ANDREW WARING
ASSOCIATES

STRUCTURAL CALCULATIONS

FOR PROPOSED ALTERATIONS

TO KITCHEN AT

WRAXALL MANOR

HIGHER WRAXALL DORSET

CLIENT :
Mr & Mrs R Boileau
Rampisham Manor
Rampisham
Dorchester

Architect:
TFH Reeve
Shaftesbury
SP7 9EP

Scanned to File	
Job No. 10545	

The Old Brewery House
Portersbridge Street
Romsey
Hampshire
SO51 8DJ

Date: May 2020

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CONTENTS

RESIDUAL DESIGN RISKS TO BE MANAGED BY CONTRACTOR

- Beam levels subject to opening up of ceilings, contractor to advise of difficulties in maintaining connections whist maximising headroom
- Foundation and/or concrete spreader details subject to size of exiting foundations and opening up of under croft. Allow for safe opening up and inspection.
- Safe propping and temporary stability to be managed by contractor

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ANDREW WARING ASSOCIATES Consulting Civil & Structural Engineers	Project: Wraxall Manor	Job No. 10545	Sheet No. L 1	Rev.
	Element: LOADINGS	Date 24.04.20	By dgs	Checked

			SERVICE kN/m²	γf	ULTIMATE kN/m²
ROOF					
SLATE			0.35		
BOARDING FELT AND BATTENS			0.10		
RAFTERS PURLINS AND TRUSSES			0.20		
	Σ LOAD ON SLOPE	=	0.65		
LOAD ON PLAN Roof slope = 45 °	LOAD ON PLAN	=	0.92	1.4	1.29
IMPOSED LOAD		=	0.30	1.6	0.48
	TOTAL	=	**1.22**		**1.77**
CEILING					
BOARDING			0.10		
BEAMS & JOISTS			0.15		
INSULATION			0.05		
SERVICES			0.10		
LATHE & PLASTER			0.45		
	Σ LOAD ON PLAN	=	0.85	1.4	1.19
IMPOSED LOAD		=	0.25	1.6	0.40
	TOTAL	=	**1.1**		**1.59**
FLAT ROOF					
ASHPHALT			0.35		
BOARDING			0.13		
BEAMS & JOISTS			0.17		
INSULATION			0.05		
SERVICES			0.10		
LATHE & PLASTER			0.45		
	Σ LOAD ON PLAN	=	1.25	1.4	1.75
IMPOSED LOAD (Access)		=	0.75	1.6	1.20
	TOTAL	=	**2.00**		**2.95**
TIMBER FLOOR					
BOARDING			0.13		
BEAMS & JOISTS			0.15		
SERVICES			0.10		
PARTITIONS			1.00		
LATHE & PLASTER			0.45		
	Σ LOAD ON PLAN	=	1.83	1.4	2.56
IMPOSED LOAD (Domestic)		=	1.50	1.6	2.40
	TOTAL	=	**3.33**		**4.96**

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WRAXALL MANOR	10545	2	
Element	**Date**	**By**	**Checked**
NEW KITCHEN OPENINGS	30/4/20	DGS	

ROOF LOAD ON INTERNAL MASONRY WALL

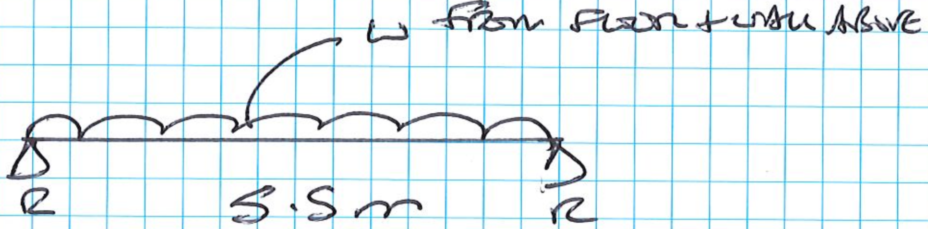
$$gk = \frac{5m}{2} \times (0.72 + 0.85) + \frac{4.8}{2}(1.25) = 7.43\,kN/m$$

$$qk = \text{–''–} \times (0.3 + 0.25) + \text{–''–}(0.75) = 3.18\,kN/m$$

SW WALL @ FIRST

$$gk = 3.5m \times 0.235 \times 21\,kN/m^3 = 17.27\,kN/m$$

BEAM 1



$$w_{gk} = 7.43 + 17.27 + \frac{6.3m}{2} \times 1.83 = 30.5\,kN/m$$

$$w_{qk} = 3.18 + \text{–''–} \times 1.50 = 7.9\,kN/m$$

FROM THE FOLLOWING

USE 254 x 254 UC 107 FOR A SHALLOW BEAM

OR

533 x 210 UB FOR A DEEP BEAM (TOO DEEP)

$R_{gk} = 86.1\,kN$ $R_{qk} = 21.7\,kN$ (SLS)

