Structural
Engineers
Cambridge Ltd

Calculations
for
STOCKBRIDGE MEADOWS
BOARDWALK
MELBOURN

21-494

August 2021

Structural Engineers Cambridge Ltd
The White Horse
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Cambridge CB22 3EF

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	Ref. No. Q14494
	Date AUG 2021 **Notes**

General and Safety Notes

Building Regulations Approval

Most structural alterations will require Building Control approval and must be examined by a Building Inspector prior to concealing or covering structural members. It is the client's and contractor's responsibility to ensure that applications and inspections have been carried out.

Planning Permission

Planning permission may or may not be required in connection with the work described herein, and a suitably qualified architect or planning advisor should be consulted before commencement of work.

Party Wall Agreements

Structural alterations to a Party Wall, or excavations in the vicinity of a neighbour's property, will require the adjoining owner's consent under the Party Wall Act 1996. This will require a Party Wall Agreement to be made before commencement of the works. Advice may be obtained from the government Planning Portal www.planningportal.gov.uk or by contacting a Chartered Building Surveyor.

Safety

This information is provided in the expectation that those appointed to carry out the work are suitably qualified and experienced contractors. If there is any doubt about aspects of the specification, the engineer should be contacted before commencement of work on site. The work described should be capable of being carried out using the normal range of skills and equipment expected of a competent general contractor. If any operations are outside this norm a method statement or more detailed description of the procedure should be requested.

Excavations in excess of 1.2 metres deep or in unstable ground should not be entered by any person unless a system of shoring or ground support has been installed.

Any variation between the architect's drawings and specification and this information should be brought to the attention of the architect and engineer immediately.

Temporary Support

Installation of beams, lintels or other supporting structures should be undertaken only with the provision of suitable temporary support to the structure above. Attention should be paid to the nature of the supported loads (from the calculations) and the capacity of props, shores and needle beams as appropriate. If in doubt about the requirements, contact the engineer before commencement of work.

Dimensions etc.

The dimensions given in these documents are for design purposes only and should be checked on site for construction. Beam sizes are given for identification of the section and the span dimension is between centrelines of supports (i.e. neither the length of the beam nor the opening width).

Copyright

N.B. Dimensions given on pages of analysis are <u>not</u> for fabrication purposes.
The contractor is responsible for checking all dimensions for construction.

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Calculation LOADINGS / JOISTS

Engr.

BS 1648
BS 6399

Decking	0.15
Joists	0.20
LL	5.00
	5.35 kN/m²

Joists @ 300 c/c BS 5268

Load/m $5.35 \times 0.30 = 1.60 kN/m$

Spanning 4.0m

$M = 1.60 \times 4.0^2/8 = 3.20 kN/m$

$Z_{REQD} = \dfrac{3.20 \times 10^6}{7.5 \times 0.8 \times 1.10 \times 1.045}$
$\qquad\qquad\qquad\qquad\quad K_2 \quad K_8 \quad K_7$

(C24 Grade)

$= 465 \times 10^3 mm^3$

$K_7 = \left(\dfrac{300}{200}\right)^{0.11} = 1.045$

$K_8 = 1.10$

$K_2 = 0.80$

$I_{REQD} = \dfrac{4.34 \times 1.60 \times 4.0^3 \times 10^9}{9900 \times 0.8}$
$\qquad\qquad\qquad\qquad\qquad K_2$

$\dfrac{4.34 f L^3}{E}$

$= 56.15 \times 10^6 mm^4$

75 x 200 C24 @ 300 c/c

$(Z = 500 \times 10^3 mm^3)$
$(I = 50 \times 10^6 mm^4)$

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Calculation	
Engr.	

Deflection $= 0.003 \times 4000 \times \frac{56.25}{50}$

$= 13.50$mm Acceptable!

Shear $\frac{1.5V}{bd}$ $= \frac{1.5 \times (1.60 \times 2) \times 10^3}{75 \times 200}$

$= 0.32 N/mm^2$

Grade stress of $0.71 \times 0.9 = 0.64 N/mm^2$

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Calculation PLATFORM TRIMMERS (RSA's)	Engr.

