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STRUCTURAL CALCULATIONS

Pilsley Sports Pavilion

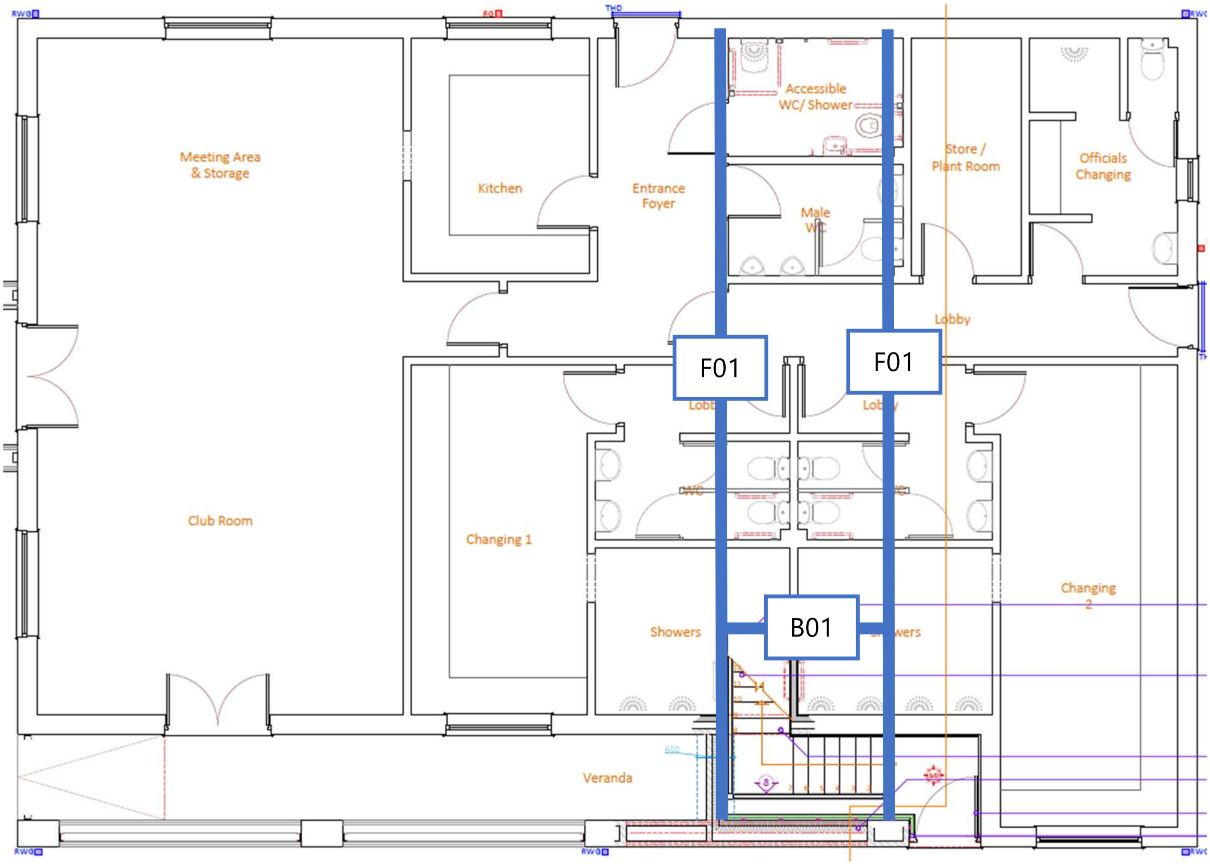
for

Pilsley Parish Council

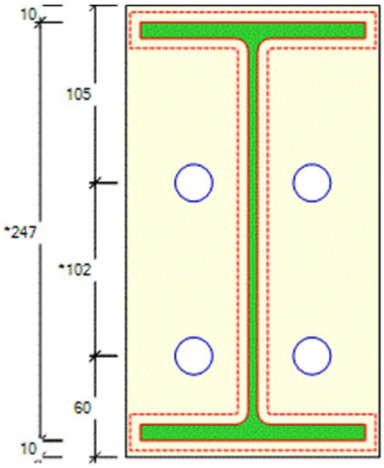
Reference: **230540-G2K-CAL-001**

Revision History				
Version	**Description**	**By**	**Chk'd**	**Date**
A01	First issue	DK	-	28/06/23

