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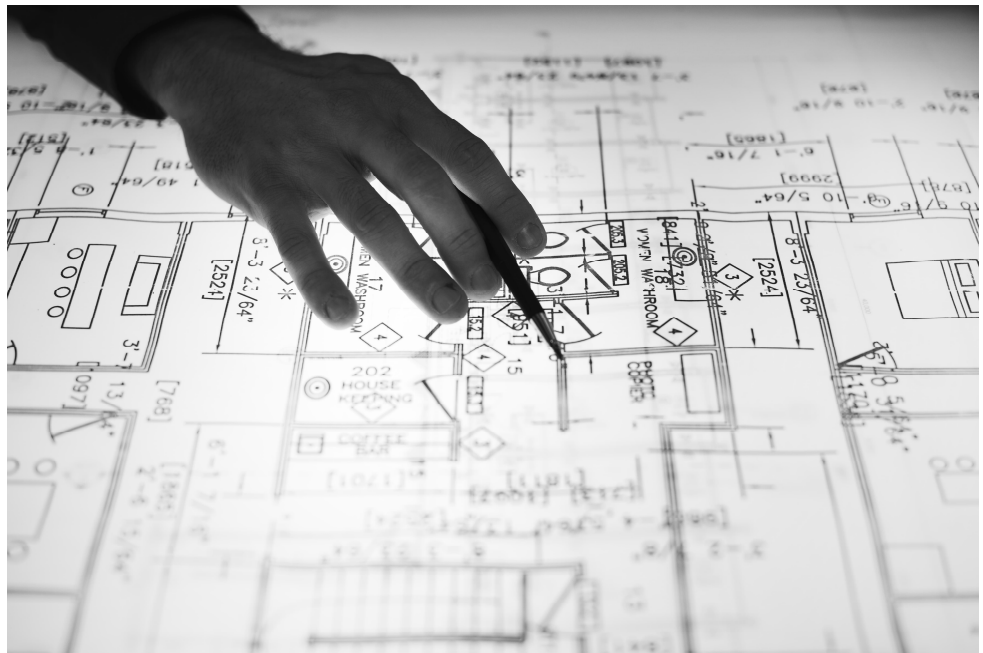
Civil & Structural
Engineers



STRUCTURAL CALCULATIONS

PROJECT

Waitrose Wokingham – Propping to
leaning Boundary wall



409-411 Brixton Road
London
SW9 7DG



Established 1972

☎: 020 7593 0088

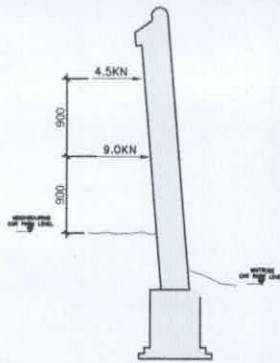
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DETAILS