Andrew Waring Associates The Old Brewery House Portersbridge Street Romsey	Project WRAXALL MANOR				Job no. 10545	
	Calcs for KICTHEN BEAM 1				Start page no./Revision 3	
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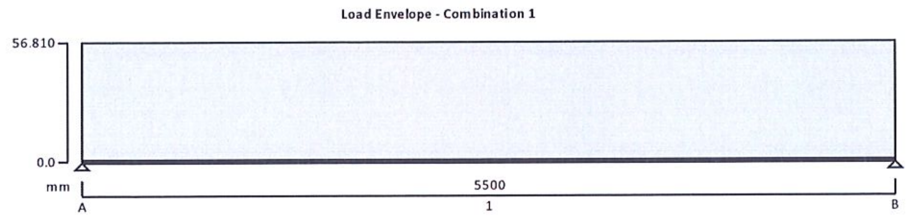
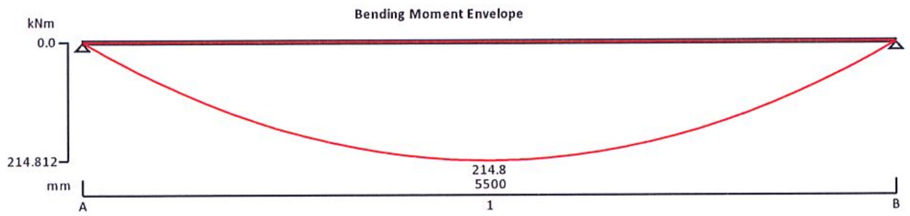
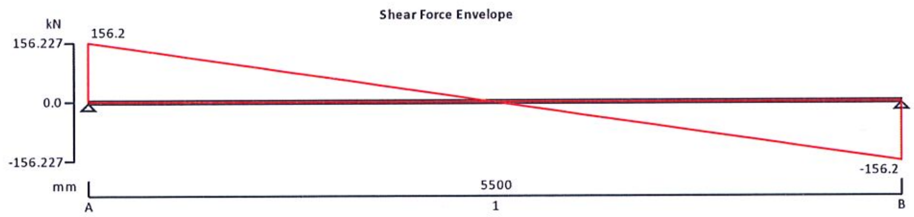
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 30.5 kN/m
	Imposed full UDL 7.9 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} = **214.8** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **156.2** kN	V_{min} = **-156.2** kN
Deflection	δ_{max} = **13.1** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **156.2** kN	R_{A_min} = **156.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **86.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **21.7** kN	
Maximum reaction at support B	R_{B_max} = **156.2** kN	R_{B_min} = **156.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **86.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **21.7** kN	

Section details

Section type — **UC 254x254x107 (BS4-1)**

Steel grade — **S355**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **20.5** mm

Design strength — p_y = **345** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2

Lateral restraint

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 — $K_{LT.A}$ = **1.00** + 2 × D

$K_{LT.B}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **0.89**

Internal compression parts - Table 11

Depth of section — d = **200.3** mm

d / t = 17.5 × ε <= 80 × ε — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **129.4** mm

b / T = 7.1 × ε <= 9 × ε — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **156.2** kN

d / t < 70 × ε

Web does not need to be checked for shear buckling

Shear area — A_v = t × D = **3414** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **706.6** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **214.8** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **512.1** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1} + 2 \times D =$ **6033** mm

Slenderness ratio — $\lambda = L_E / r_{yy} =$ **91.517**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — $u =$ **0.848**

Torsional index — $x =$ **12.393**

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

Ratio - cl.4.3.6.9 — $\beta_W =$ **1.000**

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **55.857**

Limiting slenderness - Annex B.2.2 — $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} =$ **30.632**

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

Bending strength - Section 4.3.6.5

Robertson constant — $\alpha_{LT} =$ **7.0**

Perry factor — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.177**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **648.5** N/mm²

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **554** N/mm²

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **265.6** N/mm²

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment — $M_2 =$ **161.1** kNm

Moment at centre-line of segment — $M_3 =$ **214.8** kNm

Moment at three quarter point of segment — $M_4 =$ **161.1** kNm

Maximum moment in segment — $M_{abs} =$ **214.8** kNm

Maximum moment governing buckling resistance — $M_{LT} = M_{abs} =$ **214.8** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) =$ **0.925**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment — $M_b = p_b \times S_{xx} =$ **394.2** kNm

$M_b / m_{LT} =$ **426.2** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 =$ **15.278** mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **13.094** mm

PASS - Maximum deflection does not exceed deflection limit

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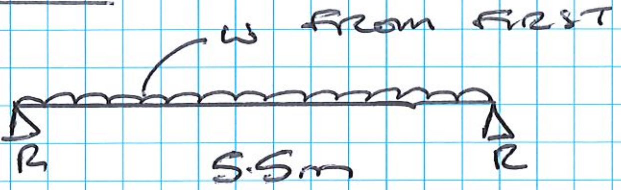
WRAXALL MANOR	10545	6	
Element	**Date**	**By**	**Checked**
NEW KITCHEN OPENINGS	30/4/20	DGS	

