/m}}$$

Vertical load at mid-height;

$$N_{Ed1} = (g_k + \gamma \cdot t \cdot (h - h_{c1} / 2)) \cdot \gamma_{fG} + q_k \cdot \gamma_{fQ} + N_{mc1} = \mathbf{20.32 \text{ kN/m}}$$

kN/m

Design moment at mid-height;

$$M_{Ed1} = g_k \cdot \gamma_{fG} \cdot e_{gmu1} + q_k \cdot \gamma_{fQ} \cdot e_{qmu1} + N_{mc1} \cdot e_{mc1} = \mathbf{0.00 \text{ kNm/m}}$$

kNm/m

Eccentricities due to loads - eq. 6.7;

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = \mathbf{5.6 \text{ mm}}$$

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Slenderness ratio limit for creep eccentricity; $\lambda_c = 27$

Eccentricity due to creep; $e_{k1} = 0.0$ mm

Eccentricity at mid-height - eq. 6.6; $e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \sqrt{t}) = 5.6$ mm

From eq. G2; $A_{11} = 1 - 2 \sqrt{e_{mk1} / t} = 0.89$

From eq. G3; $u_1 = (h_{ef} / t_{ef} \sqrt{(1 / K_E)^{1/2} - 0.063}) / (0.73 - 1.17 \sqrt{e_{mk1} / t}) = 0.85$

Capacity reduction factor - eq. G1; $\Phi_{m1} = A_{11} \exp(-(u_1^2) / 2) = 0.62$

Design vertical resistance of panel - eq.6.2; $N_{Rd1} = \Phi_{m1} \sqrt{t} \sqrt{f_d} = 105.42$ kN/m

PASS - Design value of vertical resistance exceeds applied vertical load

WALL BEARING/ UNDER BEAM1 (RIGHT SIDE OF BEAM1 INTERNAL)

MASONRY BEARING DESIGN

In accordance with EN1996-1-1:2005 + A1:2012, incorporating Corrigenda February 2006 and July 2009 and the UK National Annex.

Tedds calculation version 1.0.12

Summary table

Load	Local concentration		Spreader		Utilisation	
	Design force	Resistance	Design stress	Resistance		
1	27.8 kN	51.3 kN	1.23 N/mm ²	1.71 N/mm ²	0.717	Pass

Masonry panel details

Panel length; $L = 780$ mm

Panel height; $h = 2500$ mm

Thickness of load bearing leaf; $t = 100$ mm

Effective height; $h_{ef} = 400$ mm

Effective thickness; $t_{ef} = 126$ mm

Masonry material details

Unit type; **Aggregate concrete - Group 1**

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Compressive strength of masonry unit; $f_c = 5.2 \text{ N/mm}^2$
Height of unit; $h_u = 215 \text{ mm}$
Width of unit; $w_u = 100 \text{ mm}$
Conditioning factor; $k = 1.0$
- Conditioning to the air dry condition in accordance with cl.7.3.2
Shape factor - Table A.1; $d_{sf} = 1.38$
Mean compressive strength of masonry unit; $f_b = f_c \cdot k \cdot d_{sf} = 7.18 \text{ N/mm}^2$
Specific weight of units; $\gamma = 18 \text{ kN/m}^3$
Mortar type; **M6 - General Purpose**
Compressive strength of mortar; $f_m = 6.0 \text{ N/mm}^2$
Compressive strength factor - Tbl. NA 4; $K = 0.75$
Characteristic compressive strength - eq. 3.1; $f_k = K \cdot f_b^{0.7} \cdot f_m^{0.3} = 5.10 \text{ N/mm}^2$
Short term secant modulus of elasticity factor; $K_E = 1000$
Modulus of elasticity - cl.3.7.2; $E_w = K_E \cdot f_k = 5101 \text{ N/mm}^2$

Design compressive strength of masonry

Category of manufacturing control; **Category II**
Class of execution control; **Class 2**
Partial factor for compressive strength; $\gamma_M = 3.00$
Cross-sectional area of wall; $A = L \cdot t = 0.08 \text{ m}^2$; ($A < 0.1 \text{ m}^2$)
Design compressive strength of masonry; $f_d = f_k / \gamma_M \cdot (0.7 + 3 \cdot A / 1 \text{ m}^2) = 1.59 \text{ N/mm}^2$

Partial safety factors for design loads

Partial safety factor for permanent load; $\gamma_{FG} = 1.35$
Partial safety factor for variable load; $\gamma_{FQ} = 1.50$

