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8 BUSBY PLACE, LONDON

ENLARGED REAR OPENING

STRUCTURAL CALCULATIONS

(PAGE 01 TO 11)

Client: MS. EMILY THORNE

Job ref.: 1318

Engineer: MJW

Issue	01					
Date	05/12/2013					
Authorised						

Project	:	8 BUSBY PLACE, LONDON
Section	:	ENLARGED REAR OPENING
	:	STRUCTURAL CALCULATIONS
Job No	:	1318
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Ref. 1.

NEW OPENING TO REAR WALL.

3.5M SPAN.

SUPPORTS EXTERNAL WALL, FLOORS & ROOF ABOVE.

LOADINGS

	DEAD	LIVE	
FLOOR	0.7	1.5	
PARTITIONS	–	1.0	
	0.7	2.5	kN/m²
ROOF	0.9	0.6	kN/m²
WALL	4.5	–	kN/m²

ROOF SPANS FRONT TO BACK 9M.

9.0/2 @ 0.9, 0.6 = 4.0   2.7 kN/M

FLOORS 3NO. SPANNING 4M.

3 x 4.0/2 @ 0.7, 2.5 = 4.2   15 kN/M.

EXTERNAL WALL 8.5M HIGH.

8.5 @ 4.5   38.3   – kN/M

Σ 46.5   17.7 kN/M.

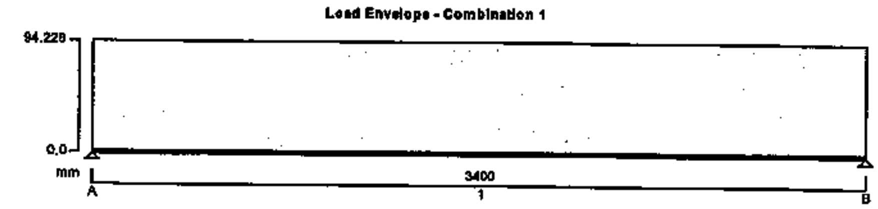
REFER TO TEDDS CALCULATIONS

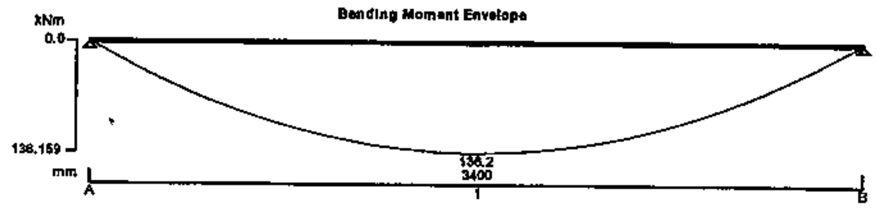
Tedds	Project: 8 BUSBY PLACE, NW5 2SR			Job no. 1318	
	Calcs for: REAR OPENING - BEAM			Start page no./Revision: R	
	Calcs by: M	Calcs date: 02/12/2013	Checked by	Checked date	Approved by	Approved date

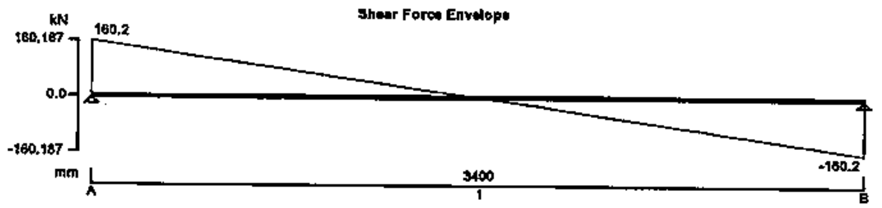
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.04







Support conditions

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

Applied loading

Beam loads	Imposed full UDL 17.7 kN/m
	Dead full UDL 46.5 kN/m
	Dead self weight of beam × 1

