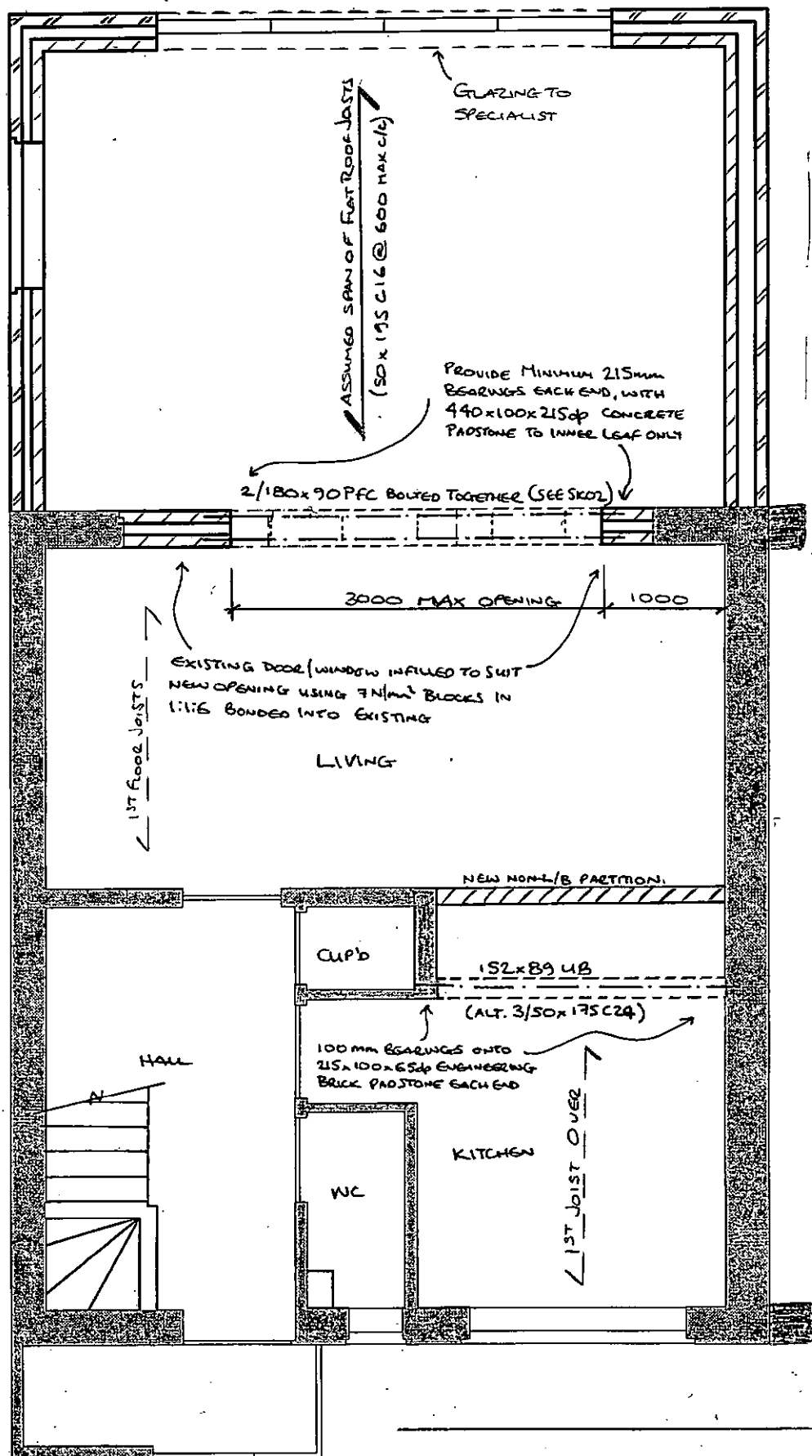


CALCULATIONS C.

	Date	Job No.	Sheet No. SK01	Rev.
Job 37 Pulborough Way				By

Ground floor
Plan (1:50)



	Date	Job No.	Sheet No.	Rev.
	FEB '09		01	

Job **37 PULBOROUGH WAY**

By

ELEMENT LOADING.