BEAM 2



$w_{gk} = \frac{4.7m}{2} \times 1.83 = 4.3kN/m$

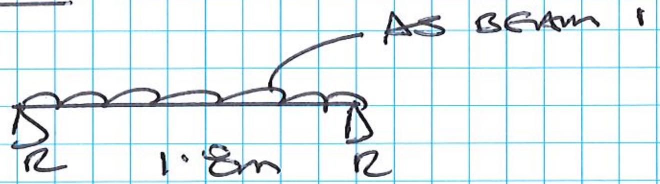
$w_{qk} = \text{—"—} \times 1.50 = 3.5kN/m$

FROM THE FOLLOWING USE

254 x 146 UB 37 (S355)

$R_{gk} = 12.8kN \quad q_k = 9.7kN$

BEAM 3



BY INSPECTION (FOR MIN 200 WIDE BEAM)

USE 203 x 203 UC 46 (S355)

$R_{gk} = (7.43 + 17.27 + 0.46)\frac{1.8}{2} = 22.6kN$

$R_{qk} = 3.18 \times \frac{1.8}{2} = 2.9kN$ (SLS)

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	Calcs for KICTHEN BEAM 2				Start page no./Revision 7	
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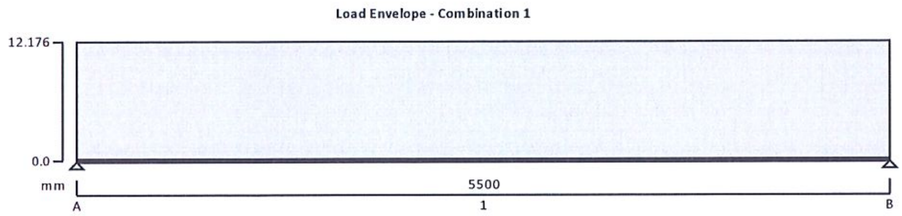
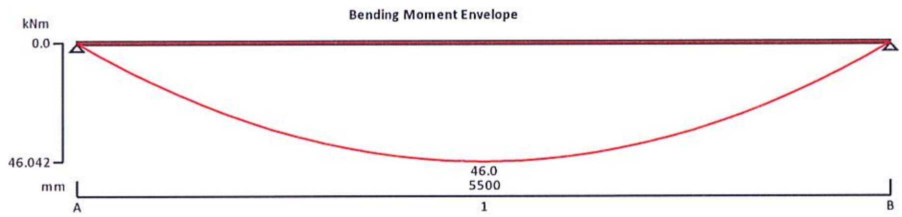
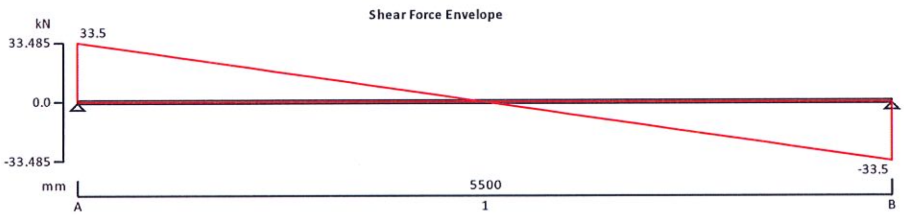
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	Dead full UDL 4.3 kN/m
	Imposed full UDL 3.53 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} = **46** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **33.5** kN	V_{min} = **-33.5** kN
Deflection	δ_{max} = **8.6** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **33.5** kN	R_{A_min} = **33.5** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **12.8** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **9.7** kN	
Maximum reaction at support B	R_{B_max} = **33.5** kN	R_{B_min} = **33.5** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **12.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **9.7** kN	

Section details

Section type — **UB 254x146x37 (BS4-1)**

Steel grade — **S355**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **10.9** mm

Design strength — p_y = **355** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2

Lateral restraint

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 — $K_{LT.A}$ = **1.00** + 2 × D

$K_{LT.B}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section — d = **219** mm

$d / t = 39.5 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **73.2** mm

$b / T = 7.6 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **33.5** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **1613** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v$ = **343.5** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min}))$ = **46** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **171.5** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = 1.0 \times L_{s1} + 2 \times D =$ **6012** mm

Slenderness ratio $\lambda = L_E / r_{yy} =$ **172.842**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter u = **0.890**

Torsional index x = **24.332**

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.730**

Ratio - cl.4.3.6.9 $\beta_W =$ **1.000**

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]} =$ **112.258**

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

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

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} =$ **7.0**

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.574**

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **160.6** N/mm²

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **303.9** N/mm²

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **115.9** N/mm²

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 =$ **34.5** kNm

Moment at centre-line of segment $M_3 =$ **46** kNm

Moment at three quarter point of segment $M_4 =$ **34.5** kNm

Maximum moment in segment $M_{abs} =$ **46** kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs} =$ **46** kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) =$ **0.925**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} =$ **56** kNm