JOB NO: 5097

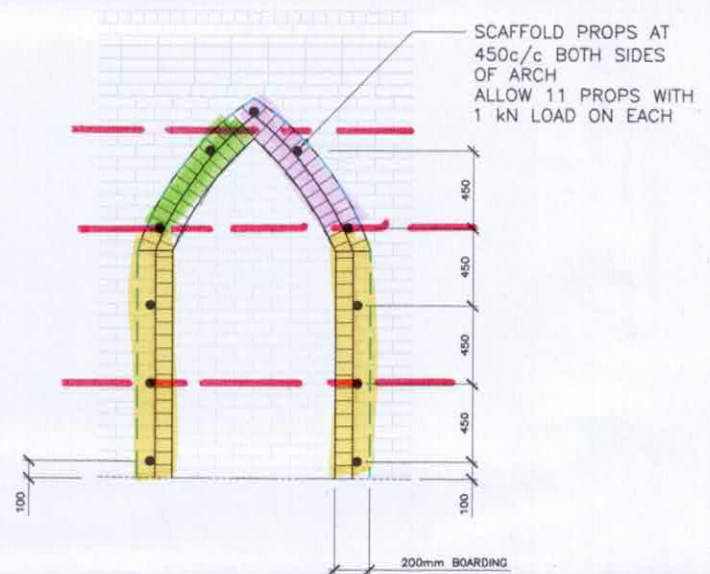
DATE: Sept 2025



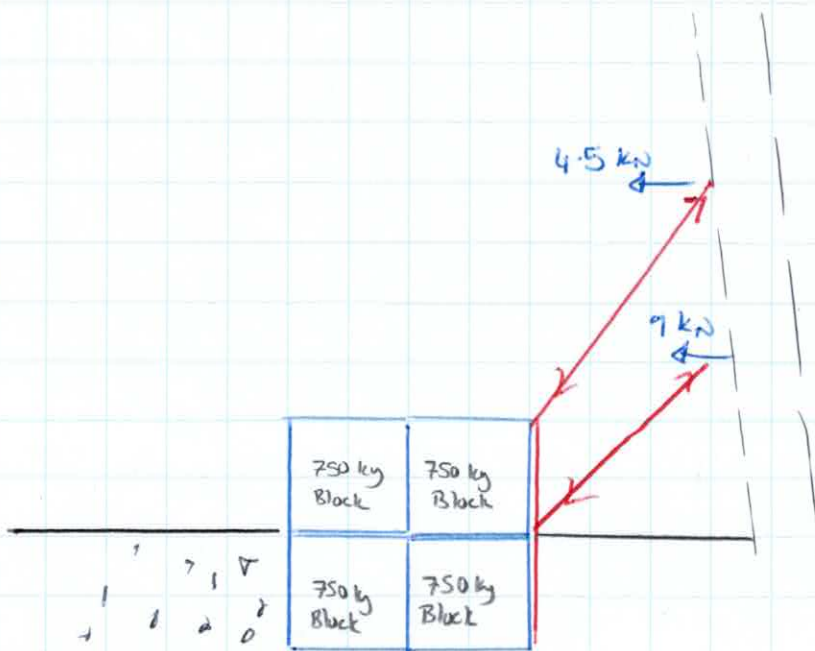
LOADS FOR TEMPORARY WORKS AT 2.8m CENTRES ALONG FULL LENGTH OF WALL H1
(not to scale)

TEMPORARY WORKS TO BE DESIGNED AND INSTALLED BY CONTRACTOR BEFORE COMMENCING WORKS TO THE WALL. DESIGN TO BE SUBMITTED TO HPM AND LOCAL AUTHORITY FOR COMMENT 14 DAYS BEFORE WORKS COMMENCE.

TEMPORARY WORKS TO BE INSTALLED + REMOVED CAREFULLY TO AVOID DISTURBING THE WALL.



LOADS FOR TEMPORARY WORKS AROUND ARCHWAY (not to scale)



$$\text{TOTAL HORIZONTAL FORCE} = 13.5 \text{ kN} \\ 27 \text{ kN/WH}$$

$$\text{WEIGHT OF BLOCKS} = 30 \text{ kN}$$

$$\text{RESISTANCE TO SLIDING} = 30 \times 0.6 \\ = 18 \text{ kN}$$

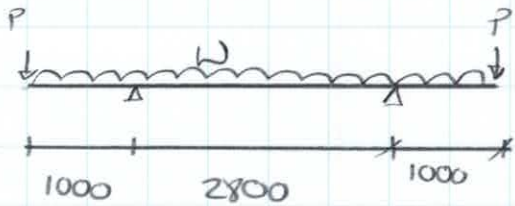
TAKE PASSIVE RESISTANCE OF POUND BASE

$$K_p = \frac{1 + \sin 45}{1 - \sin 45} = 5.83$$

$$P = K_p \gamma h^2 \times 0.5 = 10.6 \text{ kN/m}$$

$$\text{TOTAL RESISTANCE} = 28 \text{ kN (FOS 2.0)}$$

FRAME 1 WAILER DESIGN : (FRAME 2-4 SIMILAR)



$W = \text{DISTRIBUTED LOAD (Worst Case)}$
 $(9/2.8) = 3.21 \text{ kN/m}$

$P = 400\text{mm WALL} = 3.21 \times 0.4 = 1.28 \text{ kN}$

REFER TO TEDD'S

REACTION = 9 kN (AS NPM REQUIREMENTS)

USE 152UC23

$\delta_{TOT} = 0.7 \text{ mm} \therefore \text{ACCEPTABLE}$

CHECK MAX PUSH-PULL PROP LOAD

Max Prop Length = 2.3 m

Max Prop load = 14 kN (OK)