Load combinations

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

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Imposed × 1.60

Analysis results

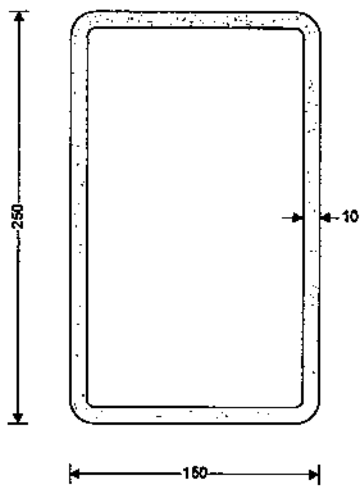
Maximum moment	M_{max} = **136.2** kNm	M_{min} = **0** kNm
Maximum shear	V_{max} = **160.2** kN	V_{min} = **-160.2** kN
Deflection	δ_{max} = **8.9** mm	δ_{min} = **0** mm
Maximum reaction at support A	R_{A_max} = **160.2** kN	R_{A_min} = **160.2** kN
Unfactored dead load reaction at support A	R_{A_Dead} = **80** kN	
Unfactored imposed load reaction at support A	$R_{A\_Imposed}$ = **30.1** kN	
Maximum reaction at support B	R_{B_max} = **160.2** kN	R_{B_min} = **160.2** kN
Unfactored dead load reaction at support B	R_{B_Dead} = **80** kN	
Unfactored imposed load reaction at support B	$R_{B\_Imposed}$ = **30.1** kN	

Section details

Section type	**RHS 250x150x10.0 (Corus Celsius)**
Steel grade	**S275**

From table 9: Design strength p_y

Thickness of element	t = **10.0** mm
Design strength	p_y = **275** 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**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]} = \mathbf{1.00}$

Web - major axis - Table 12

Depth of section $d = D - 3 \times t = \mathbf{220}$ mm

$d / t = 22.0 \times \varepsilon <= 64 \times \varepsilon$ Class 1 plastic

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Flange - major axis - Table 12

Width of section — $b = B - 3 \times t = \mathbf{120}$ **mm**

$b / t = 12.0 \times \varepsilon <= \min(28 \times \varepsilon, 80 \times \varepsilon - d / t)$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = \max(\text{abs}(V_{max}), \text{abs}(V_{min})) = \mathbf{160.2}$ **kN**

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

Web does not need to be checked for shear buckling

Shear area — $A_v = A \times D / (D + B) = \mathbf{4683}$ mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v = \mathbf{772.7}$ kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = \max(\text{abs}(M_{s1\_max}), \text{abs}(M_{s1\_min})) = \mathbf{136.2}$ kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) = \mathbf{163}$ kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1} = \mathbf{3400}$ **mm**

Slenderness ratio — $\lambda = L_E / r_{yy} = \mathbf{56.072}$

Limiting slenderness ratio - Table 15 — $435 \times (275 \text{ N/mm}^2 / p_y) = \mathbf{435}$

λ is less than limiting value, no allowance need be made for lateral-torsional buckling

PASS - Moment capacity exceeds design bending moment

Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection — $\delta_{lim} = L_{s1} / 360 = \mathbf{9.444}$ mm

Maximum deflection span 1 — $\delta = \max(\text{abs}(\delta_{max}), \text{abs}(\delta_{min})) = \mathbf{8.906}$ mm

PASS - Maximum deflection does not exceed deflection limit

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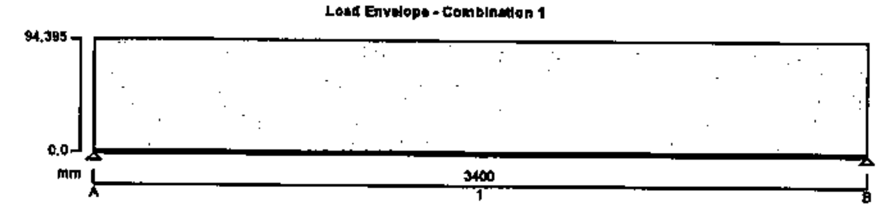
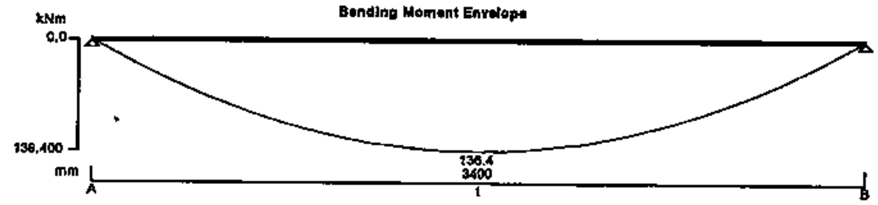
STEEL BEAM ANALYSIS & DESIGN (BS5950)