Superimposed vertical loading details

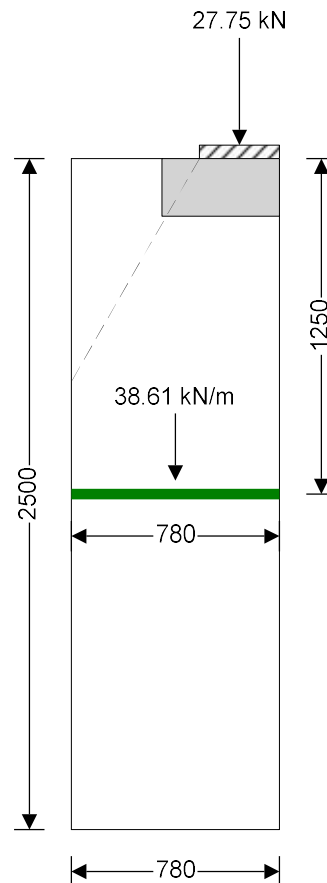
Permanent UDL at top of wall; $g_k = 0.00 \text{ kN/m}$
Variable UDL at top of wall; $q_k = 0.00 \text{ kN/m}$
Eccentricity of permanent UDL load; $e_{gu} = 0 \text{ mm}$
Eccentricity of variable UDL load; $e_{qu} = 0 \text{ mm}$

Slenderness ratio of masonry wall - Section 5.5.1.4

Slenderness ratio limit; $\lambda_{lim} = 27$
Slenderness ratio; $\lambda = h_{ef} / t_{ef} = 3.2$

PASS - Slenderness ratio is less than slenderness limit

Concentrated Load 1 details - Internal Beam 1 Load + 2No. Beam 2 Load + Existing Lintel



Permanent concentrated load;

$$G_{kc1} = 15.00 \text{ kN}$$

Variable concentrated load;

$$Q_{kc1} = 5.00 \text{ kN}$$

Eccentricity of concentrated load;

$$e_{c1} = 0 \text{ mm}$$

Length of concentrated load;

$$L_{c1} = 300 \text{ mm}$$

Width of concentrated load;

$$w_{c1} = 100 \text{ mm}$$

Height of concentrated load;

$$h_{c1} = 2500 \text{ mm}$$

Distance of load to right vertical edge;

$$r_{11} = 0 \text{ mm}$$

Distance of load to nearest vertical edge;

$$a_{11} = 0 \text{ mm}$$

Walls subjected to concentrated loads - Section 6.1.3

Eccentricity check;

$$e_{c1} \leq t / 4$$

PASS - Eccentricity of load is less than $t/4$

Area of bearing;

$$A_{b1} = L_{c1} \times w_{c1} = 30000 \text{ mm}^2$$

Effective length of bearing at mid-height;

$$l_{efm1} = L = 780 \text{ mm}$$

Effective bearing area;

$$A_{ef1} = l_{efm1} \times t = 78000 \text{ mm}^2$$

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Bearing area ratio check;	$A_{ratio1} = \text{Min}(A_{b1} / A_{ef1}, 0.45) = \mathbf{0.38}$
Initial enhancement factor;	$\beta_{init1} = \text{Max}((1 + 0.3 \sqrt{a_{11} / h_{c1}}) \sqrt{(1.5 - 1.1 \sqrt{A_{ratio1}}), 1.0) = \mathbf{1.08}$
Maximum enhancement factor;	$\beta_{max1} = \text{Min}(1.25 + a_{11} / (2 \sqrt{h_{c1}}), 1.5) = \mathbf{1.25}$
Enhancement factor for concentrated loads;	$\beta_1 = \text{Min}(\beta_{init1}, \beta_{max1}) = \mathbf{1.08}$
Design value of the concentrated load;	$N_{Edc1} = G_{kc1} \sqrt{\gamma_{fG}} + Q_{kc1} \sqrt{\gamma_{fQ}} = \mathbf{27.75 \text{ kN}}$
Design value concentrated load resistance;	$N_{Rdc1} = \beta_1 \sqrt{A_{b1}} f_d = \mathbf{51.30 \text{ kN}}$
PASS - Design resistance exceeds applied concentrated load	