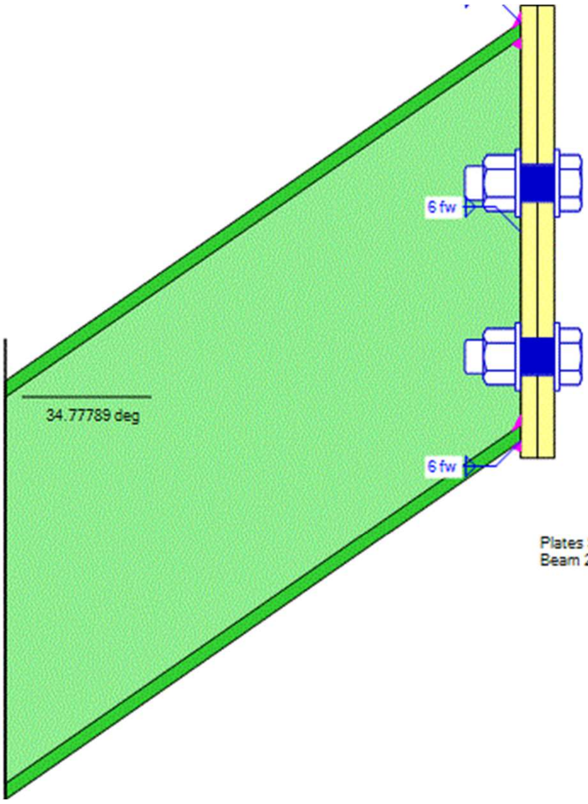
Summary



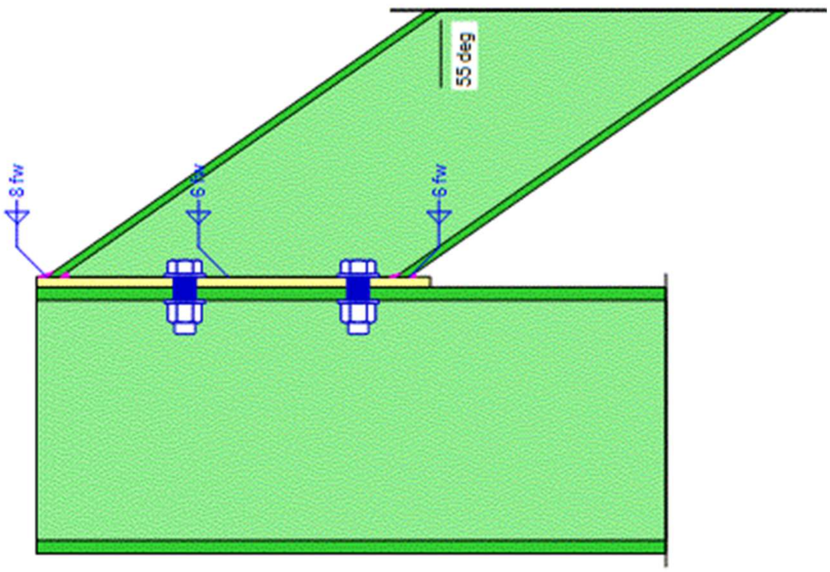
	Size	Bearing/connection details
B01	178x102x19UB	Beam to connect to bottom chords of frames with 10mm thick endplates, 6mm fillet welds and 4No. M16, 8.8 bolts at each end.
F01	Rafters: 203x133x25UBs Bottom chord: 254x146x43UB	Steel to steel connections shown below. Bottom chord to bear 100mm onto 440mm long x 100mm wide x 215mm deep padstones at walls.

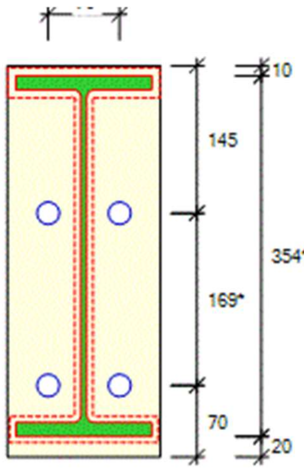


End-Plate 267 x 150 x 10 mm (3 kg)
4 No. M20 Grade 8.8 Bolts in 22 mm holes



Plates S 275
Beam 203x133 UB 25 [S275]





Plates S 275
Beam 203x133 UB 25 [S275]
Column 254x146 UB 43 [S275]
Top 10 above top flange

End-Plate 384 x 150 x 10 mm (5 kg)
4 No. M20 Grade 8.8 Bolts in 22 mm holes

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Notes

1. These calculations are based on Green2k drawing numbers:
    - 1875 201
    - 1875 202
    - 1875 203
    - 1875 204

2. All dimensions shown within these calculations are for design purposes only and are to be checked on site before ordering materials.

3. All details are subject to approval by all concerned parties which should be obtained before construction commences.

4. Any assumptions made by the designer have been highlighted within these calculations. These should be checked before ordering materials. If any findings differ from these assumptions, we are to be informed so that our design can be re-assessed.

5. In case of any doubts regarding these calculations please feel free to contact us.

Scope of Works

We have been asked to provide structural engineering services to design a steel beam and girder trusses to form an opening within an attic trussed roof.

The existing attic trusses are spaced at 600mm centres and so one of the trusses will be required to be removed to fit in the proposed staircase. In order to maintain stability of the roof it is proposed to install two steel girder trusses so that the loading on the existing trusses remains unchanged.