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

TEDDS calculation version 3.0.04





Support conditions

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

Applied loading

Beam loads	Imposed full UDL 17.7 kN/m
	Dead full UDL 46.5 kN/m
	Dead self weight of beam × 1

Load combinations

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

Tedds	Project: 8 BUSBY PLACE, NW5 2SR				Job no. 1318	
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Outstand flanges - Table 11

Width of section — $b = B / 2 =$ **103.2** mm

$b / T = 5.9 \times \varepsilon <= 9 \times \varepsilon$ — Class 1 plastic

Section is class 1 plastic

Shear capacity - Section 4.2.3

Design shear force — $F_v = max(abs(V_{max}), abs(V_{min})) =$ **160.5** kN

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D =$ **2158** mm^2

Design shear resistance — $P_v = 0.6 \times p_y \times A_v =$ **343.1** kN

PASS - Design shear resistance exceeds design shear force

Moment capacity - Section 4.2.5

Design bending moment — $M = max(abs(M_{s1\_max}), abs(M_{s1\_min})) =$ **136.4** kNm

Moment capacity low shear - cl.4.2.5.2 — $M_c = min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx}) =$ **211.7** kNm

Effective length for lateral-torsional buckling - Section 4.3.5

Effective length for lateral torsional buckling — $L_E = 1.0 \times L_{s1} =$ **3400** mm

Slenderness ratio — $\lambda = L_E / r_{yy} =$ **64.187**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — $u =$ **0.853**

Torsional index — $x =$ **11.926**

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25} =$ **0.799**

Ratio - cl.4.3.6.9 — $\beta w =$ **1.000**

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta w]} =$ **43.747**

Limiting slenderness - Annex B.2.2 — $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5} =$ **34.951**

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

Bending strength - Section 4.3.6.5

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

Perry factor — $\eta_{LT} = max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0) =$ **0.062**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2 =$ **1057.2** N/mm^2

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2 =$ **693.7** N/mm^2

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5}) =$ **245.3** N/mm^2

Equivalent uniform moment factor - Section 4.3.6.6

Moment at quarter point of segment — $M_2 =$ **102.3** kNm

Moment at centre-line of segment — $M_3 =$ **136.4** kNm

Moment at three quarter point of segment — $M_4 =$ **102.3** kNm

Maximum moment in segment — $M_{abs} =$ **136.4** kNm

Maximum moment governing buckling resistance — $M_{LT} = M_{abs} =$ **136.4** kNm

Equivalent uniform moment factor for lateral-torsional buckling

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

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment — $M_b = p_b \times S_{xx} =$ **196** kNm

$M_b / m_{LT} =$ **211.8** kNm

PASS - Moment capacity exceeds design bending moment

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Check vertical deflection - Section 2.5.2

Consider deflection due to dead and imposed loads

Limiting deflection $\delta_{lim} = L_{s1} / 360 =$ **9.444** mm

Maximum deflection span 1 $\delta = \max(\mathrm{abs}(\delta_{max}), \mathrm{abs}(\delta_{min})) =$ **7.231** mm

PASS - Maximum deflection does not exceed deflection limit

STEEL MEMBER DESIGN (BS5950)

