Consulting Structural Engineers

STRUCTURAL REPORT

Hallen Hall Village,
Moorhouse Lane,
Bristol,
BS10 7RU

6th December 2017
Ref: 17074 Hallen Hall Village mr

1.0 INTRODUCTION

This report presents RISE's report, as Structural Engineer, for alterations to the building at Moorhouse Lane, Bristol, BS10 7RU.

2.0 AIMS OF THE REPORT

The aims of this report are to:

- Record the design criteria and performance parameters to which the alterations are being designed.
- Outline our proposals and specification for the steelwork.
- Show any particulars on our proposals which might have an impact on the Health and Safety of the works carried out by the contractor.

3.0 THE SITE LOCATION

Location of the and view of the building

4.0 OVERVIEW OF PROPOSED SCHEME

The building is a recreation centre for the local football club. It is a single storey masonry building with pitched roof formed by steel trusses spanning between the external walls.

The entrance, located at the East side, leads into a reception area in the centre of the building, from which it is possible to reach four different areas. A lounge bar is located at the east area and a meeting room and bathrooms in the West area. Changing rooms occupy the North area and a large open function room/skittle alley in the South area.

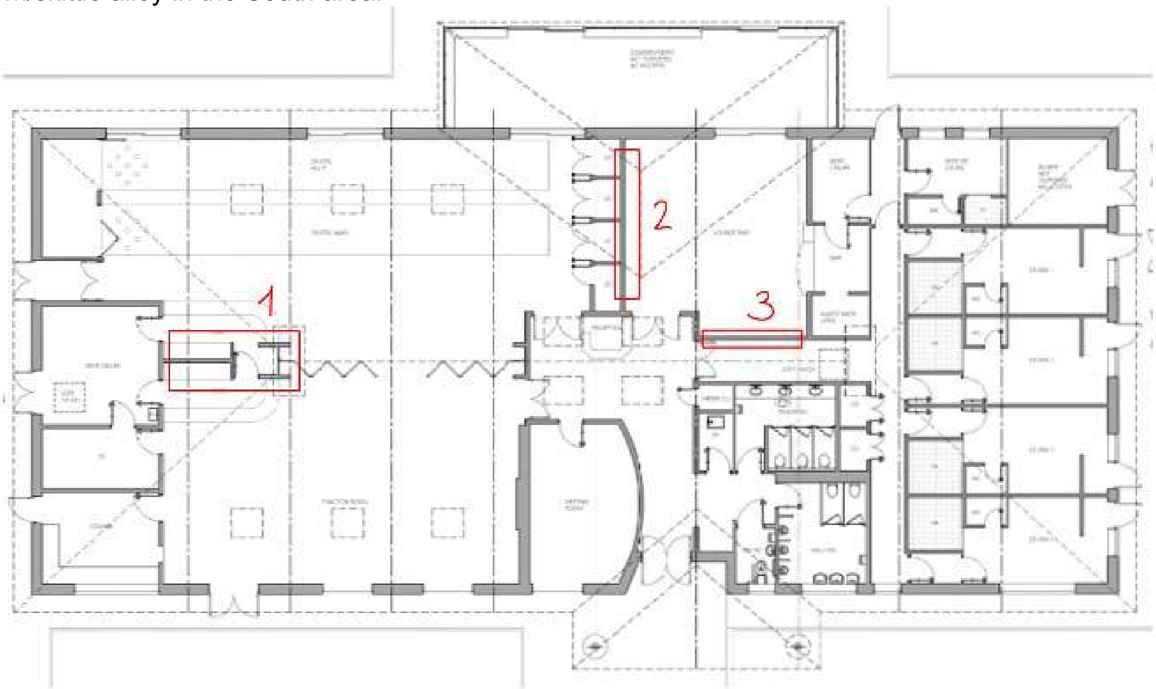


Figure 1. Existing ground floor plan with walls to be demolished

Figure 2. Bar walls No1 to be removed

It is proposed to re-arrange the layout of the building, taking down some of the walls. The walls forming the bars at the South area (Figure 1 walls No1) will be removed to create a single large open space formed by the existing function room and the skittle alley.

These walls are not load-bearing since the ceiling which covers the central part of the south area is supported by a series of steel and timber beams spanning between the trusses.

Figure 3. Wall No2

At the East area a 4m long and 2.4m high opening will be created between the lounge bar and the skittle alley (Figure 1 wall No2). The 140mm thick blockwork wall goes up until reaching the roof, following its shape. A steel beam will be provided over the opening to support the wall above, a small portion of ceiling and the portion of the roof between the trusses. A masonry pier will be left at the end of the beam towards the external wall. The pier must have a minimum length of 450mm and the beam has to bear over the full length of the pier to avoid any load eccentricity. Padstones will be provided at the ends of the beam. Removing the wall will not be a problem for the stability of the external masonry panel running perpendicular to the wall to be taken down. This is because it has the same size of the other external masonry panels in the skittle alley which have no walls or piers to act as a restraint against the wind.

Figure 4. Wall No 3

Another 4m long opening will be created in the 100mm thick blockwork wall between the lounge bar and the corridor leading to the changing rooms, in order to create a new kitchen (Figure 1 wall No3). The height of the opening will be the same as the ceiling, 2.75m. The load-bearing wall goes up until the roof. A steel beam will be located over the new opening, supporting the wall above, the ceiling and the portion of roof between the purlins. Two existing walls, perpendicular to the wall to be re-arranged, located at the ends of the opening, will be extended penetrating the corridor. The new wall portions must be load-bearing blockwork fully bonded to the existing part, creating longer unique walls. In this way the new longer walls will be able to carry the reaction produced by the beam.

A new masonry wall will be provided to close the new kitchen and a precast concrete lintel will be provided over the new door.

4.0 DESIGN AND PERFORMANCE PARAMETERS

4.1 Occupancy Loads

The structure is designed to resist the following loads. These have been calculated in accordance with NA to BS EN 1991-1.1.

Category of loaded area	Location	Dead Load (kN/m2)	Live Load (kN/m2)
See Table NA.7	Ceiling	0.8	0.6 (maintenance)
See Table NA.7	Roof	1	0.6

4.2 Permissible Deflections

The design of new constructional steel elements will limit deflection and displacement in accordance to the following criteria:

Span / 360 (Due to live loads only)

Span / 200 (Due to dead loads and live loads)

Deflection of beams under existing walls will be limited to span/600 or 6mm whichever is the lesser to limit the extent of cracking due to dead load deflection. This can never be eliminated and there may be a requirement to repair minor movements