It is assumed that the existing trusses have been designed to accommodate the proposed new loadings. This should be checked with a specialist truss manufacturer.

These calculations have been undertaken using the following design standards:

- BS EN 1990 – Basis of Structural Design
- BS EN 1991 – Actions on Structures
- BS EN 1993 – Design of Steel Structures
- BS EN 1996 – Design of Masonry Structures

Designer's Risk Assessment

Checklist of potential operations and hazards		
Potential hazards	**Present**	**Key significant hazards to be addressed**
Client operations		
Adjacent activities		
Restricted site		
Traffic		
Interface with public		
Near to highways		
Near to railways		
Near to waterways		
Tidal working		
Ground instability		No ground investigation data available.
Contamination		
Soil gas		
Ground water		
Inundation		
Sewage		
Fuel tanks		
Services		
Overhead cables		
Demolition		
Unstable structures		
Explosives		
Asbestos		
Bird droppings		
Dust		
Hazardous materials		
Radiation		
Hot working		
Confined spaces		
Working at height		
Manual handling		
Lifting operations		
Vibration		
Noise		
Other (state)		

Hazard	**Action by Designer**	**Residual Hazard**
Unknown ground conditions	To eliminate: Cannot be eliminated	
	To reduce: A ground investigation would provide information.	

Structural Design

General Loading

SECTION		OUTPUT
	Loadings - in accordance with BS EN 1991-1-1:2002 and NA	

SECTION				OUTPUT
Annex A	Roof - sloped	Pitch = 35 °		
	Dead		g_k	
	Tiles	Concrete tiles	0.68	
	Battens/felt	50x25mm C16 timbers @ 200mm c/c	0.04	
	Rafters etc.	50x150mm C24 timbers @ 400mm c/c	0.15	
	Ceiing joists etc.	50x100mm C24 timbers @ 400mm c/c	0.06	g_k =
	Ceiling/insulation	12mm plasterboard and skim	0.22	1.15 kN/m^2
	Variable		q_k	
	Roof	Table NA7	0.50	q_k =
	Ceiling	Cl. 6.3.4.2 Note 8	0.00	0.50 kN/m^2
	Floor - timber			
	Dead		g_k	
	Boards	22mm chipboard	0.18	
	Joists etc.	50x250mm C24 timbers @ 400mm c/c	0.20	g_k =
	Ceiling	12mm plasterboard and skim	0.18	0.56 kN/m^2
	Variable		q_k	
	Live	Table NA2 Sub-category C13	3.00	q_k =
	Partitions	Cl. 6.3.1.2 Note 8	0.5	3.50 kN/m^2

Beam – B01

SECTION		OUTPUT
B01	Steel beam design - in accordance with BS EN 1993	
	L = 1.8 m γg = 1.35 γq = 1.50	

	Dead	Vari.	LW		G_k	Q_k
Roof				m		
Flat roof				m		
Floor	0.56	3.0	6.5	m	6.55	35.10
Walls				m		
SW	0.19	kN/m		kN	6.9	35.1

F = 62 kN

M = 62.0 x 1.8 / 8 = 13.9 kNm

SCI P360 Ch. 3.3 — Effective length k = 1.2L+2h = 2.5 m

C_1 = 1.13

Tata 'Blue Book' & Table 7 BS EN 10025-2

Try 178x102x19UB

$M_{b,Rd}$	23.3 kNm	(5.0m)							
A_v	9.85 cm^2	t_w	4.8 mm	f_y	275 N/mm^2	I_y	1360 cm^4		
						E	210000 N/mm^2		

M is less than $M_{b,Rd}$ — OK

V_{Ed} = 62.0 / 2 = 31 kN

$V_{pl,Rd}$ = 0.577 x 275 x 9.9 x 10^2 = 156295 N — = 156 kN

V_{Ed} is less than $V_{pl,Rd}$ — OK

$$wG_k = \frac{5 \times 6.9 \times 10^3 \times 1800^3}{384 \times 210 \times 10^3 \times 1360 \times 10^4}$$ — 0.2 mm

$$wG_k = \frac{5 \times 35.1 \times 10^3 \times 1800^3}{384 \times 210 \times 10^3 \times 1360 \times 10^4}$$ — 0.9 mm

Total deflection — 1.1 mm

Cl. NA.2.23 — Deflection limit = span/360 = 5.0 mm 1.1 < 5.0 — OK

Therefore, provide 178x102x19UB

Frame – F01

Frames are to be positioned at approximately 1.8m apart from each other.