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

TEDDS calculation version 3.0.04

Section details

Section type — **UC 152x152x37 (BS4-1)**

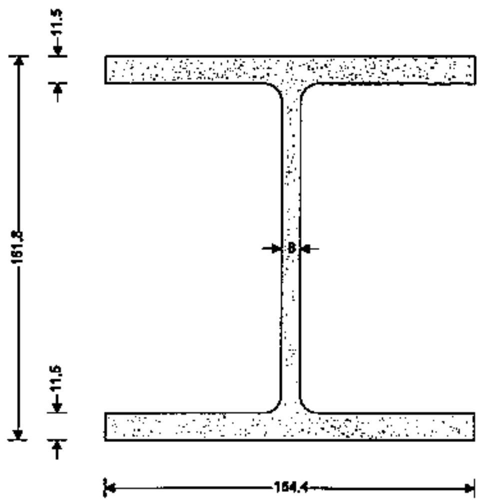
Steel grade — **S275**

From table 9: Design strength p_y

Thickness of element — max(T, t) = **11.5** mm

Design strength — p_y = **275** N/mm²

Modulus of elasticity — E = **205000** N/mm²



Lateral restraint

Distance between major axis restraints — L_x = **3000** mm

Distance between minor axis restraints — L_y = **3000** 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} = **1.00**

Classification of cross sections - Section 3.5

$\varepsilon = \sqrt{[275 \text{ N/mm}^2 / p_y]}$ = **1.00**

Internal compression parts - Table 11

Depth of section — d = **123.6** mm

Stress ratios — r1 = min(F_c / (d × t × p_{yw}), 1) = **0.588**

r2 = F_c / (A × p_{yw}) = **0.123**

d / t = 15.5 × ε <= max(80 × ε / (1 + r1), 40 × ε) — Class 1 plastic

Outstand flanges - Table 11

Width of section — b = B / 2 = **77.2** mm

b / T = 6.7 × ε <= 9 × ε — Class 1 plastic

Section is class 1 plastic

Shear capacity (parallel to y-axis) - Section 4.2.3

Design shear force — $F_{y,v}$ = **100 kN**

$d / t < 70 \times \varepsilon$

Web does not need to be checked for shear buckling

Shear area — $A_v = t \times D$ = **1294 mm²**

Design shear resistance — $P_{y,v} = 0.6 \times p_y \times A_v$ = **213.6 kN**

PASS - Design shear resistance exceeds design shear force

Shear capacity (parallel to x-axis) - Section 4.2.3

Design shear force — $F_{x,v}$ = **0 kN**

Moment capacity major (x-x) axis - Section 4.2.5

Design bending moment — M_x = **20 kNm**

Moment capacity low shear - cl.4.2.5.2 — $M_{cx} = \min(p_y \times S_{xx}, 1.2 \times p_y \times Z_{xx})$ = **84.9 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 — $\lambda = L_E / r_{yy}$ = **77.483**

Equivalent slenderness - Section 4.3.6.7

Buckling parameter — u = **0.848**

Torsional index — x = **13.334**

Slenderness factor — $v = 1 / [1 + 0.05 \times (\lambda / x)^2]^{0.25}$ = **0.781**

Ratio - cl.4.3.6.9 — β_W = **1.000**

Equivalent slenderness - cl.4.3.6.7 — $\lambda_{LT} = u \times v \times \lambda \times \sqrt{[\beta_W]}$ = **51.325**

Limiting slenderness - Annex B.2.2 — $\lambda_{L0} = 0.4 \times (\pi^2 \times E / p_y)^{0.5}$ = **34.310**

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

Bending strength - Section 4.3.6.5

Robertson constant — α_{LT} = **7.0**

Perry factor — $\eta_{LT} = \max(\alpha_{LT} \times (\lambda_{LT} - \lambda_{L0}) / 1000, 0)$ = **0.119**

Euler stress — $p_E = \pi^2 \times E / \lambda_{LT}^2$ = **768.1 N/mm²**

$\phi_{LT} = (p_y + (\eta_{LT} + 1) \times p_E) / 2$ = **567.3 N/mm²**

Bending strength - Annex B.2.1 — $p_b = p_E \times p_y / (\phi_{LT} + (\phi_{LT}^2 - p_E \times p_y)^{0.5})$ = **234.7 N/mm²**