$M_b / m_{LT} =$ **60.5** kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **15.278** mm

Maximum deflection span 1 $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) =$ **8.601** mm

PASS - Maximum deflection does not exceed deflection limit

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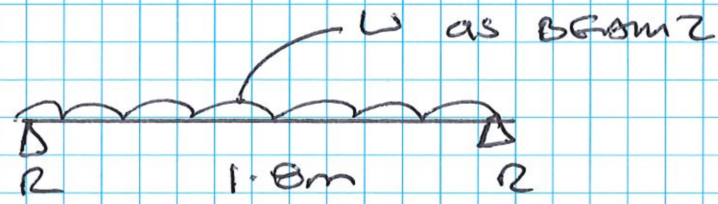
Element	**Date**	**By**	**Checked**
NEW KITCHEN OPENINGS	30/4/20	DGS	

BEAM 4

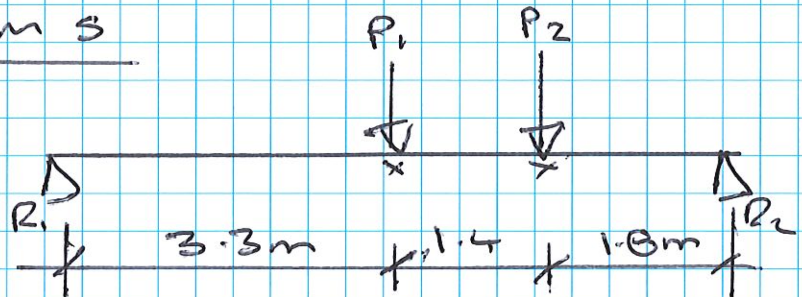


BY INSPECTION USE 203×133UB 25 (S355) FOR MIN REQ WIDTH OF BEAM

$R_{gk} = \frac{1.8m}{2} \times (4.3 + 0.25) = 4.1kN$

$R_{qk} = -''- \times 3.53 = 3.2kN$ (SLS)

BEAM 5



$P_1\ gk = R$ BEAM 2 + R BEAM 4 = 16.9 kN SLS

$P_1\ qk$ = " " = 12.9 kN

P_2 = R BEAM 1 + R BEAM 3 gk = 108.7kN

qk = 24.6kN

LIMITING TOTAL LOAD DEFLECTION ≤ 12mm

5a) USE 457×152 UB 82 (S355)

$R_1\ gk$ = 41 kN $R_1\ qk$ = 13.2 kN

$R_2\ gk$ = 89.6 qk = 24.3 (164.7kN uls)

Andrew Waring Associates The Old Brewery House Portersbridge Street Romsey	Project WRAXALL MANOR				Job no. 10545	
	Calcs for KICTHEN BEAM 5b				Start page no./Revision 1	
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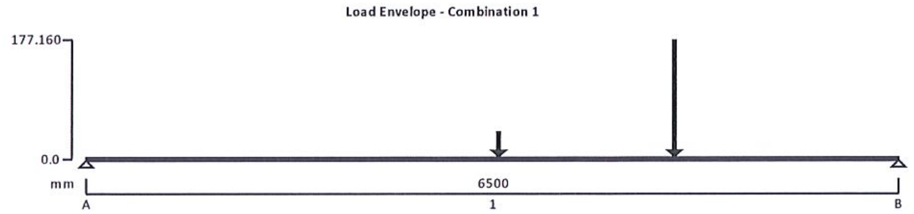
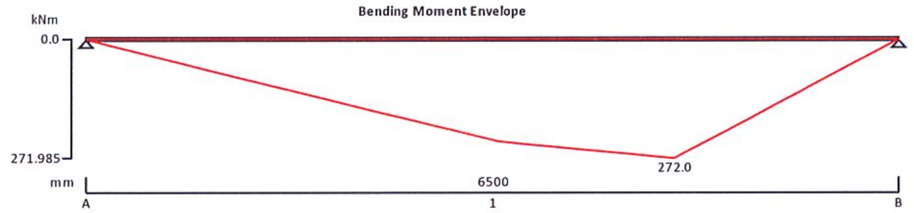
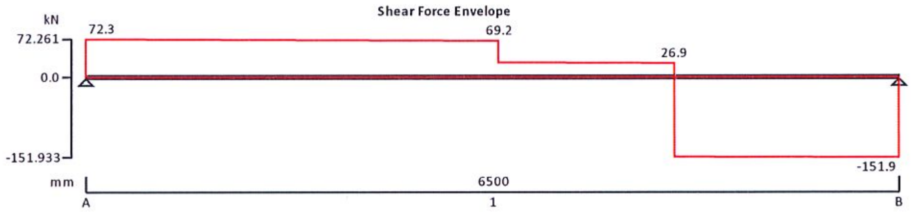
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.07







