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project :	Martello Cafe & Public Toilets
project no :	8850
by :	ICR
date :	Jan 2022
no :	1

spithurst hub spithurst road
barcombe east sussex BN8 5EE
01273 470 066 info@ingsengineering.co.uk

MARTELLO CAFE & PUBLIC TOILETS
ESPLANADE
SEAFORD
EAST SUSSEX

The following calculations and details are to be read in conjunction with all relevant drawings prepared by ABIR ARCHITECTS.

All dimensions and assumptions made within the following calculations and details are to be checked on site, and any discrepancy reported to Ings Engineering Limited.

Design Documents

Loads to	BS 648 & BS 6399
Timber to	BS 5268
Steel to	BS 5950
Masonry to	BS 5628
Concrete to	BS 8110
Foundations to	BS 8004
Building Regulations to	Part A

Loading (kN/m²)	***DL***	**+**	***LL***	***Total***
Flat Roof	0.75	+	0.75	1.50
Solar PV Allowance	0.25			0.25
Services Allowance	0.25			0.25
Cafe/ WC	2.50	+	2.00	4.50
Kitchen	2.50	+	3.00	5.50
Internal Timber Stud	0.40			0.40
External Timber Stud	1.00			1.00
215 Brick	4.70			4.70
330 Brick	7.30			7.30

General Notes

These calculations are for obtaining Building Regulation Approval only.

Note also requirements of the Building Regulations outside the scope of these calculations i.e. Building Regulations Approved Documents B to M, which should be referred to as appropriate.

Works carried out on site prior to the relevant Building Regulation approval being obtained from the Local Authority, are undertaken entirely at the Contractor and Clients own risk.

Calculations are to be read in conjunction with all accompanying specification notes and architectural drawings.

Calculations have been produced from drawings only, unless noted.

No structural investigations unless noted have been undertaken and all assumed loadbearing walls and existing foundations must be physically exposed, checked and confirmed with the BUILDING CONTROL OFFICER on site prior to commencement of the works.

Work to site dimensions only. Do not use measurement shown in the calculations - these are for design analysis only

Any discrepancies are to be reported immediately to the designer for clarification.

The Contractor is responsible for all temporary works and for the stability of the works in progress including any necessary sacrificial jacks to take up beam deflections.

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The Construction (Design and Management) Regulations 2015 (CDM 2015)

1. Wherever possible all foreseeable risks to Health and Safety have been considered at Design Stage and suitable measures have been taken to combat such risks at source.

   Where it has not been possible to reduce risk element to a minimum due to practical or physical considerations, the Contractor must employ competent people and establish safe methods of work to reduce the risk accordingly.

2. **Construction Stage Associated Risks**

   The following hazards and/or risks have been identified and are to be addressed by the Contractor with regards to Health and Safety accordingly.

   - Contractor is to assess existing structural configurations and provide all necessary temporary support works
   - The contractor is to provide all necessary lifting equipment to install heavy building components
   - Contractor is to provide all necessary signage, fencing, hoarding etc to protect staff, vistors for the duration of the works

   Normal Construction Hazards/Risks have been excluded from the above which are considered to be covered by the use competent Contractors working in accordance with good working practice and compliance with current Health and Safety legislation.

3. **Maintenance, Cleaning or Demolition Associated Risks**

   The following hazards and/or risks have been identified and are to be addressed by any person or Contractor undertaking maintenance, cleaning or demolition works to the above property.

   - All future works are to be carried out after consultation with a structural engineer or suitably qualified building professional

   Normal Construction Hazards/Risks have been excluded from the above which are considered to be covered by the use of competent Contractors working in accordance with good working practice and compliance with current Health and Safety legislation.

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project : Martello Cafe & Public Toilets
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barcombe east sussex BN8 5EE
01273 470 066 info@ingsengineering.co.uk

Beam: FRJ1 **Span: 4.3 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.1	0		L	0.22	0.22	0.45
U	L	flat roof	0.75*0.4	0		L	0.65	0.65	1.34
U	D	flat roof	0.75*0.4	0		L	0.65	0.65	1.34
U	D	solar PV	0.25*0.4	0		L	0.22	0.22	0.45
U	D	services	0.25*0.4	0		L	0.22	0.22	0.45
					Total load (unfactored):	**3.87 kN**	**1.94**	**1.94**	4.01
					Dead/Permanent (unfactored):	2.58 kN	1.29	1.29	2.67
					Live/Variable (unfactored):	1.29 kN	0.65	0.65	1.34

Load types: O:Beam o.w.

Maximum B.M. = 2.08 kNm (unfactored) at 2.15 m. from R1
Maximum S.F. = 1.94 kN (unfactored) at R1
Mid-span deflections: Dead: $2.67 \times 10^8/EI$ ***(E in N/mm², I in cm⁴)***
Live: $1.34 \times 10^8/EI$
Total: $4.01 \times 10^8/EI$

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 47 x 220 C24 ***4.3 kg/m approx***

z = 379 cm^3 I = 4,170 cm^4

Timber grade: C24 Load sharing system: K_8 = 1.1 [§2.10.11]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/220)^{0.11}$ = 1.03 [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

E = 10,800 N/mm^2 (E_{mean})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.03 x 1.1 = 8.54 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 2.08 x 1000/379 = 5.49 N/mm^2 OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm^2

Applied shear stress, τ_a = 1.94 x 1000 x 3/(2 x 47 x 220) = 0.28 N/mm^2 OK

Deflection

Bending deflection = $4.01 \times 10^8/(10,800 \times 4,170)$ = 8.9 mm

Mid-span shear deflection = $1.2M_0/GA$ (G=E/16) = $1.2 \times 2.08 \times 10^6/((10800/16) \times 47 \times 220)$ = 0.4 mm

Total deflection = 8.9 + 0.4 = 9.3 mm (0.0022 L) <= 0.003L OK

provide flat roof joists @ 400 centres, min size as noted above

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01273 470 066 info@ingsengineering.co.uk

Beam: FRJ2 **Span: 2.6 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.1	0		L	0.13	0.13	0.06
U	L	flat roof	0.75*0.4	0		L	0.39	0.39	0.18
U	D	flat roof	0.75*0.4	0		L	0.39	0.39	0.18
U	D	services	0.25*0.4	0		L	0.13	0.13	0.06
P	D	condensor unit	0.95	L/2			0.47	0.47	0.35
				Total load (unfactored):		**3.03 kN**	**1.51**	**1.51**	0.82
				Dead/Permanent (unfactored):		2.25 kN	1.13	1.13	0.65
				Live/Variable (unfactored):		0.78 kN	0.39	0.39	0.18

Load durations: D: Dead

Maximum B.M. = 1.29 kNm (unfactored) at 1.30 m. from R1
Maximum S.F. = 1.51 kN (unfactored) at R1
Mid-span deflections: Dead: $0.65 \times 10^8/EI$ ***(E in N/mm², I in cm⁴)***
Live: $0.18 \times 10^8/EI$
Total: $0.82 \times 10^8/EI$

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 47 x 220 C24 ***4.3 kg/m approx***

$z = 379\ cm^3$ $I = 4{,}170\ cm^4$

Timber grade: C24 Load sharing system: $K_8 = 1.1$ [§2.10.11]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/220)^{0.11} = 1.03$ [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

$E = 10{,}800\ N/mm^2$ (E_{mean})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8 = 7.5 \times 1.00 \times 1.03 \times 1.1 = 8.54\ N/mm^2$

Applied bending stress, $\sigma_{m,a} = 1.29 \times 1000/379 = 3.41\ N/mm^2$ OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8 = 0.71 \times 1.00 \times 1.1 = 0.78\ N/mm^2$

Applied shear stress, $\tau_a = 1.51 \times 1000 \times 3/(2 \times 47 \times 220) = 0.22\ N/mm^2$ OK

Deflection

Bending deflection = $0.824 \times 10^8/(10{,}800 \times 4{,}170) = 1.8$ mm

Mid-span shear deflection = $1.2M_0/GA$ ($G=E/16$) = $1.2 \times 1.29 \times 10^6/((10800/16) \times 47 \times 220) = 0.2$ mm

Total deflection = 1.8 + 0.2 = 2.1 mm (0.0008 L) <= 0.003L OK

provide flat roof joists @ 400 centres, min size as noted above

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project : Martello Cafe & Public Toilets
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date : Jan 2022
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Beam: FRJ3 **Span: 3.7 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.1	0		L	0.19	0.19	0.24
U	L	flat roof	0.75*0.4	0		L	0.56	0.56	0.73
U	D	flat roof	0.75*0.4	0		L	0.56	0.56	0.73
U	D	solar PV	0.25*0.4	0		L	0.19	0.19	0.24
U	D	services	0.25*0.4	0		L	0.19	0.19	0.24
P	D	HRV unit	2.5/5	L/2			0.25	0.25	0.53
				Total load (unfactored):		**3.83 kN**	**1.91**	**1.91**	2.72
			Dead/Permanent (unfactored):			2.72 kN	1.36	1.36	1.99
			Live/Variable (unfactored):			1.11 kN	0.56	0.56	0.73

Load durations: D: Dead

Maximum B.M. = 2.00 kNm (unfactored) at 1.85 m. from R1
Maximum S.F. = 1.91 kN (unfactored) at R1
Mid-span deflections: Dead: 1.99 x 10^8/EI ***(E in N/mm², I in cm⁴)***
Live: 0.73 x 10^8/EI
Total: 2.72 x 10^8/EI

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 47 x 220 C24 ***4.3 kg/m approx***

z = 379 cm^3 I = 4,170 cm^4

Timber grade: C24 Load sharing system: K_8 = 1.1 [§2.10.11]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/220)^{0.11}$ = 1.03 [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

E = 10,800 N/mm^2 (E_{mean})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.03 x 1.1 = 8.54 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 2.00 x 1000/379 = 5.28 N/mm^2 OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm^2

Applied shear stress, τ_a = 1.91 x 1000 x 3/(2 x 47 x 220) = 0.28 N/mm^2 OK

Deflection

Bending deflection = 2.72 x 10^8/(10,800 x 4,170) = 6.0 mm

Mid-span shear deflection = 1.2M_0/GA (G=E/16) = 1.2 x 2.00 x 10^6/((10800/16) x 47 x 220) = 0.3 mm

Total deflection = 6.0 + 0.3 = 6.4 mm (0.0017 L) <= 0.003L OK

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spithurst hub spithurst road
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project : Martello Cafe & Public Toilets
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by : ICR
date : Jan 2022
no : 6

Beam: FRJ4 **Span: 1.9 m.**
+ RH Cantilever: 1.2 m.

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	SpanDef
O	D	o.w.	0.1	0		3.1	0.09	0.09	0.02
U	D	solar PV	0.25*0.4	0		L	0.09	0.09	0.02
U	D	services	0.25*0.4	0		L	0.09	0.09	0.02
R	L	flat roof	0.75*0.4	0		3.1	0.17	0.76	0.05
R	D	flat roof	0.75*0.4	0		3.1	0.17	0.76	0.00
		Total load (unfactored, all loads applied):				**2.43 kN**	**0.63**	**1.80**	0.10
		Dead/Permanent (unfactored):				1.50 kN	0.46	1.04	0.05
		Live/Variable (unfactored):				0.93 kN	0.17	0.76	0.05
		R1: DL+LL on span, DL on RH cant				0.74			
		R2: DL+LL on span and RH cant					1.80		

Reactions are calculated ignoring live loads on opposing cantilever(s)

Check for uplift at R1 with worst case loading
DL on span
Uplift at R1 with worst case loading not checked

Load durations: D: Dead

Span:
Maximum B.M. = 0.31 kNm at 0.82 m. from R1 (unfactored) ***(Cant LLs ignored)***
N.B. Span B.M. may be greater if span unloaded and cantilever loaded
Maximum S.F. = -1.08 kN at R2 (unfactored)
Mid-span deflections: Dead = $0.05 \times 10^8/EI$ ***(E in N/mm², I in cm⁴)***
Live = $0.05 \times 10^8/EI$ ***(Live loads on span only)***
Total = $0.10 \times 10^8/EI$

RH cantilever:
B.M. at root = -0.43 kNm (unfactored)
S.F. at root = 0.72 kN (unfactored)
End deflections: Dead = $0.04 \times 10^8/EI$ x ***(E in N/mm², I in cm⁴)***
Live = $0.24 \times 10^8/EI$ ***(Live loads on cantilever only)***
Total = $0.28 \times 10^8/EI$

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 47 x 147 C24 ***2.9 kg/m approx***

z = 169 cm³ I = 1,244 cm⁴
Timber grade: C24 Load sharing system: K_8 = 1.1 [§2.10.11]
K_3 (loading duration factor) = 1.00 (long term)
K_7 (depth factor) = $(300/147)^{0.11}$ = 1.08 [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]
Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.08 x 1.1 = 8.92 N/mm²
Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm²
E = 10,800 N/mm² (E_{mean})

Span

Bending

Applied bending stress, $\sigma_{m,a}$ = 0.432 x 1000/169 = 2.55 N/mm² OK

Shear

Applied shear stress, τ_a = 1.08 x 1000 x 3/(2 x 47 x 147) = 0.23 N/mm² OK

Deflection

Bending deflection (LL on span only) = $0.104 \times 10^8/(10{,}800 \times 1{,}244)$ = 0.8 mm
Mid-span shear deflection = $1.2M_0/GA$ (G=E/16) = $1.2 \times 0.298 \times 10^6/((10800/16) \times 47 \times 147)$ = 0.1 mm
Total deflection = 0.8 + 0.1 = 0.9 mm (0.0004 L) <= 0.003L OK

RH cantilever

Bending

Applied bending stress at cantilever root, $\sigma_{m,a}$ = 0.432 x 1000/169 = 2.55 N/mm² OK

Shear

Applied shear stress at cantilever root, τ_a = 0.72 x 1000 x 3/(2 x 47 x 147) = 0.16 N/mm² OK

Deflection

RH cantilever deflection (LL on cantilevers only) = $0.278 \times 10^8/(10{,}800 \times 1{,}244)$ = 2.1 mm (0.0017 L) OK

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ings engineering limited
consulting structural engineers

spithurst hub spithurst road
barcombe east sussex BN8 5EE
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no : 7

Beam: **FRJ5** **Span: 1.3 m.**

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O D	o.w.	0.1	0		L	0.06	0.06	0.004
U L	flat roof	0.75*0.4	0		L	0.20	0.20	0.011
U D	flat roof	0.75*0.4	0		L	0.20	0.20	0.011
			Total load (unfactored):		**0.91 kN**	**0.45**	**0.45**	0.026
			Dead/Permanent (unfactored):		0.52 kN	0.26	0.26	0.015
			Live/Variable (unfactored):		0.39 kN	0.20	0.20	0.011

Load types: O:Beam o.w.