Design of spreader beam

Type of spreader;	Concrete padstone
Type of bearing onto spreader;	Point load
Location of load from RHS of spreader;	$P_{11} = \mathbf{150 \text{ mm}}$
Length of spreader;	$L_{sp1} = \mathbf{440 \text{ mm}}$
Height of spreader;	$h_{sp1} = \mathbf{215 \text{ mm}}$
Width of spreader;	$w_{sp1} = \mathbf{100 \text{ mm}}$
Eccentricity of load on spreader;	$e_{sp1} = \mathbf{0 \text{ mm}}$
Modulus of elasticity;	$E_{sp1} = \mathbf{29962 \text{ N/mm}^2}$
Second moment of area;	$I_{sp1} = 1/12 \sqrt{w_{sp1}} \sqrt{h_{sp1}}^3 = \mathbf{82819792 \text{ mm}^4}$
Modulus of the wall;	$k_0 = E_w / h = \mathbf{2.04 \text{ N/mm}^2/\text{mm}}$
Winkler's constant;	$K_{c1} = k_0 \sqrt{w_{sp1}} = \mathbf{204.03 \text{ N/mm/mm}}$
Characteristic of the system;	$\alpha_1 = (K_{c1} / (4 \sqrt{E_{sp1}} \sqrt{I_{sp1}}))^{1/4} = \mathbf{0.00213 \text{ mm}^{-1}}$
Classification of spreader;	$\alpha L_1 = \alpha_1 \sqrt{L_{sp1}} = \mathbf{0.94; \text{Medium}}$
Krilov's functions for the spreader length;	$B_{\alpha11} = 1/2 \sqrt{(\cosh(\alpha L_1) \sqrt{\sin(180 \sqrt{\alpha L_1} / \pi)} + \sinh(\alpha L_1) \sqrt{\cos(180 \sqrt{\alpha L_1} / \pi)})} = \mathbf{0.91}$ $C_{\alpha11} = 1/2 \sqrt{\sinh(\alpha L_1) \sqrt{\sin(180 \sqrt{\alpha L_1} / \pi)}} = \mathbf{0.44}$ $D_{\alpha11} = 1/4 \sqrt{(\cosh(\alpha L_1) \sqrt{\sin(180 \sqrt{\alpha L_1} / \pi)} - \sinh(\alpha L_1) \sqrt{\cos(180 \sqrt{\alpha L_1} / \pi)})} = \mathbf{0.14}$
Krilov's functions at the point load;	$A_{\alpha P11} = \cosh(\alpha_1 \sqrt{P_{11}}) \sqrt{\cos(180 \sqrt{\alpha_1} \sqrt{P_{11}} / \pi)} = \mathbf{1.00}$ $B_{\alpha P11} = 1/2 \sqrt{(\cosh(\alpha_1 \sqrt{P_{11}}) \sqrt{\sin(180 \sqrt{\alpha_1} \sqrt{P_{11}} / \pi)} + \sinh(\alpha_1 \sqrt{P_{11}}) \sqrt{\cos(180 \sqrt{\alpha_1} \sqrt{P_{11}} / \pi)})} = \mathbf{0.32}$
Using method of initial conditions	
Initial moment of LH edge;	$M_{01} = \mathbf{0 \text{ kNm}}$
Initial shear of LH edge;	$V_{01} = \mathbf{0 \text{ kN}}$

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Which gives;

$$(4 - \alpha_1^2 - C_{\alpha 11} - \delta_{01} + 4 - \alpha_1 - D_{\alpha 11} - \Phi_{01}) - E_{sp1} - I_{sp1} - B_{\alpha P11} / \alpha_1 - N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

and;

$$(4 - \alpha_1^3 - B_{\alpha 11} - \delta_{01} + 4 - \alpha_1^2 - C_{\alpha 11} - \Phi_{01}) - E_{sp1} - I_{sp1} - A_{\alpha P11} - N_{Edc1} = \mathbf{0.00 \text{ kN}}$$

Therefore,

Initial deflection of LH edge;

$$\delta_{01} = \mathbf{0.01167 \text{ mm}}$$

Initial rotation of LH edge;

$$\Phi_{01} = \mathbf{0.001363}$$

Location of maximum deflection;

$$x_{def1} = \mathbf{440 \text{ mm}}$$

Krilov's functions at the spreader length;

$$A_{\alpha xdef1} = \cosh(\alpha_1 - x_{def1}) - \cos(180 - \alpha_1 - x_{def1} / \pi) = \mathbf{0.87}$$

$$B_{\alpha xdef1} = 1/2 - (\cosh(\alpha_1 - x_{def1}) - \sin(180 - \alpha_1 - x_{def1} / \pi) + \sinh(\alpha_1 - x_{def1}) - \cos(180 - \alpha_1 - x_{def1} / \pi)) = \mathbf{0.91}$$

Distance of point load right of loaction;

$$p_{1def1} = \mathbf{150 \text{ mm}}$$

Krilov's functions at the spreader length;

$$D_{\alpha p1def1} = 1/4 - (\cosh(\alpha_1 - p_{1def1}) - \sin(180 - \alpha_1 - p_{1def1} / \pi) - \sinh(\alpha_1 - p_{1def1}) - \cos(180 - \alpha_1 - p_{1def1} / \pi)) = \mathbf{0.01}$$