Support conditions

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

Applied loading

Beam loads	Dead self weight of beam × 1
	P1 - Dead point load 15.6 kN at 3300 mm
	P1 - Imposed point load 12 kN at 3300 mm
	P2 - Dead point load 100.6 kN at 4700 mm
	P2 - Imposed point load 22.7 kN at 4700 mm

Load combinations

Load combination 1	Support A	Dead × 1.40
		Imposed × 1.60
		Dead × 1.40
		Imposed × 1.60
	Support B	Dead × 1.40

Andrew Waring Associates The Old Brewery House Portersbridge Street Romsey	Project WRAXALL MANOR			Job no. 10545	
	Calcs for KICTHEN BEAM 5b			Start page no./Revision 12	
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Imposed × 1.60

Analysis results

Maximum moment	M_{max} = **272** kNm	M_{min} = **0** kNm
Maximum moment span 1 segment 1	$M_{s1\_seg1\_max}$ = **233.4** kNm	$M_{s1\_seg1\_min}$ = **0** kNm
Maximum moment span 1 segment 2	$M_{s1\_seg2\_max}$ = **272** kNm	$M_{s1\_seg2\_min}$ = **0** kNm
Maximum moment span 1 segment 3	$M_{s1\_seg3\_max}$ = **272** kNm	$M_{s1\_seg3\_min}$ = **0** kNm
Maximum shear	V_{max} = **72.3** kN	V_{min} = **-151.9** kN
Maximum shear span 1 segment 1	$V_{s1\_seg1\_max}$ = **72.3** kN	$V_{s1\_seg1\_min}$ = **0** kN
Maximum shear span 1 segment 2	$V_{s1\_seg2\_max}$ = **28.2** kN	$V_{s1\_seg2\_min}$ = **-150.3** kN
Maximum shear span 1 segment 3	$V_{s1\_seg3\_max}$ = **0** kN	$V_{s1\_seg3\_min}$ = **-151.9** kN
Deflection segment 4	δ_{max} = **11.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **72.3** kN	R_{A_min} = **72.3** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **37.7** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **12.2** kN	
Maximum reaction at support B	R_{B_max} = **151.9** kN	R_{B_min} = **151.9** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **82.8** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **22.5** kN	

Section details

Section type — **UB 457x152x67 (BS4-1)**

Steel grade — **S355**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **15.0** mm

Design strength — p_y = **355** N/mm^2

Modulus of elasticity — E = **205000** N/mm^2

Lateral restraint

Span 1 has lateral restraint at supports plus 3300 mm and 4700 mm

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** + 2 × D

$K_{LT.B}$ = **1.00** + 2 × D

Classification of cross sections - Section 3.5

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

Internal compression parts - Table 11

Depth of section — d = **407.6** mm

$d / t = 51.5 \times \varepsilon <= 80 \times \varepsilon$ — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **76.9** mm

$b / T = 5.8 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min}))$ = **151.9** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **4122** mm^2

Design shear resistance $P_v = 0.6 \times p_y \times A_v = \mathbf{878}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity at span 1 segment 1 - Section 4.2.5

Design bending moment $M = \max(\mathrm{abs}(M_{s1\_seg1\_max}), \mathrm{abs}(M_{s1\_seg1\_min})) = \mathbf{233.4}$ kNm

Moment capacity low shear - cl.4.2.5.2 $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{515.8}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling $L_E = ((1.0 + 1.0) \times L_{s1\_seg1} + 2 \times D) / 2 = \mathbf{3758}$ mm

Slenderness ratio $\lambda = L_E / r_{yy} = \mathbf{115.065}$

Equivalent slenderness - Section 4.3.6.7

Buckling parameter $u = \mathbf{0.869}$

Torsional index $x = \mathbf{33.587}$

Slenderness factor $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} = \mathbf{0.891}$

Ratio - cl.4.3.6.9 $\beta w = \mathbf{1.000}$

Equivalent slenderness - cl.4.3.6.7 $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta w]} = \mathbf{89.058}$

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

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

Bending strength - Section 4.3.6.5

Robertson constant $\alpha_{LT} = \mathbf{7.0}$

Perry factor $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) = \mathbf{0.412}$

Euler stress $p_E = \pi^2 \times E / \lambda_{LT}^2 = \mathbf{255.1}$ N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 = \mathbf{357.6}$ N/mm^2

Bending strength - Annex B.2.1 $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) = \mathbf{164.4}$ N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment $M_2 = \mathbf{59.3}$ kNm

Moment at centre-line of segment $M_3 = \mathbf{118}$ kNm

Moment at three quarter point of segment $M_4 = \mathbf{176}$ kNm

Maximum moment in segment $M_{abs} = \mathbf{233.4}$ kNm

Maximum moment governing buckling resistance $M_{LT} = M_{abs} = \mathbf{233.4}$ kNm

