

Classical Design

Byways, Slab Lane, Downton, Salisbury, SP5 3PS
Tel: 01725 512065

Date: 18/03/08

Job No:1090

Client: Mr N Hindle

Sheet: 1 / 7

Address: 175 Tytherington, Warminster, BA12 7AD

Calculation

Output

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:

Pitched Roof:

Tiles	0.48
Felt and Battens	0.05
Rafters	0.10
Insulation	0.03
P/Board + skim	<u>0.15</u>
	0.81
45° slope	<u>1.15</u>
	x 1.4
	<u>1.60 kN/m²</u>

Wind:

$$v=40\text{m/s} \quad s_1 = s_3 = 1.0 \quad \text{GR} = 1 \quad H = 5.1\text{m}$$

$$\text{Building class A} \quad s_2 = 0.88$$

$$V_s = v s_1 s_2 s_3 = 40 \times 0.88 = 35.2\text{m/s}$$

$$q = k V_s^2 = 0.613 \times 37.6^2 = \quad \underline{0.76 \text{ kN/m}^2}$$

Live Load	0.6
	x <u>1.6</u>
	<u>0.96 kN/m²</u>

$$\underline{\text{TOTAL ROOF LOAD}} \quad \underline{3.32 \text{ kN/m}^2}$$

Existing roof structure provides no lateral restraint to the rafter ends at wall plate level.
Therefore introduce steel ridge beam.

Loading to proposed ridge beam:

$$1.93 \times 3.32 = 6.4 \text{ kN/m}$$

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Job No:1090

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Sheet: 2 / 7

Address: 175 Tytherington, Warminster, BA12 7AD

Calculation

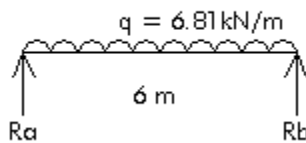
Output

Design Values

Clear Span = 6m

Beam Self Weight = 0.41 kN/m

Uniformly distributed load $q = 6.4 + 0.41 = 6.81$ kN/m



Moment Calculation

$$R_a = R_b = 6.81 \times 6/2 = 20.44 \text{ kN}$$

$$M_{\max} = 6.81 \times 6^2 / 8 = 30.65 \text{ kNm}$$

Try 203x133x30 UB

$$P_y = 275 \text{ N/mm}^2$$

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

$$b/T = 6.97 < 9\varepsilon \quad (\text{Table 11})$$

$$d/t = 26.94 < 80\varepsilon \quad (\text{Table 11})$$

Therefore section is Class 1 Plastic

Shear Check (cl 4.2.3)

$$A_v = tD = 6.4 \times 206.8 = 1324 \text{ mm}^2$$

$$P_v = 0.6P_y A_v = 0.6 \times 275 \times 1324 = 218 \text{ kN}$$

$$P_v > R \text{ -----} > \text{OK}$$

For low shear $R < 0.6P_v$ (cl4.2.5.2)

$$0.6 \times 218 = 131 > 20.4$$

Moment Capacity

$$S_{xx} = 314 \text{ cm}^3$$

$$I_{xx} = 2896 \text{ cm}^4$$

$$M_p = 314 \times 10^3 \times 275 = 86.35 \text{ kNm}$$

$q = 6.81$ kN/m

$R = 20.44$ kN

$M_{\max} = 30.7$ kNm

OK in Shear

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Sheet: 3 / 7

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

$$M_p < 1.2P_yZ \text{ -----} > \text{OK}$$

$$M_b = 33.3 \quad (\text{From section tables})$$

$$M_b > M_{\max} \text{ -----} \text{OK for LTB}$$

OK for LTB

Deflection Check

$$\text{Unfactored Imposed Load} = 4.27\text{kN}$$

$$\text{Allowable Deflection} = 6000/360 = 16.7\text{mm}$$

$$\delta = (5/384)(4.3 \times 10^3 \times 6^4)/(205 \times 10^9 \times 28.96 \times 10^{-6}) \times 1000 = 12.1\text{mm}$$

$$\delta_{\text{allow}} = 16.7\text{mm}$$

$$\delta = 12.1\text{mm}$$

$$\text{Beam deflection -----} > \text{OK}$$

Bearing Stress

Provide 200 long offcut of 100 x 65 P.S. Concrete Lintel Padstone.

