



Structural Design Calculations

for

Internal Structural Alterations - Basement

Site address

Shrewton House
Elston Lane,
Salisbury,
Wiltshire, SP3 4HJ

March 2026
Ref: 12675/EO/SC01

Registered Office Address:

C/O Accrue Accounting, Unit E4 Arena Business Centre, Holyrood Close, Poole, Dorset, United Kingdom, BH17 7FP

 STRUCTURAL AND CIVIL ENGINEERS Unit 7, Boscombe Centre, Mills Way, Amesbury, Wiltshire, SP4 7SD  01980 677722  admin@jcpengineers.co.uk	Project Ref:	12675	
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Project type:	Proposed Internal Structural Alterations - Basement	Date:	March 26

Brief:

JCP Engineers was appointed to carry out the structural design for the proposed internal structural.

Basis of the Design:

The scope of work covers the following structural design works:

Part 1 – Basement (Reduced Floor Level Works):

- Underpinning design, typical details and sequencing notes;
- Reinforced concrete slab (reduced level) with slab edge (toe) details;
- Steel beam for load-bearing wall removal;
- Preparation of structural calculations;
- Preparation of general arrangement drawings to indicate the structural design

Part 2 – Option A – Stairs (Curved Ceiling Trimming):

- Structural drawings and calculations for the new stair arrangement and associated openings (trimming/lintels/local strengthening as required).

Part 3 – Option B – Lightwells:

- Prepare structural details/calculations for both:
 1. Existing coal shaft window openings (baseline local detailing); and
 2. Extended/enlarged lightwells taken down to the reduced basement floor level (additional retaining/edge/interface detailing as required).

Load Paths & Lateral Stability:

Vertical loads: Gravity loads from the ground level are supported on the load-bearing masonry walls and transferred down to the existing chalk bearing strata.

Horizontal loads: The reduced basement floor level is offset from the existing load-bearing masonry walls forming the basement; therefore, no surcharges are incurred.

Robustness & Avoidance of Disproportionate Collapse:

The building is classed as a Consequence Class 1 building under the Building Regulations Part A, with design guidance extract as follows:

“For Class 1 buildings – Provided the building has been designed and constructed in accordance with the rules given in this Approved Document, or other guidance referenced under Section 1, for meeting compliance with requirement A1 and A2 in normal use, no additional measures are likely to be necessary.”

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Design codes used:

BS 5628: 1992 "Code of Practice for use of Masonry"
BS 5950: 2000 "The Structural Use of Steel in Building"
BS 5977: 1983 "Lintels"
BS 6399: 1997 "Loadings for Buildings"
BS 8002: 1994 "Code of Practice for Earth Retaining Structures"
BS 8004: 1986 "Code of Practice for Foundations"
BS 8110: 1997 "Structural Use of Concrete"
BS 8500: 2006 "Concrete"
BS 4449: 2005 "Steel reinforcement"

Design software used:

Tekla Tedds 2025

Structural Analysis & Design

Notes:

These calculations only apply to the structural elements included in these documents; if any discrepancies are found on site, the Engineer is to be informed. All architectural and building design requirements are to be provided by others.

Dimensions in these calculations are for design purposes only, having been scaled from copies of drawings. The building contractor is to obtain detailed dimensions from site measurements and is not to rely on those provided herein for fabrication purposes or procurement of materials.

The contractor is responsible for ensuring the stability of the structure at all times and that the works are carried out in strict compliance with all relevant Codes of Practice, Building Regulations and good building practice. All temporary support works required during the course of construction are the responsibility of the contractor.

Throughout the works the contractor / client is to be responsible for ensuring that the requirements of the Construction, Design and Management Regulations are complied with. Before any demolition works are undertaken, an R&D asbestos survey is recommended.

The Engineer believes that there are no foreseeable unusual risks that may arise during construction, maintenance, or use that a competent contractor could not foresee; however, due attention is to be given to ensuring a safe method of working and risk assessment is planned.

All structural steelwork and components are to be manufactured in accordance with the requirements of execution class EXC2 to BS EN 1090-2.

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Architectural drawings:

863-E100	-	Existing Floor Plans
863-E120	-	Existing Sections
863-P100	-	Proposed Floor Plans
863-P120	-	Proposed Sections

Structural drawings:

12675-SK01	-	Structural General Arrangement
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1. General loading data:

Timber floor load

Joists	=	0.25	kN/m ²
Boards	=	0.25	kN/m ²
Plasterboard ceiling & skim coat	=	0.25	kN/m ²
Total floor dead load	=	0.75	kN/m ²
Floor imposed load (domestic / residential to BS 6399)	=	1.50	kN/m ²
Total floor load	=	2.25	kN/m ²

Concrete floor

150mm thick concrete floor	=	3.60	kN/m ²
Finishes	=	0.10	kN/m ²
Total floor dead load	=	3.70	kN/m ²
Floor imposed load (residential / domestic to BS 6399)	=	1.50	kN/m ²
Total floor load	=	5.20	kN/m ²

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2. Beams:

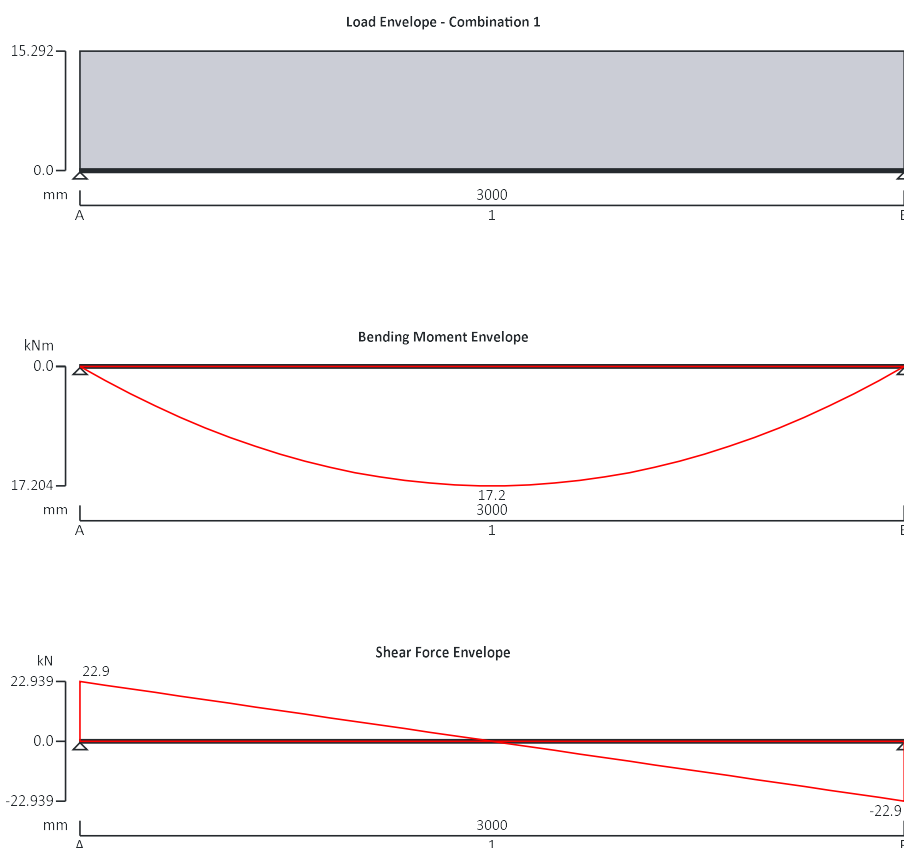
(Refer to the Engineer's general structural arrangement drawings, which indicate the structural design).

2.1. Beam BX:

Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

Support A Vertically restrained

Rotationally free

Support B Vertically restrained

Rotationally free

Applied loading

Beam loads Dead self weight of beam ' 1

2m floor width - Dead full UDL 7.2 kN/m

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2m floor width - Imposed full UDL 3 kN/m

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; $M_{max} = 17.2$ kNm; $M_{min} = 0$ kNm

Maximum shear; $V_{max} = 22.9$ kN; $V_{min} = -22.9$ kN

Deflection; $d_{max} = 0.9$ mm; $d_{min} = 0$ mm

Maximum reaction at support A; $R_{A_{max}} = 22.9$ kN; $R_{A_{min}} = 22.9$ kN

Unfactored dead load reaction at support A; $R_{A_{Dead}} = 11.2$ kN

Unfactored imposed load reaction at support A; $R_{A_{Imposed}} = 4.5$ kN

Maximum reaction at support B; $R_{B_{max}} = 22.9$ kN; $R_{B_{min}} = 22.9$ kN

Unfactored dead load reaction at support B; $R_{B_{Dead}} = 11.2$ kN

Unfactored imposed load reaction at support B; $R_{B_{Imposed}} = 4.5$ kN

Section details

Section type; **UC 152x152x30 (British Steel Section Range 2022 (BS4-1))**

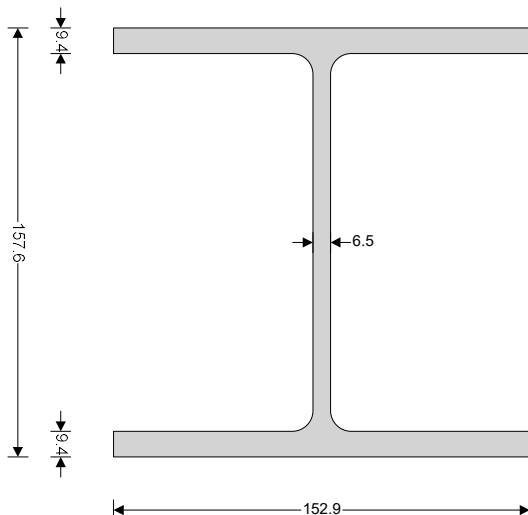
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; $\max(T, t) = 9.4$ mm

Design strength; $p_y = 355$ N/mm²

Modulus of elasticity; $E = 205000$ N/mm²



Lateral restraint

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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; $K_{LT,A} = 1.20$; + 2 ' D

$K_{LT,B} = 1.40$; + 2 ' D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = 123.6 \text{ mm}$

$d / t = 21.6$ ' $e \leq 80$ ' e ; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 76.5 \text{ mm}$

$b / T = 9.2$ ' $e \leq 10$ ' e ; Class 2 compact

Section is class 2 compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 22.9 \text{ kN}$

$d / t < 70$ ' e

Web does not need to be checked for shear buckling

Shear area; $A_v = t$ ' $D = 1024 \text{ mm}^2$

Design shear resistance; $P_v = 0.6$ ' p_y ' $A_v = 218.2 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 17.2 \text{ kNm}$