4.3 Design codes and standards

BS EN 1990 – Eurocode 0: Basis of Structural Design

BS EN 1991 – Eurocode 1: Actions on Structures

BS EN 1993 – Eurocode 3: Design of Steel Structures

BS EN 1996 – Eurocode 6: Design of Masonry Structures

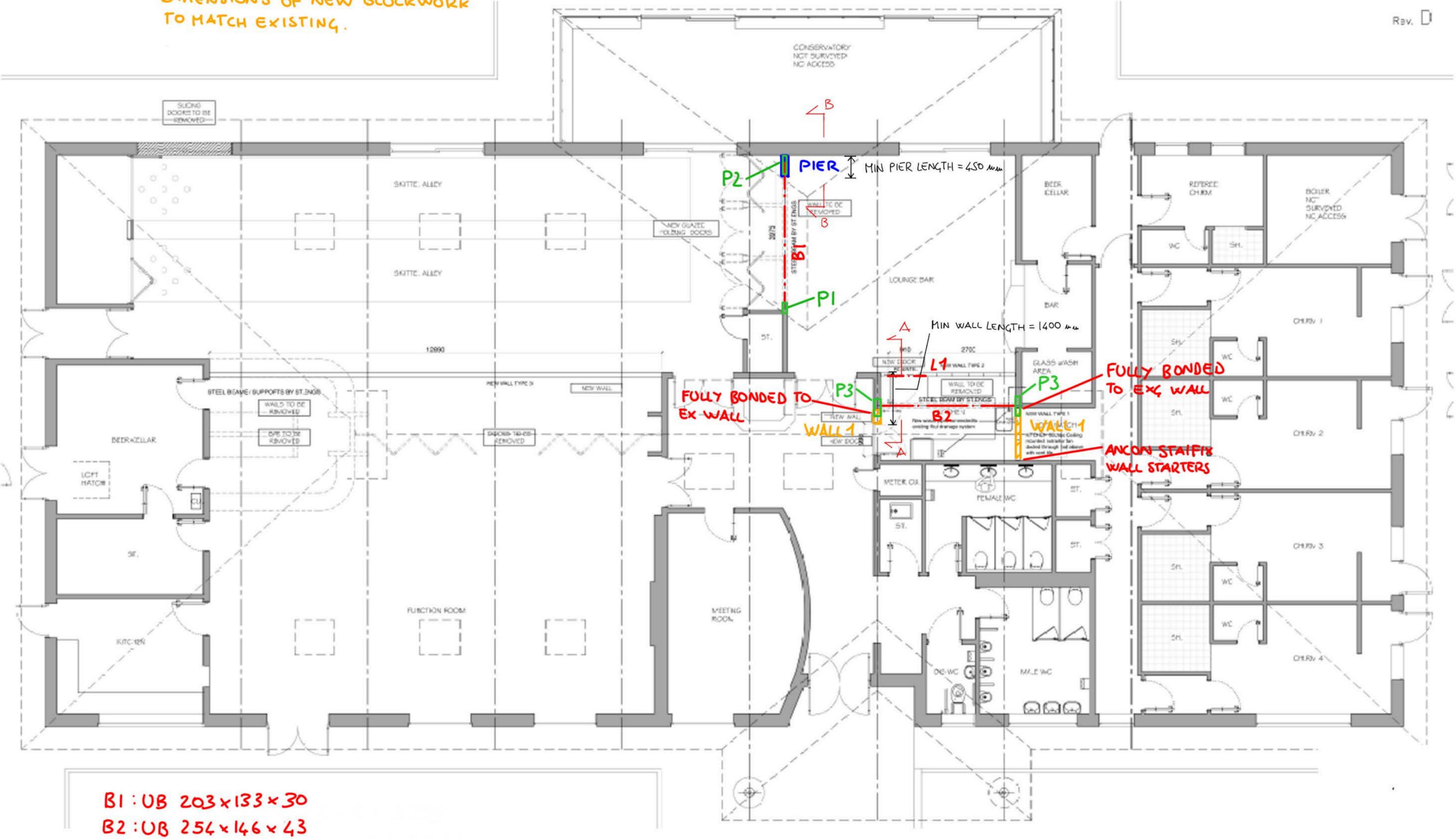
4.4 Temporary Works

It is noted that a specific temporary works design as well as a designed sequence of works will be required in order to safely carry the works. This is outside of RISE's original scope of works.

5.0 APPENDIX A: DRAWINGS

WALL 1 : NEW 100mm BLOCK LOAD-BEARING WALL MIN 7,3 $\frac{N}{mm^2}$
DIMENSIONS OF NEW BLOCKWORK
TO MATCH EXISTING.

Rev. D



B1 : UB 203 × 133 × 30
B2 : UB 254 × 146 × 43
P1 : 215 LONG × 140 WIDE × 215 DEEP PRECAST PADSTONE
P2 : 440 LONG × 140 WIDE × 215 DEEP PRECAST PADSTONE
P3 : 440 LONG × 100 WIDE × 215 DEEP PRECAST PADSTONE
PIER : MINIMUM 450 mm LONG
L1 : NAYLOR P100 LINTEL OVER NEW DOOR

Job: 17074 HALLEN HALL VILLAGE	Title: GROUND FLOOR PLAN			
	Scale: /	Date: 01/12/17	By: MR	Checked: IC
Drawing Status: CONSTRUCTION	Drawing Number: 17074/ SK01			Revision: C1

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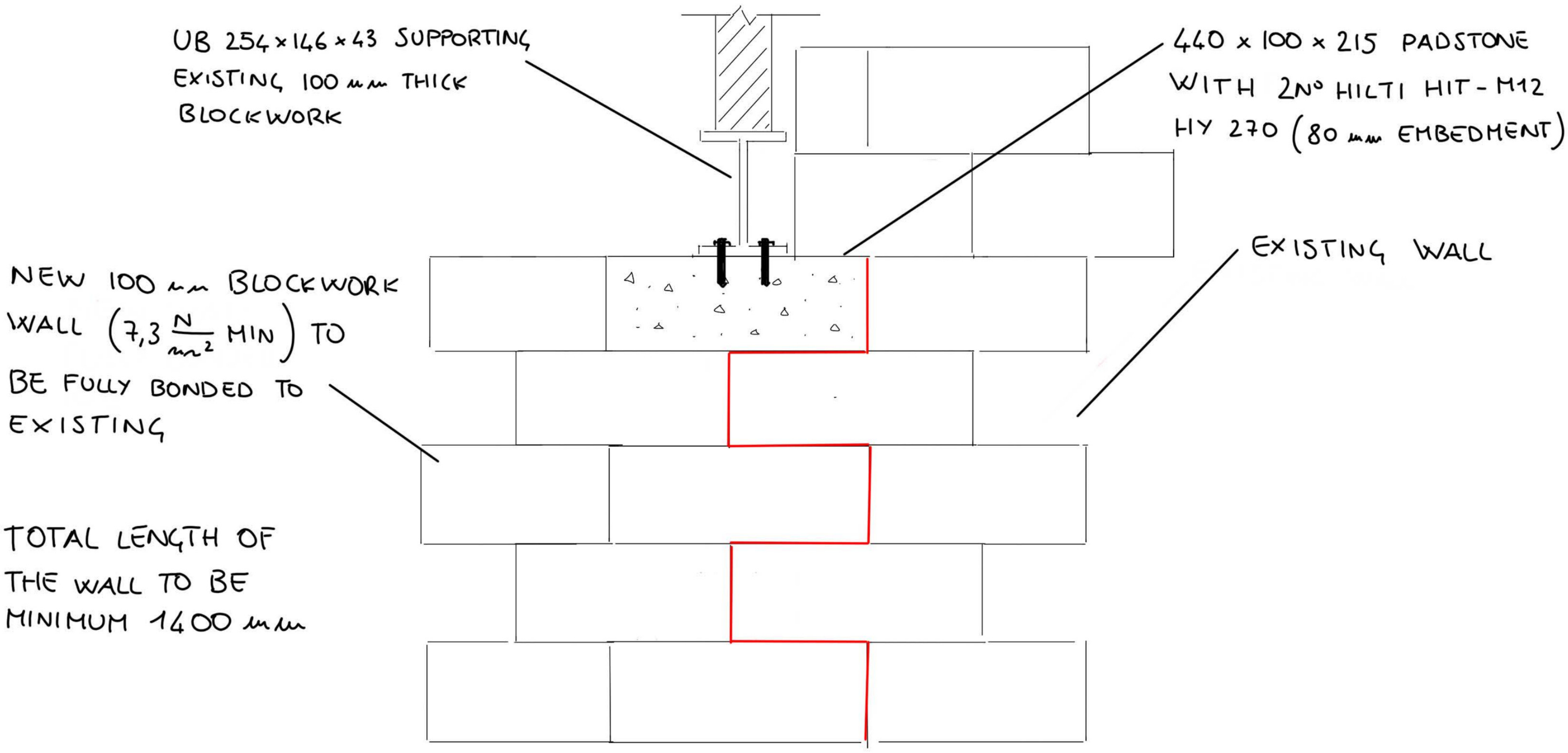
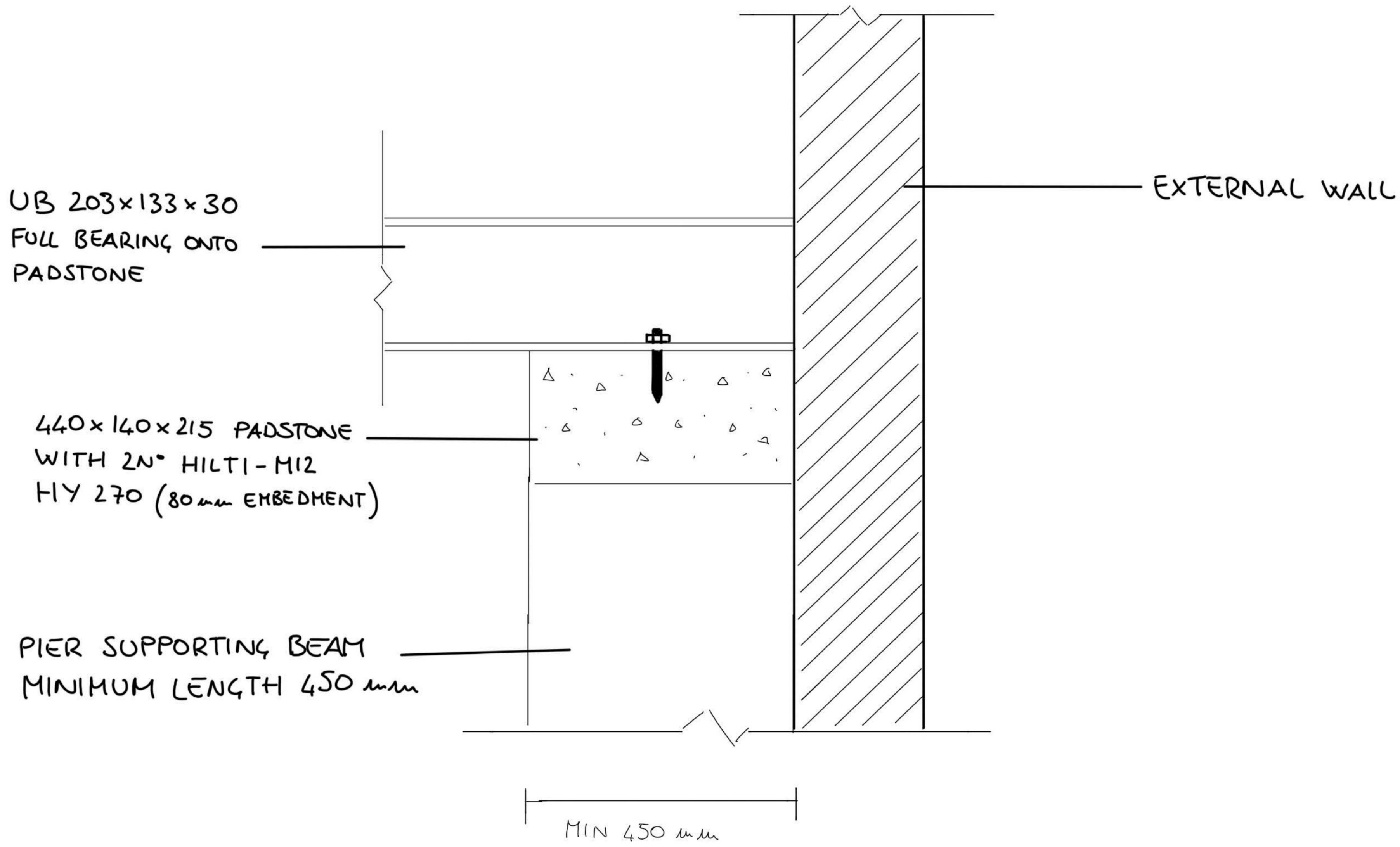


