James Shipton B.Eng(Hons)

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Tel: 01725 512065

Date: 04/10/18		Job No: 3102
Client: Mr & Mrs A B Cee		Sheet: 1 / 9
Address: The Barn, Sometown, Borsetshire,		

Calculation

Output

<u>Job Description</u> Introduce larger dormer window so re-work calculations:

Basis for Design

Loading: Materials BS 648:1964
Imposed BS 6339:1996

Materials: Structural Use of Concrete BS 8110-1:1997
Structural Use of Steelwork BS 5950-1:2000
Structural Use of Masonry BS 5628-1:1992
Structural Use of Timber BS 5268-2:2002

LOADING: See previous calculations sheets JN1315.

Beam SF-2

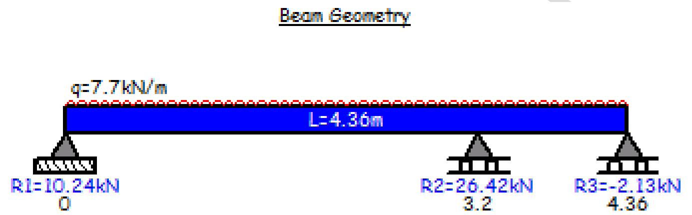
Loading as per Beam SF-1

See Structural Engineer's drawing for fabrication details.

Design Values

Clear Span = 4.36m

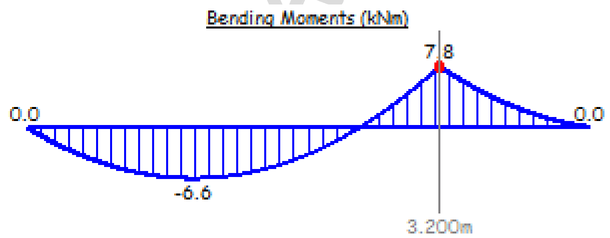
Beam Self Weight = 1.4 x 16 x 9.81/1000 = 0.22 kN/m



Maximum Bending Moment

M_{max} = -7.79 kNm @ 3.2m

M_{max}=7.8kNm



Try 152x89x16 UB

P_y = 275N/mm^2

$\varepsilon = (275/P_y)^{1/2} = 1$ (Table 11)

$b/T = 5.76 < 9\varepsilon$ (Table 11)

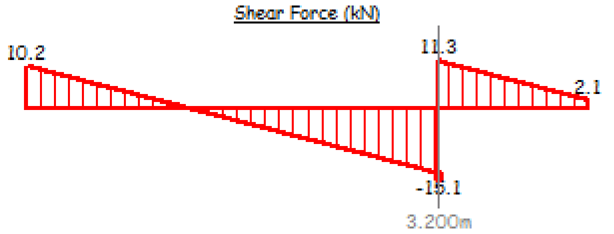
$d/t = 27.1 < 80\varepsilon$ (Table 11)

Calculation	Output

$d/t < 70\varepsilon$ Therefore no need to check for shear buckling. (cl 4.2.3)

Section is Class 1 Plastic

Shear Capacity Check (cl 4.2.3)



$A_v = tD = 4.5 \times 152.4 = 686mm^2$

$P_v = 0.6P_y A_v = 0.6 \times 275 \times 686 = 113kN$

Maximum Shear = 15.11kN at 3.2m

15.11kN < Pv = 113kN Shear Capacity ------> OK

For low shear R < 0.6Pv (cl4.2.5.2)

0.6 x 113=68kN > 15.1kN

OK IN SHEAR

Moment Capacity

$S_{xx} = 123cm^3$

$M_p = 123 \times 10^3 \times 275 = 33.83$ kNm

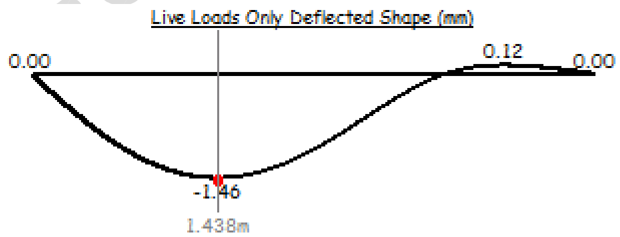
Zxx = $109cm^3$

$1.5P_yZ = 44.96kNm$

$M_{max} < M_p$ --------> OK

$M_p < 1.2P_yZ$ --------> OK

Deflection Check



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Calculation	Output
Use Unfactored Live Loads for Deflections.	
Allowable Deflection = 4360/360 = 12.1mm	
Max deflection occurs at x = 1.438m	
δ = 1.5mm	δ=1.5mm
Beam deflection ------> OK	