Mortar designation(iii), Bearing Type 1

Stress on the padstone

$$\sigma_{\text{allow}} = 1.25 \times 20 / 3.5 = 7.1 \text{ N/mm}^2$$

$$\sigma_{\text{pad}} = 20.44 \times 10^3 / (133.9 \times 100) = 1.5 \text{ N/mm}^2$$

Padstone bearing stress ----- > OK

Stress on the brickwork

$$\sigma_{\text{allow}} = 1.25 \times 3.5 / 3.5 = 1.2 \text{ N/mm}^2$$

$$\sigma_{\text{brick}} = 20.44 \times 10^3 / (100 \times 200) = 1 \text{ N/mm}^2$$

Brickwork bearing stress ----- > OK

LTB CHECK

$$M_b = 33.3\text{kNm} \quad (\text{Section Tables})$$

$$M_b > M_{\max} = 30.7\text{kNm}$$

Therefore beam OK for LTB

Therefore Provide a 203x133x30 UB

END OF CALCULATION

Classical Design

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Job No:1090

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Sheet: 4 / 7

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Connection Design

Introduce beam cover plate splice at 1/3 span (2m).

$$M @ 2m = 27.26 \text{ kNm}$$

$$V @ 2m = 13.53 \text{ kN}$$

$$\text{Beam depth } D = 206.8\text{mm}$$

$$\text{Horizontal force in bottom flange } F = 27.26/0.2068 = 132 \text{ kN}$$

Check Beam Flange

$$\text{Strength of flange plate } P_t = P_y A_e > 132 \text{ kN} \quad (\text{cl } 4.6.1)$$

$$A_e = K_e A_{\text{net}}$$

$$\text{Assuming S275 steel and M20 bolts } K_e = 1.2 \quad (\text{cl } 3.4.3)$$

$$A_{\text{net}} = [133.9 - (2 \times 22)] \times 9.6 = 863 \text{ mm}^2$$

$$T < 16\text{mm therefore } P_y = 275 \text{ N/mm}^2$$

Strength of flange plate:

$$(275 \times 1.2 \times 863) / 10^3 = 285 \text{ kN} > 132 \text{ -----} > \text{OK}$$

Design Flange Cover Plates

$$\text{Minimum area required } > 132 \times 10^3 / 275 = 480 \text{ mm}^2$$

$$A_e = 1.2 A_{\text{net}} = 480$$

$$A_{\text{net}} = 400 \text{ mm}^2$$

Assume cover plate 130mm wide

$$A_{\text{net}} = [130 - (2 \times 22)] \times T_{\text{cover plate}} = 400$$

$$T_{\text{cover plate}} = 4.65 \text{ mm}$$

Provide 130 x 10mm thick cover plate.

Design of Flange Bolts.

Assuming M20 H.S.F.G bolts

$$\text{Slip resistance } P_{\text{SL}} = 1.1 K_s u P_0 \quad (\text{cl } 6.4.2)$$

$$P_{\text{SL}} = 1.1 \times 1 \times 0.5 \times 144 = 79.2$$

$$\text{Bearing resistance } P_{\text{bg}} = d t p_{\text{bs}} \quad (\text{cl } 6.4.4)$$

Classical Design

Byways, Slab Lane, Downton, Salisbury, SP5 3PS
Tel: 01725 512065

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Job No:1090

Client: Mr N Hindle

Sheet: 5 / 7

Address: 175 Tytherington, Warminster, BA12 7AD

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$$p_{bs} = 460 \text{ N/mm}^2 \quad (\text{Table 32})$$

$$P_{bg} = 1.5 \times 20 \times 10 \times 460/10^3 = 138 \text{ kN/bolt}$$

$$(F = 132\text{kN})$$

$$\text{Therefore minimum number of bolts required} \geq 132/79.2 = 1.6$$

Adopt 8 No M20 HSFG bolts

$$\text{Minimum spacing} \geq 2.5d \quad 2.5 \times 20 = 50\text{mm} \quad (\text{cl 6.2.1})$$

$$\text{Maximum spacing} \leq 14t \quad 14 \times 9.6 = 134\text{mm} \quad (\text{cl 6.2.1.2})$$

$$\text{Minimum edge distance} \geq 1.25D \quad 1.25 \times 22 = 27.5\text{mm} \quad (\text{cl 6.2.2.4})$$

$$\text{Maximum edge distance} \leq 11t \quad 11 \times 9.6 = 105.6 \quad (\text{cl 6.2.2.5})$$

For HSFG bolts, outer splice should not be thinner than the lesser of:

$$1) \text{ bolt diameter}/2 \quad 20/2 = 10\text{mm}$$

$$2) 10\text{mm}$$

Design of web cover plate

Try 240 wide x 160 deep by 10mm thick.