SECTION A-A

Job	Title	
17074 HALLEN HALL VILLAGE	SECTION A-A Scale: / · Date: 01/12/17 · By: MR · Checked: IC	RISE Structural Engineers Ltd. 26A Oakfield Road Clifton Bristol BS8 2AT
Drawing Status: CONSTRUCTION	Drawing Number: 17074 / SK02 · Revision: C1	t: +44 (0) 117 317 9801 e: info@risestructures.com



SECTION B-B

Job: 17074 HALLEN HALL VILLAGE	Title: SECTION B-B	RISE Structural Engineers Ltd. 26A Oakfield Road Clifton Bristol BS8 2AT
	Scale: / · Date: 01/12/17 · By: MR · Checked: IC	
Drawing Status: CONSTRUCTION	Drawing Number: 17074 / SK03 · Revision: C1	t: +44 (0) 117 317 9801 e: info@risestructures.com

6.0 APPENDIX B: CALCULATIONS

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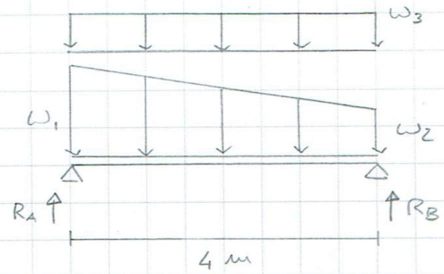
Job Title	Job No.	Page No.	Rev.
HALLEN HALL VILLAGE	17074	1	C1
Calculation/Sketch Title	**Date**	**Author**	**Checked**
BEAMS CALCULATIONS	29/11/17	MR	IC

B1) $L = 4\ m$ Height of opening $= 2{,}4\ m$

Supports 140 mm thick masonry wall with plasterboard on both faces, ceiling and roof.



$$W_D = 0{,}14 \cdot 22 + 2 \cdot 0{,}15 = 3{,}4\ kN/m^2$$

$$\omega_{1D} = 3{,}4 \cdot (4{,}92 - 2{,}4) = 8{,}6\ kN/m$$

$$\omega_{2D} = 3{,}4 \cdot (2{,}7 - 2{,}4) = 1\ kN/m$$

$$\omega_{3D} = \frac{1}{\cos 30} \cdot \frac{3{,}5}{2} + 0{,}8 \cdot \frac{1}{2} = 2{,}42\ kN/m$$

$$\omega_{3L} = 0{,}6 \cdot \frac{3{,}5}{2} + 0{,}6 \cdot \frac{1}{2} = 1{,}35\ kN/m$$

<u>UB 203×133×30 S355</u> Refer to calculation page 5

$R_{AD} = 17{,}6\ kN$ $R_{AL} = 2{,}7\ kN$

$R_{BD} = 12{,}5\ kN$ $R_{BL} = 2{,}7\ kN$

PADSTONE B1) Compressive resistance of blocks $= \frac{3{,}6}{\gamma_m} = \frac{3{,}6}{3} = 1{,}2\ \frac{N}{mm^2}$

$$L = \frac{17600 + 2700}{140 \cdot 1{,}2} = 121\ mm$$

215 Long × 140 Thick × 215 Deep

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Job Title	Job No.	Page No.	Rev.
HALLEN HALL VILLAGE	17074	2	C1
Calculation/Sketch Title	**Date**	**Author**	**Checked**
BEAMS CALCULATION	29/11/17	MR	IC

PIER UNDER SUPPORT B OF BEAM B1) H = 2,4 m

$R_{B_D} = 12,5$ kN $R_{B_L} = 2,7$ kN

$N_{wall\ above} = 3,4 \cdot 0,4 \cdot 0,6 = 0,82$ kN

$N_{Roof\ D} = \frac{1}{\cos 30°} \cdot \frac{3,5}{2} \cdot 0,4 = 0,81$ kN

$N_{ROOF\ L} = 0,6 \cdot \frac{3,5}{2} \cdot 0,4 = 0,42$ kN

$N_{ULS} = 1,35 \left(12,5 + 0,82 + 0,81\right) + 1,5 \cdot \left(2,7 + 0,42\right) = 23,8$ kN