USE STANDARD PUSH-PULL ACROSS PROP

SWL @ 2.5m = 17 kN $\therefore$ ok ✓

FIXED WITH 4 NO. MR CONCRETE SCREWS (100mm Embedment)
IZAW R-LX-UF

$S_{WEEK} = 30 \text{ kN per Bolt @ 100mm}$
 $= 6.25 \text{ kN per Bolt @ 60mm}$

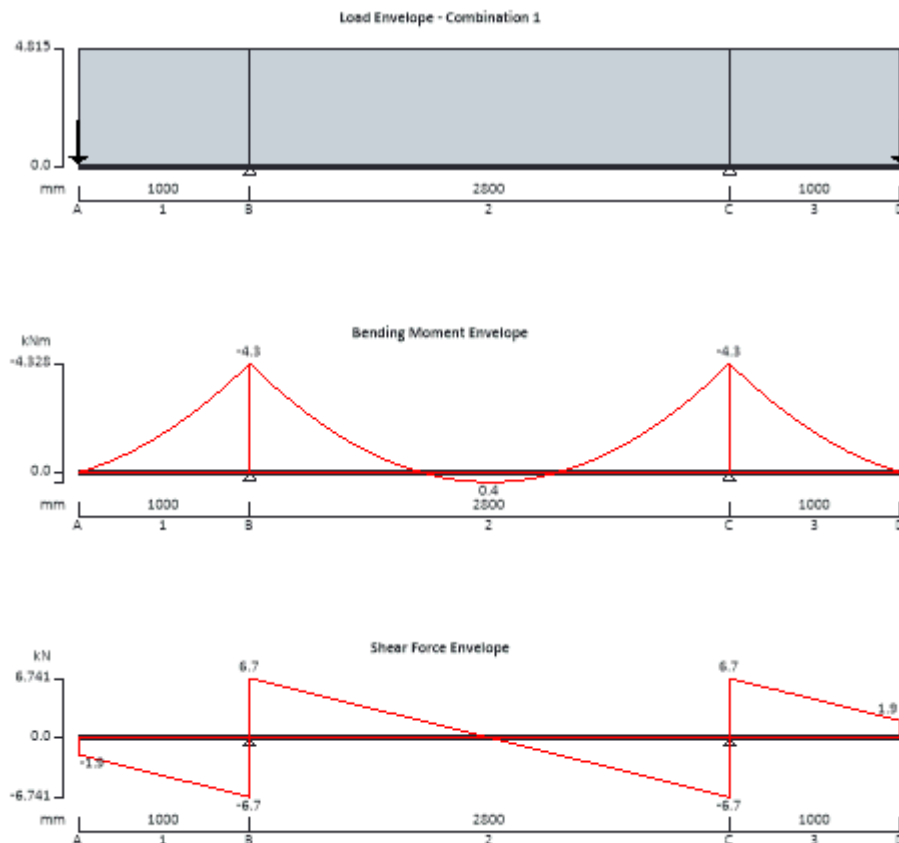
Min Edge Dist = 80mm

Project Waitrose Wokingham				Job no. 5097	
Calcs for Standard Frame 1 wailer				Start page no./Revision 1	
Calcs by KT	Calcs date 18/09/2025	Checked by	Checked date	Approved by	Approved date

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 free Rotationally free
Support B	Vertically restrained Rotationally free
Support C	Vertically restrained Rotationally free
Support D	Vertically free Rotationally free

Applied loading

Beam loads	Variable full UDL 3.21 kN/m Variable point load 1.28 kN at 0 mm Variable point load 1.28 kN at 4800 mm
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Load combinations

Load combination 1	Support A	Permanent $\times 1.35$ Variable $\times 1.50$ Permanent $\times 1.35$
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Project Waitrose Wokingham				Job no. 5097	
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Support B

Variable $\times 1.50$
Permanent $\times 1.35$
Variable $\times 1.50$
Permanent $\times 1.35$
Variable $\times 1.50$
Permanent $\times 1.35$
Variable $\times 1.50$
Permanent $\times 1.35$
Variable $\times 1.50$
Permanent $\times 1.35$
Variable $\times 1.50$