Maximum B.M. = 0.148 kNm (unfactored) at 0.65 m. from R1
Maximum S.F. = 0.455 kN (unfactored) at R1
Mid-span deflections: Dead: 0.015 x 10^8/EI ***(E in N/mm², I in cm⁴)***
Live: 0.011 x 10^8/EI
Total: 0.026 x 10^8/EI

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 47 x 97 C24 ***1.9 kg/m approx***

z = 73.7 cm^3 I = 357 cm^4
Timber grade: C24 Load sharing system: K_8 = 1.1 [§2.10.11]
K_3 (loading duration factor) = 1.00 (long term)
K_7 (depth factor) = $(300/97)^{0.11}$ = 1.13 [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]
E = 10,800 N/mm² (E_{mean})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.13 x 1.1 = 9.34 N/mm²
Applied bending stress, $\sigma_{m,a}$ = 0.148 x 1000/73.7 = 2.01 N/mm² OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm²
Applied shear stress, τ_a = 0.455 x 1000 x 3/(2 x 47 x 97) = 0.15 N/mm² OK

Deflection

Bending deflection = 0.026 x 10^8/(10,800 x 357) = 0.7 mm
Mid-span shear deflection = $1.2M_0$/GA (G=E/16) = 1.2 x 0.148 x 10^6/((10800/16) x 47 x 97) = 0.1 mm
Total deflection = 0.7 + 0.1 = 0.7 mm (0.0006 L) <= 0.003L OK

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Beam: T1 **Span: 1.1 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.1	0		L	0.06	0.06	0.002
U	L	flat roof	0.75*3.7/2	0		L	0.76	0.76	0.026
U	D	flat roof	0.75*3.7/2	0		L	0.76	0.76	0.026
U	D	solar PV	0.25*3.7/2	0		L	0.25	0.25	0.009
U	D	services	0.25*3.7/2	0		L	0.25	0.25	0.009
				Total load (unfactored):		**4.18 kN**	**2.09**	**2.09**	0.072
			Dead/Permanent (unfactored):			2.65 kN	1.33	1.33	0.046
			Live/Variable (unfactored):			1.53 kN	0.76	0.76	0.026

Load types: O:Beam o.w.

Maximum B.M. = 0.575 kNm (unfactored) at 0.55 m. from R1
Maximum S.F. = 2.09 kN (unfactored) at R1
Mid-span deflections: Dead: 0.046 x 10^8/EI ***(E in N/mm², I in cm⁴)***
Live: 0.026 x 10^8/EI
Total: 0.072 x 10^8/EI

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 47 x 220 C24 ***4.3 kg/m approx***

z = 379 cm^3 I = 4,170 cm^4

Timber grade: C24 Single member: No load sharing

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/220)^{0.11}$ = 1.03 [§2.10.6] K_8 (load sharing factor) = 1.0 [§2.9,2.10]

E = 7,200 N/mm² (E_{min})

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.03 x 1.0 = 7.76 N/mm²

Applied bending stress, $\sigma_{m,a}$ = 0.57 x 1000/379 = 1.52 N/mm² OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.0 = 0.71 N/mm²

Applied shear stress, τ_a = 2.09 x 1000 x 3/(2 x 47 x 220) = 0.30 N/mm² OK

Deflection

Bending deflection = 0.072 x 10^8/(7,200 x 4,170) = 0.2 mm

Mid-span shear deflection = $1.2M_0/GA$ (G=E/16) = 1.2 x 0.57 x 10^6/((7200/16) x 47 x 220) = 0.1 mm

Total deflection = 0.2 + 0.1 = 0.4 mm (0.0004 L) <= 0.003L OK

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spithurst hub spithurst road
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Beam: T2 **Span: 3.7 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.1	0		L	0.19	0.19	0.24
U	L	flat roof	0.75*0.8	0		L	1.11	1.11	1.46
U	D	flat roof	0.75*0.8	0		L	1.11	1.11	1.46
U	D	solar PV	0.25*0.8	0		L	0.37	0.37	0.49
U	D	services	0.25*0.8	0		L	0.37	0.37	0.49
P	D	HRV unit	1.25	L/2			0.63	0.63	1.32
					Total load (unfactored):	**7.54 kN**	**3.77**	**3.77**	5.46
					Dead/Permanent (unfactored):	5.32 kN	2.66	2.66	4.00
					Live/Variable (unfactored):	2.22 kN	1.11	1.11	1.46

Load durations: D: Dead

Maximum B.M. = 4.07 kNm (unfactored) at 1.85 m. from R1
Maximum S.F. = 3.77 kN (unfactored) at R1
Mid-span deflections: Dead: $4.00 \times 10^8/EI$ ***(E in N/mm², I in cm⁴)***
Live: $1.46 \times 10^8/EI$
Total: $5.46 \times 10^8/EI$

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 2no 47 x 220 C24 ***8.7 kg/m approx***

z = 758 cm³ I = 8,341 cm⁴

Timber grade: C24 2 members acting together: K_8 = 1.1 [§2.9]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/220)^{0.11}$ = 1.03 [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

E = 7,200 x 1.14 = 8,208 N/mm² ($E_{min}.K_9$)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.03 x 1.1 = 8.54 N/mm²

Applied bending stress, $\sigma_{m,a}$ = 4.07 x 1000/758 = 5.36 N/mm² OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm²

Applied shear stress, τ_a = 3.77 x 1000 x 3/(2 x 94 x 220) = 0.27 N/mm² OK

Deflection

Bending deflection = $5.46 \times 10^8/(8{,}208 \times 8{,}341)$ = 8.0 mm

Mid-span shear deflection = $1.2M_0/GA$ (G=E/16) = $1.2 \times 4.07 \times 10^6/((8208/16) \times 94 \times 220)$ = 0.5 mm

Total deflection = 8.0 + 0.5 = 8.4 mm (0.0023 L) <= 0.003L OK

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Beam: B1 **Span: 3.2 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	D	o.w.	0.2	0		L	0.32	0.32	0.27
U	L	flat roof	0.75*2.2^2/2*1.0	0		L	2.90	2.90	2.48
U	D	flat roof	1.0*2.2^2/2*1.0	0		L	3.87	3.87	3.30
				Total load (unfactored):		**14.19 kN**	**7.10**	**7.10**	6.06
			Dead/Permanent (unfactored):			8.38 kN	4.19	4.19	3.58
			Live/Variable (unfactored):			5.81 kN	2.90	2.90	2.48
			Total load (factored):			**21.03 kN**	**10.52**	**10.52**	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 8.41 kNm (factored) at 1.60 m. from R1

Maximum S.F. = 10.52 kN (factored) at R1

Mid-span deflections: Dead: $3.58 \times 10^8/EI$ ***(E in N/mm², I in cm⁴)***
Live: $2.48 \times 10^8/EI$
Total: $6.06 \times 10^8/EI$

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : **152 x 89 x 16 UB** S355 (compact)

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_x=834 cm^4 r_y=2.10 cm S_x=123 cm^3 x=19.6

Section classification: T = 7.7mm p_y = 355 N/mm^2 $\varepsilon = \sqrt{(275/355)} = 0.88$
(Table 11) Flange: b/T = 44.3/7.7 = 5.76 (<=9ε: Class 1 plastic)
Web: d/t = 121.8/4.5 = 27.1 (<=80ε: Class 1, plastic)
For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 11 kN

Shear capacity = 0.6 p_y.t.D = 0.6 x 355 x 4.5 x 152.4/1000 = 146 kN OK

Bending

Maximum B.M. = 8.41 kNm

Moment capacity, $M_c = p_y.S_x$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

Effective length = 1.2L+2D = 4.14 m. [Table 13]

Bending strength, p_b = 117 N/mm^2

Maximum moment within segment, M_x = 8.41 kNm

Equivalent uniform moment factor, m_{LT} = 0.925 (M_2 = 6.31, M_3 = 8.41, M_4 = 6.31)

Equivalent uniform moment = 0.925 x 8.41 = 7.78 kNm

Buckling resistance moment, $M_b = p_b.S_x$ = 117 x 123/1000 = 14.35 kNm OK

Web buckling and crushing have not been checked

Deflection

LL deflection = 2.48 x 1e8/(205,000 x 834) = 1.5 mm (L/2206) OK
TL deflection = 6.06 x 1e8/(205,000 x 834) = 3.5 mm (L/903)

Bearings

R1: User-defined

provide 8mm MS plate 6mm FW to end of beam fixed to web of supporting beam using 4No. M16 (8.8) bolts

R2: User-defined

provide multiple studs under beam bearing (min 2No.)

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project no : 8850
by : ICR
date : Jan 2022
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01273 470 066 info@ingsengineering.co.uk

Beam: B2 **Span: 3.3 m.**

		Load name	*Loading w1*	*Start x1*	*Loading w2*	*End x2*	*R1comp*	*R2comp*	*Defl.*
O	D	o.w.	0.2	0		L	0.33	0.33	0.31
U	L	flat roof	0.75*4.7/2	0		L	2.91	2.91	2.72
U	D	flat roof	0.75*4.7/2	0		L	2.91	2.91	2.72
U	D	solar PV	0.25*4.7/2	0		L	0.97	0.97	0.91
U	D	services	0.25*4.7/2	0		L	0.97	0.97	0.91
				Total load (unfactored):		**16.17 kN**	**8.09**	**8.09**	7.56
				Dead/Permanent (unfactored):		10.35 kN	5.18	5.18	4.84
				Live/Variable (unfactored):		5.82 kN	2.91	2.91	2.72
				Total load (factored):		**23.80 kN**	**11.90**	**11.90**	

Load types: O:Beam o.w.; U:UDL; Load positions: m. from R1; Load durations: D: Dead; L: Live

Maximum B.M. = 9.82 kNm (factored) at 1.65 m. from R1

Maximum S.F. = 11.90 kN (factored) at R1

Mid-span deflections: Dead: $4.84 \times 10^8/EI$ ***(E in N/mm², I in cm⁴)***
Live: $2.72 \times 10^8/EI$
Total: $7.56 \times 10^8/EI$

Beam calculation to BS5950-1:2000 using S355 steel

SECTION SIZE : **152 x 89 x 16 UB** S355 (compact)

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm I_x=834 cm^4 r_y=2.10 cm S_x=123 cm^3 x=19.6

Section classification: T = 7.7mm p_y = 355 N/mm^2 $\varepsilon = \sqrt{(275/355)} = 0.88$
(Table 11) Flange: b/T = 44.3/7.7 = 5.76 (<=9ε: Class 1 plastic)
Web: d/t = 121.8/4.5 = 27.1 (<=80ε: Class 1, plastic)
For design purposes section classification is Class 2, compact

Shear

Maximum S.F. = 12 kN

Shear capacity = 0.6 p_y.t.D = 0.6 x 355 x 4.5 x 152.4/1000 = 146 kN OK

Bending

Maximum B.M. = 9.82 kNm

Moment capacity, $M_c = p_y.S_x$ = 355 x 123/1000 = 43.7 kNm OK

Lateral-torsional buckling

Beam is laterally restrained at supports only

Restraint condition at R1 and R2: Compression flange laterally unrestrained; both flanges free to rotate on plan. Partial torsional restraint by dead bearing of bottom flange to support (1.2L+2D)

Effective length = 1.2L+2D = 4.26 m. [Table 13]

Bending strength, p_b = 114 N/mm^2

Maximum moment within segment, M_x = 9.82 kNm

Equivalent uniform moment factor, m_{LT} = 0.925 (M_2 = 7.36, M_3 = 9.82, M_4 = 7.36)

Equivalent uniform moment = 0.925 x 9.82 = 9.08 kNm

Buckling resistance moment, $M_b = p_b.S_x$ = 114 x 123/1000 = 13.97 kNm OK

Web buckling and crushing have not been checked

Deflection

LL deflection = 2.72 x 1e8/(205,000 x 834) = 1.6 mm (L/2074) OK
TL deflection = 7.56 x 1e8/(205,000 x 834) = 4.4 mm (L/746)

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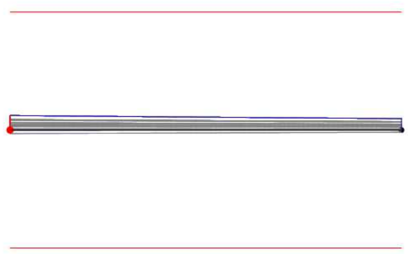
Axial with Moments (Member)
Member SB B\2L5Id 400 @ Level 5 in Load Case 8 : Beam B3