$N_{D\ TOT} = 12,5 + 0,82 + 0,81 = 14,1$ kN

$N_{L\ TOT} = 2,7 + 0,42 = 3,1$ kN

Refer calculation page 9

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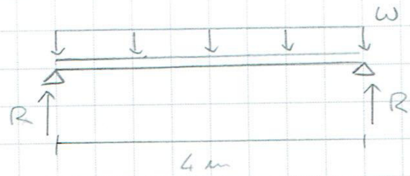
Job Title	Job No.	Page No.	Rev.
HALLEN HALL VILLAGE	17074	3	C1
Calculation/Sketch Title	**Date**	**Author**	**Checked**
BEAMS CALCULATION	29/11/17	MR	IC

B2) $L = 4\,m$ $\quad H_{OPENING} = 2{,}7\,m$ $\quad H_{WALL} = 6{,}5\,m$ $\quad t_{WALL} = 0{,}1\,m$

Supports 100 mm thick masonry wall with plasterboard on both faces, ceiling (2,5 m total ceiling joists length) and roof (4 m between purlins)

$$W_D = 0{,}1 \cdot 22 + 2 \cdot 0{,}15 = 2{,}5\ kN/m^2$$



$$\omega_D = 2{,}5\,(6{,}5 - 2{,}7) + \frac{1}{\cos 30^\circ} \cdot \frac{4}{2} + 0{,}8 \cdot \frac{2{,}5}{2} = 12{,}8\ kN/m$$

$$\omega_L = 0{,}6 \cdot \frac{4}{2} + 0{,}6 \cdot \frac{2{,}5}{2} = 1{,}95\ kN/m$$

$R_D = 26{,}4\ kN$ $\qquad R_L = 3{,}9\ kN$

<u>UB 254 × 146 × 43 S355</u> Refer calculation page 12

PADSTONE B2)

$$L = \frac{(26400 + 3900)}{100 \cdot 1{,}2} = 253\ mm$$

440 Long × 100 Thick × 215 Deep

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	Calculation/Sketch Title BEAMS CALCULATION	Date 01/12/17	Author MR	Checked IC

WALL UNDER B2)

$L = 1{,}4\ m \qquad t = 0{,}1\ m$

$R_D = 26{,}4\ kN \qquad R_L = 3{,}9\ kN$

$N_{wall\ above} = 2{,}5\left(6{,}5 - 2{,}7\right) \cdot 1{,}4 = 13{,}3\ kN$

$N_{D\ TOT} = 26{,}4 + 13{,}3 = 39{,}7\ kN$

$N_{L\ TOT} = 3{,}9\ kN$

Refer calculation page 1C

LINTEL OVER NEW DOOR)

$L = 0{,}9\ m$

$w_D = 2{,}5\left(2{,}75 - 2{,}1\right) + 0{,}8 \cdot \frac{1}{2} = 2\ kN/m$

$w_L = 0{,}6 \cdot \frac{1}{2} = 0{,}3\ kN/m$

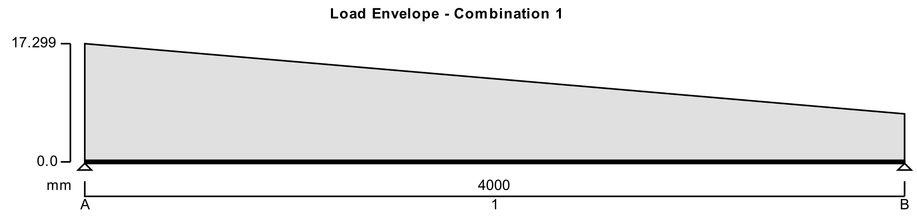
$w_{ULS} = 2{,}3\ kN/m < w_{RD} = 7{,}96\ kN/m$ THEREFORE PASS

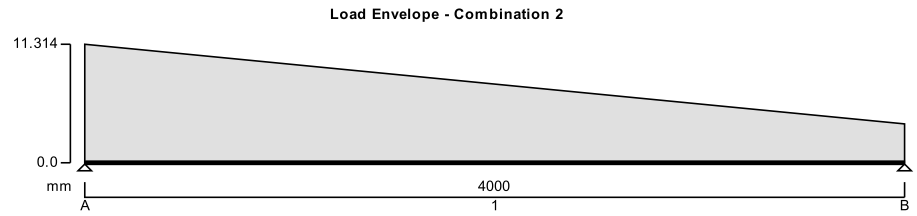
NAYLOR P100 LINTEL

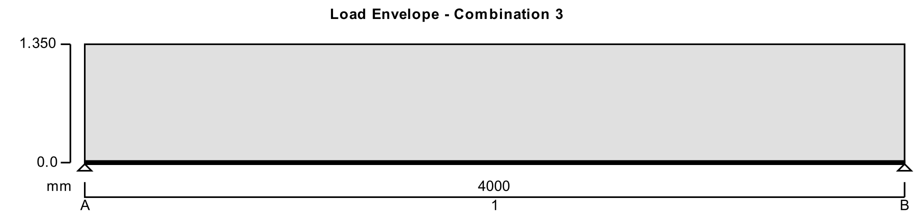
STEEL BEAM ANALYSIS & DESIGN (EN1993-1-1:2005)

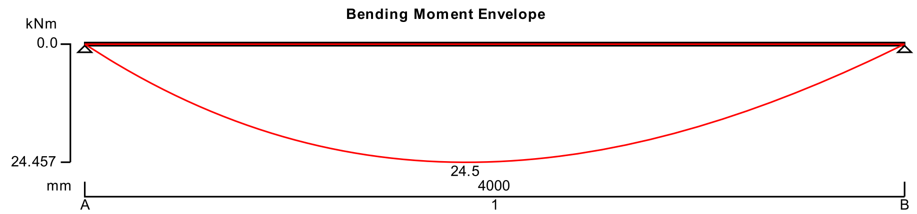
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

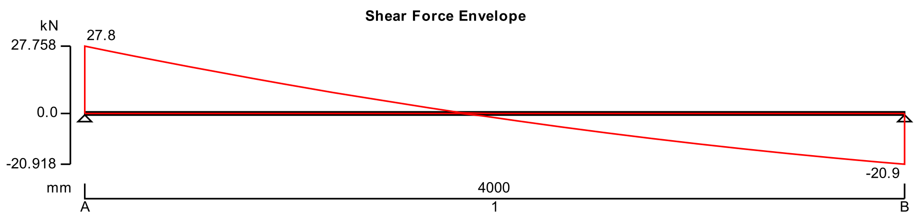
TEDDS calculation version 3.0.13











Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1	
	Permanent full VDL 8.6 kN/m to 1 kN/m	
	Permanent full UDL 2.42 kN/m	
	Variable full UDL 1.35 kN/m	

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50
Load combination 2	Support A	Permanent × 1.00
	Span 1	Permanent × 1.00
	Support B	Permanent × 1.00
		Variable × 1.00
		Variable × 1.00
		Variable × 1.00

Analysis results

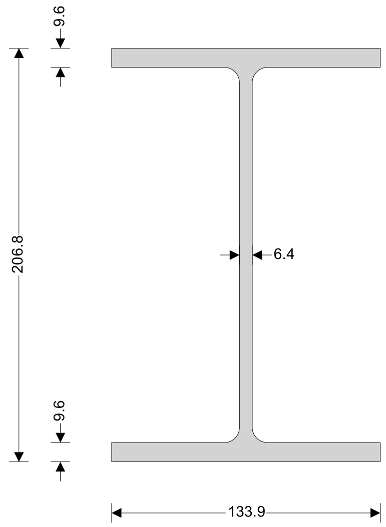
Maximum moment	M_{max} = **24.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **27.8** kN	V_{min} = **-20.9** kN
Deflection	δ_{max} = **4.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **27.8** kN	R_{A_min} = **2.7** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **17.6** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **2.7** kN	
Maximum reaction at support B	R_{B_max} = **20.9** kN	R_{B_min} = **2.7** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **12.5** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **2.7** kN	