Support C

Support D

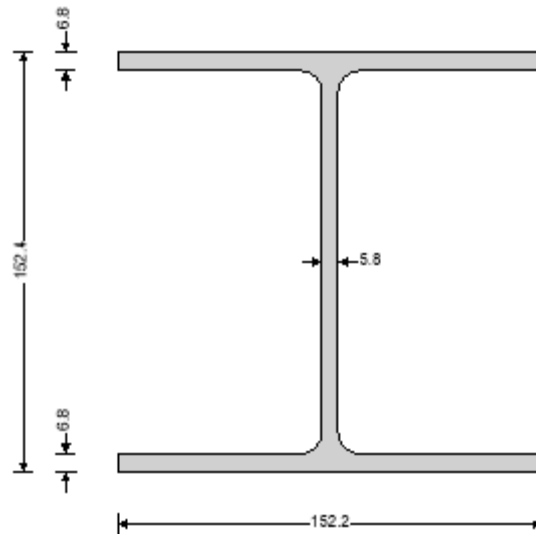
Analysis results

Maximum moment	$M_{max} = 0.4 \text{ kNm}$	$M_{min} = -4.3 \text{ kNm}$
Maximum moment span 1	$M_{s1_max} = 0 \text{ kNm}$	$M_{s1_min} = -4.3 \text{ kNm}$
Maximum moment span 2	$M_{s2_max} = 0.4 \text{ kNm}$	$M_{s2_min} = -4.3 \text{ kNm}$
Maximum moment span 3	$M_{s3_max} = 0 \text{ kNm}$	$M_{s3_min} = -4.3 \text{ kNm}$
Maximum shear	$V_{max} = 6.7 \text{ kN}$	$V_{min} = -6.7 \text{ kN}$
Maximum shear span 1	$V_{s1_max} = -1.9 \text{ kN}$	$V_{s1_min} = -6.7 \text{ kN}$
Maximum shear span 2	$V_{s2_max} = 6.7 \text{ kN}$	$V_{s2_min} = -6.7 \text{ kN}$
Maximum shear span 3	$V_{s3_max} = 6.7 \text{ kN}$	$V_{s3_min} = 1.9 \text{ kN}$
Deflection	$\delta_{max} = 0.7 \text{ mm}$	$\delta_{min} = 0.1 \text{ mm}$
Deflection span 1	$\delta_{s1_max} = 0.7 \text{ mm}$	$\delta_{s1_min} = 0 \text{ mm}$
Deflection span 2	$\delta_{s2_max} = 0 \text{ mm}$	$\delta_{s2_min} = 0.1 \text{ mm}$
Deflection span 3	$\delta_{s3_max} = 0.7 \text{ mm}$	$\delta_{s3_min} = 0 \text{ mm}$
Maximum reaction at support A	$R_{A_max} = 0 \text{ kN}$	$R_{A_min} = 0 \text{ kN}$
Maximum reaction at support B	$R_{B_max} = 13.5 \text{ kN}$	$R_{B_min} = 13.5 \text{ kN}$
Unfactored variable load reaction at support B	$R_{B_Variable} = 9 \text{ kN}$	
Maximum reaction at support C	$R_{C_max} = 13.5 \text{ kN}$	$R_{C_min} = 13.5 \text{ kN}$
Unfactored variable load reaction at support C	$R_{C_Variable} = 9 \text{ kN}$	
Maximum reaction at support D	$R_{D_max} = 0 \text{ kN}$	$R_{D_min} = 0 \text{ kN}$

Section details

Section type	UC 152x152x23 (British Steel Section Range 2022 (BS4-1))
Steel grade	S275
EN 10025-2:2004 - Hot rolled products of structural steels	
Nominal thickness of element	$t = \max(t_r, t_w) = 6.8 \text{ mm}$
Nominal yield strength	$f_y = 275 \text{ N/mm}^2$
Nominal ultimate tensile strength	$f_u = 410 \text{ N/mm}^2$
Modulus of elasticity	$E = 210000 \text{ N/mm}^2$

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Calcs for Standard Frame 1 wailer				Start page no./Revision 3	
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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
Span 2 has lateral restraint at supports only
Span 3 has lateral restraint at supports only
Cantilever tip is free
Cantilever support is continuous with lateral and torsional restraint