Therefore dead UDL on rafters = (0.9+0.4) x $0.92_{(roof)}$ = 1.2kN/m (sls)

And variable UDL on rafters = (0.9+0.4) x $0.5_{(roof)}$ = 0.65kN/m (sls)

Attic trusses run parallel with proposed frames – Beam B01 applies point loads to the bottom chords

Dead PL = 3.45kN (sls)

Variable PL = 17.55kN (sls)

MASTERFRAME DATA FILE

LOADING CASES AND LOAD COMBINATION

Load Group Labels

Load Group UT	Unity Load Factor (All Cases)
Load Group D1	Dead Load
Load Group L1	Live Load

Load Case 001 : 1.35 Dead + 1.5 Variable

Load Combination	+ 1.00 UT + 1.35 D1 + 1.50 L1

Load Case 002 : 1.0 Dead + 1.0 Variable

Load Combination	+ 1.00 UT + 1.00 D1 + 1.00 L1

THE NODAL CO-ORDINATES

Node	X (m)	Y (m)	Z (m)	Node	X (m)	Y (m)	Z (m)
1	0.000	0.000	0.000	2	13.000	0.000	0.000
3	6.500	4.514	0.000				

MEMBER PROPERTIES

Member 1

```
M  ...  ...          254x146 UB 43 [S 275]
 A 54.77E-4           Iy 6545E-8          Iz 678.3E-8          It 23.88E-8
 E 210.0E6            G 80.77E6

  Members 2-3
M  ...  ...          203x133 UB 25 [S 275]
 A 31.96E-4           Iy 2341E-8          Iz 308.5E-8          It 5.96E-8
 E 210.0E6            G 80.77E6
```

MEMBER LOADING

Member Self Weight Density Load Included in Load Group D1, defined by Modulus of Elasticity

E kN/mm²	Density kN/m³
>= 200.00	77.01
>= 20.00	24.00
>= 2.00	10.00

Member 1

```
D1 PY   -003.450 2.000       ( kN,m )
L1 PY   -017.550 2.000       ( kN,m )
```

Members 1-3 - MasterFrame Pro Loads

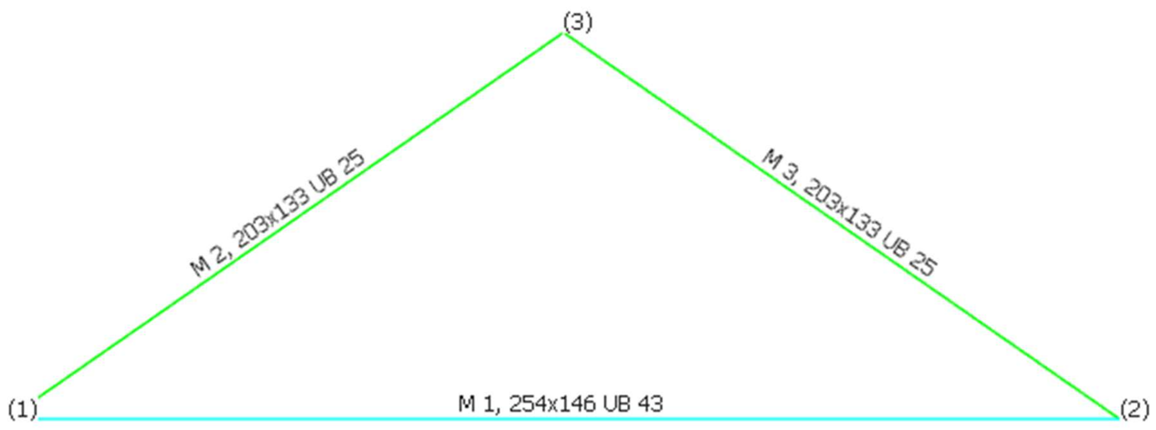
```
D1 D      077.010              ( kN/m³)

 Member 2
D1 UDLY -001.200              ( kN/m )
L1 UDLY -000.650              ( kN/m )
```

NODAL LOADING AND SUPPORT CONDITIONS

NODES 1-2

```
UT Rs 1 1 1 0 0 0 (Pinned)
```

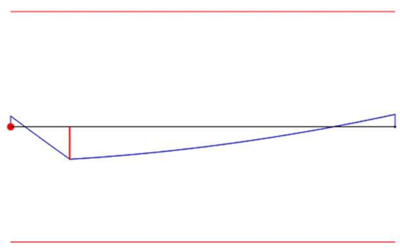


AXIAL WITH MOMENTS (MEMBER)
Member SBL1Id 1 @ Level 1 in Load Case 1

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 D     077.010          ( kN/m³)
D1 PY   -003.450 2.000    ( kN,m )
L1 PY   -017.550 2.000    ( kN,m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2						
Member No.	Node End1 End2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
1	1	0.000C	29.736	-14.510	43.824	26.930
	2	0.000C	-8.648	-16.859	@ 2.000	@ 5.439

Classification and Effective Area (EN 1993: 2006)

Section (42.99 kg/m)	254x146 UB 43 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	5.8, 30.42, 275, 0, 43.82, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Shear Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	29.736 / 320.728 =	0.093	OK