Section details

Section type	**UKB 203x133x30 (Tata Steel Advance)**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **9.6** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{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 × h

$K_{LT.B}$ = **1.200** + 2 × 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 = **172.4** mm

$c / t_w = 33.1 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **56.2** mm

$c / t_f = 7.2 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **187.6** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **27.8** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **1458** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **298.7** 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(abs(M_{s1_max}), abs(M_{s1_min})) = **24.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **111.6** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **1**

$C_1 = 1 / k_c^2$ = **1**

Destabilised load condition factor D = **1.5**

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.931**

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.2 \times L_{s1} + 2 \times h$ = **5214** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **61.5** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **2.021**

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 α_{LT} = **0.34**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **2.307**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.245**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.245**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **27.3** 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 $\delta_{lim} = L_{s1} / 600$ = **6.7** mm

Maximum deflection span 1 δ = max(abs(δ_{max}), abs(δ_{min})) = **4.861** mm

PASS - Maximum deflection does not exceed deflection limit

MASONRY COLUMN DESIGN

In accordance with EN1996-1-1:2005 incorporating corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.0.04

Geometry

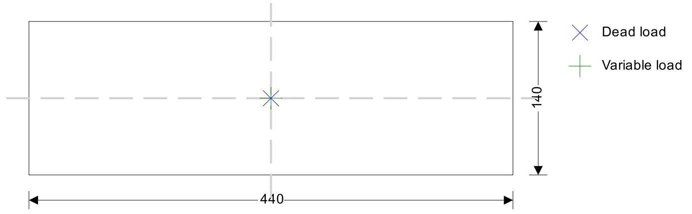
Width of column $b = \mathbf{440}$ mm

Thickness of column $t = \mathbf{140}$ mm

Height of column $h = \mathbf{2400}$ mm

Reduction factor for effective height $\rho_2 = \mathbf{1.0}$

Effective height of column (cl 5.5.1.2) $h_{eff} = h \times \rho_2 = \mathbf{2400}$ mm



Plan on column

Loading

Vertical dead load $G_k = \mathbf{14.1}$ kN

Eccentricity of dead load in x-direction $e_{Gb} = \mathbf{0}$ mm

Eccentricity of dead load in y-direction $e_{Gt} = \mathbf{0}$ mm

Vertical live load $Q_k = \mathbf{3.1}$ kN

Eccentricity of variable load in x-direction $e_{Qb} = \mathbf{0}$ mm

Eccentricity of variable load in y-direction $e_{Qt} = \mathbf{0}$ mm

Characteristic wind loading $W_k = \mathbf{0.0}$ kN/m^2

Vertical wind loading $W_v = \mathbf{0.0}$ kN

Masonry details

Masonry type **Aggregate concrete brick - Group 2**

Compressive strength of masonry unit $f_c = \mathbf{3.6}$ N/mm^2

Height of unit $h_u = \mathbf{215}$ mm

Width of unit $w_u = \mathbf{140}$ mm

Conditioning factor $k = \mathbf{1.0}$

- Conditioning to the air dry condition in accordance with cl.7.3.2

Shape factor - Table A.1 $d_{sf} = \mathbf{1.3}$

Mean compressive strength of masonry unit $f_b = f_c \times k \times d_{sf} = \mathbf{4.68}$ N/mm^2

Density of masonry $\gamma = \mathbf{18}$ kN/m^3

Mortar type **M2 - General purpose mortar**

Compressive strength of masonry mortar $f_m = \mathbf{2}$ N/mm^2

Compressive strength factor - Table NA.4 $K = \mathbf{0.70}$

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3} = \mathbf{2.539}$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

$f_{xk1} = \mathbf{0.2}$ N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

$f_{xk2} = \mathbf{0.6}$ N/mm^2

Partial factors for material strength

Category of manufacturing control — **Category II**

Class of execution control — **Class 2**

Partial factor for masonry in compressive flexure — γ_{Mc} = **3.00**

Slenderness ratio

Slenderness ratio minor axis (cl.5.5.2.1) — $\lambda_t = h_{eff} / t$ = **17.14**

Slenderness ratio major axis (cl.5.5.2.1) — $\lambda_b = h_{eff} / b$ = **5.45**

Maximum slenderness — $\lambda = \max(\lambda_t, \lambda_b)$ = **17.14**

PASS - Slenderness ratio is less than 27

Load combinations derived from Eq.6.10a and Eq.6.10b for lateral and vertical loading (utilisation)

Combination 1 — 1.35 × perm unfav + 1 × perm fav + 1.5 × 0.7 × variable + 1.5 × 0.5 × wind (0.748)

Combination 2 — 0.925 × 1.35 × perm unfav + 1 × perm fav + 1.5 × variable + 1.5 × 0.5 × wind (0.743)

Combination 3 — 0.925 × 1.35 × perm unfav + 1 × perm fav + 1.5 × 0.7 × variable + 1.5 × wind (0.700)

The following output relates to combination 1

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design bending moment top or bottom of column — $M_{idb} = \text{abs}(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb})$ = **0** kNm

Design vertical load at top or bottom of column — $N_{idb} = \text{abs}(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k)$ = **22.29** kN

Initial eccentricity - cl.5.5.1.1 — $e_{init} = h_{eff} / 450$ = **5.3** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load — M_{Edb} = **0** kNm

Eccentricity due to horizontal load — e_{hb} = **0.0** mm

Eccentricity at top or bottom of column - eq.6.5 — $e_{ib} = \max(M_{idb} / N_{idb} + e_{hb} + e_{init}, 0.05 \times b)$ = **22.0** mm

Reduction factor top or bottom of column - eq.6.4 — $\Phi_{ib} = \max(1 - 2 \times e_{ib} / b, 0)$ = **0.9**

Ratio of top and middle mnts due to eccentricity — α_{mdb} = **1.0**

Design bending moment at middle of column — $M_{mdb} = \alpha_{mdb} \times \text{abs}(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb})$ = **0** kNm

Design vertical load at middle of column — $N_{mdb} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2$ = **24.086** kN

Eccentricity due to horizontal load — e_{hmb} = **0.0** mm

Eccentricity middle of column due to loads - eq.6.7 — $e_{mb} = M_{mdb} / N_{mdb} + e_{hmb} + e_{init}$ = **5.3** mm

Eccentricity at middle of column due to creep — e_{kb} = **0.0** mm

Eccentricity at middle of column - eq.6.6 — $e_{mkb} = \max(e_{mb} + e_{kb}, 0.05 \times b)$ = **22.0** mm

From eq.G.2 — $A_{1b} = 1 - 2 \times e_{mkb} / b$ = **0.9**

Short term secant modulus of elasticity factor — K_E = **1000**

Modulus of elasticity - cl.3.7.2 — $E = K_E \times f_k$ = **2539** N/mm^2

Slenderness - eq.G.4 — $\lambda_b = (h_{eff} / b) \times \sqrt{(f_k / E)}$ = **0.172**

From eq.G.3 — $u_b = (\lambda_b - 0.063) / (0.73 - 1.17 \times e_{mkb} / b)$ = **0.163**

Reduction factor at middle of column - eq.G.1 — $\Phi_{mb} = \max(A_{1b} \times e^{-(u_b \times u_b)/2}, 0)$ = **0.888**

Reduction factor for slenderness and eccentricity — $\Phi_b = \min(\Phi_{ib}, \Phi_{mb})$ = **0.888**

Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design bending moment top or bottom of column — $M_{idt} = \text{abs}(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt})$ = **0** kNm

Design vertical load at top or bottom of column — $N_{idt} = \text{abs}(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k)$ = **22.29** kN

Initial eccentricity - cl.5.5.1.1 — $e_{init} = h_{eff} / 450$ = **5.3** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load — M_{Edt} = **0** kNm