<u>Bearing Stress</u>

Mortar designation(iii), Bearing Type 1

Bearing Stress on Padstones:

σ_{allow} = 1.25 x 20 / 3.5 = 7.1 N/mm^2

σ_{R1} = 10.2 x 10^3/ (88.7 x 100) = 1.2 N/mm^2

σ_{R2} = 26.4 x 10^3/ (88.7 x 100) = 3 N/mm^2

σ_{R3} = -2.1 x 10^3/ (88.7 x 100) = -0.2 N/mm^2

Bearing Stress on Brickwork:

σ_{allow} = 1.25 x 3.5 / 3.5 = 1.25 N/mm^2

σ_{R1} = 10.2 x 10^3/ (100 x 300) = 0.34 N/mm^2 Padstone L=300mm

σ_{R2} = 26.4 x 10^3/ (100 x 300) = 0.88 N/mm^2 Padstone L=300mm

σ_{R3} = -2.1 x 10^3/ (100 x 300) = -0.07 N/mm^2 Padstone L=300mm

<u>LTB Check</u>

L_E = 5.5368m (Table 13)

r_y = 2.1cm (Section Tables)

$\lambda = L_e / r_y$ = 5.5368 / 0.021 = 263.7

υ = 0.89 (Section Tables)

x = 19.6 (Section Tables)

λ/x = 263.7/19.6 = 13.45

v = 0.56 (Table 19)

$\lambda_{LT} = \upsilon v \lambda \beta_w^{0.5}$ = 0.89x0.56x263.7 = 131.8 (β_w = 1 for Class 1 & 2 Sections.)

P_b = 82.4 N/mm^2 (Table 16)

Calculation	Output

$M_b = p_b S_{xx}$ = 82.4 x 123/1000 = 10.1 kNm

m_{LT} = 1 (Table 18)

M_b/m_{LT} =10.1/1 = 10.1 kNm> M_{max} = 7.8kNm

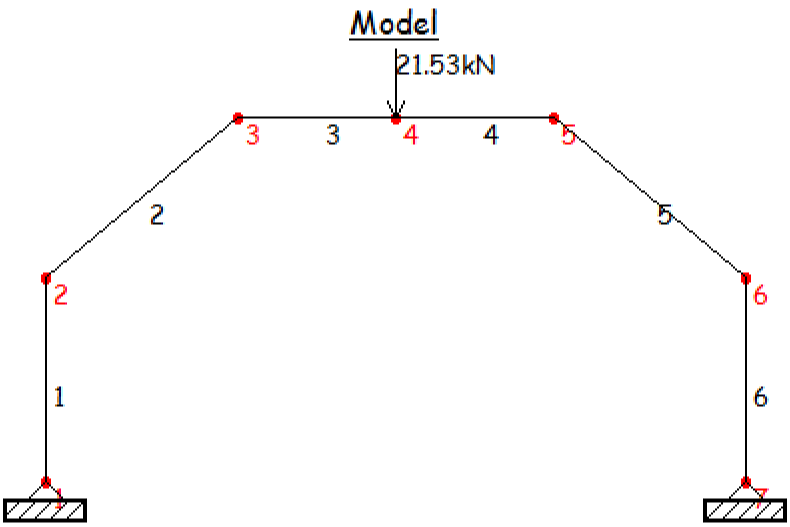
Therefore LTB OK

Therefore Provide a 152x89x16 UB for Beam SF-2

END OF CALCULATIONS

Ridge Beam Frames

```
PROJECT NAME  :- Roof Frame
FILE NAME  :- K:\CLIENTS\3102\Calcs\RoofFrame.epa
FRAME TYPE :- Rigid Jointed
```




```
DATA SUPPLIED
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NUMBER OF JOINTS  =     7
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NUMBER OF MEMBERS =     6
-----------------

JOINT COORDINATES
-----------------
```

JOINT	(X)	(Y)	R=Restrained (X)	(Y)	F=Free Rot	Support Settlements (X)	(Y)
1	0.00	0.00	R	R	F	*	*
2	0.00	1.23	F	F	F	*	*
3	1.15	2.19	F	F	F	*	*
4	2.11	2.19	F	F	F	*	*
5	3.06	2.19	F	F	F	*	*
6	4.21	1.23	F	F	F	*	*
7	4.21	0.00	R	R	F	*	*

```
MEMBER CONNECTIVITY
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```

MEMBER	END 1	END 2
1	1	2
2	2	3
3	3	4
4	4	5
5	5	6
6	6	7