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y$ ' S_{xx} , 1.2 ' p_y ' $Z_{xx}) = 87.9 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.4$ ' $L_{s1} + 2$ ' $D = 4515 \text{ mm}$

Slenderness ratio; $I = L_E / r_{yy} = 117.976$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = 0.849$

Torsional index; $x = 15.999$

Slenderness factor; $v = 1 / [1 + 0.05 \times (I / x)^2]^{0.25} = 0.720$

Ratio - cl.4.3.6.9; $b_W = 1.000$

Equivalent slenderness - cl.4.3.6.7; $I_{LT} = u \times v \times I \times \sqrt{b_W} = 72.086$

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

$I_{LT} > I_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $a_{LT} = 7.0$

Perry factor; $h_{LT} = \max(a_{LT} \times (I_{LT} - I_{L0}) / 1000, 0) = 0.293$

Euler stress; $p_E = p^2 \times E / I_{LT}^2 = 389.4 \text{ N/mm}^2$

$f_{LT} = (p_y + (h_{LT} + 1) \times p_E) / 2 = 429.3 \text{ N/mm}^2$

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (f_{LT} + (f_{LT}^2 - p_E \times p_y)^{0.5}) = 214.7 \text{ N/mm}^2$

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Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB; $m_{LT} = 1.000$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \cdot S_{xx} = 53.2 \text{ kNm}$

$$M_b / m_{LT} = 53.2 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;; $d_{lim} = L_{s1} / 360 = 8.333 \text{ mm}$

Maximum deflection span 1; $d = \max(\text{abs}(d_{max}), \text{abs}(d_{min})) = 0.883 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

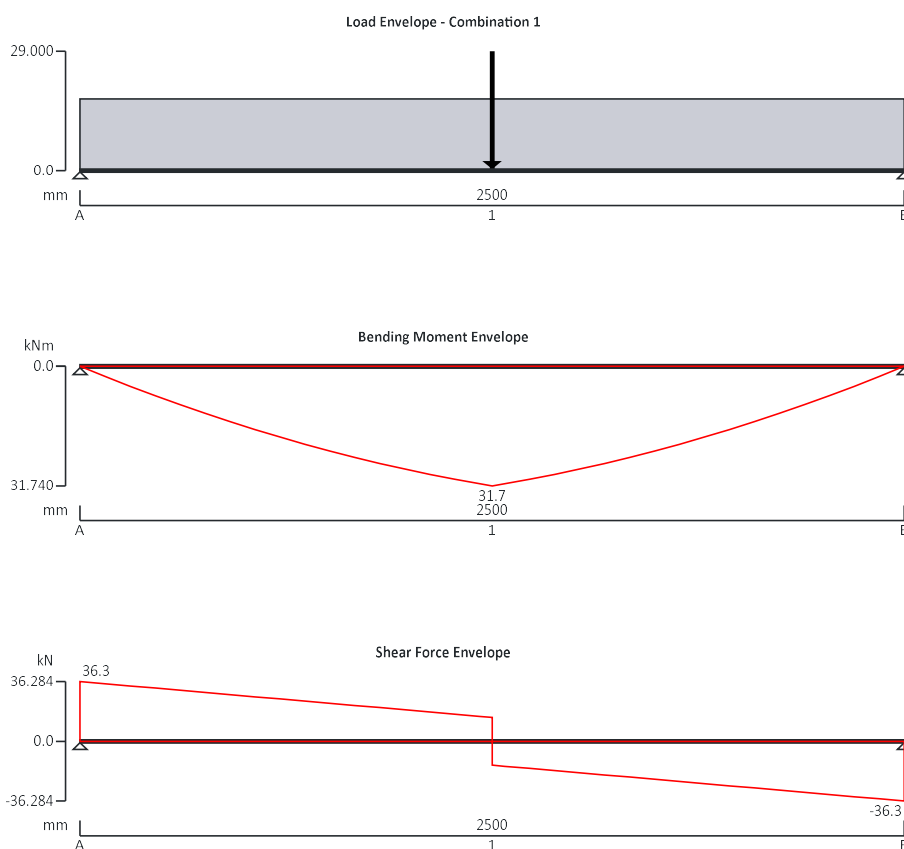
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2.2. Beam B2:

Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

Support A Vertically restrained

Rotationally free

Support B Vertically restrained

Rotationally free

Applied loading

Beam loads Dead self weight of beam ' 1

End reaction from B1 - Dead point load 15 kN at 1250 mm

End reaction from B1 - Imposed point load 5 kN at 1250 mm

2m floor width over - Dead full UDL 8.28 kN/m

2m floor width over - Imposed full UDL 3.45 kN/m

Load combinations

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Load combination 1 Support A Dead ' 1.40