Eccentricity due to horizontal load — e_{ht} = **0.0** mm

Eccentricity at top or bottom of column - eq.6.5	$e_{it} = \max(M_{idt} / N_{idt} + e_{ht} + e_{init}, 0.05 \times t)$ = **7.0** mm
Reduction factor top or bottom of column - eq.6.4	$\Phi_{it} = \max(1 - 2 \times e_{it} / t, 0)$ = **0.9**
Ratio of top and middle mnts due to eccentricity	α_{mdt} = **1.0**
Design bending moment at middle of column	$M_{mdt} = \alpha_{mdt} \times abs(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt})$ = **0** kNm
Design vertical load at middle of column	$N_{mdt} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2$ = **24.086** kN
Eccentricity due to horizontal load	e_{hmt} = **0.0** mm
Eccentricity middle of column due to loads - eq.6.7	$e_{mt} = M_{mdt} / N_{mdt} + e_{hmt} + e_{init}$ = **5.3** mm
Eccentricity at middle of column due to creep	e_{kt} = **0.0** mm
Eccentricity at middle of column - eq.6.6	$e_{mkt} = \max(e_{mt} + e_{kt}, 0.05 \times t)$ = **7.0** mm
From eq.G.2	$A_{1t} = 1 - 2 \times e_{mkt} / t$ = **0.9**
Short term secant modulus of elasticity factor	K_E = **1000**
Modulus of elasticity - cl.3.7.2	$E = K_E \times f_k$ = **2539** N/mm^2
Slenderness - eq.G.4	$\lambda_t = (h_{eff} / t) \times \sqrt{(f_k / E)}$ = **0.542**
From eq.G.3	$u_t = (\lambda_t - 0.063) / (0.73 - 1.17 \times e_{mkt} / t)$ = **0.713**
Reduction factor at middle of column - eq.G.1	$\Phi_{mt} = \max(A_{1t} \times e_e^{-(u_t \times u_t)/2}, 0)$ = **0.698**
Reduction factor for slenderness and eccentricity	$\Phi_t = \min(\Phi_{it}, \Phi_{mt})$ = **0.698**

Columns subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load	$N_{Ed} = \max(N_{idb}, N_{mdb}, N_{idt}, N_{mdt})$ = **24.086** kN
Design compressive strength of masonry	$f_d = (0.7 + 3\ m^{-2} \times t \times b) \times f_k / \gamma_{Mc}$ = **0.749** N/mm^2
Vertical resistance of column - eq.6.2	$N_{Rd} = \min(\Phi_t, \Phi_b) \times t \times b \times f_d$ = **32.180** kN
	N_{Ed} / N_{Rd} = **0.748**

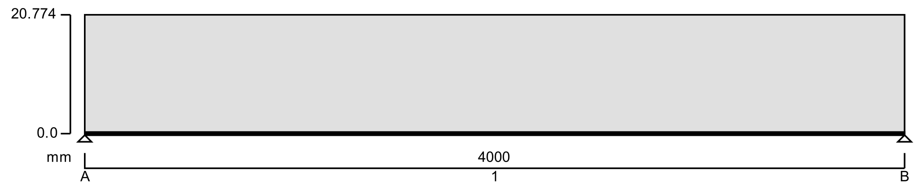
PASS - Design vertical resistance exceeds applied design vertical load

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

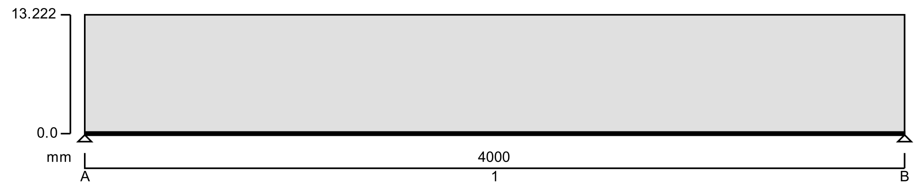
In accordance with EN1993-1-1:2005 incorporating Corrigenda February 2006 and April 2009 and the UK national annex

TEDDS calculation version 3.0.13

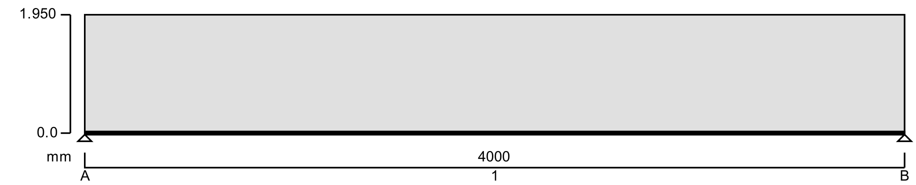
Load Envelope - Combination 1



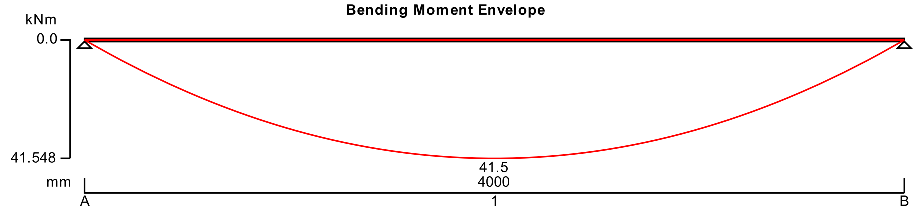
Load Envelope - Combination 2



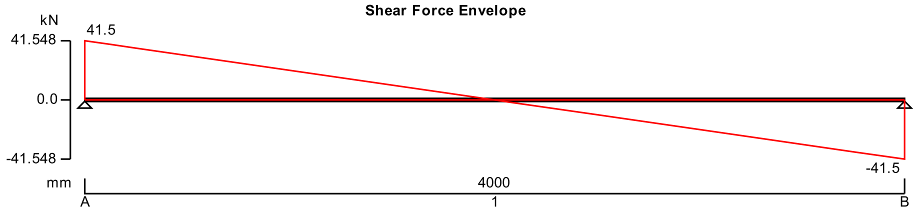
Load Envelope - Combination 3



Bending Moment Envelope



Shear Force Envelope



Support conditions

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

Applied loading

Beam loads	Permanent self weight of beam × 1	
	Permanent full UDL 12.8 kN/m	
	Variable full UDL 1.95 kN/m	

Load combinations

Load combination 1	Support A	Permanent × 1.35
		Variable × 1.50
	Span 1	Permanent × 1.35
		Variable × 1.50
	Support B	Permanent × 1.35
		Variable × 1.50
Load combination 2	Support A	Permanent × 1.00
	Span 1	Permanent × 1.00
	Support B	Permanent × 1.00
		Variable × 1.00
		Variable × 1.00
		Variable × 1.00

Analysis results

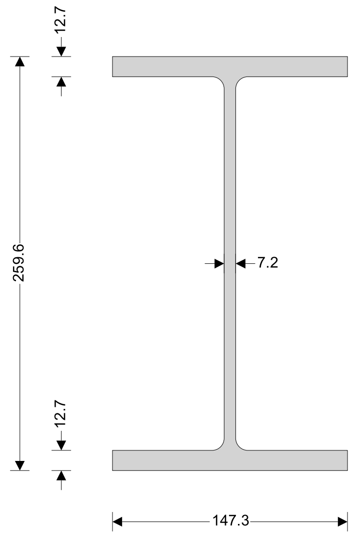
Maximum moment	M_{max} = **41.5** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **41.5** kN	V_{min} = **-41.5** kN
Deflection	δ_{max} = **3.7** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **41.5** kN	R_{A_min} = **3.9** kN
Unfactored permanent load reaction at support A	$R_{A\_Permanent}$ = **26.4** kN	
Unfactored variable load reaction at support A	$R_{A\_Variable}$ = **3.9** kN	
Maximum reaction at support B	R_{B_max} = **41.5** kN	R_{B_min} = **3.9** kN
Unfactored permanent load reaction at support B	$R_{B\_Permanent}$ = **26.4** kN	
Unfactored variable load reaction at support B	$R_{B\_Variable}$ = **3.9** kN	