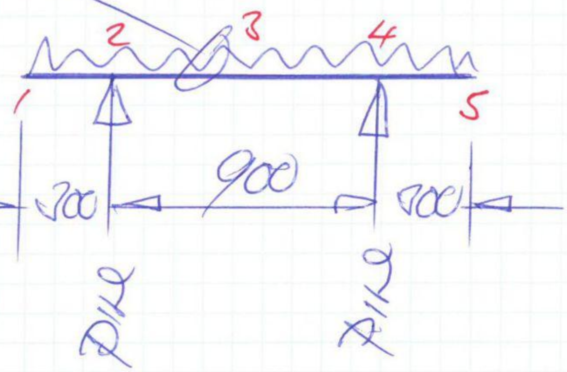
Loading

$5.35 \times 4.0 = 21.40$

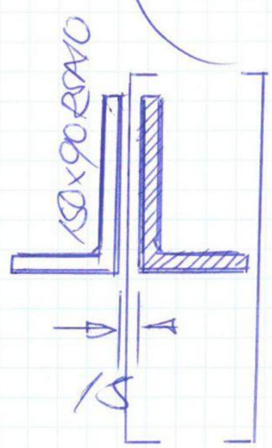
SW $\quad 0.35$

21.75 kN/m

'LL' $\quad 20.00$ kN/m



2/150x90RSA10



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Office: 1060 1

2149H.DAT

```
TABULATE DISPLACEMENTS FORCES REACTIONS
PRINT DATA RESULTS FROM 0
TYPE PLANE FRAME
METHOD ELASTIC NODES
NUMBER OF JOINTS 5
NUMBER OF MEMBERS 4
NUMBER OF SUPPORTS 2
NUMBER OF LOADINGS 1
NUMBER OF SEGMENTS 1
JOINT COORDINATES
1 0 0
2 0.30 0 S
3 0.75 0
4 1.20 0 S
5 1.50 0
JOINT RELEASES
2 MOMENT Z
4 MOMENT Z FORCE X
MEMBER INCIDENCES
1 THRU 4 CHAIN 1,2,3,4,5
CONSTANTS E 205E6 ALL G 79E6 ALL
MEMBER PROPERTIES
1 THRU 4 AX 2*23.2E-4 IZ 2*533E-8
LOADING
MEMBER LOADS
1 THRU 4 FORCE Y UNIFORM W -21.75
SOLVE
```

2/150×90×SA10

LOADING
JOINT DISPLACEMENTS

JOINT	X DISPLACEMENT	Y DISPLACEMENT	Z ROTATION
1	0.000000000	0.000020155	-0.000055985
2	0.000000000	0.000000000	-0.000100773
3	0.000000000	-0.000039679	0.000000000
4	0.000000000	0.000000000	0.000100773
5	0.000000000	0.000020155	0.000055985

LOADING
MEMBER FORCES

MEMBER	JOINT	AXIAL FORCE	SHEAR FORCE	BENDING MOMENT
1	1	0.0000	0.0000	0.0000
	2	0.0000	6.5250	-0.9787
2	2	0.0000	9.7875	0.9787
	3	0.0000	0.0000	1.2234
3	3	0.0000	0.0000	-1.2234
	4	0.0000	9.7875	-0.9787
4	4	0.0000	6.5250	0.9787
	5	0.0000	0.0000	0.0000

LOADING

SUPPORT REACTIONS

JOINT	X FORCE	Y FORCE	Z MOMENT
2	0.0000	16.3125	0.0000
4	0.0000	16.3125	0.0000

EQUILIBRIUM CHECK	SUM OF FORCES	REACTION
FORCES IN DIRECTION X	0.0000	0.0000
FORCES IN DIRECTION Y	-32.6250	32.6250
MOMENTS ABOUT AXIS Z	-24.4688	24.4688