Moment Capacity Check M.c.y.Rd

$V_{y.Ed}/V_{pl.y.Rd}$	28.598 / 320.728 =	0.089	Low
Shear			
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 566.3/1	155.733 kN.m	
$M_{y.Ed}/M_{c.y.Rd}$	43.819 / 155.733 =	0.281	OK

Equivalent Uniform Moment Factor C1

C_1= fn(M_1, M_2, M_o, ψ,μ)	-14.5, -16.9, 55.5, 0.859, -3.291	1.220	Uniform

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 13 =	13 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.220, 13.000, 678.3, 23.88, 0.1031, 210000	50.455 kN.m	
λ_{LT}= $\sqrt{W.f_y/M_{cr}}$	$\sqrt{566.3 \times 275 / 50.455}$	1.757	
χ_{LT}= Fn(λ_{LT}, ϕ_{LT}, β, λ_{LT0})	1.757, 1.888, 0.750, 0.400	0.333	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.333, 1.757, 0.905, 1.000	0.324	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.324 x 566.3 x 275 ≤ 155.733 =	50.455 kN.m	
$M_{y.Ed}/M_{b.Rd}$	43.819 / 50.455	0.868	OK

Deflection Check - Load Case 2

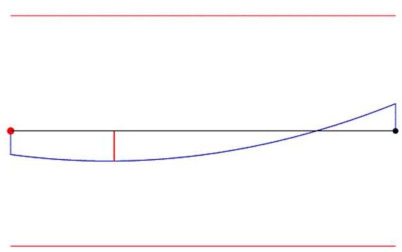
In-span δ ≤ Span/360	26.93 ≤ 13000 / 360	26.93 mm	OK

AXIAL WITH MOMENTS (MEMBER)
Member SBL1Id 3 @ Level 1 in Load Case 1

Member Loading and Member Forces

```
Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 D      077.010          ( kN/m³)
D1 UDLY -001.200           ( kN/m )
L1 UDLY -000.650           ( kN/m )
```



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2						
Mem ber No.	Node End1 End2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
2	1	21.267C	4.055	14.510	18.571	13.935
	3	10.146C	-11.960	-16.769	@ 1.978	@ 3.324

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = $Fn(b/T,d/t,f_y,N,M_y,M_z)$	8.54, 30.25, 275, 21.27, 18.56, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Shear Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	11.973 / 203.402 =	0.059	OK

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	0.257 / 203.402 =	0.001	Low
Shear			
$M_{c.y.Rd} = f_y.W_{pl.y}/\ \gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$N_{pl.Rd} = A_g.f_y/\ \gamma_{M0}$	31.96 x 275/1 =	878.9 kN	
$n = N_{Ed}/N_{pl.Rd}$	21.267 / 878.9 =	0.024	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	257.7, 12.811, 0.024	257.7 cm^3	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\ \gamma_{M0}$	257.7 x 275/1	70.868 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(18.545/70.868)^2+(0)^1=$	0.068	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x7.914 =	7.914	
$\lambda y = \sqrt{A.fy/Ncr}$	$\sqrt{31.96x275/774.8}$	1.065	
$N_{b.y.Rd}$ = Area.$\chi.f_y/\ \gamma_{M1}$	31.96x0.62x275/10/1 =	545.129 kN	Curve a
Lez = Kz.Lz	1x7.914 =	7.914	
$\lambda z = \sqrt{A.fy/Ncrz}$	$\sqrt{31.96x275/102.1}$	2.931	
$N_{b.z.Rd}$ = Area.$\chi.f_y/\ \gamma_{M1}$	31.96x0.104x275/10/1 =	91.294 kN	Curve b
Let = Kt.Lx	1x7.914 =	7.914	
$\lambda T = \sqrt{A.fy/NcrT}$	$\sqrt{31.96x275/697.78}$	1.122	
$N_{b.T.Rd}$= Area.χ.fy/ γ_{M1}	31.96x0.522x275/10/1 =	458.806 kN	Curve b

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ψ,μ)	14.5, -16.8, 15.8, -0.866, -0.944	1.263	Uniform
C_{mLT}=Max(0.1(1-ψ) -0.8α_s, 0.4)	M_h= -16.76, M_s= 14.7, ψ = -0.866, α_s= -0.877	0.888	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
C_{my}=Max(0.1(1-ψ) -0.8α_s, 0.4)	M_h= -16.77, M_s= 14.7, ψ = -0.865, α_s= -0.876	0.888	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 7.914 =	7.914 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	1.263, 7.914, 308.5, 5.964, 0.02933, 210000	30.708 kN.m	
λ_{LT}= $\sqrt{}$ W.f_y/M_{cr}	$\sqrt{}$ 257.7 x 275 / 30.708	1.519	
χ_{LT}= Fn(λ_{LT}, ϕ_{LT}, β, λ_{LT0})	1.519, 1.556, 0.750, 0.400	0.419	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.419, 1.519, 0.890, 1.000	0.419	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \leq M_{c.y.Rd}$	0.419 x 257.7 x 275 $\leq$ 70.868 =	29.702 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	21.267 / 545.129	0.039	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	21.267 / 91.294	0.233	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	18.545 / 29.702	0.624	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 19.498	0.000	OK
$k_{yy}=C_{my}\{1+0.8U_{N.y}\}$		0.915	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		1.326	
$k_{yz}=0.6\ k_{zz}$		0.796	
$k_{zy}= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		0.893	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.039+0.915x0.624+0.796x0.000	0.611	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.233+0.893x0.624+1.326x0.000	0.791	OK