Section details

Section type	**UKB 254x146x43 (Tata Steel Advance)**
Steel grade	**S355**

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

Nominal thickness of element	$t = \max(t_f, t_w)$ = **12.7** mm
Nominal yield strength	f_y = **355** N/mm^2
Nominal ultimate tensile strength	f_u = **470** N/mm^2
Modulus of elasticity	E = **210000** N/mm^2



Partial factors - Section 6.1

Resistance of cross-sections γ_{M0} = **1.00**

Resistance of members to instability γ_{M1} = **1.00**

Resistance of tensile members to fracture γ_{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 × h

$K_{LT.B}$ = **1.200** + 2 × 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 = **219** mm

$c / t_w = 37.4 \times \varepsilon <= 72 \times \varepsilon$ Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section $c = (b - t_w - 2 \times r) / 2$ = **62.5** mm

$c / t_f = 6.0 \times \varepsilon <= 9 \times \varepsilon$ Class 1

Section is class 1

Check shear - Section 6.2.6

Height of web $h_w = h - 2 \times t_f$ = **234.2** mm

Shear area factor η = **1.000**

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

Shear buckling resistance can be ignored

Design shear force $V_{Ed} = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **41.5** kN

Shear area - cl 6.2.6(3) $A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w)$ = **2020** mm^2

Design shear resistance - cl 6.2.6(2) $V_{c,Rd} = V_{pl,Rd} = A_v \times (f_y / \sqrt{[3]}) / \gamma_{M0}$ = **414.1** kN

PASS - Design shear resistance exceeds design shear force

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

Design bending moment $M_{Ed} = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **41.5** kNm

Design bending resistance moment - eq 6.13 $M_{c,Rd} = M_{pl,Rd} = W_{pl.y} \times f_y / \gamma_{M0}$ = **201** kNm

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6 k_c = **1**

$C_1 = 1 / k_c^2$ = **1**

Destabilised load condition factor D = **1.5**

Curvature factor $g = \sqrt{[1 - (I_z / I_y)]}$ = **0.947**

Poissons ratio ν = **0.3**

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

Unrestrained length $L = 1.2 \times L_{s1} + 2 \times h$ = **5319** mm

Elastic critical buckling moment $M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]}$ = **121.9** kNm

Slenderness ratio for lateral torsional buckling $\bar{\lambda}_{LT} = D \times \sqrt{(W_{pl.y} \times f_y / M_{cr})}$ = **1.926**

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 α_{LT} = **0.34**

Correction factor for rolled sections β = **0.75**

LTB reduction determination factor $\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2]$ = **2.151**

LTB reduction factor - eq 6.57 $\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2)$ = **0.270**

Modification factor $f = \min(1 - 0.5 \times (1 - k_c) \times [1 - 2 \times (\bar{\lambda}_{LT} - 0.8)^2], 1)$ = **1.000**

Modified LTB reduction factor - eq 6.58 $\chi_{LT,mod} = \min(\chi_{LT} / f, 1)$ = **0.270**

Design buckling resistance moment - eq 6.55 $M_{b,Rd} = \chi_{LT,mod} \times W_{pl.y} \times f_y / \gamma_{M1}$ = **54.2** 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 $\delta_{lim} = L_{s1} / 600$ = **6.7** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min}))$ = **3.68** mm

PASS - Maximum deflection does not exceed deflection limit

RISE Structural Engineers Ltd	Project Hallen Hall Village				Job no. 17074	
	Calcs for Pier under B2				Start page no./Revision 16 /C1	
	Calcs by MR	Calcs date 01/12/2017	Checked by IC	Checked date 05/12/2017	Approved by	Approved date

MASONRY COLUMN DESIGN

In accordance with EN1996-1-1:2005 incorporating corrigenda February 2006 and July 2009 and the UK national annex

Tedds calculation version 1.0.04

Geometry

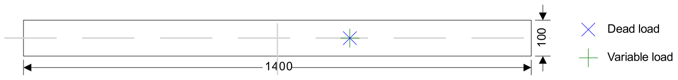
Width of column: b = **1400** mm

Thickness of column: t = **100** mm

Height of column: h = **2700** mm

Reduction factor for effective height: ρ_2 = **1.0**

Effective height of column (cl 5.5.1.2): $h_{eff} = h \times \rho_2$ = **2700** mm



Plan on column

Loading

Vertical dead load: G_k = **39.7** kN

Eccentricity of dead load in x-direction: e_{Gb} = **200** mm

Eccentricity of dead load in y-direction: e_{Gt} = **0** mm

Vertical live load: Q_k = **3.9** kN

Eccentricity of variable load in x-direction: e_{Qb} = **200** mm

Eccentricity of variable load in y-direction: e_{Qt} = **0** mm

Characteristic wind loading: W_k = **0.0** kN/m^2

Vertical wind loading: W_v = **0.0** kN

Masonry details

Masonry type: **Aggregate concrete - Group 2**

Compressive strength of masonry unit: f_c = **3.6** 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: $f_b = f_c \times k \times d_{sf}$ = **4.968** N/mm^2

Density of masonry: γ = **18** kN/m^3

Mortar type: **M4 - General purpose mortar**

Compressive strength of masonry mortar: f_m = **4** N/mm^2

Compressive strength factor - Table NA.4: K = **0.70**

Characteristic compressive strength of masonry - eq 3.1

$f_k = K \times f_b^{0.7} \times f_m^{0.3}$ = **3.259** N/mm^2

Characteristic flexural strength of masonry having a plane of failure parallel to the bed joints - Table NA.6

f_{xk1} = **0.25** N/mm^2

Characteristic flexural strength of masonry having a plane of failure perpendicular to the bed joints - Table NA.6

f_{xk2} = **0.505** N/mm^2

Partial factors for material strength

Category of manufacturing control: **Category II**

Class of execution control: **Class 2**

Partial factor for masonry in compressive flexure: γ_{Mc} = **3.00**

Slenderness ratio

Slenderness ratio minor axis (cl.5.5.2.1) $\lambda_t = h_{eff} / t =$ **27.00**

Slenderness ratio major axis (cl.5.5.2.1) $\lambda_b = h_{eff} / b =$ **1.93**

Maximum slenderness $\lambda = \max(\lambda_t, \lambda_b) =$ **27.00**

PASS - Slenderness ratio is less than 27

Load combinations derived from Eq.6.10a and Eq.6.10b for lateral and vertical loading (utilisation)

Combination 1 $1.35 \times$ perm unfav + $1 \times$ perm fav + $1.5 \times 0.7 \times$ variable + $1.5 \times 0.5 \times$ wind (0.955)

Combination 2 $0.925 \times 1.35 \times$ perm unfav + $1 \times$ perm fav + $1.5 \times$ variable + $1.5 \times 0.5 \times$ wind (0.915)

Combination 3 $0.925 \times 1.35 \times$ perm unfav + $1 \times$ perm fav + $1.5 \times 0.7 \times$ variable + $1.5 \times$ wind (0.888)

The following output relates to combination 1

Reduction factor for slenderness and eccentricity about the major axis - Section 6.1.2.2

Design bending moment top or bottom of column $M_{idb} = \text{abs}(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb}) =$ **11.538** kNm

Design vertical load at top or bottom of column $N_{idb} = \text{abs}(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k) =$ **57.69** kN

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{eff} / 450 =$ **6.0** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load $M_{Edb} =$ **0** kNm

Eccentricity due to horizontal load $e_{hb} =$ **0.0** mm

Eccentricity at top or bottom of column - eq.6.5 $e_{ib} = \max(M_{idb} / N_{idb} + e_{hb} + e_{init}, 0.05 \times b) =$ **206.0** mm