Equivalent uniform moment factor for lateral-torsional buckling

$m_{LT} = \max(0.2 + (0.15 \times M_2 + 0.5 \times M_3 + 0.15 \times M_4) / M_{abs}, 0.44) = \mathbf{0.604}$

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment $M_b = p_b \times S_{xx} = \mathbf{238.9}$ kNm

$M_b / m_{LT} = \mathbf{395.6}$ kNm

PASS - Buckling resistance moment exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = \min(13\text{ mm}, L_{s1} / 360) = \mathbf{13}$ mm

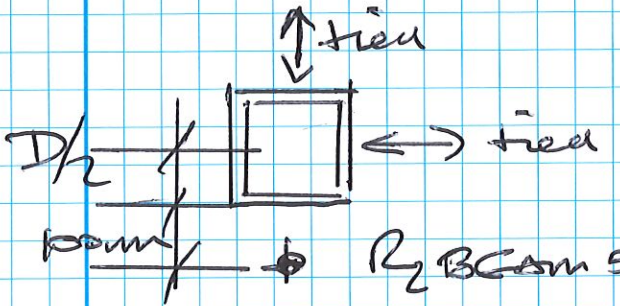
Maximum deflection span 1 $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min})) = \mathbf{11.888}$ mm

PASS - Maximum deflection does not exceed deflection limit

ANDREW WARING ASSOCIATES	Project: WRAXALL MANOR	Job Nº: 10545	Sheet Nº: 14	Rev.
Tel: 01794 524447 AWA ©	Element: KITCHEN BEAM SUPPORTS	Date: 1/8/20	By: DGS	Checked

POST IN CORNER

TRY 120 SHS



$P_{ULT} = 164.7 + (3.0m \times 0.23 \times 1.4) = 166\,kN.$

$M_{ULT} = 164.7 \times (0.06 + 0.1m) = 26.4\,kNm$

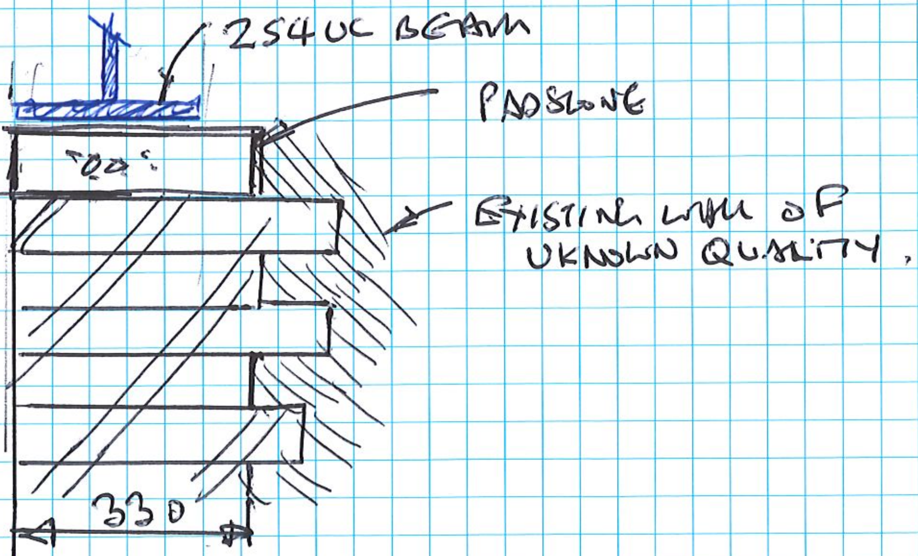
FROM THE FOLLOWING

USE 120 x 120 x 8 SHS (S355)