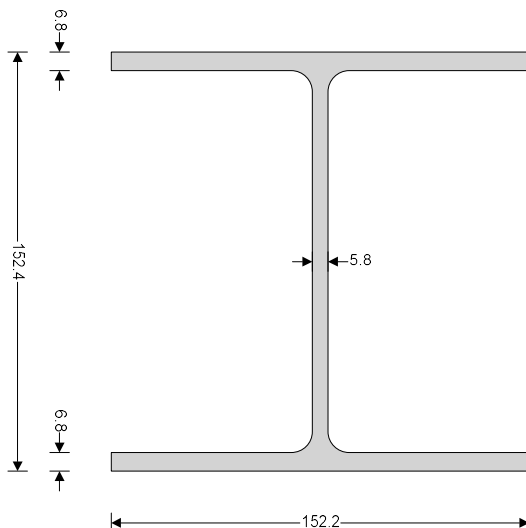
Imposed ' 1.60

Dead ' 1.40

Imposed ' 1.60

Support B Dead ' 1.40

Imposed ' 1.60

Analysis resultsMaximum moment; $M_{\max} = 31.7$ kNm; $M_{\min} = 0$ kNmMaximum shear; $V_{\max} = 36.3$ kN; $V_{\min} = -36.3$ kNDeflection; $d_{\max} = 1.3$ mm; $d_{\min} = 0$ mmMaximum reaction at support A; $R_{A_{\max}} = 36.3$ kN; $R_{A_{\min}} = 36.3$ kNUnfactored dead load reaction at support A; $R_{A_{\text{Dead}}} = 18.1$ kNUnfactored imposed load reaction at support A; $R_{A_{\text{Imposed}}} = 6.8$ kNMaximum reaction at support B; $R_{B_{\max}} = 36.3$ kN; $R_{B_{\min}} = 36.3$ kNUnfactored dead load reaction at support B; $R_{B_{\text{Dead}}} = 18.1$ kNUnfactored imposed load reaction at support B; $R_{B_{\text{Imposed}}} = 6.8$ kN**Section details**Section type; **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))**Steel grade; **S355****From table 9: Design strength p_y** Thickness of element; $\max(T, t) = 6.8$ mmDesign strength; $p_y = 355$ N/mm²Modulus of elasticity; $E = 205000$ N/mm²**Lateral restraint**

Span 1 has lateral restraint at supports only

Effective length factors



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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; $K_{LT,A} = 1.40; + 2 \cdot D$
 $K_{LT,B} = 1.40; + 2 \cdot D$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = 123.6 \text{ mm}$
 $d / t = 24.2 \cdot e \leq 80 \cdot e$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 76.1 \text{ mm}$
 $b / T = 12.7 \cdot e \leq 15 \cdot e$; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 36.3 \text{ kN}$
 $d / t < 70 \cdot e$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \cdot D = 884 \text{ mm}^2$
 Design shear resistance; $P_v = 0.6 \cdot p_y \cdot A_v = 188.3 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_max}), \text{abs}(M_{s1_min})) = 31.7 \text{ kNm}$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $b_{2f} = 10 \cdot e = 8.801$
 Limiting value for class 3 semi-compact flange; $b_{3f} = 15 \cdot e = 13.202$
 Limiting value for class 2 compact web; $b_{2w} = 100 \cdot e = 88.014$
 Limiting value for class 3 semi-compact web; $b_{3w} = 120 \cdot e = 105.617$

Effective plastic modulus - cl.3.5.6.2

$$S_{\text{eff}} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \cdot \min([(b_{3w} / (d / t))^2 - 1] / [(b_{3w} / b_{2w})^2 - 1]), [(b_{3f} / (b / T) - 1) / (b_{3f} / b_{2f} - 1)]), S_{xx}) = 170473 \text{ mm}^3$$

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \cdot S_{\text{eff}}, 1.2 \cdot p_y \cdot Z_{xx}) = 60.5 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.4 \cdot L_{s1} + 2 \cdot D = 3805 \text{ mm}$
 Slenderness ratio; $I = L_E / r_{yy} = 102.890$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = 0.840$
 Torsional index; $x = 20.701$
 Slenderness factor; $v = 1 / [1 + 0.05 \times (I / x)^2]^{0.25} = 0.818$
 Ratio - cl.4.3.6.9; $b_w = S_{\text{eff}} / S_{xx} = 0.937$

Equivalent slenderness - cl.4.3.6.7; $I_{LT} = u \times v \times I \times \sqrt{b_w} = 68.377$

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

$I_{LT} > I_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

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Robertson constant; $a_{LT} = 7.0$

Perry factor; $h_{LT} = \max(a_{LT} \times (l_{LT} - l_{L0}) / 1000, 0) = 0.267$

Euler stress; $p_E = p^2 \times E / l_{LT}^2 = 432.8 \text{ N/mm}^2$

$$f_{LT} = (p_y + (h_{LT} + 1) \times p_E) / 2 = 451.7 \text{ N/mm}^2$$

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (f_{LT} + (f_{LT}^2 - p_E \times p_y)^{0.5}) = 227.2 \text{ N/mm}^2$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB; $m_{LT} = 1.000$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff} = 38.7 \text{ kNm}$

$$M_b / m_{LT} = 38.7 \text{ kNm}$$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;; $d_{lim} = L_{s1} / 360 = 6.944 \text{ mm}$

Maximum deflection span 1; $d = \max(\text{abs}(d_{max}), \text{abs}(d_{min})) = 1.32 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit

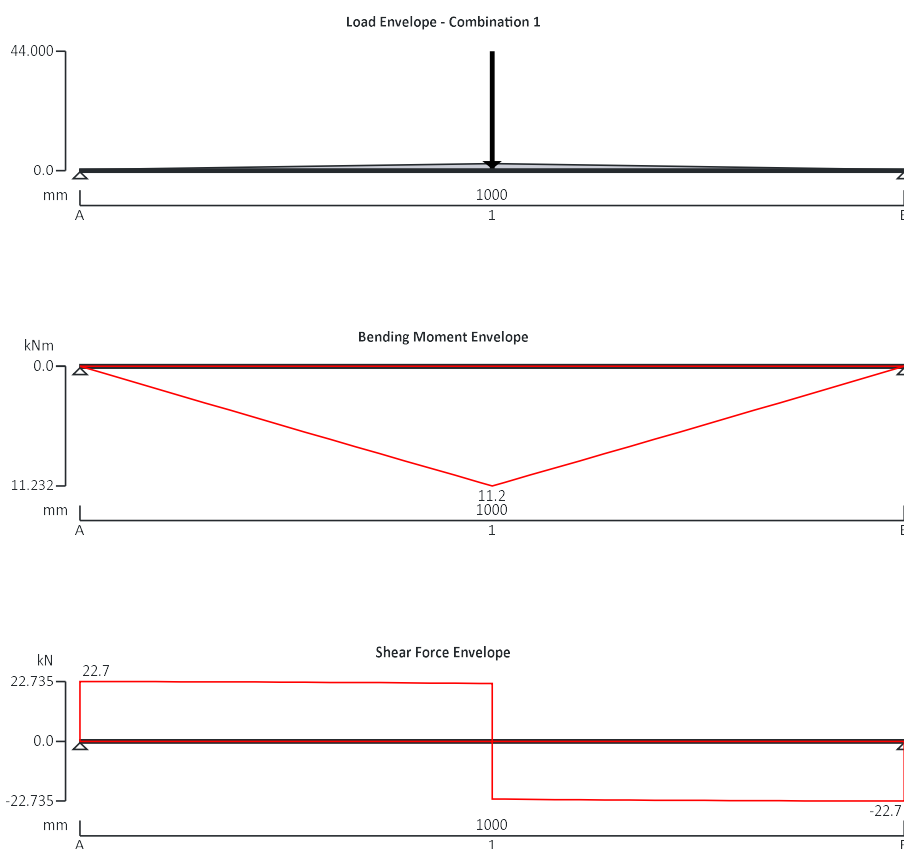
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2.3. Beam B3:

Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

Support A Vertically restrained

Rotationally free

Support B Vertically restrained

Rotationally free

Applied loading

Beam loads Dead self weight of beam ' 1

End reaction from B2 - Dead point load 20 kN at 500 mm

End reaction from B2 - Imposed point load 10 kN at 500 mm

Wall over - Dead trapezoidal load 1.65 kN/m from 500 mm to 500 mm

Load combinations

Load combination 1 Support A Dead ' 1.40

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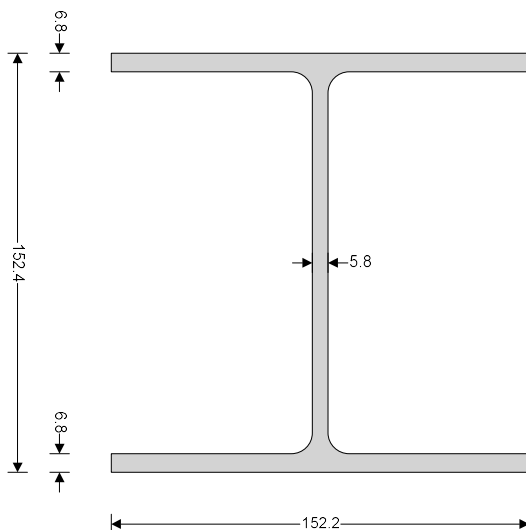
Imposed ' 1.60

Dead ' 1.40

Imposed ' 1.60

Support B Dead ' 1.40

Imposed ' 1.60

Analysis resultsMaximum moment; $M_{\max} = 11.2$ kNm; $M_{\min} = 0$ kNmMaximum shear; $V_{\max} = 22.7$ kN; $V_{\min} = -22.7$ kNDeflection; $d_{\max} = 0.1$ mm; $d_{\min} = 0$ mmMaximum reaction at support A; $R_{A_{\max}} = 22.7$ kN; $R_{A_{\min}} = 22.7$ kNUnfactored dead load reaction at support A; $R_{A_{\text{Dead}}} = 10.5$ kNUnfactored imposed load reaction at support A; $R_{A_{\text{Imposed}}} = 5$ kNMaximum reaction at support B; $R_{B_{\max}} = 22.7$ kN; $R_{B_{\min}} = 22.7$ kNUnfactored dead load reaction at support B; $R_{B_{\text{Dead}}} = 10.5$ kNUnfactored imposed load reaction at support B; $R_{B_{\text{Imposed}}} = 5$ kN**Section details**Section type; **UC 152x152x23 (British Steel Section Range 2022 (BS4-1))**Steel grade; **S355****From table 9: Design strength p_y** Thickness of element; $\max(T, t) = 6.8$ mmDesign strength; $p_y = 355$ N/mm²Modulus of elasticity; $E = 205000$ N/mm²**Lateral restraint**

Span 1 has lateral restraint at supports only

Effective length factorsEffective length factor in major axis; $K_x = 1.00$

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Effective length factor in minor axis; $K_y = 1.00$

Effective length factor for lateral-torsional buckling; $K_{LT,A} = 1.40; + 2 \cdot D$

$K_{LT,B} = 1.40; + 2 \cdot D$

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = 123.6 \text{ mm}$

$d / t = 24.2 \cdot e \leq 80 \cdot e$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 76.1 \text{ mm}$

$b / T = 12.7 \cdot e \leq 15 \cdot e$; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 22.7 \text{ kN}$