Particular integral due to load;

$$\delta'_1 = D_{\alpha p1def1} / \alpha_1^3 - N_{Edc1} / (I_{sp1} - E_{sp1}) = \mathbf{0.006 \text{ mm}}$$

Maximum deflection;

$$\delta_{max1} = A_{\alpha xdef1} - \delta_{01} + B_{\alpha xdef1} - \Phi_{01} / \alpha_1 + \delta'_1 = \mathbf{0.601 \text{ mm}}$$

Location of maximum moment;

$$x_{M1} = \mathbf{290 \text{ mm}}$$

Krilov's functions at the spreader length;

$$C_{\alpha xM1} = 1/2 - \sinh(\alpha_1 - x_{M1}) - \sin(180 - \alpha_1 - x_{M1} / \pi) = \mathbf{0.19}$$

$$D_{\alpha xM1} = 1/4 - (\cosh(\alpha_1 - x_{M1}) - \sin(180 - \alpha_1 - x_{M1} / \pi) - \sinh(\alpha_1 - x_{M1}) - \cos(180 - \alpha_1 - x_{M1} / \pi)) = \mathbf{0.04}$$

Distance of point load right of loaction;

$$p_{1M1} = \mathbf{0 \text{ mm}}$$

Krilov's functions at the spreader length;

$$B_{\alpha p1M1} = 1/2 - (\cosh(\alpha_1 - p_{1M1}) - \sin(180 - \alpha_1 - p_{1M1} / \pi) + \sinh(\alpha_1 - p_{1M1}) - \cos(180 - \alpha_1 - p_{1M1} / \pi)) = \mathbf{0.00}$$

Particular integral due to load;

$$M'_1 = -B_{\alpha p1M1} / \alpha_1 - N_{Edc1} = \mathbf{0.00 \text{ kNm}}$$

Maximum moment;

$$M_{Edsp1} = (4 - \alpha_1^2 - C_{\alpha xM1} - \delta_{01} + 4 - \alpha_1 - D_{\alpha xM1} - \Phi_{01}) - (I_{sp1} - E_{sp1}) + M'_1 = \mathbf{1.23 \text{ kNm}}$$

Location of maximum shear;

$$x_{V1} = \mathbf{290 \text{ mm}}$$

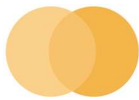
Krilov's functions at the spreader length;

$$B_{\alpha xV1} = 1/2 - (\cosh(\alpha_1 - x_{V1}) - \sin(180 - \alpha_1 - x_{V1} / \pi) + \sinh(\alpha_1 - x_{V1}) - \cos(180 - \alpha_1 - x_{V1} / \pi)) = \mathbf{0.61}$$

$$C_{\alpha xV1} = 1/2 - \sinh(\alpha_1 - x_{V1}) - \sin(180 - \alpha_1 - x_{V1} / \pi) = \mathbf{0.19}$$

Distance of point load right of loaction;

$$p_{1V1} = \mathbf{0 \text{ mm}}$$

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Krilov's functions at the spreader length; $A_{\alpha p1V1} = \cosh(\alpha_1 \cdot p_{1V1}) \cdot \cos(180 \cdot \alpha_1 \cdot p_{1V1} / \pi) = 1.00$

Particular integral due to load; $V'_1 = -A_{\alpha p1V1} \cdot N_{Edc1} = -27.75 \text{ kN}$

Shear at concentrated point load; $V_1 = (4 \cdot \alpha_1^3 \cdot B_{\alpha xV1} \cdot \delta_{01} + 4 \cdot \alpha_1^2 \cdot C_{\alpha xV1} \cdot \Phi_{01}) \cdot (I_{sp1} \cdot E_{sp1}) + V'_1 = -15.39 \text{ kN}$

Maximum shear; $V_{Edsp1} = \text{Max}(\text{Abs}(V_1), N_{Edc1} - \text{Abs}(V_1)) = 15.39 \text{ kN}$

Maximum allowable stress under spreader; $\sigma_{Rdsp1} = \beta_1 \cdot f_d = 1.71 \text{ N/mm}^2$