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D0 + 1.4 D1 + 1.6 L1

```
D1 TY2  -000.100          (  kN  )
L1 TY2  -000.060          (  kN  )
D1 D     077.010          ( kN/m³)
```



Member Forces in Load Case 8 and Maximum Deflection from Load Case 2											
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)	
				x-x	y-y	x-x	y-y	x-x	y-y		
79	134 135	0.00C 0.00C	0.00 0.00	2.18 1.82	-0.05 -0.05	-5.40 -4.24	0.01 -0.02			@	0.08 0.277

Classification and Properties (BS 5950: 2000)

Section (15.95 kg/m)	152x89 UB 16 [S 355]		
Class = Fn(b/T,d/t,py,F,Mx,My)	5.76, 27.07, 355, 0, 5.4, 0.02	(Axial: Non-Slender)	Plastic
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	2.184 / 146.075 =	0.015	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 123.3≤1.2 x 355 x 109.61 =	43.772 kN.m	
Fvy/Pvy	0.049 / 261.858 =	0	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 31.2≤1.2 x 355 x 20.43 =	8.703 kN.m	
Pz = Ag.py	20.32 x 355 =	721.36 kN	
n = F/Pz	0 / 721.36 =	0.000	OK
Srx = Fn(Sxx, n)	123.3, 0	123.3 cm^3	
Mrx = Srx.py	123.3 x 355	43.772 kN.m	
Sry = Fn(Syy, n)	31.2, 0	31.2 cm^3	
Mry = Sry.py	31.2 x 355	8.703 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(5.397/43.772)^2+(0.005/8.703)^1=$	0.016	OK

Compression Resistance Pc

λx = Lex/rxx	100x1x0.566/6.41 =	8.8	OK
Pcx = Area.pcx	20.32x355/10 =	721.360 kN	Table 24 a
λy = Ley/ryy	100x1x0.566/2.11 =	26.8	OK
Pcy = Area.pcy	20.32x339.19/10 =	689.239 kN	Table 24 b

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}$	0.2+(.15x5+.5x5+.15x5)/5 ≥ 0.44	0.911	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.557	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x-5+.6x-5+.1x-5)/5 ≥ .8x5/5	0.911	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.557	Table 26

Lateral Buckling Check Mb

Le = 1.2L+2D	1.2 x 0.566 + 2 x 0.152 =	0.984 m	
λ = Le/ryy	0.984 / 2.11	46.62	OK
v = Fn (x,Le,ryy,λ)	19.589, 0.984, 2.11, 46.62	0.94	Table 19
λ_{LT}= u.v.λ.$\sqrt{\beta_W}$	0.892 x 0.94 x 46.62 √ 1	39.09	
pb = Fn (py,λ_{LT})	355, 39.09	327.87 N/mm^2	Table 16
Mb = Sxx.pb ≤ Mc	123.3 x 327.87 ≤ 43.772 =	40.427 kN.m	

Combined Axial Compression and Bending to Annex I

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$r_b=m_{LT}.M_{LT}/M_b$	0.911x-5.4/40.4	0.122	
$r_c=F_c/P_{cy}$	0/689.2	0.000	
$\lambda_r=(r_b\lambda_{LT}+r_c\lambda_y)/(r_b+r_c)$	(0.122•39.1+0•26.8)/(0.122+0)	39.090	
$\lambda_{ro}=17.15\ \varepsilon\ (2r_b+r_c)/(r_b+r_c)$	17.15•0.88(2•0.122+0)/(0.122+0)	30.189	
$M_{ob}= M_b(1-F_c/P_{cy})$	40.427(1-0/689.2)	40.427	
$M_{xy}= M_{cx}(1-F_c/ P_{cy})^{½}$	$43.772(1-0/689.2)^{½}$	43.772	
$M_{ox}= M_{cx}(1-F_c/P_{cx})/(1+0.5F_c/P_{cx})$	43.772(1-0/721.4)/(1+0.5•0/721.4)	43.772	
$M_{oy}= M_{cv}(1-F_c/P_{cy})/(1+k_y(F_c/P_{cy}))$	8.703(1-0/689.2)/(1+1.0(0/689.2))	8.703	
M_{ab}=fn(λ_r, λ_{ro}, ε, M_{xy}, M_{ob})	39.090, 30.189, 0.880, 43.772, 40.427	43.115	
M_{ax}=fn(λ_x, ε, M_{rx}, M_{ox})	8.825, 0.880, 43.772, 43.772	43.772	
M_{ay}=fn(λ_y, ε, M_{ry}, M_{oy})	26.810, 0.880, 8.703, 8.703	8.703	
$m_x.M_x/M_{ax}+.5m_{yx}.M_y/M_{cy}(1-F_c/P_{cx})$	0.911x5.4/43.8+.5x0.557x0/(8.7(1-0/721.4))	0.113	OK
$m_{LT}.M_{LT}/M_{ab}+m_y.M_y/M_{ay}$	0.911x-5.4/43.1+0.557x0/8.7	0.114	OK
$m_x.M_x/M_{ax}+m_y.M_y/M_{ay}$	0.911x5.4/43.8+0.557x0/8.7	0.113	OK
Compare with Simplied to 4.8.3.3	0.127, 0.122, 0.122	0.127	
Compare with MoreExact to 4.8.3.3	0.113, 0.122, 0.113	0.122	

Deflection Check - Load Case 2

Deflection Limits (Cantilevers)	In-span $\delta \leq$ 566/180 = 3.1 mm Live (Case 11)	0.03 mm	OK
	In-span $\delta \leq$ 566/125 = 4.5 mm D+L (Case 2)	0.08 mm	OK

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barcombe east sussex BN8 5EE
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project no : 8850
by : ICR
date : Jan 2022
no : 14

Beam: B4 **Span: 1.3 m.**

		Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp	Defl.
O	D	o.w.	0.1	0		L	0.06	0.06	0.00
U	L	flat roof	0.75*4.9^2/2*3.7	0		L	1.58	1.58	0.09
U	D	flat roof	0.75*4.9^2/2*3.7	0		L	1.58	1.58	0.09
U	D	solar PV	0.25*3.7/2	0		L	0.30	0.30	0.02
U	D	services	0.25*3.7/2	0		L	0.30	0.30	0.02
P	D	HRV unit	2.5/2	L/2			0.63	0.63	0.06
				Total load (unfactored):		**8.91 kN**	**4.45**	**4.45**	0.28
				Dead/Permanent (unfactored):		5.75 kN	2.87	2.87	0.19
				Live/Variable (unfactored):		3.16 kN	1.58	1.58	0.09

Load durations: D: Dead

Maximum B.M. = 1.65 kNm (unfactored) at 0.65 m. from R1
Maximum S.F. = 4.45 kN (unfactored) at R1
Mid-span deflections: Dead: 0.19 x 10^8/EI ***(E in N/mm², I in cm⁴)***
Live: 0.09 x 10^8/EI
Total: 0.28 x 10^8/EI

Timber beam calculation to BS5268 Part 2: 2002 using C24 timber

Use 2no 47 x 147 C24 ***5.8 kg/m approx***

z = 339 cm^3 I = 2,488 cm^4

Timber grade: C24 2 members acting together: K_8 = 1.1 [§2.9]

K_3 (loading duration factor) = 1.00 (long term)

K_7 (depth factor) = $(300/147)^{0.11}$ = 1.08 [§2.10.6] K_8 (load sharing factor) = 1.1 [§2.9,2.10]

E = 7,200 x 1.14 = 8,208 N/mm^2 ($E_{min}.K_9$)

Bending

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g}.K_3.K_7.K_8$ = 7.5 x 1.00 x 1.08 x 1.1 = 8.92 N/mm^2

Applied bending stress, $\sigma_{m,a}$ = 1.65 x 1000/339 = 4.88 N/mm^2 OK

Shear

Permissible shear stress, $\tau_{adm,//} = \tau_{g,//}.K_3.K_8$ = 0.71 x 1.00 x 1.1 = 0.78 N/mm^2

Applied shear stress, τ_a = 4.45 x 1000 x 3/(2 x 94 x 147) = 0.48 N/mm^2 OK

Deflection

Bending deflection = 0.276 x 10^8/(8,208 x 2,488) = 1.4 mm

Mid-span shear deflection = $1.2M_0$/GA (G=E/16) = 1.2 x 1.65 x 10^6/((8208/16) x 94 x 147) = 0.3 mm

Total deflection = 1.4 + 0.3 = 1.6 mm (0.0013 L) <= 0.003L OK

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barcombe east sussex BN8 5EE
01273 470 066 info@ingsengineering.co.uk

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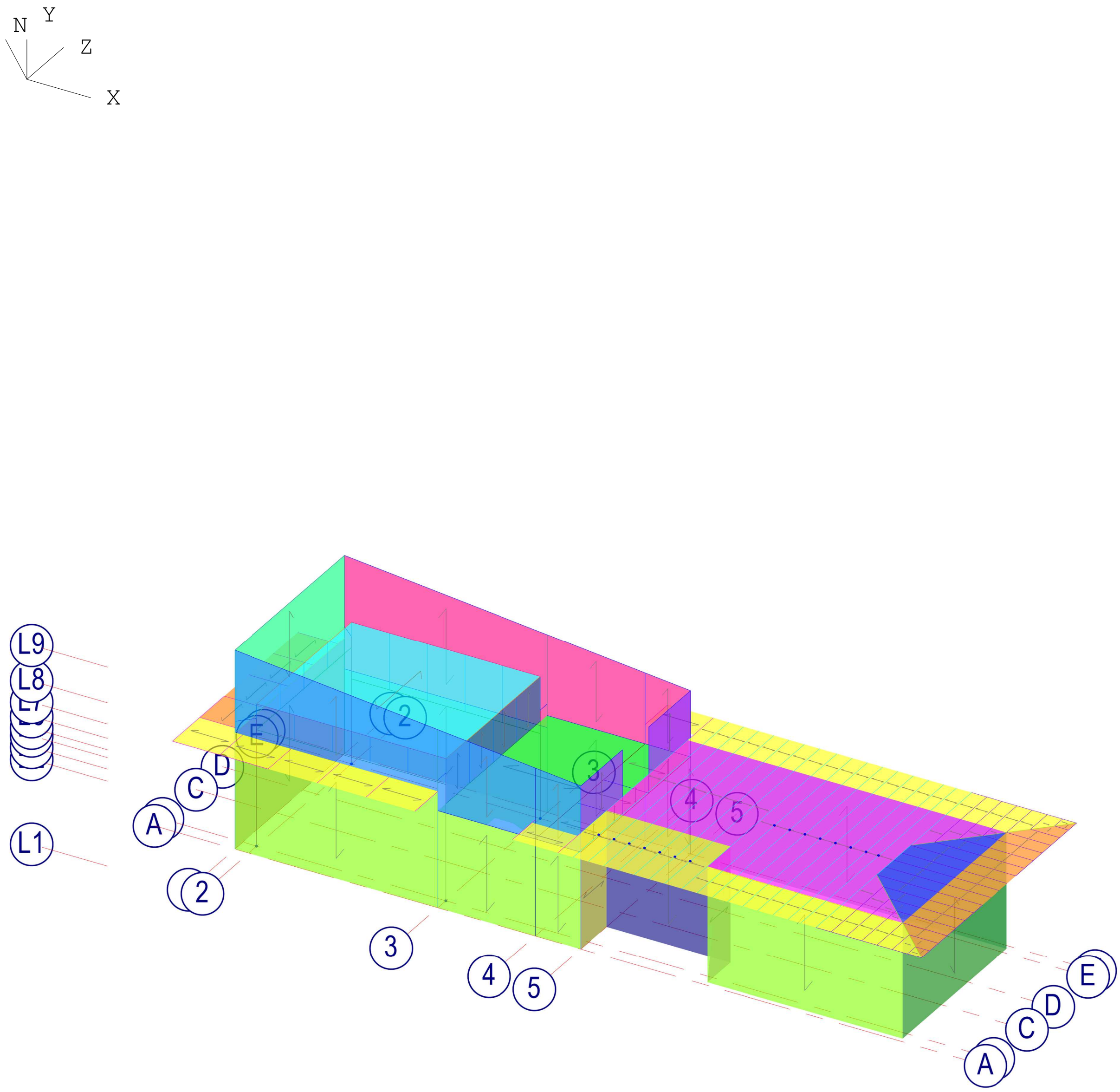