```
MEMBER PROPERTIES
-----------------
```

MEMBER	E(kN/mm^2)	I(cm^4)	A(cm^2)	L(m)
1	205.00	834.00	20.30	1.230
2	205.00	834.00	20.30	1.498

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Calculation | Output

3	205.00	834.00	20.30	0.960
4	205.00	834.00	20.30	0.950
5	205.00	834.00	20.30	1.498
6	205.00	834.00	20.30	1.230

JOINT LOADS

JOINT NO	Q(kN)	DIRECTION	MOMENT(kNm)
4	21.53	270	0

MEMBER LOADS

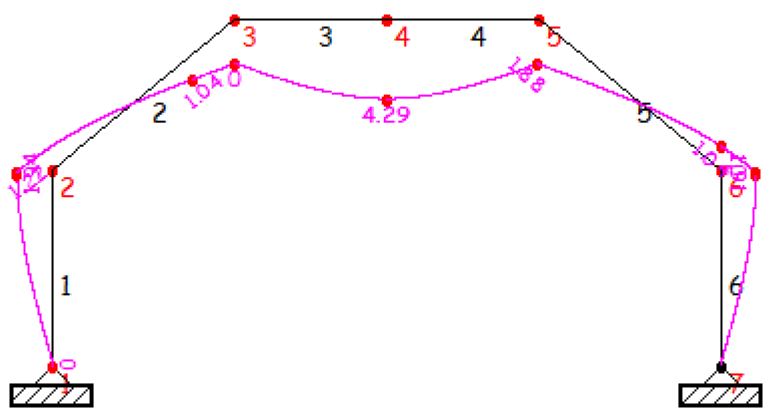
LOAD	MEMBER	LOAD TYPE	w1(kN or kN/m)	w2(kN or kN/m)	a(m)	b(m)

ANALYSIS RESULTS (Axial forces positive to the Right, Shears positive up and Moments are Anticlockwise Positive)

JOINT DISPLACEMENTS

JOINT	LX(mm)	LY(mm)	RZ(rad)
1	0.000	0.000	0.00239425
2	-1.936	-0.032	-0.00006562
3	-0.011	-2.400	-0.00298506
4	-0.024	-4.293	0.00000744
5	-0.037	-2.420	0.00298509
6	1.905	-0.032	0.00009138
7	0.000	0.000	-0.00236850

Deflected Shape (mm)



MEMBER DEFLECTIONS

Member 1

Min d = 0mm @ 0mm
Max d = 1.94mm @ 1213mm

Member 2

Min d = -1.04mm @ 1230mm
Max d = 1.22mm @ 0mm

Member 3

Min d = -4.29mm @ 959mm
Max d = 0mm @ 0mm

Member 4

Min d = -4.29mm @ 0mm
Max d = 0mm @ 0mm

Member 5

Min d = -1.88mm @ 0mm
Max d = 1.04mm @ 1230mm

Member 6

Min d = 0mm @ 0mm
Max d = 1.91mm @ 23mm

LOCAL MEMBER FORCES

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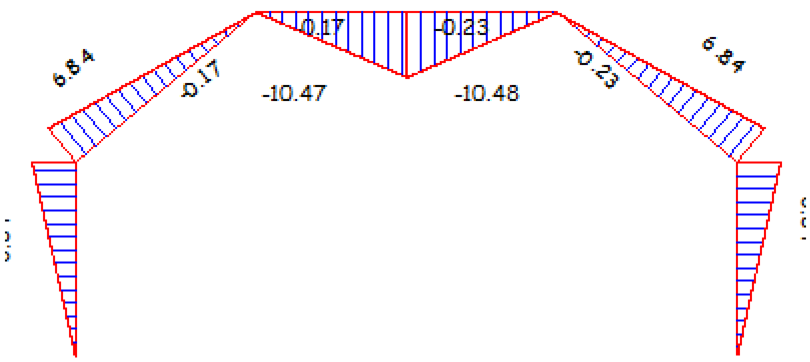
Calculation | Output

MEMBER / JOINT	PMX(kN)	PMY(kN)	CMZ(kNm)
1 / 1	10.739	-5.560	0.000
1 / 2	-10.739 Comp'	5.560	-6.838
2 / 2	11.150	4.681	6.838
2 / 3	-11.150 Comp'	-4.681	0.175
3 / 3	5.560	10.739	-0.175
3 / 4	-5.560 Comp'	-10.739	10.484
4 / 4	5.560	-10.791	-10.484
4 / 5	-5.560 Comp'	10.791	0.233
5 / 5	11.183	-4.721	-0.233
5 / 6	-11.183 Comp'	4.721	-6.838
6 / 6	10.791	5.560	6.838
6 / 7	-10.791 Comp'	-5.560	0.000

GLOBAL FORCES AND REACTIONS

JOINT	PX(kN)	PY(kN)	CZ(kNm)
1	5.560	10.739	0.000
2	0.000	0.000	0.000
3	0.000	0.000	0.000
4	0.000	-21.530	0.000
5	0.000	0.000	0.000
6	0.000	0.000	0.000
7	-5.560	10.791	0.000

Bending Moments (kNm)

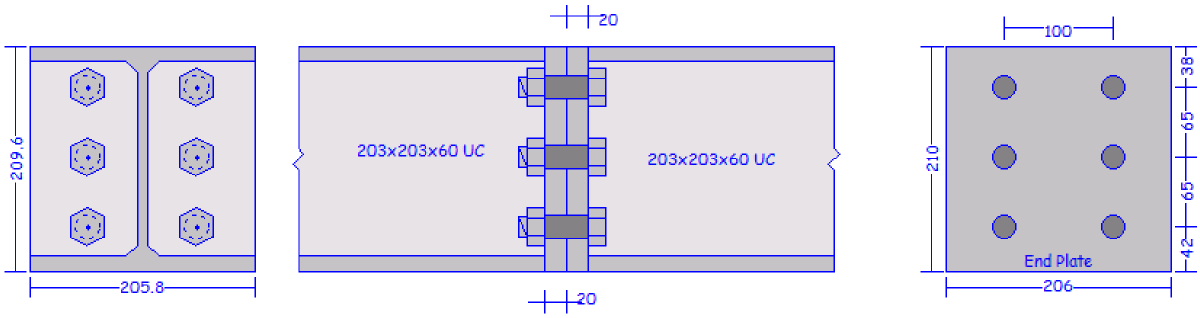


End of Output.

Beam Splice for Beam FF-3

End Plate Beam Splice

See Structural Engineer's drawing for fabrication details.



Basis of Design:

The following calculation is based on the SCI publication Joints in Steel

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Calculation	Output