Maximum reaction; $N_{Edsp1} = K_{c1} \cdot \delta_{max1} = 122.54 \text{ kN/m}$

Design stress; $\sigma_{Edsp1} = N_{Edsp1} / w_{sp1} = 1.23 \text{ N/mm}^2$

PASS - Design stress under spreader is less than the allowable bearing stress

Walls subjected to mainly vertical loading - Section 6.1.2

Eccentricity of permanent UDL at mid-height below concentrated load

$$e_{gmu1} = e_{gu} \cdot h_{c1} / (2 \cdot h) = 0.0 \text{ mm}$$

Eccentricity of variable UDL at mid-height below concentrated load

$$e_{qmu1} = e_{qu} \cdot h_{c1} / (2 \cdot h) = 0.0 \text{ mm}$$

Eccentricity of concentrated load at mid-height;

$$e_{mc1} = e_{c1} / 2 = 0.0 \text{ mm}$$

Initial eccentricity - cl.5.5.1.1(4);

$$e_{init} = h_{ef} / 450 = 0.9 \text{ mm}$$

Concentrated load at mid-height as UDL;

$$N_{mc1} = N_{Edc1} / l_{efm1} = 35.58 \text{ kN/m}$$

Vertical load at mid-height;

$$N_{Ed1} = (g_k + \gamma \cdot t \cdot (h - h_{c1} / 2)) \cdot \gamma_{fG} + q_k \cdot \gamma_{fQ} + N_{mc1} = 38.61 \text{ kN/m}$$

Design moment at mid-height;

$$M_{Ed1} = g_k \cdot \gamma_{fG} \cdot e_{gmu1} + q_k \cdot \gamma_{fQ} \cdot e_{qmu1} + N_{mc1} \cdot e_{mc1} = 0.00 \text{ kNm/m}$$

Eccentricities due to loads - eq. 6.7;

$$e_{m1} = \text{Abs}(M_{Ed1}) / N_{Ed1} + e_{init} = 0.9 \text{ mm}$$

Slenderness ratio limit for creep eccentricity;

$$\lambda_c = 27$$

Eccentricity due to creep;

$$e_{k1} = 0.0 \text{ mm}$$

Eccentricity at mid-height - eq. 6.6;

$$e_{mk1} = \text{Max}(e_{m1} + e_{k1}, 0.05 \cdot t) = 5.0 \text{ mm}$$

From eq. G2;

$$A_{11} = 1 - 2 \cdot e_{mk1} / t = 0.90$$

From eq. G3;

$$u_1 = (h_{ef} / t_{ef} \cdot (1 / K_E)^{1/2} - 0.063) / (0.73 - 1.17 \cdot e_{mk1} / t) = 0.06$$

Capacity reduction factor - eq. G1;

$$\Phi_{m1} = A_{11} \cdot \exp(-(u_1^2) / 2) = 0.90$$

Design vertical resistance of panel - eq.6.2;

$$N_{Rd1} = \Phi_{m1} \cdot t \cdot f_d = 142.70 \text{ kN/m}$$

PASS - Design value of vertical resistance exceeds applied vertical load



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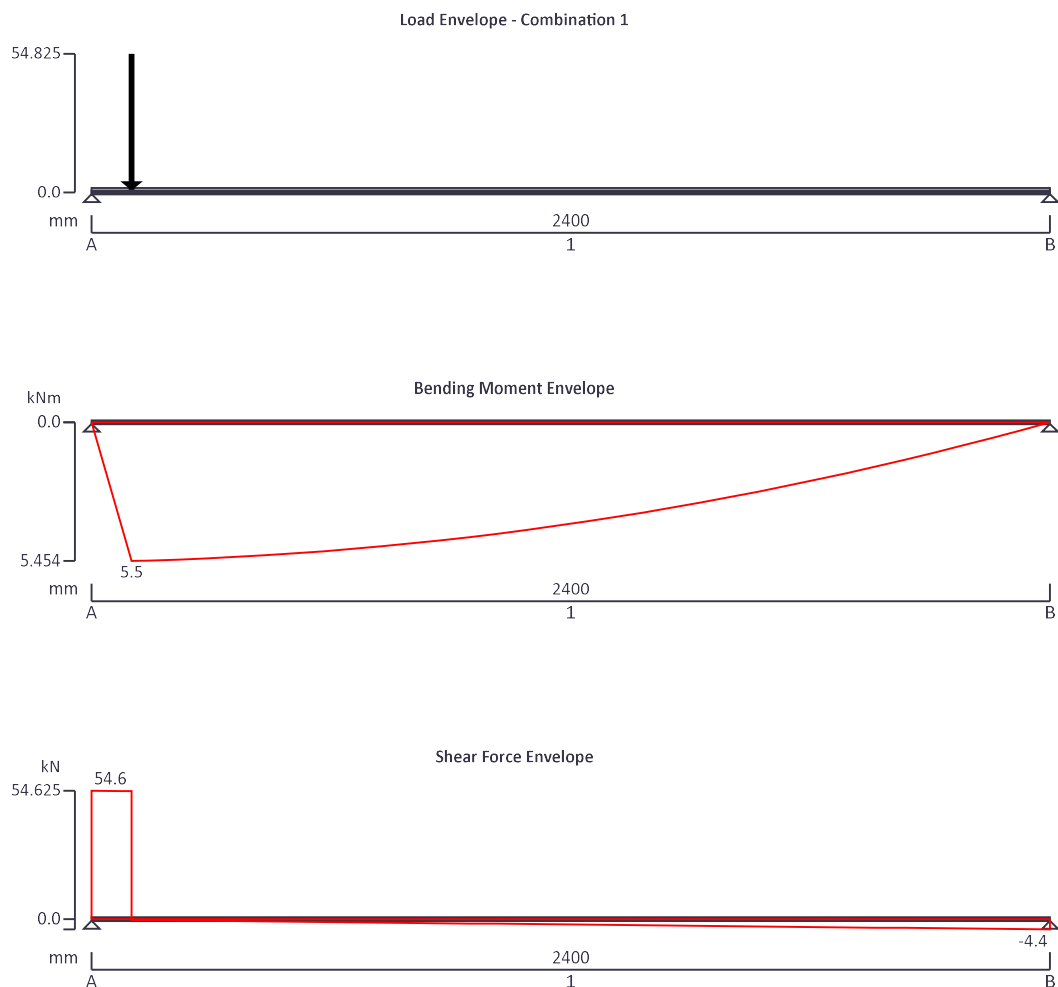
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BEAM 2