PIER FOR R_1 BEAM 1

$R_{ult} = 86.1\,kN$ $R_{gk} = 21.7\,kN$

CONSIDER REBUILDING REVEAL IN CLASS B ENG BRK



STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.07

Section details

Section type **SHS 120x120x8.0 (Tata Steel Celsius)**

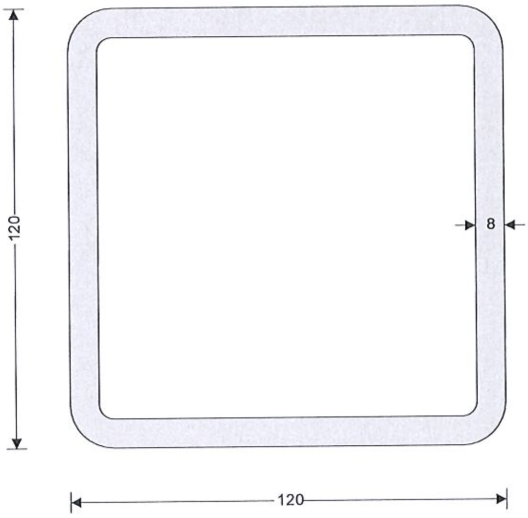
Steel grade **S355**

From table 9: Design strength p_y

Thickness of element $t = \mathbf{8.0}$ mm

Design strength $p_y = \mathbf{355}$ N/mm^2

Modulus of elasticity $E = \mathbf{205000}$ N/mm^2



Lateral restraint

Distance between major axis restraints $L_x = \mathbf{3000}$ mm

Distance between minor axis restraints $L_y = \mathbf{3000}$ mm

Effective length factors

Effective length factor in major axis $K_x = \mathbf{1.00}$

Effective length factor in minor axis $K_y = \mathbf{1.00}$

Effective length factor for lateral-torsional buckling $K_{LT} = \mathbf{1.00}$

Classification of cross sections - Section 3.5

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

Web - major axis - Table 12

Depth of section $d = D - 3 \times t = \mathbf{96}$ mm

Stress ratios $r1 = \min(F_c / (2 \times d \times t \times p_{yw}), 1) = \mathbf{0.304}$

$r2 = F_c / (A \times p_{yw}) = \mathbf{0.133}$

$d / t = 13.6 \times \varepsilon <= \max(64 \times \varepsilon / (1 + r1), 40 \times \varepsilon)$ Class 1 plastic

Flange - major axis - Table 12

Width of section $b = B - 3 \times t = \mathbf{96}$ mm

$b / t = 13.6 \times \varepsilon <= 40 \times \varepsilon$ Class 3 semi-compact

Section is class 3 semi-compact

Shear capacity - Section 4.2.3

Design shear force — $F_{y,v}$ = **75** kN

$(D - 3 \times t) / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = A \times D / (D + B)$ = **1758** mm^2

Design shear resistance — $P_{y,v} = 0.6 \times p_y \times A_v$ = **374.4** kN

PASS - Design shear resistance exceeds design shear force

Shear capacity - Section 4.2.3

Design shear force — $F_{x,v}$ = **75** kN

Shear area — $A_v = A_x$ = **1758** mm^2

Design shear resistance — $P_{x,v} = 0.6 \times p_y \times A_v$ = **374.4** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — M = **26.4** kNm

Effective plastic modulus - Section 3.5.6

Limiting value for class 2 compact flange — $\beta_{2f} = \min(32 \times \varepsilon, 62 \times \varepsilon - 0.5 \times d / t)$ = **28.165**

Limiting value for class 3 semi-compact flange — $\beta_{3f} = 40 \times \varepsilon$ = **35.206**

Limiting value for class 2 compact web — $\beta_{2w} = \max(80 \times \varepsilon / (1 + r1), 40 \times \varepsilon)$ = **53.979**

Limiting value for class 3 semi-compact web — $\beta_{3w} = \max(120 \times \varepsilon / (1 + 2 \times r2), 40 \times \varepsilon)$ = **83.423**

Effective plastic modulus - cl.3.5.6.3

$S_{eff} = \min(Z + (S - Z) \times \min([(\beta_{3w} / (d / t) - 1) / (\beta_{3w} / \beta_{2w} - 1)], [(\beta_{3f} / (b / t) - 1) / (\beta_{3f} / \beta_{2f} - 1)]), S)$ = **146457** mm^3

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{eff}, 1.2 \times p_y \times Z)$ = **51.6** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_y$ = **3000** mm

Slenderness ratio — $\lambda = L_E / r_{yy}$ = **66.000**

Equivalent slenderness - Annex B.2.6.1

Torsion constant — J = **11601421** mm^4

$\gamma_b = (1 - I_{yy} / I_{xx}) \times (1 - J / (2.6 \times I_{xx}))$ = **0.000**

$\phi_b = [S_{xx}^2 \times \gamma_b / (A \times J)]^{0.5}$ = **0.000**

Ratio - cl.4.3.6.9 — $\beta_W = S_{eff} / S_{xx}$ = **1.000**

Equivalent slenderness — $\lambda_{LT} = 2.25 \times \sqrt{[\phi_b \times \lambda \times \beta_W]}$ = **0.000**

Limiting slenderness - Annex B.2.2 — $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **30.198**

$\lambda_{LT} < \lambda_{L0}$ - No allowance need be made for lateral-torsional buckling

Buckling resistance moment - Section 4.3.6.4

Bending strength — $p_b = p_y$ = **355** N/mm^2

Buckling resistance moment — $M_b = p_b \times S_{eff}$ = **52** kNm

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force — F_c = **166** kN

Effective length for major (x-x) axis buckling - Section 4.7.3

Effective length for buckling — $L_{Ex} = L_x \times K_x$ = **3000** mm

Slenderness ratio - cl.4.7.2 — $\lambda_x = L_{Ex} / r_{xx}$ = **66.000**

Compressive strength - Section 4.7.5

Limiting slenderness — $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **15.099**

Strut curve - Table 23 — a

Robertson constant — α_x = **2.0**

Perry factor — $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000$ = **0.102**

Euler stress — $p_{Ex} = \pi^2 \times E / \lambda_x^2$ = **464.5** N/mm^2

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2$ = **433.4** N/mm^2

Compressive strength - Annex C.1 — $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5})$ = **282** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 — $P_{cx} = A \times p_{cx}$ = **991.2** kN