Construction Moment Connections (1995). The compression zone is taken as acting at the centre of the top beam flange. The top bolt row acts in shear only. The bending moment at the splice is resisted by the other bolt rows.

Input Data

Section: 203x203x60 UC

Beam Length = 7.1m

Splice at 1m

Section Dimension: D = 209.6mm B = 205.8mm T = 14.2mm t = 9.4mm

End Plate 1 Dimensions: D = 210mm B = 206mm t = 20mm py = 265N/mm^2

End Plate 2 Dimensions: D = 0mm B = 206mm t = 20mm py = 265N/mm^2

Provide S265 End Plates 210 x 206 x 20mm

Provide 3 pairs of M20 8.8 bolts in ϕ22mm holes at 100 Horizontal Centres.

Distance to top bolt row from top of End Plate = 38mm

Distance to second bolt row from top of End Plate = 103mm

Pitch of remaining bolts = 65mm

Basic Detailing Checks

Tension Flange Weld

Tb/1.4 = 14.2/1.4 = 10.1mm (say 11mm)

Web Weld

tb/1.4 = 9.4/1.4 = 6.7mm (say 7mm)

Hole Diameter

D = d + 2 = 22mm

Epsilon

$\varepsilon = (275/P_y)^{0.5} = 1.02$

Minimum Bolt Spacing (6.2.1.1):

$s_{min} > 2.5d$

$s_{min} = 2.5x20 = 50$

e2-et = 103-38 = 65 > 50 --> OK

P = 65 > 50 --> OK

Maximum Bolt Spacing (6.2.1.2):

$s_{max} < 14tp$

$s_{max} = 14x20 = 280$

e2-et = 103-38 = 65 < 280 --> OK

P = 65 < 280 --> OK

Minimum End & Edge Distance (6.2.2.4):

$e_{min} > 1.25D = 1.25x22 = 27.5mm$

et = 38 > 25 --> OK
eb = 42 > 25 --> OK
e = 53 > 25 --> OK

Maximum End & Edge Distance (6.2.2.5):

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Calculation	Output

$e_{max} < 11t\varepsilon$

e_{max} = 11x20x1.02 = 224.11mm

et = 38 < 224.11 --> OK
eb = 42 < 224.11 --> OK
e = 53 < 224.11 --> OK

Horizontal bolt spacing should not exceed 55% of end plate width.

0.55B = 0.55x205.8 = 113.3mm

100 < 113.3 --> OK

--------------- All Dimensional Checks Satisfied ---------------

Bolt bearing capacity on the End Plate $P_{bb} = dt_p p_b$ = 20x20x460/1000 = 184kN

Bolt Shear Capacity $P_s = p_s A_s$ = 375x245/1000 = 91.9kN

P_{ss} = the lesser of P_{bb} and P_s

P_{ss} = 91.9kN

Enhanced bolt tension capacity $P_t' = p_t A_t$

P_t' = 560x245/1000 = 137.2kN

Maximum tensile length of beam web per bolt row:

L_t = (g/2)tan(60) = (100/2)tan(60) = 86.6mm above and below each bolt row.

N.B. the length below may not extend into the length above from the row below.

m, bolt centre to 20% into weld

$m_1 = (g/2) - (t/2) - 0.8s_w$ = 39.7mm

$m_2 = e_b - T - 0.8s_f$ = 19mm

$\lambda_1 = (m_1/(m_1+e))$ = 0.43mm

$\lambda_2 = (m_2/(m_1+e))$ = 0.2mm

α = 6.283 (P207 Appendix iii)