Design factors for one half of composite section

SERVICE EFFECTS

$M = \frac{2.20}{2} = \cancel{1.10}$ Allow 1.15kNm

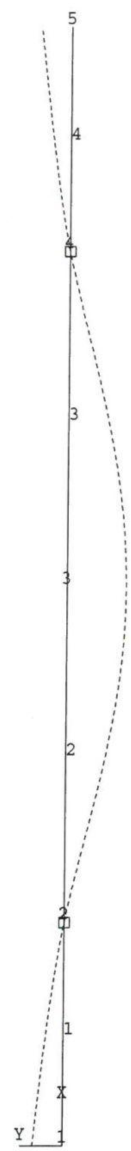
$V = \frac{9.80}{2} = \cancel{4.90}$ Allow 5.00kN

Apply $\gamma_f = 1.6$

($M_{ULT} = 1.85$ kNm
V_{ULT} 8.00 kN)

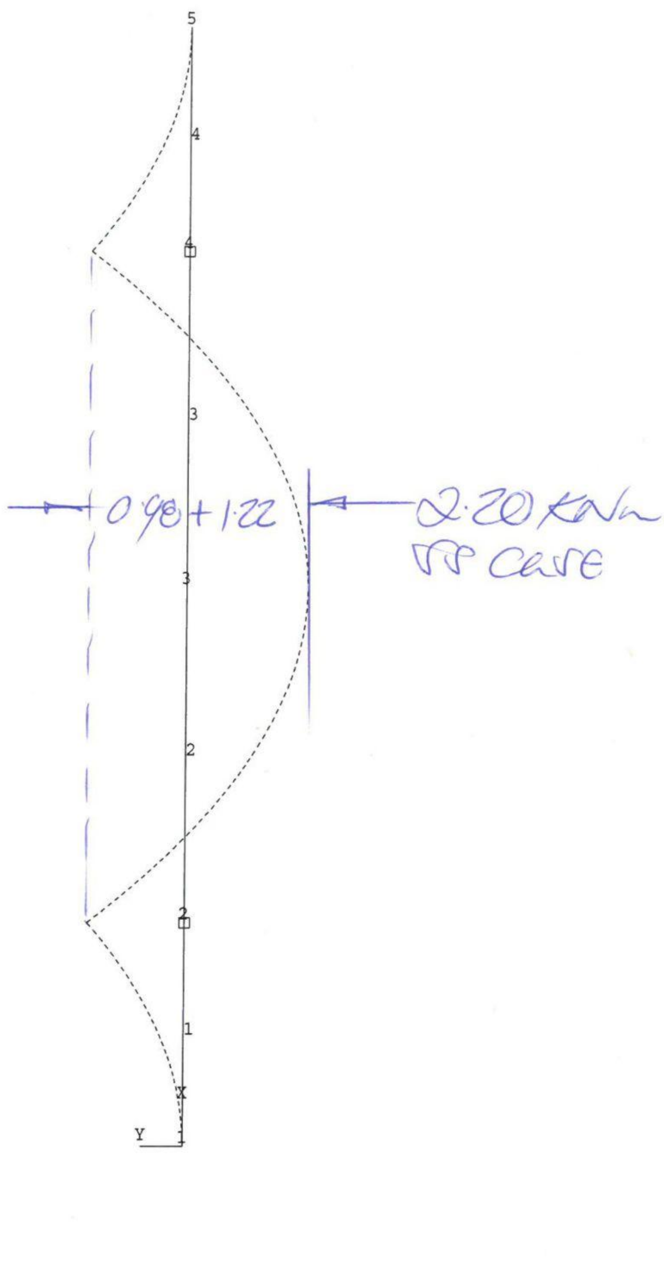
Structure scale 1 cm = 0.080 □ = supports