ROOF - PITCHED

$$\begin{aligned}
 \text{(DL) TILES} &= 0.76 \text{ kN/m}^2 \\
 \text{BATTENS + FELT} &= 0.11 \text{ " } \\
 \text{(40° PITCH) ON PLAN} &= 1.14 \text{ kN/m}^2 \\
 \text{TRUSSED RAFTERS} &= 0.35 \text{ " } \\
 \text{CEILING + INSULATION} &= 0.20 \text{ " } \\
 &= \underline{\underline{1.69 \text{ kN/m}^2}}
 \end{aligned}$$

$$\text{(IL) STORAGE + SNOW} = 0.25 \text{ kN/m}^2 + 0.60 \text{ kN/m}^2 = \underline{\underline{0.85 \text{ kN/m}^2}} \quad (\underline{\underline{3.73 \text{ kN/m}^2 \text{ WT}}})$$

1ST FLOOR

$$\begin{aligned}
 \text{(DL) BOARDING} &= 0.15 \text{ kN/m}^2 \\
 \text{JOISTS} &= 0.15 \text{ " } \\
 \text{CEILING} &= 0.20 \text{ " } \\
 &= \underline{\underline{0.50 \text{ kN/m}^2}}
 \end{aligned}$$

PARTITIONS (STUD) SAY 0.5 kN/m² GROSS

$$\text{(IL) DOMESTIC} = \underline{\underline{1.50 \text{ kN/m}^2}} \quad (\underline{\underline{3.8 \text{ kN/m}^2 \text{ WT}}})$$

CAVITY WALL

$$\begin{aligned}
 \text{(DL) BRICK} &= 2.20 \text{ kN/m}^2 \\
 \text{INSULATION} &= 0.05 \text{ " } \\
 \text{BLOCK} &= 2.00 \text{ " } \\
 \text{PLASTER} &= 0.15 \text{ " } \\
 &= \underline{\underline{4.40 \text{ kN/m}^2}}
 \end{aligned}$$

(6.16 kN/m² WT)

FLAT ROOF

$$\begin{aligned}
 \text{(DL) CHIPPINGS + FELT} &= 0.35 \text{ kN/m}^2 \\
 \text{BOARDS, JOISTS + FIERINGS} &= 0.30 \text{ " } \\
 \text{CEILING + INSULATION} &= 0.20 \text{ " } \\
 &= \underline{\underline{0.85 \text{ kN/m}^2}}
 \end{aligned}$$

$$\text{(IL) SNOW} = \underline{\underline{0.60 \text{ kN/m}^2}} \quad (\underline{\underline{2.15 \text{ kN/m}^2 \text{ WT}}})$$

	Date Feb '09	Job No.	Sheet No. 02	Rev.
Job 37 PULBOROUGH WAY				By

WALL LOADING AT GRAND FLOOR DOOR HEAD LEVEL

INNER LEAF LOAD/m = PITCHED ROOF + WALL + 1st FLOOR + S/Wt Beam

$$(DL) (1.69 \text{ kN/m}^2 \times 7.5 \text{ m}/2) + (2.2 \text{ kN/m}^2 \times 2.7 \text{ m}) + (1.0 \text{ kN/m}^2 \times 3 \text{ m}/2) + \text{S/Wt } 0.4 \text{ kN/m} = \underline{14.2 \text{ kN/m}}$$

$$(IL) (0.85 \text{ kN/m}^2 \times 7.5 \text{ m}/2) + 0 + (1.5 \text{ kN/m}^2 \times 3 \text{ m}/2) + 0 = \underline{5.5 \text{ kN/m}} \quad (28.7 \text{ kN/m wt})$$

OUTER LEAF LOAD/m = FLAT ROOF + WALL (BRICK) + S/Wt Beam

$$(DL) (0.85 \text{ kN/m}^2 \times 4 \text{ m}/2) + (2.2 \text{ kN/m}^2 \times 2.8 \text{ m}) + \text{S/Wt } 0.5 \text{ kN/m} = \underline{8.4 \text{ kN/m}}$$

$$(IL) (0.60 \text{ kN/m}^2 \times 4 \text{ m}/2) + 0 + 0 = \underline{1.2 \text{ kN/m}} \quad (13.7 \text{ kN/m wt})$$