Frame Geometry - (Full Frame) - 3D Front View

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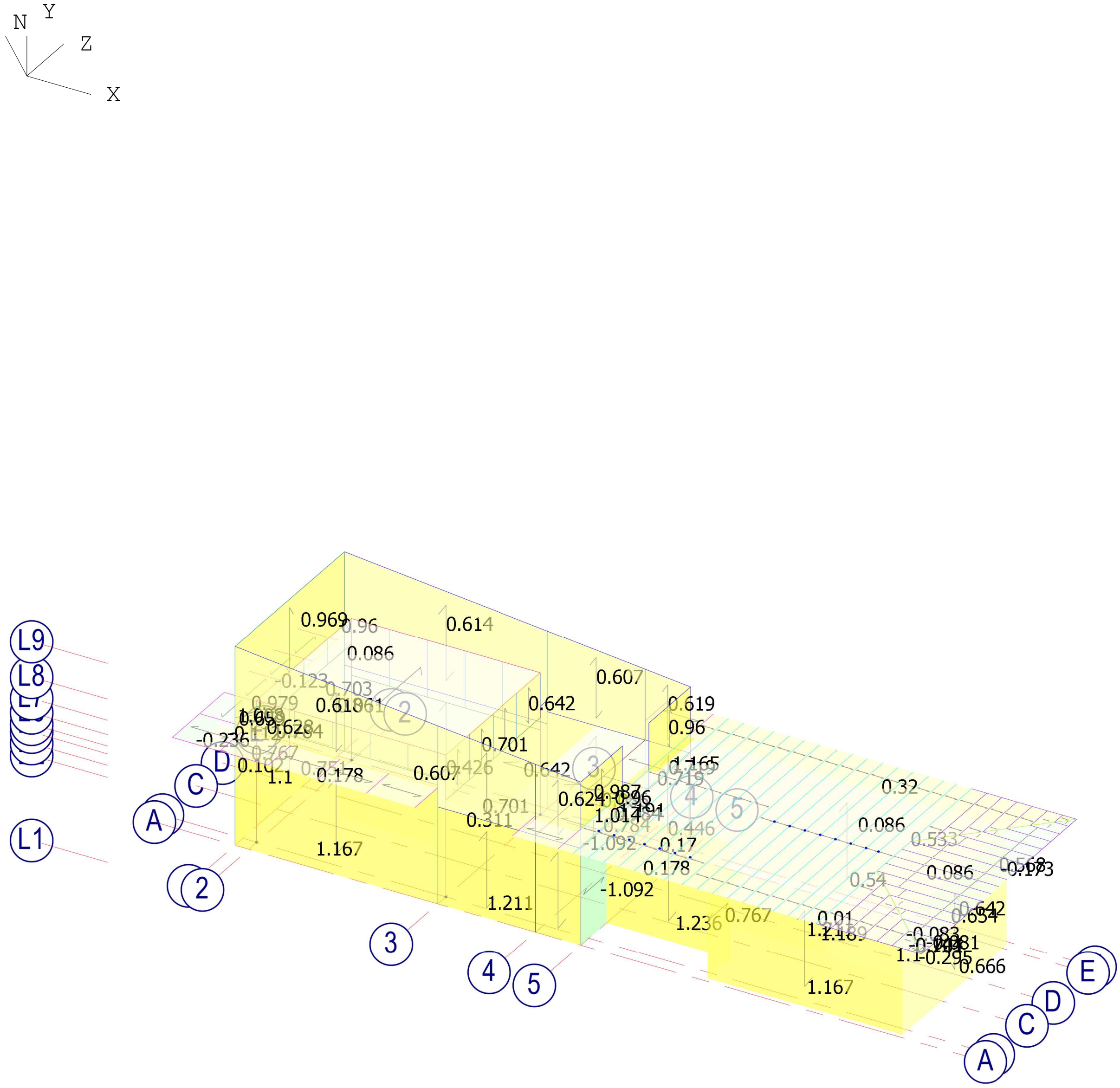

Frame Geometry - (Full Frame) - 3D Front View

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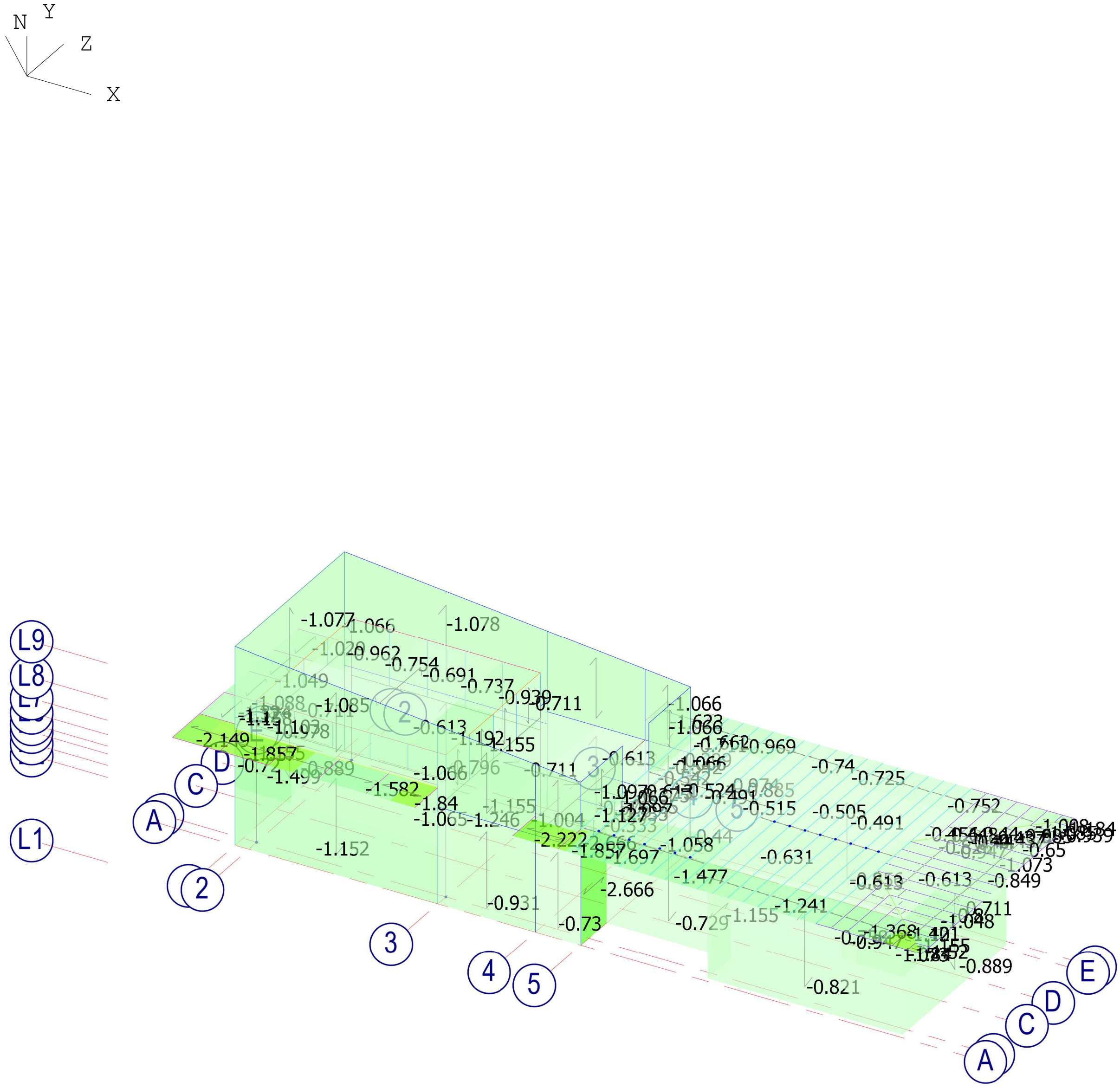

Frame Geometry - (Full Frame) - 3D Front View

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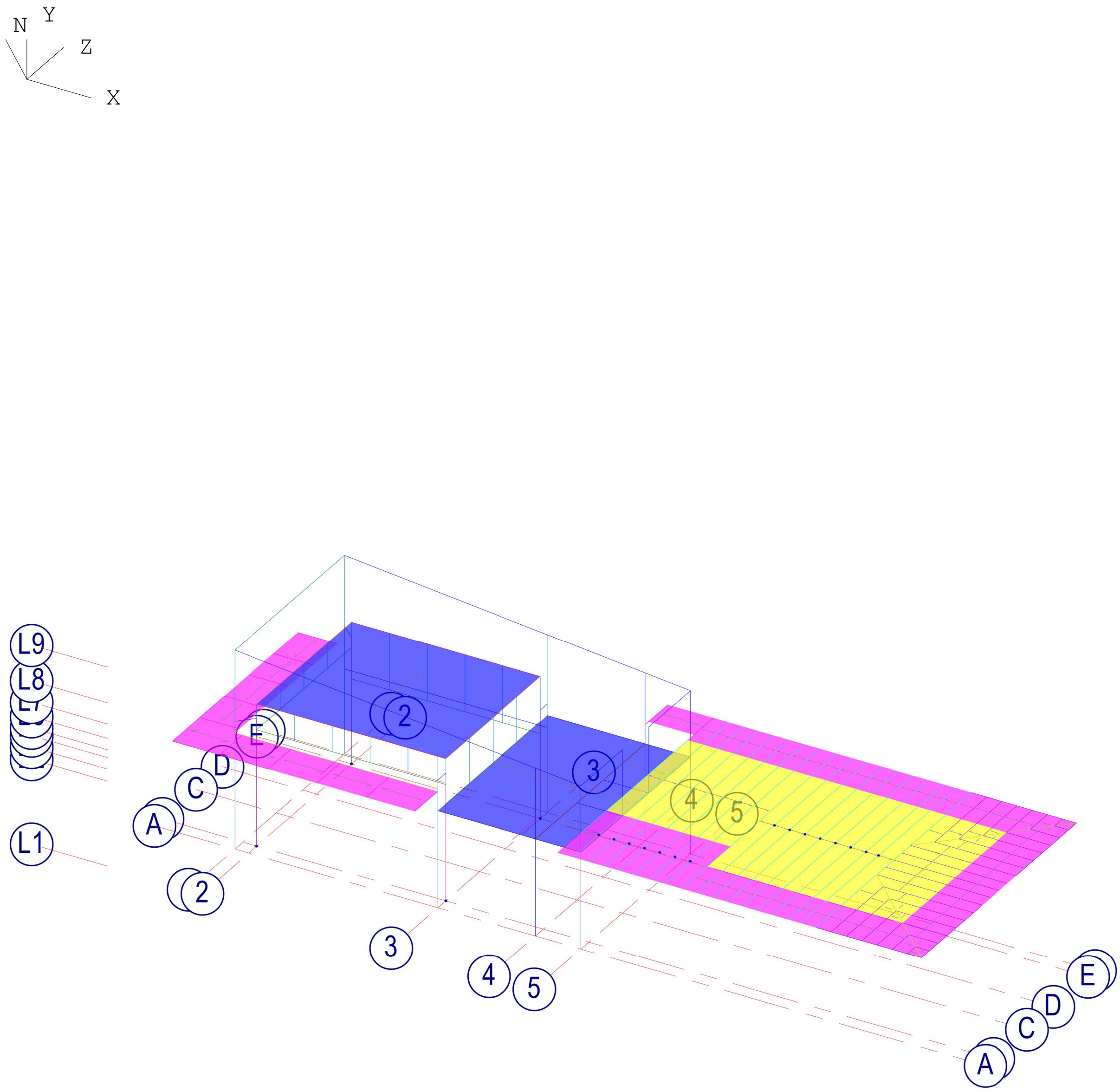

Dead D	Live L
1.250	1.500
1.250	0.750
1.000	0.750

Frame Geometry - (Full Frame) - 3D Front View

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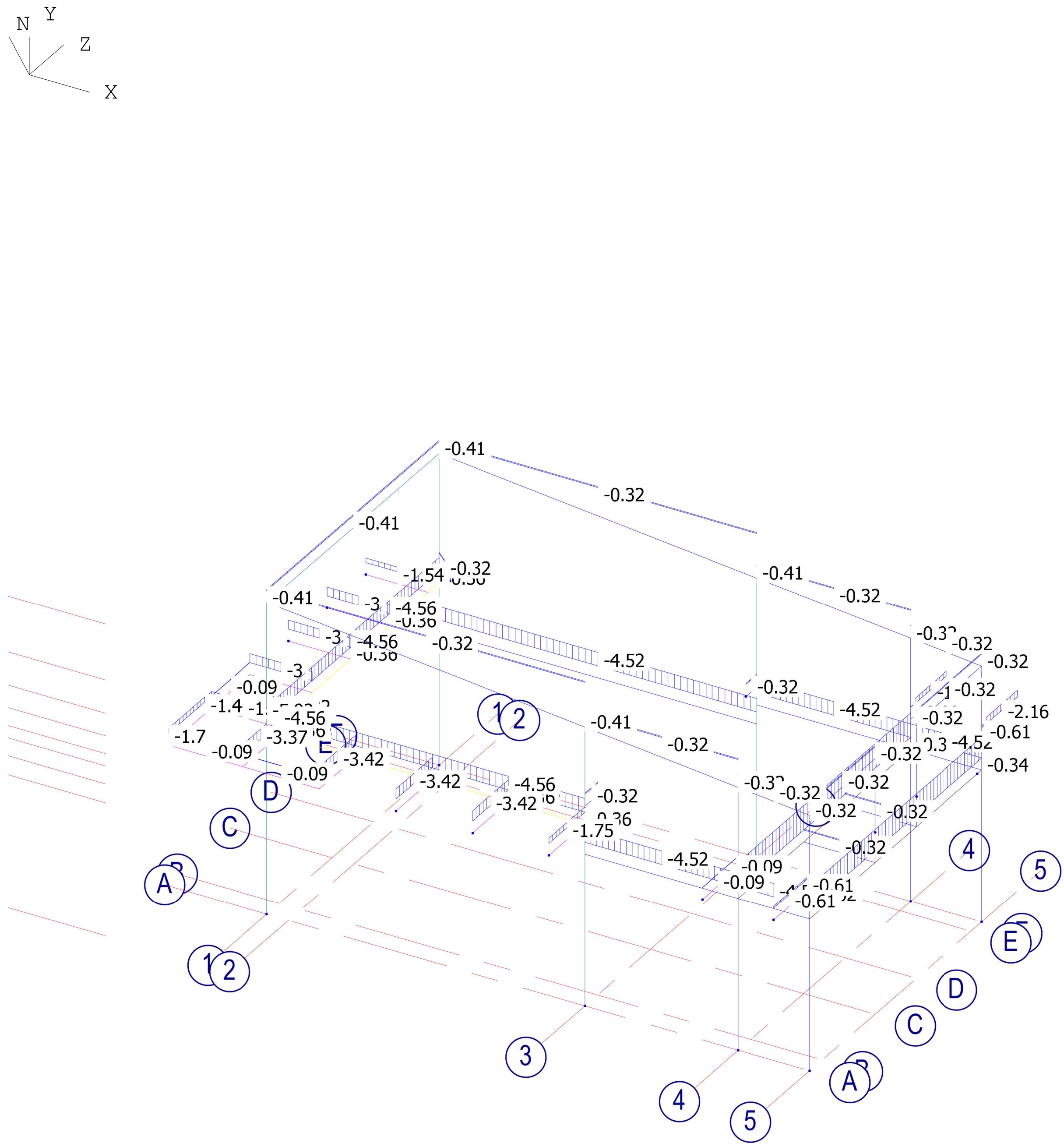