Deflectn scale 1 cm = 0.00004000000 1



Structure scale 1 cm = 0.080 □ = supports

Moment Z scale 1 cm = 0.60 1



STOCKBRIDGE MEADOWS BOARDWALK

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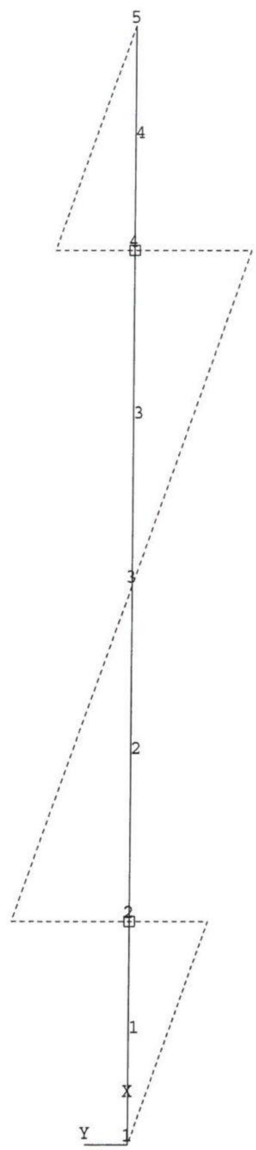
Structure scale 1 cm = 0.080 □ = supports

Force Y scale 1 cm = 5.00 1



Tekla Tedds	Project STOCKBRIDGE MEADOWS BOARDWALK			Job no. 21494	
	Calcs for RSA PLATFORM TRIMMERS			Start page no./Revision 9	
	Calcs by GF	Calcs date 17/08/2021	Checked by	Checked date	Approved by	Approved date

STEEL ANGLE DESIGN (BS5950-1:2000)

TEDDS calculation version 1.0.04

Element definition

Element being designed	**PLATFORM TRIMMERS**
Section	**RSA 150x90x10**
Steel grade	**S355**
Design strength (Table 9)	p_y = **355** N/mm^2

Angle properties

Gross area	A_g = **23.21** cm^2
Radius of gyration about v axis	r_v = **1.96** cm
Minimum section modulus about x axis	Z_{x_min} = **53.65** cm^3
Maximum section modulus about x axis	Z_{x_max} = **107.21** cm^3
Minimum section modulus about y axis	Z_{y_min} = **21.19** cm^3
Maximum section modulus about y axis	Z_{y_max} = **72.16** cm^3
Inertia about major axis	I_u = **594.6** cm^4
Inertia about minor axis	I_v = **89.1** cm^4

Design forces and moments

Shear force parallel to y-y axis	F_{vy} = **8.00** kN
Shear force parallel to x-x axis	F_{vx} = **0.00** kN
Axial force	**F = 0.0 kN**
Maximum moment about x-x axis	M_x = **1.85** kNm
Maximum moment about y-y axis	M_y = **0.00** kNm

Section classification (Table 11)

Parameter epsilon	$\varepsilon = (275\ \text{N/mm}^2/p_y)^{0.5}$ = **0.880**
Ratio for leg dimension b	$ratio_b = b/t$ = **9.000**
Ratio for leg dimension d	$ratio_d = d/t$ = **15.000**

The section is Class 4 (slender) for bending

Reduced design strength in bending for Class 4 (slender) section (cl 3.6.5)

Maximum value of beta	$\beta = \max(ratio_b, ratio_d)$ = **15.000**
Limiting value of β_3 for Class 3 section	$\beta_3 = 15 \times \varepsilon$ = **13.202**
Reduced design strength for bending	$p_{yr\_b} = (\beta_3/\beta)^2 \times p_y$ = **275.0** N/mm^2

Design for shear

For shear force parallel to y axis (cl. 4.2.3)

Shear area	$A_{vy} = 0.9 \times d \times t$ = **1350** mm^2
Shear capacity	$P_{vy} = 0.6 \times p_y \times A_{vy}$ = **287.55** kN
Shear capacity for 'low shear'	$P_{vy\_low} = 0.6 \times P_{vy}$ = **172.53** kN

PASS - The angle is in low shear parallel to y axis

Design for bending

The angle is not restrained against lateral torsional buckling

Moment capacities

Min moment capacity about x-x axis	$M_{cx\_min} = p_{yr\_b} \times Z_{x\_min}$ = **14.75** kNm
Max moment capacity about x-x axis	$M_{cx\_max} = p_{yr\_b} \times Z_{x\_max}$ = **29.48** kNm

PASS - The moment capacity about the x-x axis exceeds the applied moment

Design moments about principal axes

Angle between x-x and u-u axes — α = **19.9** deg

Resultant major (u-u) axis moment — $M_u = M_x \times \cos(\alpha) + M_y \times \sin(\alpha) =$ **1.74** kNm