Equivalent uniform moment factor - Section 4.3.6.6

Equivalent uniform moment factor for LTB — m_{LT} = **1.000**

Buckling resistance moment - Section 4.3.6.4

Buckling resistance moment — $M_b = p_b \times S_{xx}$ = **72.5 kNm**

M_b / m_{LT} = **72.5 kNm**

PASS - Buckling resistance moment exceeds design bending moment

Moment capacity minor (y-y) axis - Section 4.2.5

Design bending moment — M_y = **8 kNm**

Moment capacity low shear - cl.4.2.5.2 — $M_{cy} = \min(p_y \times S_{yy}, 1.2 \times p_y \times Z_{yy})$ = **30.2 kNm**

PASS - Moment capacity exceeds design bending moment

Compression members - Section 4.7

Design compression force — F_c = **160 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**

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Slenderness ratio - cl.4.7.2 $\lambda_x = L_{Ex} / r_{xx} = \mathbf{43.797}$

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{17.155}$

Strut curve - Table 23 **b**

Robertson constant $\alpha_x = \mathbf{3.5}$

Perry factor $\eta_x = \alpha_x \times (\lambda_x - \lambda_0) / 1000 = \mathbf{0.093}$

Euler stress $p_{Ex} = \pi^2 \times E / \lambda_x^2 = \mathbf{1054.8}$ N/mm²

$\phi_x = (p_y + (\eta_x + 1) \times p_{Ex}) / 2 = \mathbf{714.1}$ N/mm²

Compressive strength - Annex C.1 $p_{cx} = p_{Ex} \times p_y / (\phi_x + (\phi_x^2 - p_{Ex} \times p_y)^{0.5}) = \mathbf{245.2}$ N/mm²

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cx} = A \times p_{cx} = \mathbf{1155.2}$ kN

PASS - Compression resistance exceeds design compression force

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

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

Slenderness ratio - cl.4.7.2 $\lambda_y = L_{Ey} / r_{yy} = \mathbf{77.483}$

Compressive strength - Section 4.7.5

Limiting slenderness $\lambda_0 = 0.2 \times (\pi^2 \times E / p_y)^{0.5} = \mathbf{17.155}$

Strut curve - Table 23 **c**

Robertson constant $\alpha_y = \mathbf{5.5}$

Perry factor $\eta_y = \alpha_y \times (\lambda_y - \lambda_0) / 1000 = \mathbf{0.332}$

Euler stress $p_{Ey} = \pi^2 \times E / \lambda_y^2 = \mathbf{337}$ N/mm²

$\phi_y = (p_y + (\eta_y + 1) \times p_{Ey}) / 2 = \mathbf{361.9}$ N/mm²

Compressive strength - Annex C.1 $p_{cy} = p_{Ey} \times p_y / (\phi_y + (\phi_y^2 - p_{Ey} \times p_y)^{0.5}) = \mathbf{166.2}$ N/mm²

Compression resistance - Section 4.7.4

Compression resistance - cl.4.7.4 $P_{cy} = A \times p_{cy} = \mathbf{783}$ 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 $F_c / (A \times p_y) + M_x / M_{cx} + M_y / M_{cy} = \mathbf{0.624}$

PASS - Combined bending and compression check is satisfied

Member buckling resistance - Section 4.8.3.3

Max major axis moment governing M_b $M_{LT} = M_x = \mathbf{20.00}$ kNm

Equivalent uniform moment factor for major axis flexural buckling

$m_x = \mathbf{1.000}$

$m_y = \mathbf{1.000}$

Buckling resistance checks - cl.4.8.3.3.2 $F_c / P_{cx} + m_x \times M_x / M_{cx} \times (1 + 0.5 \times F_c / P_{cx}) + 0.5 \times m_{yx} \times M_y / M_{cy} = \mathbf{0.523}$

$F_c / P_{cy} + m_{LT} \times M_{LT} / M_b + m_y \times M_y / M_{cy} \times (1 + F_c / P_{cy}) = \mathbf{0.799}$

Interactive buckling $m_x \times M_x \times (1 + 0.5 \times (F_c / P_{cx})) / (M_{cx} \times (1 - F_c / P_{cx})) + m_y \times M_y \times (1 + (F_c / P_{cy})) / (M_{cy} \times (1 - F_c / P_{cy})) = \mathbf{0.693}$

PASS - Member buckling resistance checks are satisfied