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 3.0.14



Support conditions

Support A

Vertically restrained

Rotationally free

Support B

Vertically restrained

Rotationally free

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Applied loading

Beam loads

Permanent self weight of beam $\gamma = 1$

Wall - Permanent full UDL 1.1 kN/m

Extg Beam PL - Permanent point load 29.5 kN at 100 mm

Extg Beam PL - Variable point load 10 kN at 100 mm

Load combinations

Load combination 1

Support A

Permanent $\gamma = 1.35$

Variable $\gamma = 1.50$

Permanent $\gamma = 1.35$

Variable $\gamma = 1.50$

Support B

Permanent $\gamma = 1.35$

Variable $\gamma = 1.50$

Analysis results

Maximum moment;

$M_{\max} = 5.5$ kNm;

$M_{\min} = 0$ kNm

Maximum shear;

$V_{\max} = 54.6$ kN;

$V_{\min} = -4.4$ kN

Deflection;

$\delta_{\max} = 0.7$ mm;

$\delta_{\min} = 0$ mm

Maximum reaction at support A;

$R_{A_{\max}} = 54.6$ kN;

$R_{A_{\min}} = 54.6$ kN

Unfactored permanent load reaction at support A; $R_{A_{\text{Permanent}}} = 29.8$ kN

Unfactored variable load reaction at support A; $R_{A_{\text{Variable}}} = 9.6$ kN

Maximum reaction at support B;

$R_{B_{\max}} = 4.4$ kN;

$R_{B_{\min}} = 4.4$ kN

Unfactored permanent load reaction at support B; $R_{B_{\text{Permanent}}} = 2.8$ kN

Unfactored variable load reaction at support B; $R_{B_{\text{Variable}}} = 0.4$ kN

Section details

Section type;

UB 178x102x19 (BS4-1)

Steel grade;

S355

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

Nominal thickness of element;

$t = \max(t_f, t_w) = 7.9$ mm

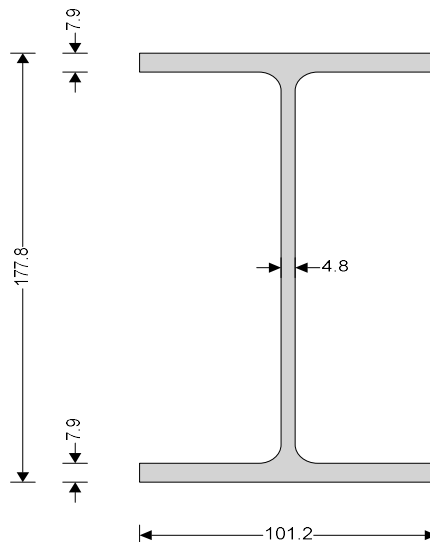
Nominal yield strength;

$f_y = 355$ N/mm²

Nominal ultimate tensile strength;

$f_u = 470$ N/mm²

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;

 $K_{LT,A} = 1.200; + 2 \sqrt{h}$
 $K_{LT,B} = 1.200; + 2 \sqrt{h}$

Classification of cross sections - Section 5.5

 $\varepsilon = \sqrt{[235 \text{ N/mm}^2 / f_y]} = 0.81$

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

Width of section;