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	$K_{LT,A} = 1.400$
	$K_{LT,B} = 1.000$
	$K_{LT,C} = 1.000$
	$K_{LT,D} = 1.000$

Classification of cross sections - Section 5.5

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

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

Width of section	$c = d = 123.6 \text{ mm}$	
	$c / t_w = 23.1 \times \varepsilon \leq 72 \times \varepsilon$	Class 1

Outstand flanges - Table 5.2 (sheet 2 of 3)

Width of section	$c = (b - t_w - 2 \times r) / 2 = 65.6 \text{ mm}$	
	$c / t_f = 10.4 \times \varepsilon \leq 14 \times \varepsilon$	Class 3

Section is class 3

Check shear - Section 6.2.6

Height of web	$h_w = h - 2 \times t_f = 138.8 \text{ mm}$
Shear area factor	$\eta = 1.000$

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

Shear buckling resistance can be ignored

Design shear force

$$V_{Ed} = \max(\text{abs}(V_{\max}), \text{abs}(V_{\min})) = \mathbf{6.7 \text{ kN}}$$

Shear area - cl 6.2.6(3)

$$A_v = \max(A - 2 \times b \times t_f + (t_w + 2 \times r) \times t_f, \eta \times h_w \times t_w) = \mathbf{997 \text{ mm}^2}$$

Design shear resistance - cl 6.2.6(2)

$$V_{pl,Rd} = A_v \times (f_y / \sqrt{3}) / \gamma_{M0} = \mathbf{158.4 \text{ kN}}$$

PASS - Design shear resistance exceeds design shear force

Check bending moment at span 3 major (y-y) axis - Section 6.2.5

Design bending moment

$$M_{Ed} = \max(\text{abs}(M_{s3_max}), \text{abs}(M_{s3_min})) = \mathbf{4.3 \text{ kNm}}$$

Design bending resistance moment - eq 6.14

$$M_{c,Rd} = M_{el,Rd} = W_{el,y} \times f_y / \gamma_{M0} = \mathbf{45.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

Correction factor - Table 6.6

$$k_c = \mathbf{1}$$

$$C_1 = 1 / k_c^2 = \mathbf{1}$$

Destabilised load condition factor

$$D = \mathbf{1.7}$$

Curvature factor

$$g = \sqrt{[1 - (I_z / I_y)]} = \mathbf{0.825}$$

Poissons ratio

$$\nu = \mathbf{0.3}$$

Shear modulus

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

Unrestrained length

$$L = 3.0 \times L_{s3} = \mathbf{3000 \text{ mm}}$$

Elastic critical buckling moment

$$M_{cr} = C_1 \times \pi^2 \times E \times I_z / (L^2 \times g) \times \sqrt{[I_w / I_z + L^2 \times G \times I_t / (\pi^2 \times E \times I_z)]} = \mathbf{108.1 \text{ kNm}}$$

Slenderness ratio for lateral torsional buckling

$$\bar{\lambda}_{LT} = D \times \sqrt{(W_{el,y} \times f_y / M_{cr})} = \mathbf{1.098}$$

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

$$b$$

Imperfection factor - Table 6.3

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

Correction factor for rolled sections

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

LTB reduction determination factor

$$\phi_{LT} = 0.5 \times [1 + \alpha_{LT} \times (\bar{\lambda}_{LT} - \bar{\lambda}_{LT,0}) + \beta \times \bar{\lambda}_{LT}^2] = \mathbf{1.071}$$

LTB reduction factor - eq 6.57

$$\chi_{LT} = \min(1 / [\phi_{LT} + \sqrt{(\phi_{LT}^2 - \beta \times \bar{\lambda}_{LT}^2)}], 1, 1 / \bar{\lambda}_{LT}^2) = \mathbf{0.640}$$

Modification factor

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

Modified LTB reduction factor - eq 6.58

$$\chi_{LT,mod} = \min(\chi_{LT} / f, 1) = \mathbf{0.640}$$

Design buckling resistance moment - eq 6.55

$$M_{b,Rd} = \chi_{LT,mod} \times W_{el,y} \times f_y / \gamma_{M1} = \mathbf{28.9 \text{ kNm}}$$