Load Diagram - Load Case 001 - All Groups - Including Automatic Selfweight (Density)

Frame Geometry - (Full Frame) - 3D Front View

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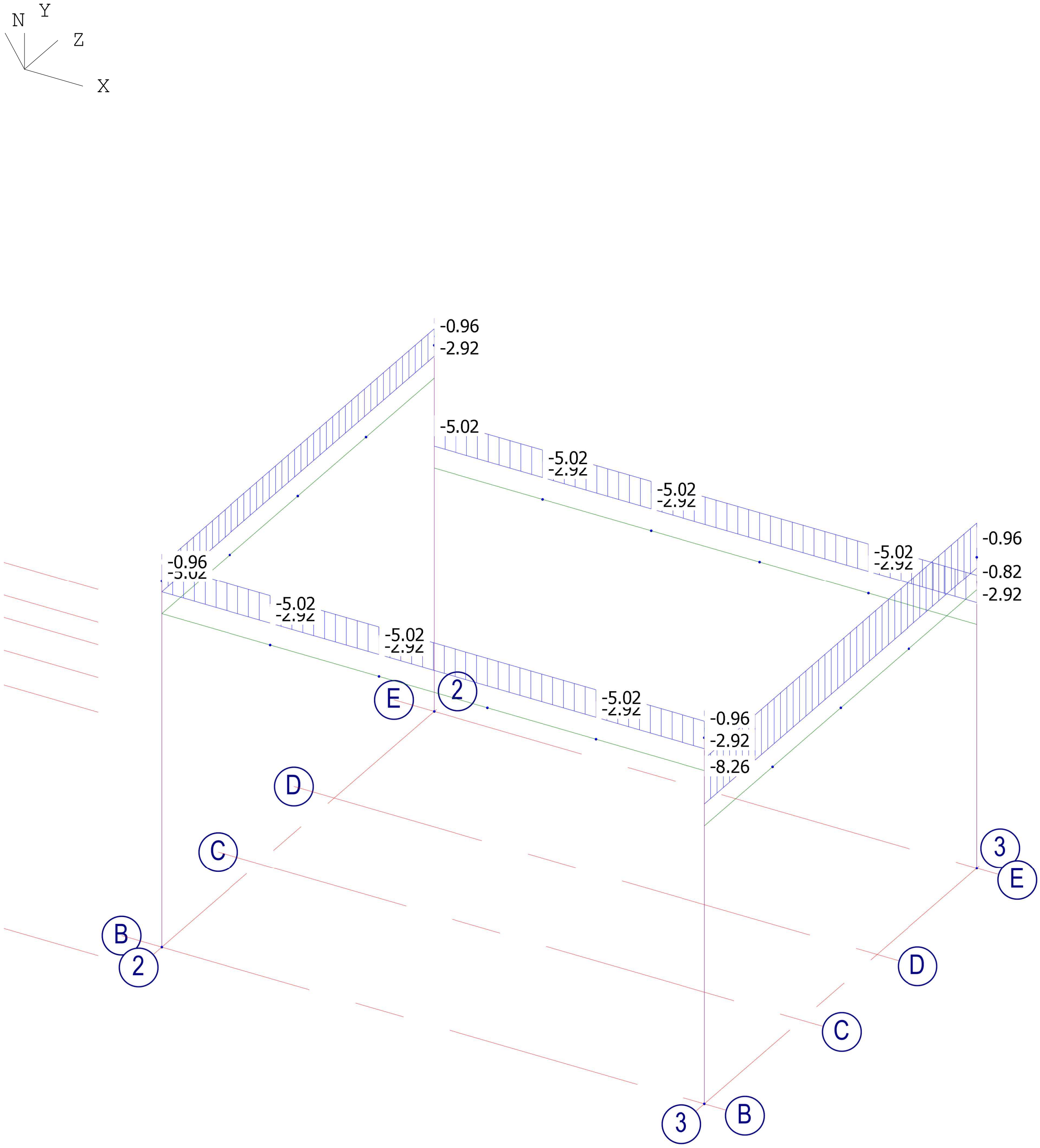

Load Diagram - Load Case 001 - All Groups - Including Automatic Selfweight (Density)
Frame Geometry - (Full Frame) - 3D Front View

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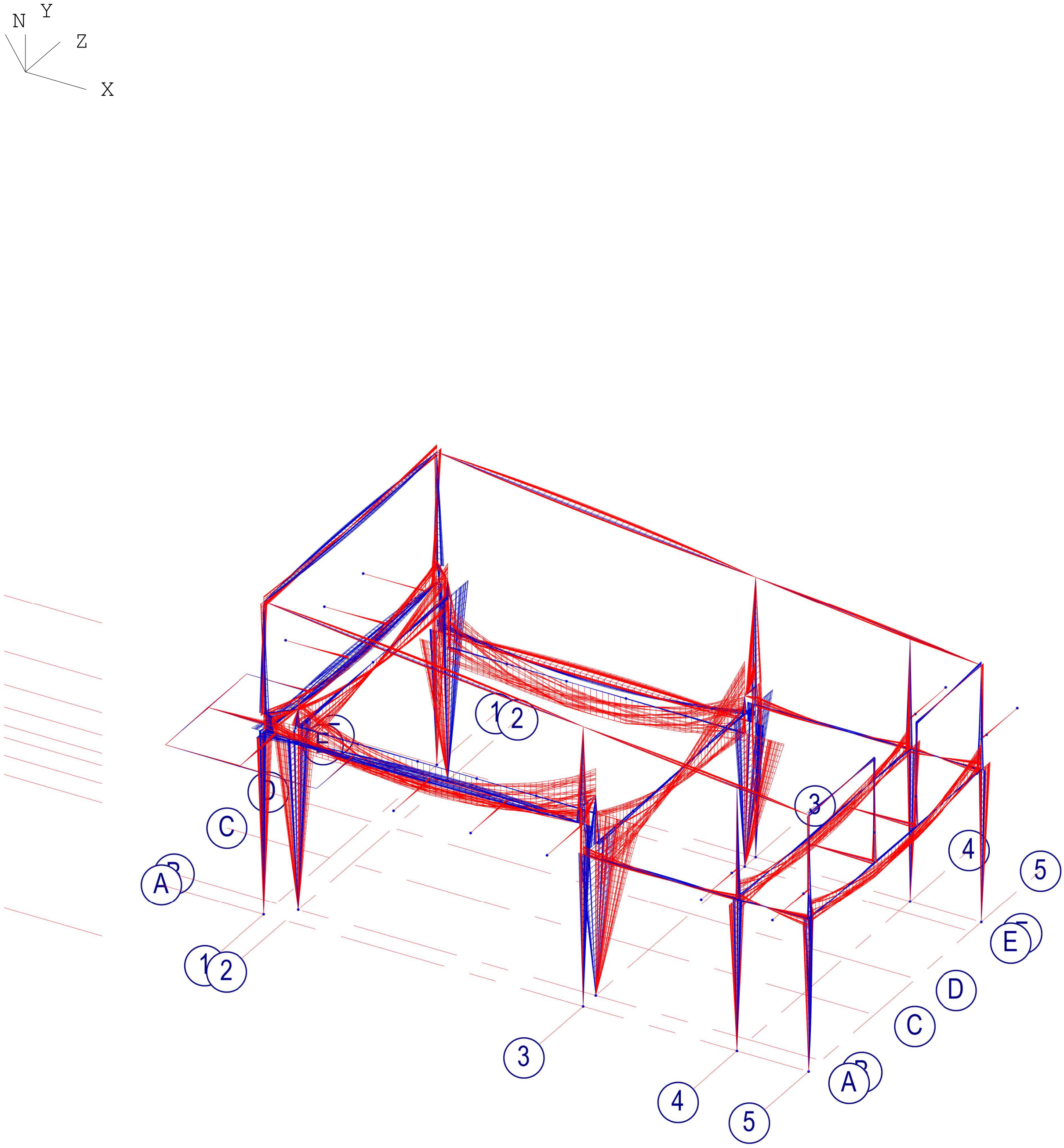

Envelope (Ultimate Cases)

Bending Moment Envelope (Ultimate Cases) - (Full Frame) - 3D Front View

Not to Scale **50 kN.m = 1m**

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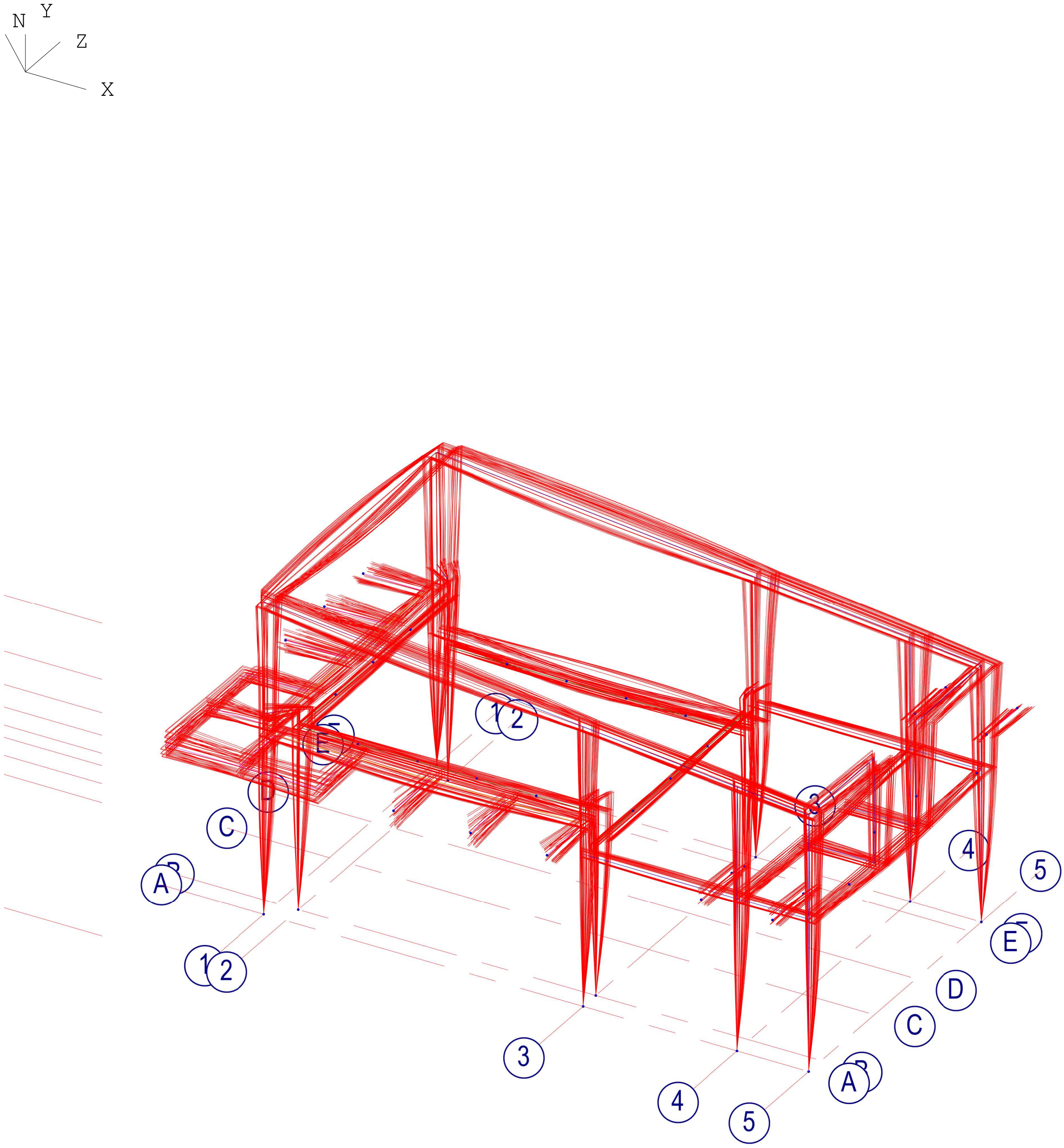

Envelope (Serviceability Cases)
Deflected Shape - (Full Frame) - 3D Front View

Not to Scale **20 Magnification**

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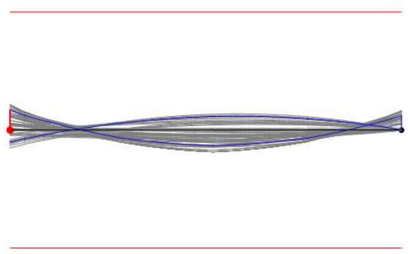
Axial with Moments (Member)
Includes Design for Torsion
Member SB A\1-3L5Id 3 @ Level 5 in Load Case 39 : 200x150 RHS lintel beams

Member Loading and Member Forces

Loading Combination : 1 UT + 1.2 D0 + 1.2 D1 + 1.2 L1 + 1.2 S3

```
D1 UDLY -003.000              ( kN/m )
D1 D     077.010              ( kN/m³)
S3 PTRM +001.888 0.000 2.764 +001.888
S3 PTRZ +001.301 0.000 0.200 +001.289
S3 PTRZ +001.289 0.200 1.482 +001.210
S3 PTRZ +001.210 1.482 2.764 +001.130
S3 PTRM +002.002 1.482 1.482 +002.002
S3 PTRZ +001.130 2.764 4.046 +001.051
S3 PTRM +002.078 2.764 2.764 +002.078
S3 PTRZ +001.051 4.047 5.328 +000.972
S3 PTRM +002.078 4.047 4.047 +002.078
```