	Date	Job No.	Sheet No. 03	Rev.
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Job 37 RULBOROUGH WAY

By

750 PIER TO PARTY WALL - ASSUMING DENSE BLOCKWORK (MIN. 7N/m²)

$$\text{INNER LEAF REACTION} = 28.7 \text{ kN/m} \times 3.5/2 = 50.3 \text{ kN wt.}$$

$$E_h = 2600 \text{ mm (ASSUMES SIMPLE RESISTANCE)}$$

$$E_t = (100 + 102)^{2/3} = 134.6 \text{ mm}$$

$$SR = 2600 / 134.6 = 19.3 \quad \beta = 0.66 \quad (\text{ELL} = 0.1 \text{ t})$$

ASSUMING 7.0N BLOCKS (DENSE BLOCK LOAD TAKEN)

$$\text{TRY 750 Ig PIER} \quad SPA = 0.7 + (0.1 \times 0.75 \times 1.5) = 0.812$$

$$\text{PIER CAPACITY} = \frac{0.66 \times 0.812 \times 100 \times 750 \times 6.4}{3.5} = 73.5 \text{ kN wt.}$$

$$\text{TOTAL LOAD @ PIER} = 50.3 \text{ kN BEAM REACTION} + (28.7 \text{ kN/m} \times 0.75 \text{ m}) = 71.9 \text{ kN wt.}$$

∴ OK.

CONSERVATIVE SINCE:-

- 1) SIMPLE RESISTANCE @ FLOORS
- 2) ASSUMES NO SPREAD INTO PARTY WALL
- 3) γ_m OF 3.5

PROVIDE MINIMUM 750 Ig REVEAL AT PARTY WALL

SEE SHEET 04 FOR CRITICAL REVEAL CALC./SIZE

	Date	Job No.	Sheet No. 04	Rev.
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Job 37 PULBOROUGH WAY

By

REVEAL TO PARTY WALL - WITH LIGHTWEIGHT BLOCKS

$$\text{INNER LEAF LOAD/m} = \overset{\text{DENSE BLOCK}}{(2.2 \text{ kN/m}^2 \times 2.7 \text{ m})} + \overset{\text{L/W BLOCKS}}{(1.2 \text{ kN/m}^2 \times 2.7 \text{ m})} = 11.5 \text{ kN/m (DL)}$$

$$(IL) = 5.5 \text{ kN/m} \quad (WT) 24.9 \text{ kN/m}$$

$$\text{Beam Reaction} = 24.9 \text{ kN/m}^2 \times 3.5 \text{ m} / 2 = \underline{43.6 \text{ kN WT}}$$

TRY 7501g PIER - ASSUMING 3.5 N/mm^2

$$\text{PIER CAPACITY} = \frac{0.66 \times 0.812 \times 100 \times 750 \times 3.5}{3.1} = \underline{45.4 \text{ kN WT}}$$

FAILS WITH 2W/m OVER $\therefore$ INCREASE

1000mm LONG REVEAL CAPACITY

$$\text{SPA} = 0.7 + (1.5 \times 0.1 \times 1.0) = 0.85$$

$$\text{PIER CAPACITY} = \frac{0.66 \times 0.85 \times 100 \times 1000 \times 3.5}{3.1} = \underline{63.3 \text{ kN WT}}$$

$$\therefore \text{MAX LENGTH OF WALL OVER} = 63.3 \text{ kN} / 24.9 \text{ kN/m} = \underline{2.54 \text{ m}}$$

$$2.54 \text{ m TOTAL} - 1.0 \text{ m BEARING} = 1.54 \text{ m CLEAR} \times 2 = \underline{3.0 \text{ m CLEAR}}$$

$\therefore$ LIGHTWEIGHT BLOCKS CRITICAL

PROVIDE MIN. 1.0m REVEAL TO PARTY WALL & MAX. 3.0m OPENING

Site:

Job: 37 PULBOROUGH WAY

Job number:

ProSteel 5.31b 532023

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Page 05

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Beam: EXISTING REAR WALL - INNER LEAF