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

$$\delta_{lim} = L_{s3} / 360 = \mathbf{2.8 \text{ mm}}$$

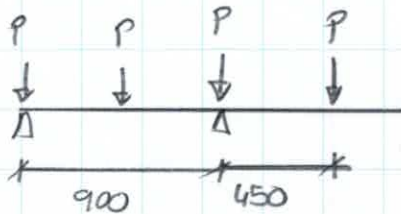
Maximum deflection span 3

$$\delta = \max(\text{abs}(\delta_{\max}), \text{abs}(\delta_{\min})) = \mathbf{0.736 \text{ mm}}$$

PASS - Maximum deflection does not exceed deflection limit

TIMBER AREA SUPPORT.

$P = 1 \text{ kN}$ SUPPORT LOAD (AS PER WPM DWG)



REFER TO TEDD'S OUTPUT

USE 200x75 C24 TIMBER,

$$\delta_{TOT} = 1.4 \text{ mm.}$$

CHECK RESISTANCE TO TOP 2 NO. THRUST BLOCKS.

$$\text{LOAD} = 15 \text{ kN.}$$

$$\text{COF of CAST CONCRETE ON CAST CONCRETE} = 0.4$$

$$\text{RESISTANCE} = 6 \text{ kN.}$$

$$(\text{FOS} = 1.3)$$

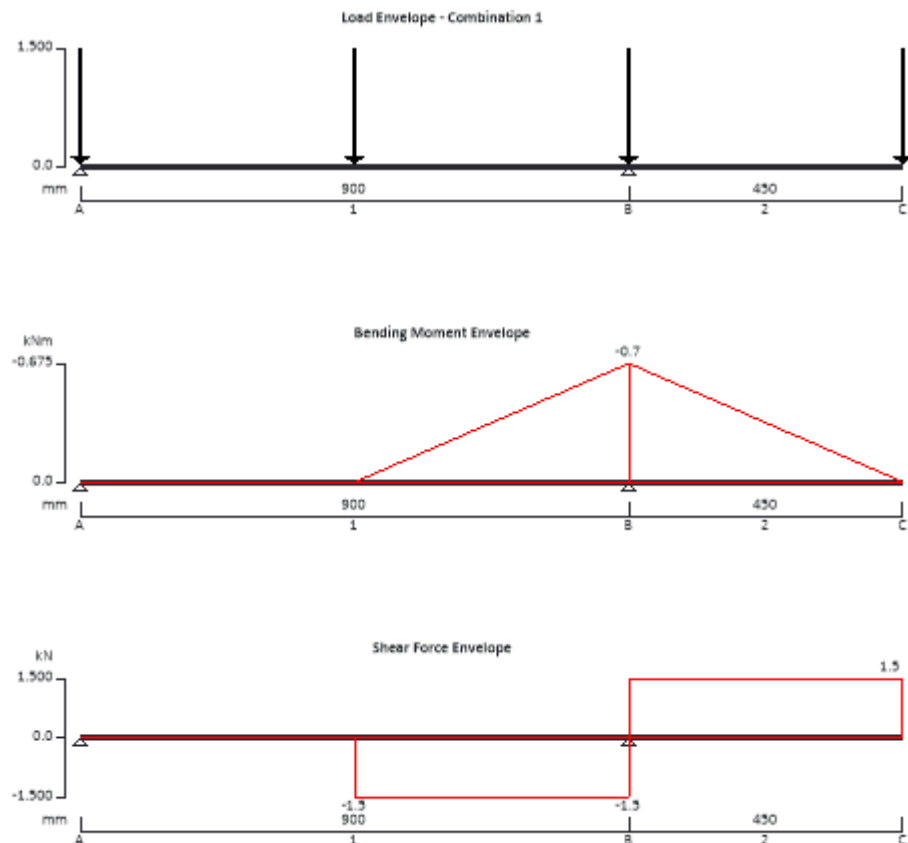
$\therefore$ ENGAGE lower block,
TO ACHIEVE FOS of 2.