Reduction factor top or bottom of column - eq.6.4 $\Phi_{ib} = \max(1 - 2 \times e_{ib} / b, 0) =$ **0.706**

Ratio of top and middle mnts due to eccentricity $\alpha_{mdb} =$ **1.0**

Design bending moment at middle of column $M_{mdb} = \alpha_{mdb} \times \text{abs}(\gamma_{fGv} \times G_k \times e_{Gb} + \gamma_{fQv} \times Q_k \times e_{Qb}) =$ **11.538** kNm

Design vertical load at middle of column $N_{mdb} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2 =$ **62.283** kN

Eccentricity due to horizontal load $e_{hmb} =$ **0.0** mm

Eccentricity middle of column due to loads - eq.6.7 $e_{mb} = M_{mdb} / N_{mdb} + e_{hmb} + e_{init} =$ **191.3** mm

Eccentricity at middle of column due to creep $e_{kb} =$ **0.0** mm

Eccentricity at middle of column - eq.6.6 $e_{mkb} = \max(e_{mb} + e_{kb}, 0.05 \times b) =$ **191.3** mm

From eq.G.2 $A_{1b} = 1 - 2 \times e_{mkb} / b =$ **0.727**

Short term secant modulus of elasticity factor $K_E =$ **1000**

Modulus of elasticity - cl.3.7.2 $E = K_E \times f_k =$ **3259** N/mm^2

Slenderness - eq.G.4 $\lambda_b = (h_{eff} / b) \times \sqrt{(f_k / E)} =$ **0.061**

From eq.G.3 $u_b = (\lambda_b - 0.063) / (0.73 - 1.17 \times e_{mkb} / b) =$ **-0.004**

Reduction factor at middle of column - eq.G.1 $\Phi_{mb} = \max(A_{1b} \times e e^{-(u_b \times u_b)/2}, 0) =$ **0.727**

Reduction factor for slenderness and eccentricity $\Phi_b = \min(\Phi_{ib}, \Phi_{mb}) =$ **0.706**

Reduction factor for slenderness and eccentricity about the minor axis - Section 6.1.2.2

Design bending moment top or bottom of column $M_{idt} = \text{abs}(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt}) =$ **0** kNm

Design vertical load at top or bottom of column $N_{idt} = \text{abs}(\gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k) =$ **57.69** kN

Initial eccentricity - cl.5.5.1.1 $e_{init} = h_{eff} / 450 =$ **6.0** mm

Conservativley assume moment due to wind load at the top of the column is equal to that at mid height

Design moment due to horizontal load $M_{Edt} =$ **0** kNm

Eccentricity due to horizontal load $e_{ht} =$ **0.0** mm

Eccentricity at top or bottom of column - eq.6.5 $e_{it} = \max(M_{idt} / N_{idt} + e_{ht} + e_{init}, 0.05 \times t) =$ **6.0** mm

Reduction factor top or bottom of column - eq.6.4 $\Phi_{it} = \max(1 - 2 \times e_{it} / t, 0) =$ **0.88**

Ratio of top and middle mnts due to eccentricity $\alpha_{mdt} =$ **1.0**

Design bending moment at middle of column $M_{mdt} = \alpha_{mdt} \times \text{abs}(\gamma_{fGv} \times G_k \times e_{Gt} + \gamma_{fQv} \times Q_k \times e_{Qt}) =$ **0** kNm

Design vertical load at middle of column	$N_{mdt} = \gamma_{fGv} \times G_k + \gamma_{fQv} \times Q_k + \gamma_{fGv} \times t \times b \times \gamma \times h / 2 =$ **62.283** kN
Eccentricity due to horizontal load	$e_{hmt} =$ **0.0** mm
Eccentricity middle of column due to loads - eq.6.7	$e_{mt} = M_{mdt} / N_{mdt} + e_{hmt} + e_{init} =$ **6.0** mm
Eccentricity at middle of column due to creep	$e_{kt} =$ **0.0** mm
Eccentricity at middle of column - eq.6.6	$e_{mkt} = \max(e_{mt} + e_{kt}, 0.05 \times t) =$ **6.0** mm
From eq.G.2	$A_{1t} = 1 - 2 \times e_{mkt} / t =$ **0.88**
Short term secant modulus of elasticity factor	$K_E =$ **1000**
Modulus of elasticity - cl.3.7.2	$E = K_E \times f_k =$ **3259** N/mm^2
Slenderness - eq.G.4	$\lambda_t = (h_{eff} / t) \times \sqrt{(f_k / E)} =$ **0.854**
From eq.G.3	$u_t = (\lambda_t - 0.063) / (0.73 - 1.17 \times e_{mkt} / t) =$ **1.199**
Reduction factor at middle of column - eq.G.1	$\Phi_{mt} = \max(A_{1t} \times e_e^{-(u_t \times u_t)/2}, 0) =$ **0.429**
Reduction factor for slenderness and eccentricity	$\Phi_t = \min(\Phi_{it}, \Phi_{mt}) =$ **0.429**

Columns subjected to mainly vertical loading - Section 6.1.2

Design value of the vertical load	$N_{Ed} = \max(N_{idb}, N_{mdb}, N_{idt}, N_{mdt}) =$ **62.283** kN
Design compressive strength of masonry	$f_d = f_k / \gamma_{Mc} =$ **1.086** N/mm^2
Vertical resistance of column - eq.6.2	$N_{Rd} = \min(\Phi_t, \Phi_b) \times t \times b \times f_d =$ **65.251** kN
	$N_{Ed} / N_{Rd} =$ **0.955**

PASS - Design vertical resistance exceeds applied design vertical load

Hi-Spec & Fair Faced Ranges

All units are prestressed to ensure optimum performance and come with 30 minutes fire rating as standard. With a range of 18 different sections sizes and with lengths available up to 4800mm on certain sections sizes.

Hi-Spec Finish
An ex-steel mould finish and made from wet cast concrete. May see small air holes on the surface, aggregate and concrete fines to bottom arrises. For use normally in plastered/ situations. **Not for use in exposed painted situations.**

Fair Faced Finish

Type C Fair Faced Finish. All corners, arrises and faces on the lintel are perfect. For use in exposed/painted situations.

Order/Specification Code
Use the Hi-Spec reference on our load Tables (e.g. S5). For Fair Faced Lintels preceed the lintel reference with the word 'Faced', ie Faced S5.

Hi-Spec Range		**P100**	**S4**	**R6**	**R9**	**R12**
Load Table Units suitable for 100mm wide walls		65, 100	100, 100	140, 100	215, 100	290, 100
Fire Resistance Available (mins)		30	30	30	30	30
Suitable for Foundation Use		yes	yes	yes	yes	yes
Maximum Stock Length Available		2400mm	3000mm	3600mm	3600mm	3600mm
					Longer lengths available on request - up to 4800mm	
Available Range Finish			Faced	Faced	Faced	Faced
		UNFACTORED LOADS IN kN/m				
Length	Clear Span	100x65	100x100	100x140	100x215	100x290
900mm	700mm	12.97	18.00	48.90	78.18	100.05
1100mm	900mm	7.96	14.20	39.00	62.44	79.90
1200mm	1000mm	6.47	12.80	34.23	56.72	72.57
1500mm	1200mm	4.50	11.25	24.38	48.57	60.85
1800mm	1500mm	2.86	7.25	15.96	36.27	49.65
2100mm	1800mm	1.95	5.02	11.21	25.78	41.91
2400mm	2100mm	1.21	3.66	8.27	19.21	31.70
2700mm	2400mm	n/a	2.77	6.32	14.83	24.53
3000mm	2700mm	n/a	2.15	4.97	11.76	19.49
3300mm	3000mm	n/a	n/a	3.99	9.53	15.83
3600mm	3200mm	n/a	n/a	3.48	8.36	13.49
Lintel Weight kg/m		16	23	34	53	70