Span: 3.2 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U D	Roof + Wall + 1st + slt	14.2	0		L	22.72	22.72
U L	Roof + Wall + 1st + slt	5.5	0		L	8.80	8.80
Unfactored reactions (kN) Total:						31.52	31.52
Dead:						22.72	22.72
Live:						8.80	8.80
						45.89	45.89

Total load: 63.04/91.78 kN Unfactored/Factored

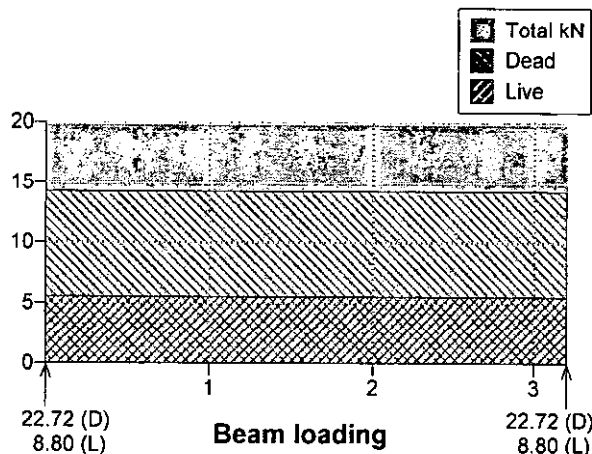
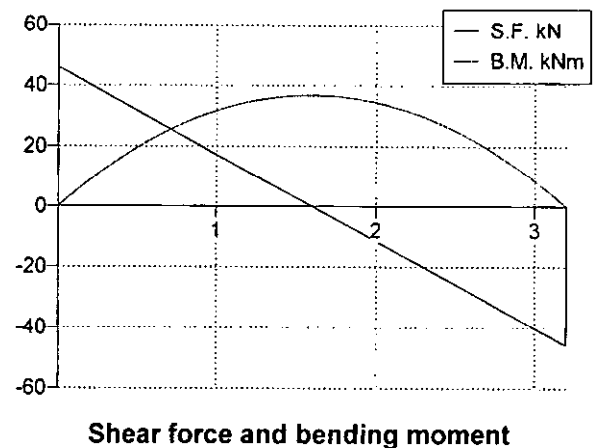
Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 36.7 kNm at 1.60 m. from R1

Maximum S.F. (factored) = 45.9 kN at R1

Live load deflection = $7.51 \times 10^8 / EI$ at 1.60 m. from R1 (E in N/mm^2 , I in cm^4)

Total deflection = $26.9 \times 10^8 / EI$ at 1.60 m. from R1



Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : 180 x 90 x 26 PFC S275

D=180.0 mm B=90.0 mm t=6.5 mm T=12.5 mm $I_x=1,820 \text{ cm}^4$ $r_y=2.89 \text{ cm}$ $S_x=232 \text{ cm}^3$

Section classification: T = 12.5mm $p_y = 275 \text{ N/mm}^2$ $\epsilon = 1$

(Table 11)

Flange: $b/T = 90.0/12.5 = 7.20$ ($\leq 9\epsilon$: Class 1 plastic)

Web: $d/t = 131.0/6.5 = 20.2$ ($\leq 40\epsilon$: Class 1 plastic)

For design purposes section classification is Class 2, compact

Shear capacity = $0.6 p_y t D = 0.6 \times 275 \times 6.5 \times 180.0/1000 = 193 \text{ kN}$ (> 45.9) OK

Maximum moment = 36.71 kNm at 1.60 m. from R1

Moment capacity, $M_c = p_y S_x = 275 \times 232/1000 = 63.80 \text{ kNm}$ OK

Beam is laterally restrained at supports only: effective length = $1.2L + 2D$

Effective length (L_E) = 4.20m

Slenderness, λ (L_E/r_y) = $4.20 \times 100/2.89 = 145.3$

Buckling parameter (u) = 0.950

Slenderness factor (v) = 0.605 ($x = 12.8$; $\lambda/x = 11.4$)

$\beta_w = 1.000$ (Class 1/2 compact)

Equivalent slenderness (λ_{LT}) = $u.v.\lambda/\beta_w = 83.58$

Bending strength, $p_b = 156.9 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 36.71 \text{ kNm}$

Equivalent uniform moment factor, $m_{1T} = 0.925$ ($M_2=27.5$, $M_3=36.7$, $M_4=27.5$)