Deflection Check - Load Case 2

In-span $\delta \leq$ Span/360	13.93 $\leq$ 7914 / 360	13.93 mm	OK

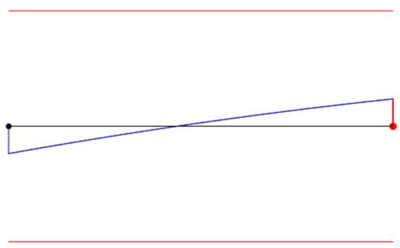
AXIAL WITH MOMENTS (MEMBER)

Member SBL1Id 4 @ Level 1 in Load Case 1

Member Loading and Member Forces

Loading Combination : 1 UT + 1.35 D1 + 1.5 L1

D1 D 077.010 (kN/m³)



Member Forces in Load Case 1 and Maximum Deflection from Load Case 2						
Mem ber No.	Node End1 End2	Axial Force (kN)	Shear Force (kN)	Bending Moment (kN.m)	Maximum Moment (kN.m @ m)	Maximum Deflection (mm @ m)
3	2	16.250C	3.169	-16.859		3.825
	3	14.750C	5.329	16.769		@ 2.137

Classification and Effective Area (EN 1993: 2006)

Section (25.09 kg/m)	203x133 UB 25 [S 275]		
Class = Fn(b/T,d/t,f_y,N,M_y,M_z)	8.54, 30.25, 275, 16.25, 16.86, 0	(Axial: Non-Slender)	Class 1
Auto Design Load Cases	1		

Shear Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	5.329 / 203.402 =	0.026	OK

Local Capacity Check

$V_{y.Ed}/V_{pl.y.Rd}$	3.169 / 203.402 =	0.016	Low
Shear			
$M_{c.y.Rd} = f_y.W_{pl.y}/\gamma_{M0}$	275 x 257.7/1	70.868 kN.m	
$N_{pl.Rd} = A_g.f_y/\gamma_{M0}$	31.96 x 275/1 =	878.9 kN	
$n = N_{Ed}/N_{pl.Rd}$	16.25 / 878.9 =	0.018	OK
$W_{pl.N.y} = Fn(W_{pl.y}, A_{vy}, n)$	257.7, 12.811, 0.018	257.7 cm³	
$M_{N.y.Rd} = W_{pl.N.y}.f_y/\gamma_{M0}$	257.7 x 275/1	70.868 kN.m	
$(M_{y.Ed}/M_{N.y.Rd})+(M_{z.Ed}/M_{N.z.Rd})$	$(16.859/70.868)^2+(0)^1=$	0.057	OK

Compression Resistance N.b.Rd

Ley = Ky.Ly	1x7.914 =	7.914	
λy = √A.fy/Ncr	√31.96x275/774.8	1.065	
$N_{b.y.Rd} = Area.\chi.f_y/\gamma_{M1}$	31.96x0.62x275/10/1 =	545.129 kN	Curve a
Lez = Kz.Lz	1x7.914 =	7.914	
λz = √A.fy/Ncrz	√31.96x275/102.1	2.931	
$N_{b.z.Rd} = Area.\chi.f_y/\gamma_{M1}$	31.96x0.104x275/10/1 =	91.294 kN	Curve b
Let = Kt.Lx	1x7.914 =	7.914	
λT = √A.fy/NcrT	√31.96x275/697.78	1.122	
$N_{b.T.Rd}= Area.\chi.fy/\gamma_{M1}$	31.96x0.522x275/10/1 =	458.806 kN	Curve b

Equivalent Uniform Moment Factors C1, C.mLT, C.mz, and C.my

C_1= fn(M_1, M_2, M_o, ψ,μ)	16.9, -16.8, 2.1, -0.995, 0.126	2.390	Uniform
C_{mLT}=Max(0.2+0.8α_s, 0.4)	M_h= 16.86, M_s= 2.17, ψ = -0.995, α_s= 0.129	0.4	Table B.3
C_{mz}=Max(0.6+0.4ψ, 0.4)	M = 0, ψ = 1.000	1	Table B.3
C_{my}=Max(0.2+0.8α_s, 0.4)	M_h= 16.86, M_s= 2.17, ψ = -0.995, α_s= 0.129	0.4	Table B.3