Project Waitrose Wokingham				Job no. 5097	
Calcs for Arch Timber support beam				Start page no./Revision 1	
Calcs by KT	Calcs date 18/09/2025	Checked by	Checked date	Approved by	Approved date

TIMBER BEAM ANALYSIS & DESIGN TO EN1995-1-1:2004

In accordance with EN1995-1-1:2004 + A2:2014 and Corrigendum No.1 and the UK National Annex incorporating National Amendment No.1

Tedds calculation version 1.7.05



Applied loading

Beam loads

Variable point load 1.000 kN at 0 mm
Variable point load 1.000 kN at 450 mm
Variable point load 1.000 kN at 900 mm
Variable point load 1.000 kN at 1350 mm

Load combinations

Load combination 1

Support A	Permanent $\times$ 1.35 Variable $\times$ 1.50
Span 1	Permanent $\times$ 1.35 Variable $\times$ 1.50
Support B	Permanent $\times$ 1.35 Variable $\times$ 1.50
Span 2	Permanent $\times$ 1.35 Variable $\times$ 1.50
Support C	Permanent $\times$ 1.35

Variable × 1.50

Analysis results

Maximum moment

 $M_{max} = 0.000 \text{ kNm}$
 $M_{min} = -0.675 \text{ kNm}$

Design moment

 $M = \max(\text{abs}(M_{max}), \text{abs}(M_{min})) = 0.675 \text{ kNm}$

Maximum shear

 $F_{max} = 1.500 \text{ kN}$
 $F_{min} = -1.500 \text{ kN}$

Design shear

 $F = \max(\text{abs}(F_{max}), \text{abs}(F_{min})) = 1.500 \text{ kN}$

Total load on beam

 $W_{tot} = 6.000 \text{ kN}$

Reactions at support A

 $R_{A_max} = 1.500 \text{ kN}$
 $R_{A_min} = 1.500 \text{ kN}$

Unfactored variable load reaction at support A

 $R_{A_Variable} = 1.000 \text{ kN}$

Reactions at support B

 $R_{B_max} = 4.500 \text{ kN}$
 $R_{B_min} = 4.500 \text{ kN}$

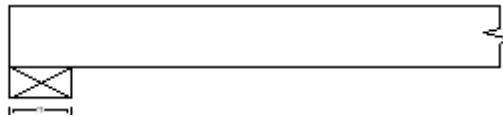
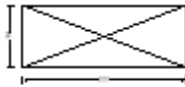
Unfactored variable load reaction at support B

 $R_{B_Variable} = 3.000 \text{ kN}$

Reactions at support C

 $R_{C_max} = 0.000 \text{ kN}$
 $R_{C_min} = 0.000 \text{ kN}$

Unfactored variable load reaction at support C

 $R_{C_Variable} = 0.000 \text{ kN}$


Timber section details

Breadth of timber sections

 $b = 200 \text{ mm}$

Depth of timber sections

 $h = 75 \text{ mm}$

Number of timber sections in member

 $N = 1$

Overall breadth of timber member

 $b_b = N \times b = 200 \text{ mm}$

Timber strength class - EN 338:2016 Table 1

C24

Member details

Load duration - cl.2.3.1.2

Permanent

Service class of timber - cl.2.3.1.3

3

Length of span 1

 $L_{s1} = 900 \text{ mm}$

Length of span 2

 $L_{s2} = 450 \text{ mm}$

Length of bearing

 $L_b = 75 \text{ mm}$

Section properties

Cross sectional area of member

 $A = N \times b \times h = 15000 \text{ mm}^2$

Section modulus

 $W_y = N \times b \times h^2 / 6 = 187500 \text{ mm}^3$
 $W_z = h \times (N \times b)^2 / 6 = 500000 \text{ mm}^3$

Second moment of area

 $I_y = N \times b \times h^3 / 12 = 7031250 \text{ mm}^4$
 $I_z = h \times (N \times b)^3 / 12 = 50000000 \text{ mm}^4$

Radius of gyration

 $r_y = \sqrt{I_y / A} = 21.7 \text{ mm}$
 $r_z = \sqrt{I_z / A} = 57.7 \text{ mm}$