Equivalent uniform moment = $0.925 \times 36.71 = 33.96 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 156.9 \times 232/1000 = 36.40 \text{ kNm}$ OK

Web buckling and crushing have not been checked

LL deflection = $7.508 \times 10^8 / 205,000 \times 1820 = 2.0 \text{ mm}$ ($L/1590$) OK

TL deflection = $26.89 \times 10^8 / 205,000 \times 1820 = 7.2 \text{ mm}$ ($L/444$)

Site:

Job: 37 PULBOROUGH WAY

Job number:

ProSteel 5.31b 532023

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Page 06

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Beam: EXISTING REAR WALL - OUTER LEAF

Span: 3.2 m.

Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U D Roof + Wall + Swt	8.4	0		L	13.44	13.44
U L Roof + Wall + Swt	1.2	0		L	1.92	1.92
Unfactored reactions (kN) Total:					15.36	15.36
Dead:					13.44	13.44
Live:					1.92	1.92
					21.89	21.89

Total load: 30.72/43.78 kN Unfactored/Factored

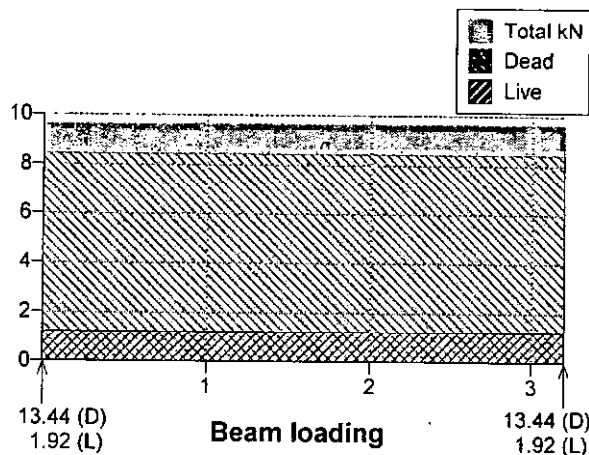
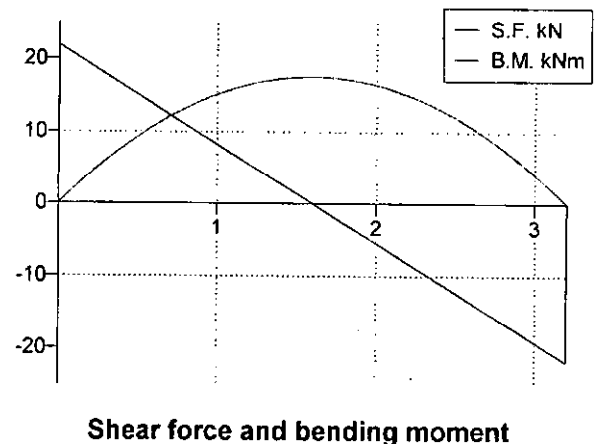
Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 17.5 kNm at 1.60 m. from R1

Maximum S.F. (factored) = 21.9 kN at R1

Live load deflection = $1.64 \times 10^8 / EI$ at 1.60 m. from R1 (E in N/mm^2 , I in cm^4)

Total deflection = $13.1 \times 10^8 / EI$ at 1.60 m. from R1



Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : 180 x 90 x 26 PFC S275

D=180.0 mm B=90.0 mm t=6.5 mm T=12.5 mm $I_x=1,820 \text{ cm}^4$ $r_y=2.89 \text{ cm}$ $S_x=232 \text{ cm}^3$

Section classification: T = 12.5mm $p_y = 275 \text{ N/mm}^2$ $\epsilon = 1$

(Table 11)

Flange: $b/T = 90.0/12.5 = 7.20$ ($\leq 9\epsilon$: Class 1 plastic)

Web: $d/t = 131.0/6.5 = 20.2$ ($\leq 40\epsilon$: Class 1 plastic)

For design purposes section classification is Class 2, compact

Shear capacity = $0.6 p_y tD = 0.6 \times 275 \times 6.5 \times 180.0/1000 = 193 \text{ kN}$ (≥ 21.9) OK