PASS - Compression resistance exceeds design compression force

Effective length for minor (y-y) axis buckling - Section 4.7.3

Effective length for buckling — $L_{Ey} = L_y \times K_y$ = **3000** mm

Slenderness ratio - cl.4.7.2 — $\lambda_y = L_{Ey} / r_{yy}$ = **66.000**

Compressive strength - Section 4.7.5

Limiting slenderness — $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5}$ = **15.099**

Strut curve - Table 23 — a

Robertson constant — α_y = **2.0**

Perry factor — $\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000$ = **0.102**

Euler stress — $p_{Ey} = \pi^2 \times E / \lambda_y^2$ = **464.5** N/mm^2

$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2$ = **433.4** N/mm^2

Compressive strength - Annex C.1 — $p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5})$ = **282** N/mm^2

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 — $P_{cy} = A \times p_{cy}$ = **991.2** kN

PASS - Compression resistance exceeds design compression force

Compression members with moments - Section 4.8.3

Comb.compression & bending check - cl.4.8.3.2 — $F_c / (A \times p_y) + M / M_c$ = **0.645**

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b — $M_{LT} = M_x$ = **26.40** kNm

Equivalent uniform moment factor for major axis flexural buckling

m_x = **1.000**

m_y = **1.000**

Buckling resistance checks - cl.4.8.3.3.3 — $F_c / P_{cx} + m_x \times M / M_c \times (1 + 0.5 \times F_c / P_{cx})$ = **0.722**

$F_c / P_{cy} + 0.5 \times m_{LT} \times M_{LT} / M_{cx}$ = **0.423**

PASS - Member buckling resistance checks are satisfied

Andrew Waring Associates The Old Brewery House Portersbridge Street Romsey	Project Wraxall Manor			Job no. 10545	
	Calcs for Pier for R1 Beam 1			Start page no./Revision 18 1	
	Calcs by DS	Calcs date 01/05/2020	Checked by	Checked date	Approved by	Approved date

VERTICAL LOADING RECTANGULAR COLUMN (BS5628-1:2005)

TEDDS calculation version 1.0.02

Compressive strength from Table 2 BS5628:Part 1 - Clay or calcium silicate bricks

Mortar designation	Mortar = **"ii"**
Brick compressive strength	p_{unit} = **75.0** N/mm^2
Characteristic compressive strength	f_k = **12.00** N/mm^2
Column width	b = **330** mm
Column thickness	t = **330** mm
Column height	h = **3.00** m
Column slenderness - minor axis (Clause 24.1)	λ_t = h/t = **9.09**
Column slenderness - major axis (Clause 24.1)	λ_b = h/b = **9.09**
Maximum slenderness	λ_{max} = max(λ_t, λ_b) = **9.09**

Slenderness < 27 - OK

Partial safety factor for material (Table 4)	γ_m = **3.5**

Load eccentricity

Eccentricity of applied load about minor axis	e_{xt} = **50.0** mm
Eccentricity of applied load about major axis	e_{xb} = **50.0** mm

Capacity reduction factor: minor axis

Eccentricity due to slenderness	e_{at} = max(0 mm, t × (((h/t)^2/2400)-0.015)) = **6.4** mm
Design eccentricity	e_{tt} = 0.6×max(abs(e_{xt}), 0.05×t)+ e_{at} = **36.4** mm
	e_{mt} = max(abs(e_{xt}),e_{tt}) = **50.0** mm
Capacity reduction factors	β_{tcalc} = max(0, 1.1 × (1 - (2 × e_{mt} / t))) = **0.77**
	β_{max} = 1.0
	β_t = min (β_{tcalc}, β_{max}) = **0.77**

Capacity reduction factor: major axis

Eccentricity due to slenderness	e_{ab} = max(0 mm, b × (((h/b)^2/2400)-0.015)) = **6.4** mm
Design eccentricity	e_{tb} = 0.6× max(abs(e_{xb}), 0.05×b)+ e_{ab} = **36.4** mm
	e_{mb} = max(abs(e_{xb}),e_{tb}) = **50.0** mm
Capacity reduction factors	β_{bcalc} = max(0, 1.1 × (1 - (2 × e_{mb} / b))) = **0.77**
	β_{max} = 1.0
	β_b = min (β_{bcalc}, β_{max}) = **0.77**
Minimum capacity reduction factor	β_{min} = min(β_t, β_b) = **0.77**

Design vertical load resistance

Compressive strength correction factor	
Plan area of column	A = t × b = **0.11** m^2
For small plan area (Clause 19.1.2)	c = min(1.0, 0.7+(1.5 m^{-2})×A) = **0.86**
Design vertical load resistance	DVLR = β_{min} × t × b × c × f_k / γ_m = **247.135** kN
Applied factored vertical load on column	V = **142.000** kN

Column - OK