Lateral Buckling Check M.b.Rd

Le = 1.00 L	1 x 7.914 =	7.914 m	
M_{cr}= Fn(C_1,L_e,I_z,I_t,I_w,E)	2.390, 7.914, 308.5, 5.964, 0.02933, 210000	58.101 kN.m	
λ_{LT}= √ W.fy/M_{cr}	√ 257.7 x 275 / 58.101	1.104	
χ_{LT}= Fn(λ_{LT}, ϕ_{LT}, β, λ_{LT0})	1.104, 1.077, 0.750, 0.400	0.636	Curve b
χ_{LT}.mod = Fn(χ_{LT},λ_{LT},k_c,f)	0.636, 1.104, 0.647, 0.856	0.743	6.3.2.3
$M_{b.Rd} = \chi\ W_{pl.y}.f_y \le M_{c.y.Rd}$	0.743 x 257.7 x 275 ≤ 70.868 =	52.636 kN.m	

Buckling Resistance

$U_{N.y} = N_{Ed}/(\chi_y.N_{Rk}/\gamma_{M1})$	16.25 / 545.129	0.030	OK
$U_{N.z} = N_{Ed}/(\chi_z.N_{Rk}/\gamma_{M1})$	16.25 / 91.294	0.178	OK
$U_{M.y} = M_{y.Ed}/(\chi_{LT}.M_{y.Rk}/\gamma_{M1})$	16.859 / 52.636	0.320	OK
$U_{M.z} = M_{z.Ed}/(M_{z.Rk}/\gamma_{M1})$	0 / 19.498	0.000	OK
$k_{yy}=C_{my}\{1+0.8U_{N.y}\}$		0.410	
$k_{zz}=C_{mz}\{1+1.4U_{N.z}\}$		1.249	
$k_{yz}=0.6\ k_{zz}$		0.750	
$k_{zy}= 1- \{0.1\lambda_z/(C_{mLT}-0.25)\}U_{N.z}$		0.652	
$U_{Ny}+k_{yy}.U_{M.y}+k_{yz}.U_{M.z}$	0.030+0.410x0.320+0.750x0.000	0.161	OK
$U_{Nz}+k_{zy}.U_{M.y}+k_{zz}.U_{M.z}$	0.178+0.652x0.320+1.249x0.000	0.387	OK

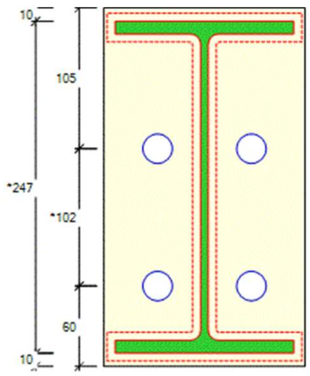
Deflection Check - Load Case 2

In-span $\delta \leq$ Span/360 $3.83 \leq 7914 / 360$ 3.83 mm OK

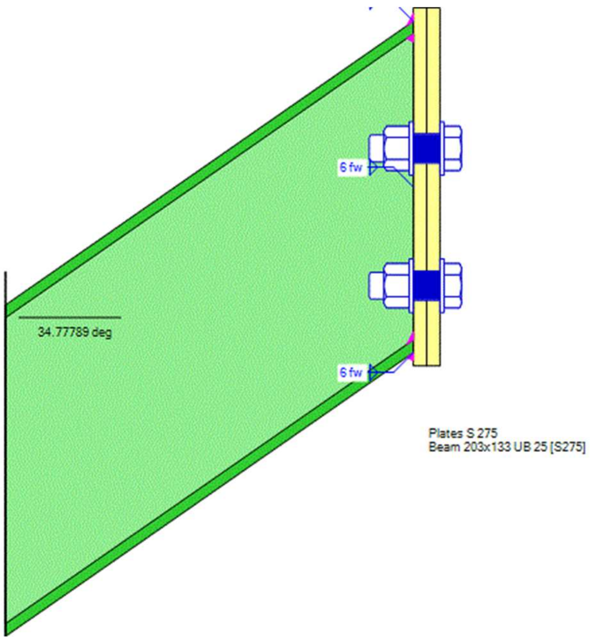
Support Reactions (Load Case 001 : 1.35 Dead + 1.5 Variable Ultimate)							
Node	Support Reactions (kN and kN.m)			Node	Support Reactions (kN and kN.m)		
	Rx ➔ (kN)	Ry ↑ (kN)	Mz ↗ (kN.m)		Rx ➔ (kN)	Ry ↑ (kN)	Mz ↗ (kN.m)
1		45.197	0.000	2		15.314	0.000
Total		60.511	0.000				

BRIEF 1

APEX JOINT AT : END 2 OF MEMBER SBL1ID 3 - LEVEL 1



End-Plate 267 x 150 x 10 mm (3 kg)
4 No. M20 Grade 8.8 Bolts in 22 mm holes



Beam to Beam End-Plated Connection to EC 3 (UK NAD)