Resultant minor (v-v) axis moment — $M_v = M_x \times \sin(\alpha) + M_y \times \cos(\alpha) =$ **0.63** kNm

Max major axis mt in segm't length governing M_b — $M_{LT} = M_u =$ **1.74** kNm

Equivalent uniform moment factor

EUM factor m_{LT} — $m_{LT} =$ **1.000**

Major and minor axis section modulii

Distance from u-u axis to extreme fibre 1 — $u_1 = (d - c_x)\times\cos(\alpha) + c_y\times\sin(\alpha) =$ **101.0** mm

Distance from u-u axis to extreme fibre 2 — $u_2 = [d\times\cos(\alpha) + b\times\sin(\alpha)] - u_1 =$ **70.7** mm

Maximum distance to extreme fibre — $u = \max(u_1, u_2) =$ **101.0** mm

Minimum major axis section modulus — $Z_u = I_u/u =$ **58.9** cm^3

Distance from v-v axis to extreme fibre 1 — $v_1 = (b-c_y)/\cos(\alpha) - [(c_x-t)+(b-c_y)\times\tan(\alpha)]\times\cos(\alpha)\times\tan(\alpha) =$ **51.8** mm

Distance from v-v axis to extreme fibre 2 — $v_2 = b\times\cos(\alpha) - v_1 =$ **32.8** mm

Maximum distance to extreme fibre — $v = \max(v_1, v_2) =$ **51.8** mm

Minimum minor axis section modulus — $Z_v = I_v/v =$ **17.2** cm^3

Equivalent slenderness (B.2.9)

Unrestrained length for lateral torsional buckling — $L_{v\_b} =$ **900** mm

Slenderness wrt v-v axis — $\lambda_v = L_{v\_b}/r_v =$ **45.9**

Gamma factor — $\gamma_a = 1 - I_v/I_u =$ **0.850**

Phi factor — $\phi_a = [Z_u^2\times\gamma_a/(A\times J)]^{0.5} =$ **4.071**

The long leg is in compression at some point within the segment length, therefore:-

Monosymmetry index — $\psi_a =$ **-9.030**

Nu factor — $\nu_a = 1/[(1+(4.5\times \psi_a /\lambda_v)^2)^{0.5}+(4.5\times \psi_a /\lambda_v)]^{0.5} =$ **1.490**

Equivalent slenderness — $\lambda_{LT} = 2.25 \times \nu_a \times (\phi_a\times\lambda_v)^{0.5} =$ **45.8**

Bending strength (B.2.1)

Limiting equivalent slenderness — $\lambda_{L0} = 0.4 \times (\pi^2 \times E/p_{yr\_b})^{0.5} =$ **34.3**

Robertson constant — $a_{LT} = 7.0$

Perry factor — $\eta_{LT} = \max(a_{LT} \times (\lambda_{LT} - \lambda_{L0})/1000, 0) =$ **0.081**

Euler buckling stress — $p_E = \pi^2 \times E/\lambda_{LT}^2 =$ **962.9** N/mm^2

Factor phi — $\phi_{LT} = [p_{yr\_b} + (\eta_{LT}+1)\times p_E]/2 =$ **657.8** N/mm^2

Bending strength — $p_b = p_E\times p_{yr\_b}/[\phi_{LT}+(\phi_{LT}^2 - p_E\times p_{yr\_b})^{0.5}] =$ **248.0** N/mm^2

Buckling resistance moment — $M_b = p_b \times Z_u =$ **14.61** kNm

Effective buckling resistance moment — $M_{beff} = M_b/m_{LT} =$ **14.61** kNm

PASS - Effective buckling resistance moment exceeds applied major axis moment

Minor axis bending resistance

Bending resistance — $M_{cv} = p_{yr\_b} \times Z_v =$ **4.73** kNm

PASS - Minor axis bending resistance moment exceeds applied minor axis moment

Equivalent uniform moment factors

EUM factor m_u — $m_u =$ **1.000**

EUM factor m_v — $m_v =$ **1.000**

Member buckling resistance (cl 4.8.3.3)

Equation 1 — $UF_1 = m_u\times M_u/(p_{yr\_b}\times Z_u) + m_v\times M_v/(p_{yr\_b}\times Z_v) =$ **0.240**

Equation 2 $UF_2 = m_{LT} \times M_{LT}/M_b + m_v \times M_v/(p_{yr\_b} \times Z_v) =$ **0.252**

PASS - Member buckling resistance is adequate

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Calculation EDGE BEAM (RSA)	Engr.