Maximum moment = 17.51 kNm at 1.60 m. from R1

Moment capacity, $M_c = p_y S_x = 275 \times 232/1000 = 63.80 \text{ kNm}$ OK

Beam is laterally restrained at supports only: effective length = $1.2L+2D$

Effective length (L_E) = 4.20m

Slenderness, λ (L_E/r_y) = $4.20 \times 100/2.89 = 145.3$

Buckling parameter (u) = 0.950

Slenderness factor (v) = 0.605 ($x = 12.8$; $\lambda/x = 11.4$)

$\beta_w = 1.000$ (Class 1/2 compact)

Equivalent slenderness (λ_{LT}) = $u.v.\lambda/\beta_w = 83.58$

Bending strength, $p_b = 156.9 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 17.51 \text{ kNm}$

Equivalent uniform moment factor, $m_{1T} = 0.925$ ($M_2=13.1$, $M_3=17.5$, $M_4=13.1$)

Equivalent uniform moment = $0.925 \times 17.51 = 16.20 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 156.9 \times 232/1000 = 36.40 \text{ kNm}$ OK

Web buckling and crushing have not been checked

LL deflection = $1.638 \times 10^8 / 205,000 \times 1820 = 0.4 \text{ mm}$ (L/7288) OK

TL deflection = $13.11 \times 10^8 / 205,000 \times 1820 = 3.5 \text{ mm}$ (L/911)

	Date	Job No.	Sheet No. 07	Rev.
Job 37 PULBOROUGH WAY				By

BEARINGS

INNER LEAF

REACTION $A=B = 46 \text{ kN/m}$

$\Sigma W/m \text{ OVER} = 28.7 \text{ kN/m}$

ASSUMING 3.5N BLOCKS $f_k = \frac{1.25 \times 3.5}{3.1} = 1.41 \text{ N/mm}^2$

$f_k \text{ OVER} = \frac{28.7 \times 10^3}{100 \times 10^3} = 0.287 \text{ N/mm}^2$

$\therefore \text{RESIDUAL} = 1.41 - 0.287 = 1.12 \text{ N/mm}^2$

BEARING LENGTH REQD = $\frac{46 \times 10^3}{1.12 \times 100} = 411 \text{ mm}$

PROVIDE MIN. 215mm BEARING ONTO 440x100x215mm DEEP PAVING STONE

OUTER LEAF

REACTION $A=B = 22 \text{ kN/m}$

$\Sigma W/m \text{ OVER} = 13.7 \text{ kN/m}$

ASSUMING 20N BRICKS $f_k = \frac{1.25 \times 5.8}{3.5} = 2.07 \text{ N/mm}^2$

$f_k \text{ OVER} = \frac{13.7 \times 10^3}{105 \times 10^3} = 0.13 \text{ N/mm}^2$

$\therefore \text{RESIDUAL } f_k = 2.07 - 0.13 = 1.94 \text{ N/mm}^2$

BEARING LENGTH REQD = $\frac{22 \times 10^3}{1.94 \times 100} = 114 \text{ mm}$

PROVIDE NOMINAL 215mm BEARING ONTO BRICKWORK

Site: 37 PULBOROUGH WAY

Job:

Job number:

SuperBeam 4.51b 0

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Page 08

File copy

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Beam: Typical Flat Roof Joist

Span: 3.9 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U D	Flat Roof	0.85x0.6	0		L	0.99	0.99
U L	Flat Roof	.60x0.6	0		L	0.70	0.70
Total load: 3.39 kN						1.70	1.70
Dead:						0.99	0.99
Live:						0.70	0.70

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. = 1.65 kNm at 1.95 m. from R1

Maximum S.F. = 1.70 kN at R1

Live load deflection = $1.08 \times 1e8/EI$ at 1.95 m. from R1 (E in N/mm², I in cm⁴)

Total deflection = $2.62 \times 1e8/EI$ at 1.95 m. from R1

Timber beam calculation to BS5268 Part 2: 2002 using C16 timber

Use 50 x 195 C16 @ 600 max c/c

$z = 316.9 \text{ cm}^3$ $I = 3090 \text{ cm}^4$

Timber grade: C16 Load sharing system: $K_8 = 1.1$

K_3 (loading duration factor) = 1.25 K_7 (depth factor) = 1.049 K_8 (load sharing factor) = 1.1