LOADING CASE 001 : 1.35 DEAD + 1.5 VARIABLE

Basic Data

Integrated Applied Forces at End-plate Interface

Left Rafter Forces M, Fvr, Fr	16.8 kNm, 12.0 kN, 10.1 kN
Resultant Forces M, Fv, F	16.8 kNm, 4.0 kN, 15.2 kN
Load directions	Top of Connection in Tension, Rafter moving Down and in Compression.
Design to	EC 3:

Part 1-8: 2005 Design of Connections

Weld Grades	All weld grades provided to suit minimum connected steel grade

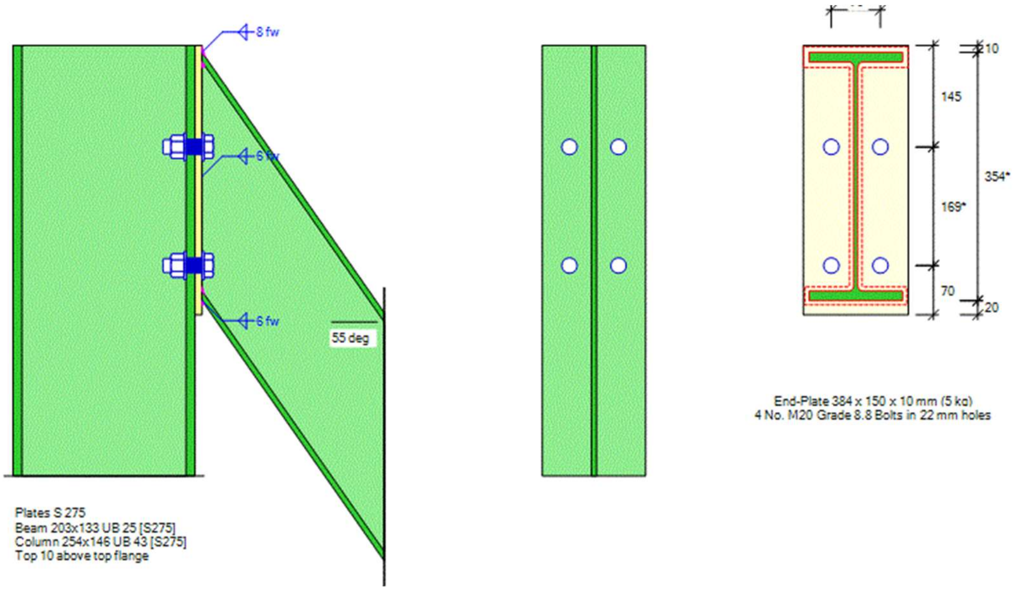
Basic Dimensions

Rafter-203x133UB25 [S 275]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	91.1 kN.m, 243.4 kN, 948.2 kN	Mc = 91.1 kN.m	OK

Summary of Results (Unity Ratios)

Moment Capacity 30.5 kNm (for 1 rows of bolts) (Modified Applied Mom. M_{mod}=15.0 kNm)		0.49	OK
Shear Capacity		0.02	OK
Flange Welds	0.35	0.35	OK
Web Welds	0.68, 0.03	0.68	OK

BRIEF 2



Beam to Column Flange End-Plated Connection to EC 3 (UK NAD)

LOADING CASE 001

Basic Data

User Defined Applied Forces at Column/Right Rafter Interface

Right Rafter Forces M, Fvr, Fr	17.9 kNm, 3.7 kN, 17.6 kN
Resultant Forces M, Fv, F	17.9 kNm, -12.3 kN, 13.1 kN
Load directions	Top of Connection in Tension, Rafter moving Up and in Compression.
Design to	EC 3: Part 1-8: 2005 Design of Connections
Weld Grades	All weld grades provided to suit minimum connected steel grade

Basic Dimensions

Column-254x146UB43 [S 275]	D=259.6, B=147.3, T=12.7, t=7.2, r=7.6, py=275		
Rafter-203x133UB25 [S 275]	D=203.2, B=133.2, T=7.8, t=5.7, r=7.6, py=275		
Bolts 20 mm Ø in 22 mm holes	Grade 8.8 Bolts		
Plates S 275	All weld grades provided to suit minimum connected steel grade		
Rafter Capacities Mc, Fvc, Fc	146.2 kN.m, 340.1 kN, 1115.7 kN	Mc = 146.2 kN.m	OK
Column Forces M, Fv, F	0.9 x 19 = 18 kN.m, 13 kN, 12 kN		
Column Capacities Mc, Fvc, Fc	155.7 kN.m, 320.7 kN, 1506.2 kN	Mc = 155.7 kN.m	OK

Summary of Results (Unity Ratios)

Moment Capacity 44.3 kNm (for 1 rows of bolts) (Modified Applied Mom. M_{mod}=15.6 kNm)		0.35	OK
Shear Capacity		0.05	OK
Flange Welds	0.19	0.19	OK
Web Welds	0.68, 0.05	0.68	OK