Span ne 4.0m

Supports edge timber.

Load/m

$5.35 \times 0.35 = 1.90\,kN/m$

$M = 1.90 \times 4.0^2/8 = 3.80\,kNm$

$V = 1.90 \times 4.0/2 = 3.80\,kN$

$M_{ult} = 5.70\,kNm$
$V_{ult} = 5.70\,kN$

150x90 RSA 10

STEEL ANGLE DESIGN (BS5950-1:2000)

TEDDS calculation version 1.0.04

Element definition

Element being designed	**EDGE BEAM**
Section	**RSA 150x90x10**
Steel grade	**S355**
Design strength (Table 9)	p_y = **355** N/mm^2

Angle properties

Gross area	A_g = **23.21** cm^2
Radius of gyration about v axis	r_v = **1.96** cm
Minimum section modulus about x axis	Z_{x_min} = **53.65** cm^3
Maximum section modulus about x axis	Z_{x_max} = **107.21** cm^3
Minimum section modulus about y axis	Z_{y_min} = **21.19** cm^3
Maximum section modulus about y axis	Z_{y_max} = **72.16** cm^3
Inertia about major axis	I_u = **594.6** cm^4
Inertia about minor axis	I_v = **89.1** cm^4

Design forces and moments

Shear force parallel to y-y axis	F_{vy} = **5.70** kN
Shear force parallel to x-x axis	F_{vx} = **0.00** kN
Axial force	F = **0.0 kN**
Maximum moment about x-x axis	M_x = **5.70** kNm
Maximum moment about y-y axis	M_y = **0.00** kNm

Section classification (Table 11)

Parameter epsilon	$\varepsilon = (275\ \text{N/mm}^2/p_y)^{0.5}$ = **0.880**
Ratio for leg dimension b	$ratio_b = b/t$ = **9.000**
Ratio for leg dimension d	$ratio_d = d/t$ = **15.000**

The section is Class 4 (slender) for bending

Reduced design strength in bending for Class 4 (slender) section (cl 3.6.5)

Maximum value of beta	$\beta = \max(ratio_b, ratio_d)$ = **15.000**
Limiting value of β_3 for Class 3 section	$\beta_3 = 15 \times \varepsilon$ = **13.202**
Reduced design strength for bending	$p_{yr\_b} = (\beta_3/\beta)^2 \times p_y$ = **275.0** N/mm^2

Design for shear

For shear force parallel to y axis (cl. 4.2.3)

Shear area	$A_{vy} = 0.9 \times d \times t$ = **1350** mm^2
Shear capacity	$P_{vy} = 0.6 \times p_y \times A_{vy}$ = **287.55** kN
Shear capacity for 'low shear'	$P_{vy\_low} = 0.6 \times P_{vy}$ = **172.53** kN

PASS - The angle is in low shear parallel to y axis

Design for bending

The angle is not restrained against lateral torsional buckling

Moment capacities

Min moment capacity about x-x axis	$M_{cx\_min} = p_{yr\_b} \times Z_{x\_min}$ = **14.75** kNm
Max moment capacity about x-x axis	$M_{cx\_max} = p_{yr\_b} \times Z_{x\_max}$ = **29.48** kNm

PASS - The moment capacity about the x-x axis exceeds the applied moment

Design moments about principal axes

Angle between x-x and u-u axes — α = **19.9** deg

Resultant major (u-u) axis moment — $M_u = M_x \times \cos(\alpha) + M_y \times \sin(\alpha)$ = **5.36** kNm

Resultant minor (v-v) axis moment — $M_v = M_x \times \sin(\alpha) + M_y \times \cos(\alpha)$ = **1.94** kNm