Member Forces in Load Case 39 and Maximum Deflection from Load Case 72

Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) x-x	Shear Force (kN) y-y	Bending Moment (kN.m) x-x	Bending Moment (kN.m) y-y	Maximum Moment (kN.m @ m) x-x	Maximum Moment (kN.m @ m) y-y	Maximum Deflection (mm @ m)
	21 68	8.56T 8.78T	3.23 1.28	16.23 -12.79	-10.38 7.18	-11.85 -8.34	7.13 -0.15	9.12 @ 2.764	-8.20 @ 3.095	7.73 @ 2.844

Classification and Properties (BS 5950: 2000)

Section (26.5 kg/m)	200x150x5 RHS 26.5 [S 355]		
Class = Fn(b/t,d/t,py,F,Mx,My)	27, 37, 355, 0, 11.855, 8.196	(Axial: Slender)	Slender
Effective Properties	Area=32.83(33.73) cm², Zxx=197.01 cm³, Zyy=163.29(168.61) cm³		
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	16.229 / 399.622 =	0.041	Low Shear
Mcx = py.Zxx	355 x 197.01 =	69.939 kN.m	
Fvy/Pvy	10.381 / 299.717 =	0.035	Low Shear
Mcy = py.Zyy	355 x 163.29 =	57.969 kN.m	
Ae = Fn(Ag,A.net,py,Us)	32.83,32.83,355,470	32.83 cm²	
Pz = Ae.py	32.83x355	1165.565 kN	
F/Ae.py+Mx/Mcx+My/Mcy	-8.78 / 1165.565 + 11.855 / 69.939 + 7.13 / 57.969 =	0.3	OK

Equivalent Uniform Moment Factors mLT, mx, my and myx

m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}	0.2+(.15x3+.5x9+.15x5)/12 ≥ 0.44	0.686	Table 18
m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}	0.2+(.1x-3+.6x-8+.1x-7)/8 ≥ .8x8/8	0.928	Table 26
m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}	0.2+(.1x3+.6x9+.1x5)/12 ≥ .8x9/12	0.726	Table 26
m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}	0.2+(.1x-3+.6x-8+.1x-7)/8 ≥ .8x8/8	0.9	Table 26

Lateral Buckling Check Mb

Mb = Mc	Section not susceptible to lateral torsional buckling	69.939 kN.m

Simplified Approach

py.Zx	355x197.01	69.939 kN.m	
py.Zy	355x163.29	57.969 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	0+0.726x11.9/69.9+0.928x7.1/58	0.237	OK
F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy	0+0.686x-11.9/69.9+0.928x7.1/58	0.230	OK

More Exact Approach

Max=Mcx/(1+.5F/Pcx)	69.9/(1+.5x0/897.7)	69.939 kN.m	
May=Mcy/(1+.5F/Pcy)	58/(1+.5x0/705)	57.969 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	0/897.7+0.726x11.9/69.9+.5x0.9x8.2/58	0.187	OK
F/Pcy+.5m_{LT}.M_{LT}/Mcx+my.My/May	0/705+.50.686x-11.9/69.9+0.928x7.1/58	0.172	OK

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Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	69.9(1-0/897.7)/(1+.5x0/897.7)	69.939 kN.m	
May=Mcy(1-F/Pcy)/(1+.5F/Pcy)	58(1-0/705)/(1+.5x0/705)	57.969 kN.m	
m.Mx/Max+m.My/May	0.726x11.855/69.939+0.928x7.13/57.969	0.230	OK

Torsion Design

J, C, Q_f, Q_w	2386 cm^4, 266.9 cm^3, 97.0 cm^3, 118.2 cm^3		
K_v= 1+0.5 m.Mx/Mb	1 + 0.5 • 0.726 • 11.86 / 69.94	1.062	

Torsion Bending Design @ 0.000

$M_{yt} = M_{xb}. \theta$	11.85 • 2.353•10^{-3}	0.028 kN.m	
$\sigma_{byt} = M_{yt} / Z_y$	0.028 / 163.3	0.17 N/mm^2	
Modified Local Capacity Check	0.3 + 0.17 / 355	0.301	OK
Modified Buckling Check	0.23 + 0.17 • 1.062 / 355	0.231	OK

Torsion Shear Design

$\tau_{bw} = F_v. Q_w / (2\, I_x.\, t)$	16.23 • 118.2 / (2 • 1970 • 5.0)	9.74 N/mm^2	
$\tau_t = T_o / C$	3.234 / 266.9	12.12 N/mm^2	
Shear Stress = $(\tau_{bw} + \tau_t.\, K_v) / p_v$	9.74 + 12.12 • 1.062 = 22.60 N/mm^2 / 213	0.106	OK

Deflection Check - Load Case 72

In-span $\delta \leq$ Span/360	7.73 $\leq$ 5329 / 360	7.73 mm	OK
Torq in Case 16 @ 2.764 m	θ_{max}= 3.218•10^{-3} Radian = 0.184° $\leq$ 2.00°	0.184°	OK

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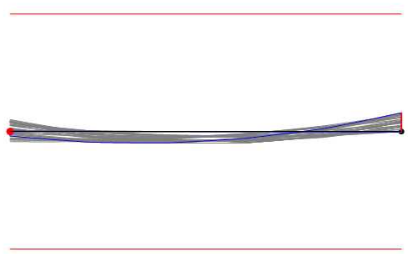
Axial with Moments (Member)
203x203 UC60 main frame beams
Member SB B\2-3L5Id 4 @ Level 5 in Load Case 35

Member Loading and Member Forces

Loading Combination : 1 UT + 1.2 D0 + 1.2 D1 + 1.2 L1 + 1.2 S2

```
D1 UDLY -001.500          ( kN/m )
D1 D     077.010          ( kN/m³)
D1 UDLY -001.500          ( kN/m )
```



Member Forces in Load Case 35 and Maximum Deflection from Load Case 2										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
	23 111	12.75C 13.12C	-0.01 0.01	14.28 -33.88	0.38 0.48	8.83 -37.01	-0.92 1.19	22.58 @ 1.866		2.73 @ 2.472

Classification and Properties (BS 5950: 2000)

Section (59.95 kg/m)	203x203 UC 60 [S 355]		
Class = Fn(b/T,d/t,py,F,Mx,My)	7.25, 17.11, 355, 13.12, 37.01, 1.19	(Axial: Non-Slender)	Plastic
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	33.875 / 419.661 =	0.081	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 656.1≤1.2 x 355 x 584.68 =	232.916 kN.m	
Fvy/Pvy	0.476 / 1120.433 =	0	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 305.3≤1.2 x 355 x 200.92 =	85.592 kN.m	
Pz = Ag.py	76.37 x 355 =	2711.135 kN	
n = F/Pz	13.115 / 2711.135 =	0.005	OK
Srx = Fn(Sxx, n)	656.1, 0.005	656.06 cm³	
Mrx = Srx.py	656.06 x 355	232.903 kN.m	
Sry = Fn(Syy, n)	305.3, 0.005	305.3 cm³	
Mry = Sry.py	305.3 x 355	85.592 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(37.013/232.903)^2+(1.192/85.592)^1=$	0.039	OK

Compression Resistance Pc

λx = Lex/rxx	100x1x4.964/8.96 =	55.4	OK
Pcx = Area.pcx	76.37x284.436/10 =	2172.239 kN	Table 24 b
λy = Ley/ryy	100x1x4.964/5.2 =	95.5	OK
Pcy = Area.pcy	76.37x150.13/10 =	1146.557 kN	Table 24 c

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}$	0.2+(.15x22+.5x19+.15x0)/37 ≥ 0.44	0.541	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x1)/1 ≥ .8x1/1	0.412	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x22+.6x19+.1x0)/37 ≥ .8x23/37	0.56	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x1)/1 ≥ .8x1/1	0.409	Table 26

Lateral Buckling Check Mb

Le = 1.00 L	1 x 4.964 =	4.964 m	
λ = Le/ryy	4.964 / 5.2	95.46	OK
v = Fn (x,Le,ryy,λ)	14.042, 4.964, 5.2, 95.46	0.741	Table 19
$\lambda_{LT}= u.v.\lambda.\sqrt{\beta_W}$	$0.847 \times 0.741 \times 95.46 \sqrt{1}$	59.9	
pb = Fn (py,λ_{LT})	355, 59.9	256.88 N/mm²	Table 16
Mb = Sxx.pb ≤ Mc	656.1 x 256.88 ≤ 232.916 =	168.537 kN.m	

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Combined Axial Compression and Bending to Annex I

$r_b=m_{LT}.M_{LT}/M_b$	0.541x-37/168.5	0.119	
$r_c=F_c/P_{cy}$	13.1/1146.6	0.011	
$\lambda_r=(r_b\lambda_{LT}+r_c\lambda_y)/(r_b+r_c)$	(0.119•59.9+0.011•95.5)/(0.119+0.011)	63.026	
$\lambda_{ro}=17.15\ \varepsilon\ (2r_b+r_c)/(r_b+r_c)$	17.15•0.88(2•0.119+0.011)/(0.119+0.011)	28.864	
$M_{ob}= M_b(1-F_c/P_{cy})$	168.537(1-13.1/1146.6)	166.609	
$M_{xy}= M_{cx}(1-F_c/ P_{cy})^{½}$	232.916(1-13.1/1146.6)^{½}	231.580	
$M_{ox}= M_{cx}(1-F_c/P_{cx})/(1+0.5F_c/P_{cx})$	232.916(1-13.1/2172.2)/(1+0.5•13.1/2172.2)	230.813	
$M_{oy}= M_{cv}(1-F_c/P_{cy})/(1+k_y(F_c/P_{cy}))$	85.592(1-13.1/1146.6)/(1+1.0(13.1/1146.6))	83.656	
$M_{ab}=fn(\ \lambda_r,\ \lambda_{ro},\ \varepsilon,\ M_{xy},\ M_{ob})$	63.026, 28.864, 0.880, 231.580, 166.609	184.358	
$M_{ax}=fn(\ \lambda_x,\ \varepsilon,\ M_{rx},\ M_{ox})$	55.398, 0.880, 232.903, 230.813	231.508	
$M_{ay}=fn(\ \lambda_y,\ \varepsilon,\ M_{ry},\ M_{oy})$	95.456, 0.880, 85.592, 83.656	83.656	
$m_x.M_x/M_{ax}+.5m_{yx}.M_y/M_{cy}(1-F_c/P_{cx})$	0.56x37/231.5+.5x0.409x1.2/(85.6(1-13.1/2172.2))	0.092	OK
$m_{LT}.M_{LT}/M_{ab}+m_y.M_y/M_{ay}$	0.541x-37/184.4+0.412x1.2/83.7	0.115	OK
$m_x.M_x/M_{ax}+m_y.M_y/M_{ay}$	0.56x37/231.5+0.412x1.2/83.7	0.095	OK
Compare with Simplied to 4.8.3.3	0.118, 0.137, 0.137	0.137	
Compare with MoreExact to 4.8.3.3	0.098, 0.136, 0.093	0.136	

Deflection Check - Load Case 2

In-span $\delta \leq$ Span/360	$2.73 \leq 4964 / 360$	2.73 mm	OK

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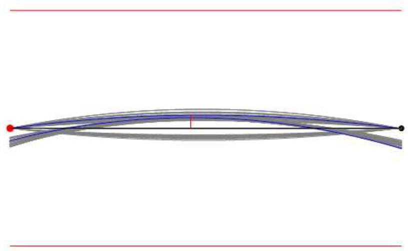
Axial with Moments (Member)

Member SB F\1-3L2Id 6 @ Level 2 in Load Case 35 : 152x152 UC23 steel frame

Member Loading and Member Forces

Loading Combination : 1 UT + 1.2 D0 + 1.2 D1 + 1.2 L1 + 1.2 S2

```
D1 UDLY -003.000             ( kN/m )
D1 D     077.010             ( kN/m³)
S2 PTRZ +001.814 0.000 5.329 +001.481
S2 UDLN +000.218             ( Co/m )
```



Member Forces in Load Case 35 and Maximum Deflection from Load Case 62											
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
18	27 28	0.69T 0.69T	0.00 0.00	-5.27 5.02	-9.54 11.09	0.05 0.19	6.28 10.41	-6.74 @ 2.582	-5.47 @ 2.472	13.57 @ 2.582

Classification and Properties (BS 5950: 2000)

Section (22.95 kg/m)	152x152 UC 23 [S 355]		
Class = Fn(b/T,d/t,py,F,Mx,My)	11.19, 21.31, 355, 0, 6.74, 10.41	(Axial: Non-Slender)	SemiComp
Effective Properties	Area=29.24 cm², Sxx=170.55(182) cm³, Syy=62.56(80.2) cm³		
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	5.021 / 188.275 =	0.027	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 170.55≤1.2 x 355 x 164.13 =	60.546 kN.m	
Fvy/Pvy	11.085 / 396.804 =	0.028	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 62.56≤1.2 x 355 x 52.67 =	22.21 kN.m	
Ae = Fn(Ag,A.net,py,Us)	29.24,29.24,355,470	29.24 cm²	
Pz = Ae.py	29.24x355	1038.02 kN	
n = F/Pz	-0.688 / 1038.02 =	0.001	OK
F/Ae.py+Mx/Mcx+My/Mcy	-0.688 / 1038.02 + 0.192 / 60.546 + 10.409 / 22.21 =	0.472	OK