$d / t < 70 \cdot e$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \cdot D = 884 \text{ mm}^2$

Design shear resistance; $P_v = 0.6 \cdot p_y \cdot A_v = 188.3 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 11.2 \text{ kNm}$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $b_{2f} = 10 \cdot e = 8.801$

Limiting value for class 3 semi-compact flange; $b_{3f} = 15 \cdot e = 13.202$

Limiting value for class 2 compact web; $b_{2w} = 100 \cdot e = 88.014$

Limiting value for class 3 semi-compact web; $b_{3w} = 120 \cdot e = 105.617$

Effective plastic modulus - cl.3.5.6.2

$S_{\text{eff}} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \cdot \min(\frac{[(b_{3w} / (d / t))^2 - 1]}{[(b_{3w} / b_{2w})^2 - 1]}, \frac{[(b_{3f} / (b / T) - 1]}{[(b_{3f} / b_{2f}) - 1]}]), S_{xx}) = 170473 \text{ mm}^3$

Moment capacity low shear - cl.4.2.5.2; $M_c = \min(p_y \cdot S_{\text{eff}}, 1.2 \cdot p_y \cdot Z_{xx}) = 60.5 \text{ kNm}$

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling; $L_E = 1.4 \cdot L_{s1} + 2 \cdot D = 1705 \text{ mm}$

Slenderness ratio; $I = L_E / r_{yy} = 46.102$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter; $u = 0.840$

Torsional index; $x = 20.701$

Slenderness factor; $v = 1 / [1 + 0.05 \times (I / x)^2]^{0.25} = 0.946$

Ratio - cl.4.3.6.9; $b_w = S_{\text{eff}} / S_{xx} = 0.937$

Equivalent slenderness - cl.4.3.6.7; $I_{LT} = u \times v \times I \times \sqrt{b_w} = 35.443$

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

$I_{LT} > I_{L0}$ - Allowance should be made for lateral-torsional buckling

Bending strength - Section 4.3.6.5

Robertson constant; $a_{LT} = 7.0$

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Perry factor; $h_{LT} = \max(a_{LT} \times (l_{LT} - l_{L0}) / 1000, 0) = \mathbf{0.037}$

Euler stress; $p_E = p^2 \times E / l_{LT}^2 = \mathbf{1610.6 \text{ N/mm}^2}$

$f_{LT} = (p_y + (h_{LT} + 1) \times p_E) / 2 = \mathbf{1012.4 \text{ N/mm}^2}$

Bending strength - Annex B.2.1; $p_b = p_E \times p_y / (f_{LT} + (f_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{339.2 \text{ N/mm}^2}$

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB; $m_{LT} = \mathbf{1.000}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment; $M_b = p_b \times S_{eff} = \mathbf{57.8 \text{ kNm}}$

$M_b / m_{LT} = \mathbf{57.8 \text{ kNm}}$

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection;; $d_{lim} = L_{s1} / 360 = \mathbf{2.778 \text{ mm}}$

Maximum deflection span 1; $d = \max(\text{abs}(d_{max}), \text{abs}(d_{min})) = \mathbf{0.081 \text{ mm}}$

PASS - Maximum deflection does not exceed deflection limit

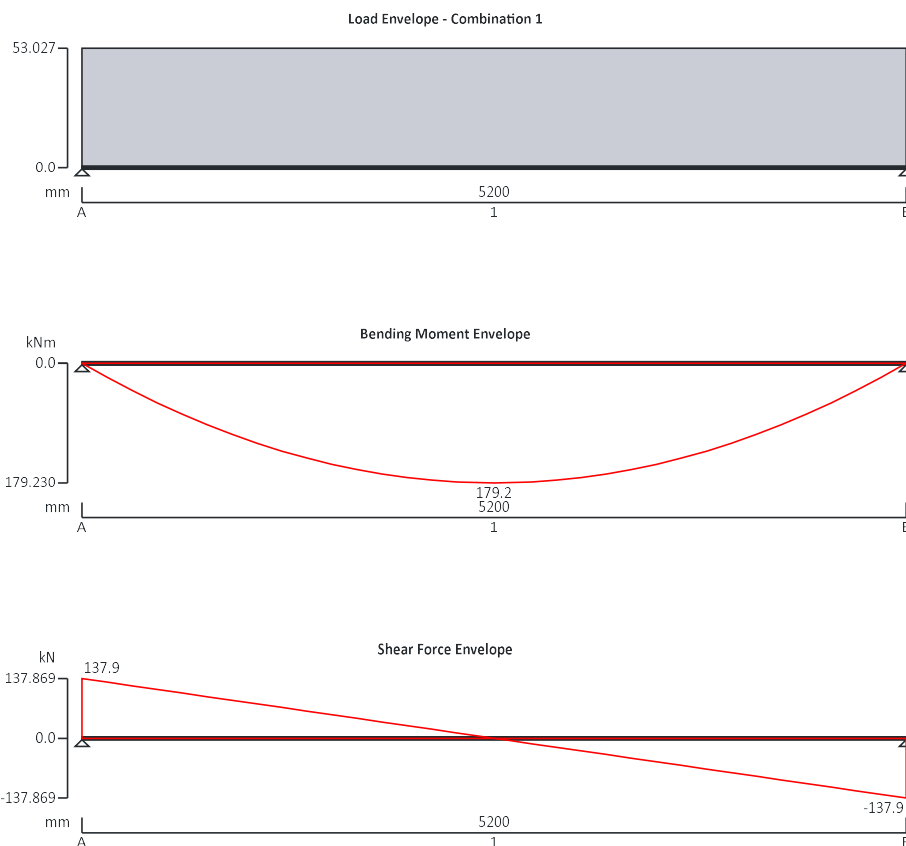
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2.4. Beam B4:

Steel beam analysis & design (BS5950)

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

TEDDS calculation version 3.0.08



Support conditions

Support A Vertically restrained

Rotationally free

Support B Vertically restrained

Rotationally free

Applied loading

Beam loads Dead self weight of beam ' 1

5m supported floor width over - Dead full UDL 3.75 kN/m

5m supported floor width over - Imposed full UDL 7.5 kN/m

5m supported floor width over - Dead full UDL 3.75 kN/m

5m supported floor width over - Imposed full UDL 7.5 kN/m

5m supported floor width over - Dead full UDL 3.75 kN/m

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5m supported floor width over - Imposed full UDL 7.5 kN/m

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; $M_{max} = 179.2$ kNm; $M_{min} = 0$ kNm

Maximum shear; $V_{max} = 137.9$ kN; $V_{min} = -137.9$ kN

Deflection; $d_{max} = 11.3$ mm; $d_{min} = 0$ mm

Maximum reaction at support A; $R_{A_{max}} = 137.9$ kN; $R_{A_{min}} = 137.9$ kN

Unfactored dead load reaction at support A; $R_{A_{Dead}} = 31.6$ kN

Unfactored imposed load reaction at support A; $R_{A_{Imposed}} = 58.5$ kN

Maximum reaction at support B; $R_{B_{max}} = 137.9$ kN; $R_{B_{min}} = 137.9$ kN

Unfactored dead load reaction at support B; $R_{B_{Dead}} = 31.6$ kN

Unfactored imposed load reaction at support B; $R_{B_{Imposed}} = 58.5$ kN

Section details

Section type; **2 x UC 203x203x46**

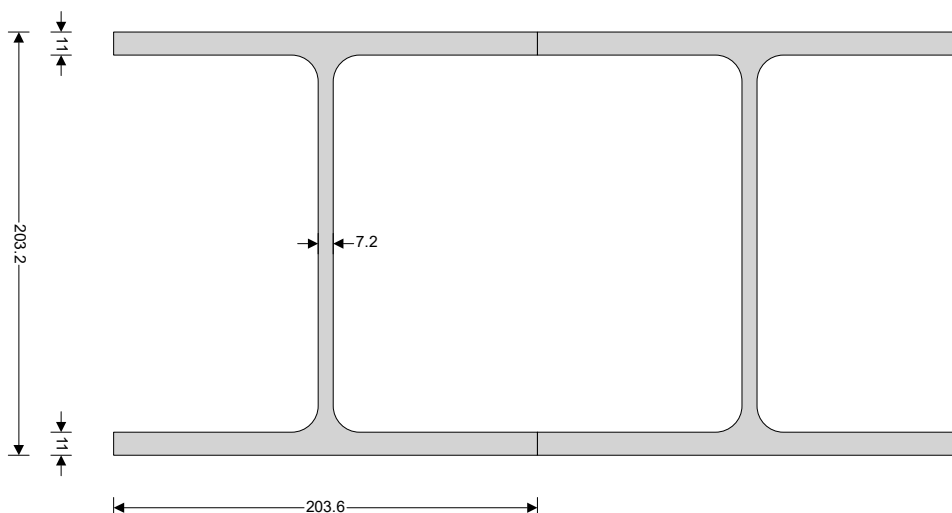
Steel grade; **S355**

From table 9: Design strength p_y

Thickness of element; $\max(T, t) = 11.0$ mm

Design strength; $p_y = 355$ N/mm²

Modulus of elasticity; $E = 205000$ N/mm²



Lateral restraint

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Span 1 has full lateral restraint

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; $K_{LT,A} = 1.00$;

$K_{LT,B} = 1.00$;

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section; $d = 155.8 \text{ mm}$

$d / t = 24.6 \leq 80 \leq e$; Class 1 plastic

Outstand flanges - Table 11

Width of section; $b = B / 2 = 101.8 \text{ mm}$

$b / T = 10.5 \leq 15 \leq e$; Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force; $F_v = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = 137.9 \text{ kN}$

$d / t < 70 \leq e$

Web does not need to be checked for shear buckling

Shear area; $A_v = t \cdot D = 1463 \text{ mm}^2$

Design shear resistance; $P_v = 0.6 \cdot N \cdot p_y \cdot A_v = 623.3 \text{ kN}$

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment; $M = \max(\text{abs}(M_{s1_{\max}}), \text{abs}(M_{s1_{\min}})) = 179.2 \text{ kNm}$

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange; $b_{2f} = 10 \leq e = 8.801$

Limiting value for class 3 semi-compact flange; $b_{3f} = 15 \leq e = 13.202$

Limiting value for class 2 compact web; $b_{2w} = 100 \leq e = 88.014$

Limiting value for class 3 semi-compact web; $b_{3w} = 120 \leq e = 105.617$

Effective plastic modulus - cl.3.5.6.2

$S_{\text{eff}} = \min(Z_{xx} + (S_{xx} - Z_{xx}) \cdot \min(\frac{[(b_{3w} / (d / t))^2 - 1]}{[(b_{3w} / b_{2w})^2 - 1]}, \frac{[(b_{3f} / (b / T) - 1]}{[(b_{3f} / b_{2f}) - 1]}]), S_{xx}) = 494583 \text{ mm}^3$