n, effective edge distance = min[edge distance = 53, 1.25m = 49.6] = 49.6mm

Effective Lengths

L_{eff}(i) = 2πm = 249.4mm

L_{eff}(ii) = 4m + 1.25e = 225mm

L_{eff}(iii) = αm = 6.283x39.7 = 249.4mm

L_{eff} for bolts acting alone (Table 2.5):

Bottom Row: L_{eff} = min{max(ii,iii),i} = 237.2mm

Other Rows: L_{eff} = min{ii,i} = 225mm

L_{eff} for bolts acting as a group (Table 2.6):

Bottom Row: L_{eff} = max{ii/2,iii-ii/2}+p/2 = 157.2mm

Intermediate Rows: L_{eff} = P = 65mm

Top Row: L_{eff} = ii/2+p/2} = 145mm

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Calculation | Output

$M_p = L_{eff} \times t^2 \times p_y/4$

Row Capacity

Mode 1 (End plate yielding): $P_r=4M_p/m$

Mode 2 (End plate yielding and bolts fail in tension): $P_r = (2M_p + n(\Sigma P_t'))/(m+n)$

Mode 3 (Bolt failure): $P_r = \Sigma P_t'$

Bolts rows are numbered from the bottom up (bottom pair = 1; top pair = 3)

Row	L_{eff}	M_p	Mode1	Mode2	Mode3	$L_{t,web}$	$P_{t,web}$	$P_{t,group}$	P_t(kN)	
1	237	6287	633	293	274	_	_	_	274.4	<<
2	225	5964	601	286	274	173	448	_	274.4	
[1+2]	302	8010	807	484	549	_	_	484	_	
[1+2]-1	_	_	_	_	_	_	_	_	209.8	<<

<< indicates the lesser value

Check Moment and Shear Capacity of Splice

Compressive Resistance of Compression (top) Flange:

$P_c = 1.4 x P_y x T_b x B_b = 1.4x275x14.2x205.8/1000 = 1125.1kN$

Bolt Line	Bolt LA(mm)	P_r(kN)	RM(kNm)	P_{ss} or P_{ts}	F_v(kN)
1	160	274.4	43.9	36.8	73.5
2	95	209.8	19.93	36.8	73.5
3	31	0	0	91.9	183.75
Totals		484.2	Mc=63.8		330.75

Beam section ultimate moment capacity $M_p = P_y \times S_{xx}$ = 275x656/1000 = 180.4kNm

Splice Moment Capacity M_c = 63.8kNm

Design Moment = 52.81kNm < Mc = 63.8kNm --> OK

Design Shear Force = 53kN < F_v = 330.75kN --> OK

Therefore Beam Splice as Designed is Satisfactory

END OF CALCULATIONS

Notes: Beam lengths used in these calculations are clear spans. The contractor should check, on site, required beam lengths before ordering.

All temporary works are the responsibility of the contractor. If in doubt you should seek the engineer's advice.

Brickwork and foundations below beam bearings should be checked to confirm ability to carry additional loading. If brickwork is inadequate, it should be rebuilt back a minimum of 450mm and down to ground level.

Building works should not commence until these calculation sheets have been approved by the local authority building control surveyor.