Equivalent Uniform Moment Factors mLT, mx, my and myx

m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}	0.2+(.15x5+.5x7+.15x5)/7 ≥ 0.44	0.923	Table 18
m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}	0.2+(.1x-3+.6x-5+.1x-1)/10 ≥ .8x5/10	0.548	Table 26
m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}	0.2+(.1x-5+.6x-7+.1x-5)/7 ≥ .8x7/7	0.949	Table 26
m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}	0.2+(.1x-3+.6x-5+.1x-1)/10 ≥ .8x5/10	0.548	Table 26

Lateral Buckling Check Mb

Le = 1.00 L	1 x 5.329 =	5.329 m	
λ = Le/ryy	5.329 / 3.7	144.02	OK
v = Fn (x,Le,ryy,λ)	20.292, 5.329, 3.7, 144.02	0.73	Table 19
λ_{LT}= u.v.λ.√βw	0.84 x 0.73 x 144.02 √ 0.937	85.52	
pb = Fn (py,λ_{LT})	355, 85.52	173.83 N/mm²	Table 16
Mb = Sxx.pb ≤ Mc	170.55 x 173.83 ≤ 60.546 =	29.647 kN.m	

Simplified Approach

py.Zx	355x164.13	58.266 kN.m	
py.Zy	355x52.67	18.698 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	0+0.949x6.7/58.3+0.548x-5.5/18.7	0.270	OK
F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy	0+0.923x-6.7/29.6+0.548x-5.5/18.7	0.369	OK

More Exact Approach

Max=Mcx/(1+.5F/Pcx)	60.5/(1+.5x0/603.5)	60.546 kN.m	
May=Mcy/(1+F/Pcy)	22.2/(1+0/228.2)	22.21 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	0/603.5+0.949x6.7/60.5+.5x0.548x10.4/22.2	0.234	OK

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$F/Pcy+m_{LT}.M_{LT}/Mb+my.My/May$	0/228.2+0.923x-6.7/29.6+0.548x-5.5/22.2	0.344	OK
Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	60.5(1-0/603.5)/(1+.5x0/603.5)	60.546 kN.m	
May=Mcy(1-F/Pcy)/(1+F/Pcy)	22.2(1-0/228.2)/(1+0/228.2)	22.21 kN.m	
m.Mx/Max+m.My/May	0.949x6.714/60.546+0.548x5.462/22.21	0.237	OK

Deflection Check - Load Case 62

In-span $\delta \leq$ Span/360	13.57 $\leq$ 5329 / 360	13.57 mm	OK

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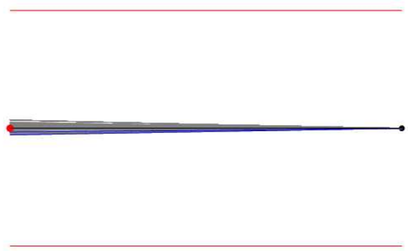
Axial with Moments (Member)

Member SB A\2L5Id 8 @ Level 5 in Load Case 67 : 70x70 SHS canopy edge beam

Member Loading and Member Forces

Loading Combination : 1 UT + 1 D0 + 1 D1 + 1.4 S2

```
D1 D     077.010            ( kN/m³)
```



Member Forces in Load Case 67 and Maximum Deflection from Load Case 16										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) x-x	Shear Force (kN) y-y	Bending Moment (kN.m) x-x	Bending Moment (kN.m) y-y	Maximum Moment (kN.m @ m) x-x	Maximum Moment (kN.m @ m) y-y	Maximum Deflection (mm @ m)
20	35 37	0.44T 0.44T	0.00 0.00	-0.13 -0.21	-0.30 -0.30	0.22 0.00	0.39 0.00			0.36 @ 0.539

Classification and Properties (BS 5950: 2000)

Section (6.24 kg/m)	70x70x3 SHS 6.24 [S 355]		
Class = Fn(b/t,d/t,py,F,Mx,My)	20.33, 20.33, 355, 0, 0.215, 0.391	(Axial: Non-Slender)	Plastic
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	0.129 / 84.561 =	0.002	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 19.87≤1.2 x 355 x 16.86 =	7.054 kN.m	
Fvy/Pvy	0.305 / 84.561 =	0.004	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 19.87≤1.2 x 355 x 16.86 =	7.054 kN.m	
Ae = Fn(Ag,A.net,py,Us)	7.94,7.94,355,470	7.94 cm²	
Pz = Ae.py	7.94x355	281.87 kN	
n = F/Pz	-0.436 / 281.87 =	0.002	OK
Srx = Fn(Sxx, n)	19.87, 0.002	19.87 cm³	
Mrx = Srx.py	19.87 x 355	7.054 kN.m	
Sry = Fn(Syy, n)	19.87, 0.002	19.87 cm³	
Mry = Sry.py	19.87 x 355	7.054 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(0.215/7.054)^{1.667}+(0.391/7.054)^{1.667}=$	0.011	OK

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}$	0.2+(.15x0+.5x0+.15x0)/0 ≥ 0.44	0.643	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.601	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.64	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.599	Table 26

Lateral Buckling Check Mb

Mb = Mc	Section not susceptible to lateral torsional buckling	7.054 kN.m

Simplified Approach

py.Zx	355x16.86	5.985 kN.m	
py.Zy	355x16.86	5.985 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	0+0.64x0.2/6+0.601x0.4/6	0.062	OK
$F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy$	0+0.643x0.2/7.1+0.601x0.4/6	0.059	OK

More Exact Approach

Max=Mcx/(1+.5F/Pcx)	7.1/(1+.5x0/256.6)	7.054 kN.m	
May=Mcy/(1+.5F/Pcy)	7.1/(1+.5x0/256.6)	7.054 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	0/256.6+0.64x0.2/7.1+.5x0.599x0.4/7.1	0.036	OK
$F/Pcy+.5m_{LT}.M_{LT}/Mcx+my.My/May$	0/256.6+.50.643x0.2/7.1+0.601x0.4/7.1	0.043	OK
Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	7.1(1-0/256.6)/(1+.5x0/256.6)	7.054 kN.m	

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May=Mcy(1-F/Pcy)/(1+.5F/Pcy)	7.1(1-0/256.6)/(1+.5x0/256.6)	7.054 kN.m	
m.Mx/Max+m.My/May	0.64x0.215/7.054+0.601x0.391/7.054	0.053	OK

Deflection Check - Load Case 16

In-span $\delta \leq$ Span/360	$0.36 \leq 1282 / 360$	0.36 mm	OK

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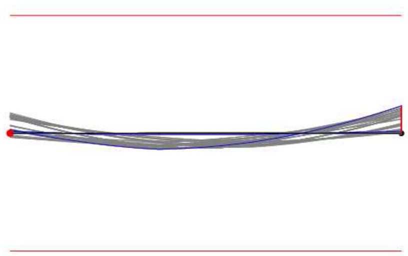
Axial with Moments (Member)

Member SB 5\A-FL3Id 169 @ Level 3 in Load Case 35 : 203x133 UB25 steel frame

Member Loading and Member Forces

Loading Combination : 1 UT + 1.2 D0 + 1.2 D1 + 1.2 L1 + 1.2 S2

```
D1 UDLY -003.746              ( kN/m )
L1 UDLY  -000.895             ( kN/m )
D1 D      077.010             ( kN/m³)
S2 PTRY +001.612 0.000 1.169 +001.612
S2 PTRX -000.088 0.000 1.169 -000.088
S2 PTRY -000.330 1.169 1.896 -000.330
S2 PTRX +000.139 1.169 1.896 +000.139
S2 PTRY -000.330 1.896 3.096 -000.330
S2 PTRX +000.139 1.896 3.096 +000.139
S2 PTRY -000.330 3.096 4.863 -000.330
S2 PTRX +000.139 3.096 4.863 +000.139
S2 PTRY -000.060 4.863 4.992 -000.060
S2 PTRM +000.249 4.863 4.863 +000.249
```



Member Forces in Load Case 35 and Maximum Deflection from Load Case 32

Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) x-x	Shear Force (kN) y-y	Bending Moment (kN.m) x-x	Bending Moment (kN.m) y-y	Maximum Moment (kN.m @ m) x-x	Maximum Moment (kN.m @ m) y-y	Maximum Deflection (mm @ m)
	17	18.65T	0.00	12.30	1.97	-2.86	-1.51	12.40	0.86	4.03
	114	18.93T	0.00	-20.66	-1.35	-21.32	-1.57	@ 1.896	@ 1.169	@ 2.280

Classification and Properties (BS 5950: 2000)

Section (25.09 kg/m)	203x133 UB 25 [S 355]		
Class = Fn(b/T,d/t,py,F,Mx,My)	8.54, 30.25, 355, 0, 21.32, 1.57	(Axial: Non-Slender)	Compact
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	20.655 / 246.705 =	0.084	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 257.7≤1.2 x 355 x 230.42 =	91.484 kN.m	
Fvy/Pvy	1.352 / 398.337 =	0.003	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 70.9≤1.2 x 355 x 46.32 =	19.732 kN.m	
Ae = Fn(Ag,A.net,py,Us)	31.96,31.96,355,470	31.96 cm^2	
Pz = Ae.py	31.96x355	1134.58 kN	
n = F/Pz	-21.712 / 1134.58 =	0.019	OK
Srx = Fn(Sxx, n)	257.7, 0.019	257.54 cm^3	
Mrx = Srx.py	257.54 x 355	91.425 kN.m	
Sry = Fn(Syy, n)	70.9, 0.019	70.9 cm^3	
Mry = Sry.py	70.9 x 355	19.732 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(21.318/91.425)^2+(1.573/19.732)^1=$	0.134	OK

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}$	0.2+(.15x9+.5x11+.15x0)/21 ≥ 0.44	0.518	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x1+.6x1+.1x0)/2 ≥ .8x1/2	0.582	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x9+.6x11+.1x0)/21 ≥ .8x12/21	0.542	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x1+.6x1+.1x0)/2 ≥ .8x1/2	0.58	Table 26

Lateral Buckling Check Mb

Le = 1.00 L	1 x 4.992 =	4.992 m	
λ = Le/ryy	4.992 / 3.11	160.51	OK
v = Fn (x,Le,ryy,λ)	25.404, 4.992, 3.11, 160.51	0.76	Table 19
$\lambda_{LT}= u.v.\lambda.\sqrt{\beta_W}$	0.878 x 0.76 x 160.51 $\sqrt{1}$	107.1	
pb = Fn (py,λ_{LT})	355, 107.1	124.87 N/mm^2	Table 16
Mb = Sxx.pb ≤ Mc	257.7 x 124.87 ≤ 91.484 =	32.179 kN.m	

Simplified Approach

py.Zx	355x230.42	81.799 kN.m

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py.Zy	355x46.32	16.444 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	0+0.542x21.3/81.8+0.582x-1.6/16.4	0.197	OK
F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy	0+0.518x-21.3/32.2+0.582x-1.6/16.4	0.399	OK

More Exact Approach

Max=Mcx/(1+.5F/Pcx)	91.5/(1+.5x0/965.1)	91.484 kN.m	
May=Mcy/(1+F/Pcy)	19.7/(1+0/220.2)	19.732 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	0/965.1+0.542x21.3/91.5+.5x0.58x1.6/19.7	0.150	OK
F/Pcy+m_{LT}.M_{LT}/Mb+my.My/May	0/220.2+0.518x-21.3/32.2+0.582x-1.6/19.7	0.390	OK
Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	91.5(1-0/965.1)/(1+.5x0/965.1)	91.484 kN.m	
May=Mcy(1-F/Pcy)/(1+F/Pcy)	19.7(1-0/220.2)/(1+0/220.2)	19.732 kN.m	
m.Mx/Max+m.My/May	0.542x21.318/91.484+0.582x1.573/19.732	0.167	OK

Deflection Check - Load Case 32

In-span $\delta \leq$ Span/360	$4.03 \leq 4992 / 360$	4.03 mm	OK

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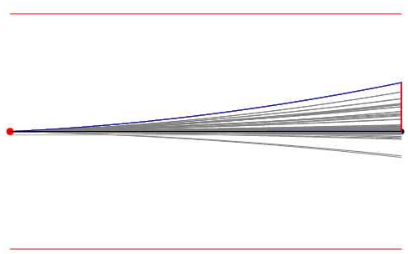
Axial with Moments (Member)
70x70 SHS canopy cantilever
Member SB 2-1\AL5Id 181 @ Level 5 in Load Case 3

Member Loading and Member Forces

Loading Combination : 1 UT + 1.4 D0 + 1.4 D1 + 1.6 L1

```
D1 UDLY -001.263          ( kN/m )
L1 UDLY -000.947          ( kN/m )
D1 D     077.010          ( kN/m³)
```



Member Forces in Load Case 3 and Maximum Deflection from Load Case 16										
Member No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
218	35 52	0.02C 0.02C	0.20 -0.20	-1.06 -4.57	-0.02 -0.02	0.00 -2.94	0.00 -0.02			0.87 @ 0.626

Classification and Properties (BS 5950: 2000)

Section (6.24 kg/m)	70x70x3 SHS 6.24 [S 355]		
Class = Fn(b/t,d/t,py,F,Mx,My)	20.33, 20.33, 355, 0.021, 2.938, 0.019	(Axial: Non-Slender)	Plastic
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	4.573 / 84.561 =	0.054	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 19.87≤1.2 x 355 x 16.86 =	7.054 kN.m	
Fvy/Pvy	0.018 / 84.561 =	0	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 19.87≤1.2 x 355 x 16.86 =	7.054 kN.m	
Pz = Ag.py	7.94 x 355 =	281.87 kN	
n = F/Pz	0.021 / 281.87 =	0.000	OK
Srx = Fn(Sxx, n)	19.87, 0	19.87 cm^3	
Mrx = Srx.py	19.87 x 355	7.054 kN.m	
Sry = Fn(Syy, n)	19.87, 0	19.87 cm^3	
Mry = Sry.py	19.87 x 355	7.054 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(2.938/7.054)^{1.667}+(0.019/7.054)^{1.667}=$	0.232	OK