 $c = d = 146.8 \text{ mm}$
 $c / t_w = 37.6 \sqrt{\varepsilon} \leq 72 \sqrt{\varepsilon}; \quad \text{Class 1}$

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section;

 $c = (b - t_w - 2 \sqrt{r}) / 2 = 40.6 \text{ mm}$

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$$c / t_f = 6.3 < \varepsilon = 9 < \varepsilon; \quad \text{Class 1}$$

Section is class 1

Check shear - Section 6.2.6

Height of web; $h_w = h - 2 t_f = \mathbf{162 \text{ mm}}$

Shear area factor; $\eta = \mathbf{1.000}$

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

Shear buckling resistance can be ignored

Design shear force; $V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{54.6 \text{ kN}}$

Shear area - cl 6.2.6(3); $A_v = \max(A - 2 b t_f + (t_w + 2 r) t_f, \eta h_w t_w) = \mathbf{985 \text{ mm}^2}$

Design shear resistance - cl 6.2.6(2); $V_{c,Rd} = V_{pl,Rd} = A_v (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{201.9 \text{ kN}}$

PASS - Design shear resistance exceeds design shear force

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

Design bending moment; $M_{Ed} = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = \mathbf{5.5 \text{ kNm}}$

Design bending resistance moment - eq 6.13; $M_{c,Rd} = M_{pl,Rd} = W_{pl,y} f_y / \gamma_{M0} = \mathbf{60.8 \text{ kNm}}$

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; $g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.948}$

Poissons ratio; $\nu = \mathbf{0.3}$

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

Unrestrained length; $L = 1.2 L_{s1} + 2 h = \mathbf{3236 \text{ mm}}$

Elastic critical buckling moment; $M_{cr} = C_1 \pi^2 E I_z / (L^2 g) \sqrt{[I_w / I_z + L^2 G I_t / (\pi^2 E I_z)]} = \mathbf{46.1 \text{ kNm}}$

Slenderness ratio for lateral torsional buckling; $\bar{\lambda}_{LT} = \sqrt{(W_{pl,y} f_y / M_{cr})} = \mathbf{1.148}$

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

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Imperfection factor - Table 6.3;

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

Correction factor for rolled sections;

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

LTB reduction determination factor;

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.122}$$

LTB reduction factor - eq 6.57;

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.610}$$

Modification factor;

$$f = \min(1 - 0.5 \sqrt{(1 - k_c) [1 - 2 \sqrt{(\bar{\lambda}_{LT} - 0.8)^2}]}, 1) = \mathbf{0.977}$$

Modified LTB reduction factor - eq 6.58;

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.624}$$

$$\text{Design buckling resistance moment - eq 6.55; } M_{b,Rd} = \chi_{LT,mod} \times W_{pl,y} \times f_y / \gamma_{M1} = \mathbf{37.9 \text{ kNm}}$$

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;

$$\delta_{lim} = L_{s1} / 250 = \mathbf{9.6 \text{ mm}}$$

Maximum deflection span 1;

$$\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{0.703 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

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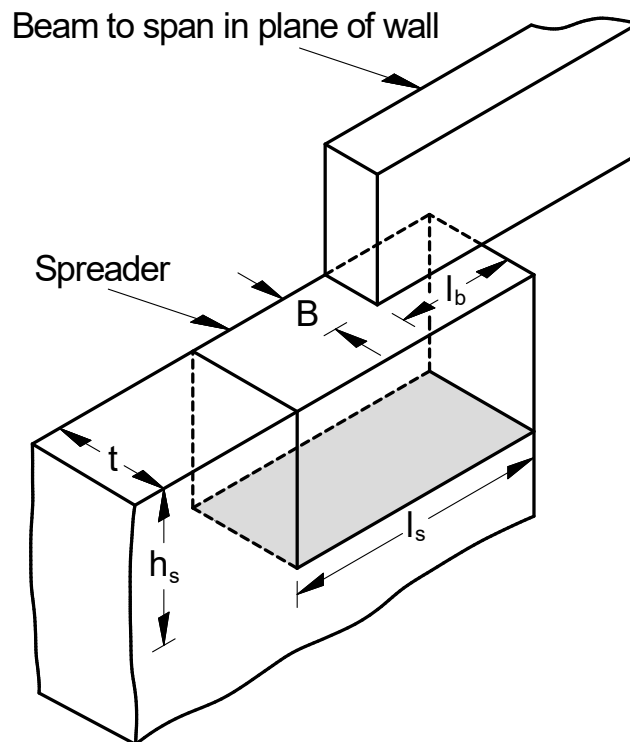
WALL BEARING/ UNDER BEAM 2