$$F_v = 13.53 \text{ kN} = 6.8 \text{ kN/plate}$$

$$P_v = 0.6P_y A_v \quad A_v = 0.9A \quad (\text{cl 4.2.3})$$

$$A_v = 0.9[(10 \times 160) - (3 \times 22)] = 1380\text{mm}^2$$

$$P_v = (0.6 \times 275 \times 1380)/10^3 = 227 > 6.8 \text{ -----} > \text{OK in shear.}$$

Web cover plates in bending

$$\text{Assuming } M_b = S_{xx} P_b$$

$$\lambda_{LT} = 2.8 \left(\frac{\beta_w L_E d}{t^2} \right)^{0.5} \quad (\text{cl B.2.7, 4.3.6.9})$$

$$\lambda_{LT} = (2.8 \times 1 \times 180 \times 120/10)^{0.5} = 78$$

$$P_b = 169 \text{ N/mm}^2 \quad (\text{Table 16})$$

$$S_{xx} = (10 \times 180^2/4) - (2[(10 \times 22 \times 50) + (10 \times 11 \times 5.5)]) \\ = 57.79 \times 10^3 \text{ mm}^3$$

$$M_b = 169 \times 10^{-6} \times 57.79 \times 10^3 = 9.8 \text{ kNm}$$

$$\text{Applied bending moment} = (13.53 \times 60)/10^3 = 0.81 \text{ kNm -----} > \text{OK}$$

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Sheet: 6 / 7

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Bolts in web cover plate

$$P_r = \sqrt{\left(\frac{P}{n}\right)^2 + \left(\frac{Pe}{2Z}\right)^2} = 4.66kN$$

Shear bolt capacity $P_s = p_s A_s$

$$P_s = 400 \times 314 = 125 \text{ kN}$$

(Table 30)

125 > 4.66 Therefore web bolts are adequate.

THEREFORE PROVIDE COVER PLATE TO TOP AND BOTTOM FLANGE AND PLATES TO BOTH SIDES OF WEB. ALL BOLTS M20 HSEFG WITH 22 ϕ HOLES.

Rafter Check

Ignoring purlins max unsupported span = 1.7m

Loading to rafter $0.4 \times 3.32 = 1.33 \text{ kN/m}$

47x97 C16 @ 400cc

$$Z = 73.7 \times 10^3 \text{ mm}^3$$

$$I = 3.57 \times 10^6 \text{ mm}^4$$

$$\sigma_{\text{allow}} = 5.3 \times 1.1 = 5.83 \text{ N/mm}^2$$

$$M = 5.83 \times 73.7 \times 10^3 / 10^6 = 0.4 \text{ kNm}$$

$$L_{\text{mzx}} (Z) = \sqrt{(8 \times 0.47 \times 10^3 / 1.33 \times 10^3)} = 1.7 \text{ m}$$

$$L_{\text{mzx}} (I) = (0.003 \times 384 EI / 5q)^{1/3} = 1.76 \text{ m}$$

Therefore rafter ok in bending.

Notes: Inner leaf brickwork above first floor window lintels should be rebuilt using minimum 25N bricks.

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

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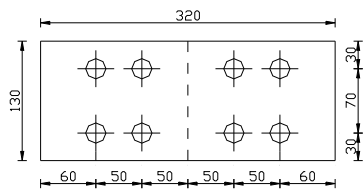
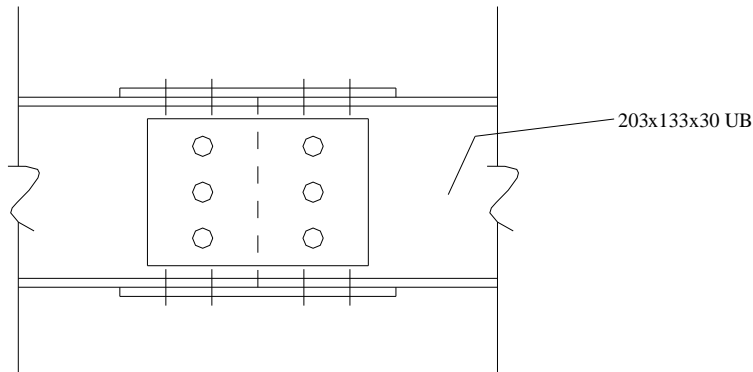
Client: Mr N Hindle

Sheet: 7 / 7

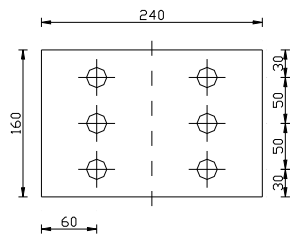
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Flange Cover Plate 130x320x10 mm



Web Cover Plate 240x180x10 mm

All bolts M20 HSF 22Ø Holes