Compression Resistance Pc

λx = Lex/rxx	100x1x1.043/2.73 =	38.2	OK
Pcx = Area.pcx	7.94x334.616/10 =	265.685 kN	Table 24 a
λy = Ley/ryy	100x1x1.043/2.73 =	38.2	OK
Pcy = Area.pcy	7.94x334.62/10 =	265.685 kN	Table 24 a

Equivalent Uniform Moment Factors mLT, mx, my and myx

m_{LT}	Cantilever	1	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.632	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x-1+.1x-2)/3 ≥ .8x2/3	0.506	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x0+.6x0+.1x0)/0 ≥ .8x0/0	0.632	Table 26

Lateral Buckling Check Mb

Mb = Mc	Section not susceptible to lateral torsional buckling	7.054 kN.m	

Simplified Approach

py.Zx	355x16.86	5.985 kN.m	
py.Zy	355x16.86	5.985 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	0.021/265.685+0.506x2.9/6+0.632x0/6	0.251	OK
$F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy$	0.021/265.685+1x-2.9/7.1+0.632x0/6	0.419	OK

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More Exact Approach

Max=Mcx/(1+.5F/Pcx)	7.1/(1+.5x0/265.7)	7.054 kN.m	
May=Mcy/(1+.5F/Pcy)	7.1/(1+.5x0/265.7)	7.054 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	0/265.7+0.506x2.9/7.1+.5x0.632x0/7.1	0.212	OK
F/Pcy+.5m_{LT}.M_{LT}/Mcx+my.My/May	0/265.7+.51x-2.9/7.1+0.632x0/7.1	0.210	OK
Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	7.1(1-0/265.7)/(1+.5x0/265.7)	7.053 kN.m	
May=Mcy(1-F/Pcy)/(1+.5F/Pcy)	7.1(1-0/265.7)/(1+.5x0/265.7)	7.053 kN.m	
m.Mx/Max+m.My/May	0.506x2.938/7.053+0.632x0.019/7.053	0.418	OK

Deflection Check - Load Case 2

Deflection Limits (Cantilevers)	Cantilever $\delta \leq$ 1043/180 = 5.8 mm Live (Case 11)	2.12 mm	OK
	Cantilever $\delta \leq$ 1043/125 = 8.3 mm D+L (Case 2)	5.05 mm	OK

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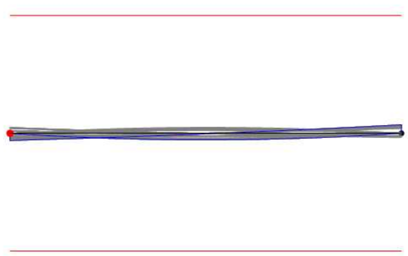
Axial with Moments (Member)

Member SB 1\A-FL9Id 331 @ Level 9 in Load Case 67 : 152x152 UC30 steel frame

Member Loading and Member Forces

Loading Combination : 1 UT + 1 D0 + 1 D1 + 1.4 S2

```
D1 D     077.010           ( kN/m³)
S2 UDLX +001.314           ( Co/m )
```



Member Forces in Load Case 67 and Maximum Deflection from Load Case 72										
Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN)		Bending Moment (kN.m)		Maximum Moment (kN.m @ m)		Maximum Deflection (mm @ m)
				x-x	y-y	x-x	y-y	x-x	y-y	
373	265 274	0.58T 0.58T	0.00 0.00	-1.61 -3.08	3.93 -3.99	5.36 -6.33	0.04 -0.11		4.91 @ 2.446	7.90 @ 2.496

Classification and Properties (BS 5950: 2000)

Section (30.03 kg/m)	152x152 UC 30 [S 355]		
Class = Fn(b/T,d/t,py,F,Mx,My)	8.13, 19.02, 355, 0, 6.33, 4.91	(Axial: Non-Slender)	Compact
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	2.341 / 218.197 =	0.011	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 247.7≤1.2 x 355 x 221.94 =	87.934 kN.m	
Fvy/Pvy	0.03 / 551.045 =	0	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 111.6≤1.2 x 355 x 73.43 =	31.281 kN.m	
Ae = Fn(Ag,A.net,py,Us)	38.26,38.26,355,470	38.26 cm^2	
Pz = Ae.py	38.26x355	1358.23 kN	
n = F/Pz	-0.582 / 1358.23 =	0.000	OK
Srx = Fn(Sxx, n)	247.7, 0	247.7 cm^3	
Mrx = Srx.py	247.7 x 355	87.933 kN.m	
Sry = Fn(Syy, n)	111.6, 0	111.6 cm^3	
Mry = Sry.py	111.6 x 355	31.281 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(0.432/87.933)^2+(4.909/31.281)^1=$	0.157	OK

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}$	0.2+(.15x3+.5x0+.15x3)/6 ≥ 0.44	0.44	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x4+.6x5+.1x4)/5 ≥ .8x5/5	0.95	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x3+.6x0+.1x-3)/6 ≥ .8x3/6	0.395	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x4+.6x5+.1x4)/5 ≥ .8x5/5	0.95	Table 26

Lateral Buckling Check Mb

Le = 1.00 L	1 x 4.992 =	4.992 m	
λ = Le/ryy	4.992 / 3.83	130.34	OK
v = Fn (x,Le,ryy,λ)	15.946, 4.992, 3.83, 130.34	0.693	Table 19
$\lambda_{LT}= u.v.\lambda.\sqrt{\beta_W}$	0.849 x 0.693 x 130.34 $\sqrt{1}$	76.67	
pb = Fn (py,λ_{LT})	355, 76.67	199.9 N/mm^2	Table 16
Mb = Sxx.pb ≤ Mc	247.7 x 199.9 ≤ 87.934 =	49.516 kN.m	

Simplified Approach

py.Zx	355x221.94	78.789 kN.m	
py.Zy	355x73.43	26.068 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	0+0.395x6.3/78.8+0.95x4.8/26.1	0.207	OK
$F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy$	0+0.44x1.3/49.5+0.95x4.8/26.1	0.187	OK

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More Exact Approach

Max=Mcx/(1+.5F/Pcx)	87.9/(1+.5x0/880.8)	87.934 kN.m	
May=Mcy/(1+F/Pcy)	31.3/(1+0/353.9)	31.281 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	0/880.8+0.395x6.3/87.9+.5x0.95x4.9/31.3	0.103	OK
F/Pcy+m_{LT}.M_{LT}/Mb+my.My/May	0/353.9+0.44x1.3/49.5+0.95x4.8/31.3	0.157	OK
Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	87.9(1-0/880.8)/(1+.5x0/880.8)	87.934 kN.m	
May=Mcy(1-F/Pcy)/(1+F/Pcy)	31.3(1-0/353.9)/(1+0/353.9)	31.281 kN.m	
m.Mx/Max+m.My/May	0.395x1.31/87.934+0.95x4.804/31.281	0.152	OK

Deflection Check - Load Case 72

In-span $\delta \leq$ Span/360	$7.9 \leq 4992 / 360$	7.9 mm	OK

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Approved	:

Axial with Moments (Member)

Member SC E\3L2Id 354 @ Level 2 in Load Case 35 : 200x200 SHS main frame col

Member Loading and Member Forces

Loading Combination : 1 UT + 1.2 D0 + 1.2 D1 + 1.2 L1 + 1.2 S2

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D1 D     077.010              ( kN/m³)
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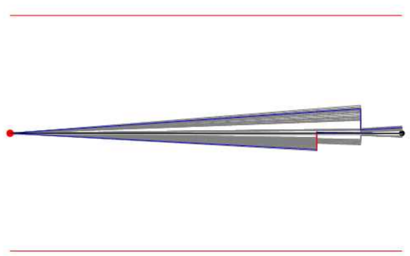

Member Forces in Load Case 35 and Maximum Deflection from Load Case 68

Mem ber No.	Node End1 End2	Axial Force (kN)	Torque Moment (kN.m)	Shear Force (kN) x-x	Shear Force (kN) y-y	Bending Moment (kN.m) x-x	Bending Moment (kN.m) y-y	Maximum Moment (kN.m @ m) x-x	Maximum Moment (kN.m @ m) y-y	Maximum Deflection (mm @ m)
	26	13.62C	2.03	13.76	-14.35	0.00	0.00	30.34	-36.31	1.84
	160	51.02T	0.00	1.92	-10.15	-0.01	0.00	@ 2.205	@ 2.541	@ 1.581

Additional Nominal Moments

MyUp	-10.529 kN.m

Classification and Properties (BS 5950: 2000)

Section (69.6 kg/m)	200x200x12 SHS 69.6 [S 355]		
Class = Fn(b/t,d/t,py,F,Mx,My)	13.67, 13.67, 355, 13.615, 30.646, 45.776	(Axial: Non-Slender)	Plastic
Auto Design Load Cases	1, 3-10, 29, 31, 33, 35, 37, 39, 41, 43, 45, 47, 49, 51, 53, 55, 57, 59, 61, 63, 65, 67, 69, 71, 73, 75, 77, 79, 81, 83, 85, 87, 89 & 91		

Local Capacity Check

Fvx/Pvx	13.761 / 944.549 =	0.015	Low Shear
Mcx = py.Sxx≤1.2 py.Zxx	355 x 620.91≤1.2 x 355 x 517.09 =	220.28 kN.m	
Fvy/Pvy	14.35 / 944.549 =	0.015	Low Shear
Mcy = py.Syy≤1.2 py.Zyy	355 x 620.91≤1.2 x 355 x 517.09 =	220.28 kN.m	
Ae = Fn(Ag,A.net,py,Us)	88.69,88.69,355,470	88.69 cm^2	
Pz = Ae.py	88.69x355	3148.495 kN	
n = F/Pz	-51.019 / 3148.495 =	0.016	OK
Srx = Fn(Sxx, n)	620.91, 0.016	620.69 cm^3	
Mrx = Srx.py	620.69 x 355	220.28 kN.m	
Sry = Fn(Syy, n)	620.91, 0.016	620.69 cm^3	
Mry = Sry.py	620.69 x 355	220.28 kN.m	
$(Mx/Mrx)^{Z1}+(My/Mry)^{Z2}$	$(30.646/220.28)^{1.667}+(40.202/220.28)^{1.667}=$	0.096	OK

Compression Resistance Pc

λx = Lex/rxx	100x1x2.844/7.64 =	37.2	OK
Pcx = Area.pcx	88.69x335.709/10 =	2977.398 kN	Table 24 a
λy = Ley/ryy	100x1x2.844/7.64 =	37.2	OK
Pcy = Area.pcy	88.69x335.71/10 =	2977.398 kN	Table 24 a

Equivalent Uniform Moment Factors mLT, mx, my and myx

$m_{LT}=0.2+(.15M_2+.5M_3+.15M_4)/M_{max}$	0.2+(.15x10+.5x20+.15x29)/31 ≥ 0.44	0.711	Table 18
$m_y=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x-13+.6x-26+.1x-39)/39 ≥ .8x39/39	0.8	Table 26
$m_x=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x10+.6x20+.1x29)/31 ≥ .8x29/31	0.766	Table 26
$m_{yx}=0.2+(.1M_2+.6M_3+.1M_4)/M_{max}$	0.2+(.1x-13+.6x-26+.1x-39)/46 ≥ .8x39/46	0.673	Table 26

Lateral Buckling Check Mb

Mb = Mc	Section not susceptible to lateral torsional buckling	220.28 kN.m

Simplified Approach

py.Zx	355x517.09	183.567 kN.m

ings engineering limited

Cloud 552e5

spithurst hub spithurst road
barcombe east sussex BN8 5EE
01273 470 066 info@ingsengineering.co.uk

py.Zy	355x517.09	183.567 kN.m	
F/Pc+mx.Mx/py.Zx+my.My/py.Zy	13.615/2977.398+0.766x30.6/183.6+0.8x-40.2/183.6	0.308	OK
F/Pcy+m_{LT}.M_{LT}/Mb+my.My/py.Zy	13.615/2977.398+0.711x30.6/220.3+0.8x-40.2/183.6	0.279	OK

More Exact Approach

Max=Mcx/(1+.5F/Pcx)	220.3/(1+.5x13.6/2977.4)	219.778 kN.m	
May=Mcy/(1+.5F/Pcy)	220.3/(1+.5x13.6/2977.4)	219.778 kN.m	
F/Pcx+mx.Mx/Max+.5myx.My/Mcy	13.6/2977.4+0.766x30.6/219.8+.5x0.673x45.8/220.3	0.181	OK
F/Pcy+.5m_{LT}.M_{LT}/Mcx+my.My/May	13.6/2977.4+.50.711x30.6/220.3+0.8x-40.2/219.8	0.200	OK
Max=Mcx(1-F/Pcx)/(1+.5F/Pcx)	220.3(1-13.6/2977.4)/(1+.5x13.6/2977.4)	218.773 kN.m	
May=Mcy(1-F/Pcy)/(1+.5F/Pcy)	220.3(1-13.6/2977.4)/(1+.5x13.6/2977.4)	218.773 kN.m	
m.Mx/Max+m.My/May	0.766x30.646/218.773+0.8x40.202/218.773	0.247	OK

Deflection Check - Load Case 68

In-span $\delta \leq$ Span/360	$1.84 \leq 2844 / 360$	1.84 mm	OK