Moment capacity low shear - cl.4.2.5.2; $M_c = N \cdot \min(p_y \cdot S_{\text{eff}}, 1.2 \cdot p_y \cdot Z_{xx}) = 351.2 \text{ kNm}$

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to imposed loads

Limiting deflection; $d_{\text{lim}} = L_{s1} / 360 = 14.444 \text{ mm}$

Maximum deflection span 1; $d = \max(\text{abs}(d_{\max}), \text{abs}(d_{\min})) = 11.346 \text{ mm}$

PASS - Maximum deflection does not exceed deflection limit



3. Masonry support:

(Refer to the Engineer's general structural arrangement drawings, which indicate the structural design).

3.1. Beam B4 supports:

masonry bearing design to bs5628-1:2005

TEDDS calculation version 1.0.08

Masonry details

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

Compressive strength of unit; $p_{unit} = 7.3 \text{ N/mm}^2$

Mortar designation; **iii**

Least horizontal dimension of masonry units; $l_{unit} = 100 \text{ mm}$

Height of masonry units; $h_{unit} = 215 \text{ mm}$

Category of masonry units; **Category II**

Category of construction control ; **Normal**

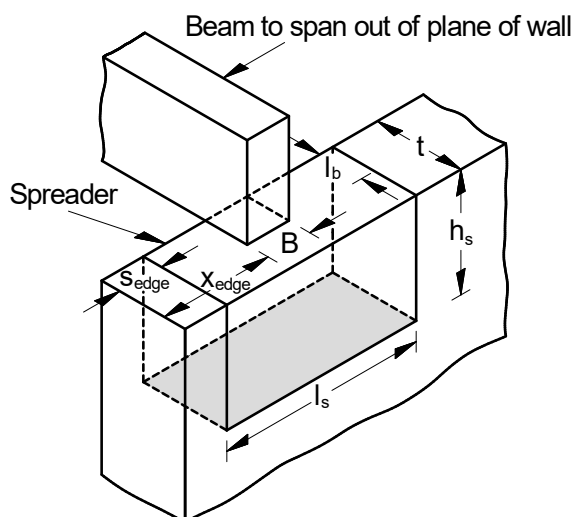
Partial safety factor for material strength; $g_m = 3.5$

Thickness of load bearing leaf; $t = 328 \text{ mm}$

Effective thickness of masonry wall; $t_{ef} = 328 \text{ mm}$

Height of masonry wall; $h = 2400 \text{ mm}$

Effective height of masonry wall; $h_{ef} = 2400 \text{ mm}$



Bearing details

Beam spanning out of plane of wall

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Width of bearing; $B = 400$ mm

Length of bearing; $l_b = 100$ mm

Edge distance; $x_{edge} = 2500$ 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/mm²

Characteristic compressive strength (Table 2c); $f_{kc} = 3.20$ N/mm²

Characteristic compressive strength (Table 2d); $f_{kd} = 6.40$ N/mm²

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

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

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

Ratio between 0.6 and 4.5 - OK

Characteristic compressive strength; $f_k = 6.40$ N/mm²

Loading details

Characteristic concentrated dead load; $G_k = 31$ kN

Characteristic concentrated imposed load; $Q_k = 58$ kN

Design concentrated load; $F = (G_k \times 1.4) + (Q_k \times 1.6) = 136.2$ kN

Characteristic distributed dead load; $g_k = 0.0$ kN/m

Characteristic distributed imposed load; $q_k = 0.0$ kN/m

Design distributed load; $f = (g_k \times 1.4) + (q_k \times 1.6) = 0.0$ kN/m

Masonry bearing type

Bearing type; **Type 2**

Bearing safety factor; $g_{bear} = 1.50$

Check design bearing without a spreader

Design bearing stress; $f_{ca} = F / (B \times l_b) + f / t = 3.405$ N/mm²

Allowable bearing stress; $f_{cp} = g_{bear} \times f_k / g_m = 2.743$ N/mm²

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

Spreader details

Length of spreader; $l_s = 400$ mm

Depth of spreader; $h_s = 215$ mm

Edge distance; $s_{edge} = \max(0 \text{ mm}, x_{edge} - (l_s - B) / 2) = 2500$ mm

Spreader bearing type

Bearing type; **Type 2**

Bearing safety factor; $g_{bear} = 1.50$

Check design bearing with a spreader

Loading acts at midpoint of spreader

Design bearing stress; $f_{ca} = F / (l_s \times t) + f / t = 1.040$ N/mm²

Allowable bearing stress; $f_{cp} = g_{bear} \times f_k / g_m = 2.743$ N/mm²

PASS - Allowable bearing stress exceeds design bearing stress

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

Slenderness ratio; $h_{ef} / t_{ef} = 7.33$

Eccentricity at top of wall; $e_x = 130.4$ mm

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From BS5628:1 Table 7

Capacity reduction factor; $b = 0.22$

Length of bearing distributed at $0.4 \times h$; $l_d = 2320$ mm

Maximum bearing stress; $f_{ca} = F / (l_d \times t) + f / t = 0.179$ N/mm²

Allowable bearing stress; $f_{cp} = b \times f_k / g_m = 0.409$ N/mm²

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