$E = 8800 \text{ N/mm}^2$ (E_{mean})

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g} \cdot K_3 \cdot K_7 \cdot K_8 = 5.3 \times 1.25 \times 1.049 \times 1.1 = 7.64 \text{ N/mm}^2$

Applied bending stress, $\sigma_{m,a} = 1.65 \times 1000/316.9 = 5.22 \text{ N/mm}^2$ OK

Permissible shear stress, $\tau_{adm,||} = \tau_{g,||} \cdot K_3 \cdot K_8 = 0.67 \times 1.25 \times 1.1 = 0.92 \text{ N/mm}^2$

Applied shear stress, $\tau_a = 1.70 \times 1000 \times 3/2 \times 50 \times 195 = 0.26 \text{ N/mm}^2$ OK

Bending deflection = $2.62 \times 1e8/8800 \times 3090 = 9.64 \text{ mm}$

Mid-span shear deflection = $1.2 \times 1.65 \times 1e6/((E/16) \times 50 \times 195) = 0.37 \text{ mm}$

Total deflection = $9.64 + 0.37 = 10.01 \text{ mm}$ (0.0026 L) $\leq 0.003L$ OK

Site: 37 PULBOROUGH WAM

Job:

Job number:

ProSteel 5.31b 532023

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Page 09

File copy

Printed 20 Feb 2009 13:08

Beam: KITCHEN / LIVING WALL

Span: 2.5 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U D	1st FLOOR	1.0x6.1x0.5	0		L	3.81	3.81
U L	1st FLOOR	1.5x6.1x0.5	0		L	5.72	5.72
U D	s/wt	0.3	0		L	0.38	0.38
Unfactored reactions (kN) Total:						9.91	9.91
Dead:						4.19	4.19
Live:						5.72	5.72
Total load: 19.81/30.02 kN Unfactored/Factored						15.01	15.01

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. (factored) = 9.38 kNm at 1.25 m. from R1

Maximum S.F. (factored) = 15.0 kN at R1

Live load deflection = $2.33 \times 1e8/EI$ at 1.25 m. from R1 (E in N/mm², I in cm⁴)

Total deflection = $4.03 \times 1e8/EI$ at 1.25 m. from R1

Beam calculation to BS5950-1:2000 using S275 steel

SECTION SIZE : 152 x 89 x 16 UB S275

D=152.4 mm B=88.7 mm t=4.5 mm T=7.7 mm $I_x=834 \text{ cm}^4$ $r_y=2.10 \text{ cm}$ $S_x=123 \text{ cm}^3$

Section classification: T = 7.7mm $p_y = 275 \text{ N/mm}^2$ $\epsilon = 1$

(Table 11)

Flange: $b/T = 44.3/7.7 = 5.76$ ($\leq 9\epsilon$: Class 1 plastic)

Web: $d/t = 121.8/4.5 = 27.1$ ($\leq 80\epsilon$: Class 1 plastic)

For design purposes section classification is Class 2, compact

Shear capacity = $0.6 p_y tD = 0.6 \times 275 \times 4.5 \times 152.4/1000 = 113 \text{ kN}$ (≥ 15.0) OK

Maximum moment = 9.383 kNm at 1.25 m. from R1

Moment capacity, $M_c = p_y S_x = 275 \times 123/1000 = 33.83 \text{ kNm}$ OK

Beam is laterally restrained at supports only: effective length = $1.2L+2D$

Effective length (L_F) = 3.30m

Slenderness, λ (L_F/r_y) = $3.30 \times 100/2.10 = 157.4$

Buckling parameter (u) = 0.889

Slenderness factor (v) = 0.698 ($x = 19.6$; $\lambda/x = 8.03$)

$\beta_w = 1.000$ (Class 1/2 compact)

Equivalent slenderness (λ_{LT}) = $u.v.\lambda/\beta_w = 97.59$

Bending strength, $p_b = 129.1 \text{ N/mm}^2$

Maximum moment within segment, $M_x = 9.383 \text{ kNm}$

Equivalent uniform moment factor, $m_{1T} = 0.925$ ($M_2=7.04$, $M_3=9.38$, $M_4=7.04$)