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;	$\rho_{\text{unit}} = 5.2 \text{ N/mm}^2$
Mortar designation;	ii
Least horizontal dimension of masonry units;	$l_{\text{unit}} = 100 \text{ mm}$
Height of masonry units;	$h_{\text{unit}} = 215 \text{ mm}$
Category of masonry units;	Category II
Category of construction control ;	Normal
Partial safety factor for material strength;	$\gamma_m = 3.5$
Thickness of load bearing leaf;	$t = 100 \text{ mm}$
Effective thickness of masonry wall;	$t_{\text{ef}} = 100 \text{ mm}$
Height of masonry wall;	$h = 2400 \text{ mm}$
Effective height of masonry wall;	$h_{\text{ef}} = 2400 \text{ mm}$



Bearing details

Beam spanning in plane of wall

Width of bearing; $B = 100 \text{ mm}$

Length of bearing; $l_b = 300 \text{ mm}$

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

;Mortar designation; Mortar = "ii"

;Block compressive strength; $p_{unit} = 5.2 \text{ N/mm}^2$

;;;;;Characteristic compressive strength (Table 2c); $f_{kc} = 2.50 \text{ N/mm}^2$

;;;;;Characteristic compressive strength (Table 2d); $f_{kd} = 5.00 \text{ N/mm}^2$

;Height of solid block; $h_{unit} = 215.0 \text{ mm}$;

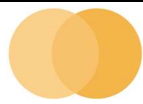
;Least horizontal dimension; $l_{unit} = 100.0 \text{ mm}$

Block ratio; $ratio = h_{unit} / l_{unit} = 2.2$

Ratio between 0.6 and 4.5 - OK

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Characteristic compressive strength;	$f_k = 5.00 \text{ N/mm}^2$
Loading details	
Characteristic concentrated dead load;	$G_k = 30 \text{ kN}$
Characteristic concentrated imposed load;	$Q_k = 10 \text{ kN}$
Design concentrated load;	$F = (G_k \times 1.4) + (Q_k \times 1.6) = 58.0 \text{ kN}$
Characteristic distributed dead load;	$g_k = 0.0 \text{ kN/m}$
Characteristic distributed imposed load;	$q_k = 0.0 \text{ kN/m}$
Design distributed load;	$f = (g_k \times 1.4) + (q_k \times 1.6) = 0.0 \text{ kN/m}$
Masonry bearing type	
Bearing type;	Type 1
Bearing safety factor;	$\gamma_{\text{bear}} = 1.25$
Check design bearing without a spreader	
Design bearing stress;	$f_{ca} = F / (B \times l_b) + f / t = 1.933 \text{ N/mm}^2$
Allowable bearing stress;	$f_{cp} = \gamma_{\text{bear}} \times f_k / \gamma_m = 1.786 \text{ N/mm}^2$
;FAIL - Design bearing stress exceeds allowable bearing stress, use a spreader	
Spreader details	
;Length of spreader;	$l_s = 440 \text{ mm}$
Depth of spreader;	$h_s = 215 \text{ mm}$
Edge distance;	$s_{\text{edge}} = \max(0 \text{ mm}, x_{\text{edge}} - (l_s - B) / 2) = 0 \text{ mm}$
Spreader bearing type	
;Bearing type;	Type 3
Bearing safety factor;	$\gamma_{\text{bear}} = 2.00$
Check design bearing with a spreader	
Loading acts eccentrically within middle third – triangular stress distribution	
Eccentricity of load;	$e = (l_s - l_b) / 2 = 70 \text{ mm}$
Maximum bearing stress;	$f_{ca} = F \times (1 + (6 \times e / l_s)) / (l_s \times t) + f / t = 2.576 \text{ N/mm}^2$
Allowable bearing stress;	$f_{cp} = \gamma_{\text{bear}} \times f_k / \gamma_m = 2.857 \text{ N/mm}^2$
PASS - Allowable bearing stress exceeds design bearing stress	



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Check design bearing at $0.4 \times h$ below the bearing level

Slenderness ratio; $h_{ef} / t_{ef} = \mathbf{24.00}$

Eccentricity at top of wall; $e_x = \mathbf{0.0}$ mm

From BS5628:1 Table 7

Capacity reduction factor; $\beta = \mathbf{0.61}$

Length of bearing distributed at $0.4 \times h$; $l_d = \mathbf{1260}$ mm

Maximum bearing stress; $f_{ca} = F / (l_d \times t) + f / t = \mathbf{0.460}$ N/mm²

Allowable bearing stress; $f_{cp} = \beta \times f_k / \gamma_m = \mathbf{0.864}$ N/mm²

PASS - Allowable bearing stress at $0.4 \times h$ below bearing level exceeds design bearing stress



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