Max major axis mt in segm't length governing M_b — $M_{LT} = M_u$ = **5.36** kNm

Equivalent uniform moment factor

EUM factor m_{LT} — m_{LT} = **1.000**

Major and minor axis section modulii

Distance from u-u axis to extreme fibre 1 — $u_1 = (d - c_x)\times\cos(\alpha) + c_y\times\sin(\alpha)$ = **101.0** mm

Distance from u-u axis to extreme fibre 2 — $u_2 = [d\times\cos(\alpha) + b\times\sin(\alpha)] - u_1$ = **70.7** mm

Maximum distance to extreme fibre — $u = \max(u_1, u_2)$ = **101.0** mm

Minimum major axis section modulus — $Z_u = I_u/u$ = **58.9** cm^3

Distance from v-v axis to extreme fibre 1 — $v_1 = (b-c_y)/\cos(\alpha) - [(c_x-t)+(b-c_y)\times\tan(\alpha)]\times\cos(\alpha)\times\tan(\alpha)$ = **51.8** mm

Distance from v-v axis to extreme fibre 2 — $v_2 = b\times\cos(\alpha) - v_1$ = **32.8** mm

Maximum distance to extreme fibre — $v = \max(v_1, v_2)$ = **51.8** mm

Minimum minor axis section modulus — $Z_v = I_v/v$ = **17.2** cm^3

Equivalent slenderness (B.2.9)

Unrestrained length for lateral torsional buckling — L_{v_b} = **4000** mm

Slenderness wrt v-v axis — $\lambda_v = L_{v\_b}/r_v$ = **204.2**

Gamma factor — $\gamma_a = 1 - I_v/I_u$ = **0.850**

Phi factor — $\phi_a = [Z_u^2\times\gamma_a/(A\times J)]^{0.5}$ = **4.071**

The long leg is in compression at some point within the segment length, therefore:-

Monosymmetry index — ψ_a = **-9.030**

Nu factor — $\nu_a = 1/[(1+(4.5\times \psi_a /\lambda_v)^2)^{0.5}+(4.5\times \psi_a /\lambda_v)]^{0.5}$ = **1.104**

Equivalent slenderness — $\lambda_{LT} = 2.25 \times \nu_a \times (\phi_a\times\lambda_v)^{0.5}$ = **71.6**

Bending strength (B.2.1)

Limiting equivalent slenderness — $\lambda_{L0} = 0.4 \times (\pi^2 \times E/p_{yr\_b})^{0.5}$ = **34.3**

Robertson constant — a_{LT} = 7.0

Perry factor — $\eta_{LT} = \max(a_{LT} \times (\lambda_{LT}- \lambda_{L0})/1000, 0)$ = **0.261**

Euler buckling stress — $p_E = \pi^2 \times E/\lambda_{LT}^2$ = **394.6** N/mm^2

Factor phi — $\phi_{LT} = [p_{yr\_b} + (\eta_{LT}+1)\times p_E]/2$ = **386.3** N/mm^2

Bending strength — $p_b = p_E\times p_{yr\_b}/[\ \phi_{LT}+(\ \phi_{LT}^2 - p_E\times p_{yr\_b})^{0.5}]$ = **184.5** N/mm^2

Buckling resistance moment — $M_b = p_b \times Z_u$ = **10.87** kNm

Effective buckling resistance moment — $M_{beff} = M_b/m_{LT}$ = **10.87** kNm

PASS - Effective buckling resistance moment exceeds applied major axis moment

Minor axis bending resistance

Bending resistance — $M_{cv} = p_{yr\_b} \times Z_v$ = **4.73** kNm

PASS - Minor axis bending resistance moment exceeds applied minor axis moment

Equivalent uniform moment factors

EUM factor m_u — m_u = **1.000**

EUM factor m_v — m_v = **1.000**

Member buckling resistance (cl 4.8.3.3)

Equation 1 — $UF_1 = m_u\times M_u/(p_{yr\_b}\times Z_u) + m_v\times M_v/(p_{yr\_b}\times Z_v)$ = **0.741**

Equation 2

$UF_2 = m_{LT} \times M_{LT}/M_b + m_v \times M_v/(p_{yr\_b} \times Z_v) = \mathbf{0.903}$

PASS - Member buckling resistance is adequate