Partial factor for material properties and resistances

Partial factor for material properties - Table 2.3

 $\gamma_M = 1.300$

Modification factors

Modification factor for load duration and moisture content - Table 3.1

 $k_{mod} = 0.500$

Deformation factor for service classes - Table 3.2

 $k_{def} = 2.000$

Depth factor for bending - exp.3.1

 $k_{h,m} = \min((150 \text{ mm} / h)^{0.2}, 1.3) = 1.149$

Depth factor for tension - exp.3.1

 $k_{h,t} = 1.000$

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Bending stress re-distribution factor - cl.6.1.6(2)

$$k_m = \mathbf{0.700}$$

Crack factor for shear resistance - cl.6.1.7(2)

$$k_{cr} = \mathbf{0.670}$$

Load configuration factor - exp.6.4

$$k_{c,90} = \mathbf{1.000}$$

System strength factor - cl.6.6

$$k_{sys} = \mathbf{1.000}$$

Effective length - Table 6.1

$$L_{ef} = 0.8 \times L_{s2} + 2 \times h = \mathbf{510 \text{ mm}}$$

Critical bending stress - exp.6.32

$$\sigma_{m,crit} = 0.78 \times (N \times b)^2 \times E_{0.05} / (h \times L_{ef}) = \mathbf{6036.078 \text{ N/mm}^2}$$

Relative slenderness for bending - exp.6.30

$$\lambda_{rel,m} = \sqrt{f_{m,k} / \sigma_{m,crit}} = \mathbf{0.063}$$

Lateral buckling factor - exp.6.34

$$k_{crit} = \mathbf{1.000}$$

Compression perpendicular to the grain - cl.6.1.5

Design compressive stress

$$\sigma_{c,90,d} = R_{B,max} / (N \times b \times L_b) = \mathbf{0.300 \text{ N/mm}^2}$$

Design compressive strength

$$f_{c,90,d} = k_{mod} \times k_{sys} \times k_{c,90} \times f_{c,90,k} / \gamma_M = \mathbf{0.962 \text{ N/mm}^2}$$

$$\sigma_{c,90,d} / f_{c,90,d} = \mathbf{0.312}$$

*PASS - Design compressive strength exceeds design compressive stress at bearing***Bending - cl 6.1.6**

Design bending stress

$$\sigma_{m,d} = M / W_y = \mathbf{3.600 \text{ N/mm}^2}$$

Design bending strength

$$f_{m,d} = k_{h,m} \times k_{mod} \times k_{sys} \times k_{crit} \times f_{m,k} / \gamma_M = \mathbf{10.603 \text{ N/mm}^2}$$

$$\sigma_{m,d} / f_{m,d} = \mathbf{0.340}$$

*PASS - Design bending strength exceeds design bending stress***Shear - cl.6.1.7**

Applied shear stress

$$\tau_d = 3 \times F / (2 \times k_{cr} \times A) = \mathbf{0.224 \text{ N/mm}^2}$$

Permissible shear stress

$$f_{v,d} = k_{mod} \times k_{sys} \times f_{v,k} / \gamma_M = \mathbf{1.538 \text{ N/mm}^2}$$

$$\tau_d / f_{v,d} = \mathbf{0.146}$$

*PASS - Design shear strength exceeds design shear stress***Deflection - cl.7.2**

Deflection limit

$$\delta_{lim} = \min(14 \text{ mm}, 0.004 \times L_{s2}) = \mathbf{1.800 \text{ mm}}$$

Instantaneous deflection due to permanent load

$$\delta_{instG} = \mathbf{0.000 \text{ mm}}$$

Final deflection due to permanent load

$$\delta_{finG} = \delta_{instG} \times (1 + k_{def}) = \mathbf{0.000 \text{ mm}}$$

Instantaneous deflection due to variable load

$$\delta_{instQ} = \mathbf{0.884 \text{ mm}}$$

Factor for quasi-permanent variable action

$$\psi_2 = \mathbf{0.3}$$

Final deflection due to variable load

$$\delta_{finQ} = \delta_{instQ} \times (1 + \psi_2 \times k_{def}) = \mathbf{1.414 \text{ mm}}$$

Total final deflection

$$\delta_{fin} = \delta_{finG} + \delta_{finQ} = \mathbf{1.414 \text{ mm}}$$

$$\delta_{fin} / \delta_{lim} = \mathbf{0.785}$$

PASS - Total final deflection is less than the deflection limit