Equivalent uniform moment = $0.925 \times 9.383 = 8.679 \text{ kNm}$

Buckling resistance moment, $M_b = p_b S_x = 129.1 \times 123/1000 = 15.88 \text{ kNm}$ OK

Check unstiffened web capacity with load of 15.01 kN

$C1 = 37.9 \text{ kN}$; $C2 = 1.24 \text{ kN/mm}$; $C4 = 129$; $K = \min\{0.5+(a_p/1.4d), 1.0\}$; $p_{ww} = 275 \text{ N/mm}^2$

(for derivation of C factors see Steelwork Design Guide to BS5950-1:2000 6th ed.)

Bearing capacity, $P_w = C1+b_1C2$ (b_1 taken as zero) Buckling capacity, $P_x = K/(C4.P_w)$

With $b_1=0$, unstiffened web buckling capacity, $P_x = 34.9 \text{ kN}$: no minimum stiff bearing length required

LL deflection = $2.327 \times 1e8/205,000 \times 834 = 1.4 \text{ mm}$ ($L/1837$) OK

TL deflection = $4.030 \times 1e8/205,000 \times 834 = 2.4 \text{ mm}$ ($L/1060$)

Site: 37 PULBOROUGH WAY

Job:

Job number:

SuperBeam 4.51b 0

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Page 10

File copy

Printed 20 Feb 2009 13:11

Beam: KITCHEN / LIVING WALL

Span: 2.5 m.

	Load name	Loading w1	Start x1	Loading w2	End x2	R1comp	R2comp
U D	1st FLOOR	1x6.1x0.5	0		L	3.81	3.81
U L	1st FLOOR	1.5x6.1x0.5	0		L	5.72	5.72
Total load: 19.06 kN						9.53	9.53
Dead:						3.81	3.81
Live:						5.72	5.72

Load types: U:UDL D: Dead; L: Live (positions in m. from R1)

Maximum B.M. = 5.96 kNm at 1.25 m. from R1

Maximum S.F. = 9.53 kN at R1

Live load deflection = $2.33 \times 10^8 / EI$ at 1.25 m. from R1 (E in N/mm², I in cm⁴)

Total deflection = $3.88 \times 10^8 / EI$ at 1.25 m. from R1

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

Use 3no 50 x 175 C24

$z = 765.6 \text{ cm}^3$ $I = 6699 \text{ cm}^4$

Timber grade: C24 3 members acting together: $K_8 = 1.1$

K_3 (loading duration factor) = 1.25 K_7 (depth factor) = 1.061 K_8 (load sharing factor) = 1.1

$E = 7200 \times 1.21 = 8712 \text{ N/mm}^2$ ($E_{\min} \cdot K_9$)

Permissible bending stress, $\sigma_{m,adm} = \sigma_{m,g} \cdot K_3 \cdot K_7 \cdot K_8 = 7.5 \times 1.25 \times 1.061 \times 1.1 = 10.94 \text{ N/mm}^2$

Applied bending stress, $\sigma_{m,a} = 5.96 \times 1000 / 765.6 = 7.78 \text{ N/mm}^2$ OK

Permissible shear stress, $\tau_{adm,||} = \tau_{g,||} \cdot K_3 \cdot K_8 = 0.71 \times 1.25 \times 1.1 = 0.98 \text{ N/mm}^2$

Applied shear stress, $\tau_a = 9.53 \times 1000 \times 3/2 \times 150 \times 175 = 0.54 \text{ N/mm}^2$ OK

Bending deflection = $3.88 \times 10^8 / 8712 \times 6699 = 6.64 \text{ mm}$

Mid-span shear deflection = $1.2 \times 5.96 \times 10^6 / ((E/16) \times 150 \times 175) = 0.50 \text{ mm}$

Total deflection = $6.64 + 0.50 = 7.14 \text{ mm}$ ($0.0029 L$) $\leq 0.003L$ OK

	Date	Job No.	Sheet No. 11	Rev.
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Job 37 Purborough Way

By

INTERNAL WALL

SAY 850mm LONG

@ 0.4m SPREAD = $2400 \times 0.4 = 960\text{mm}$ ($> 850 \therefore$ CHECK O/A CAPACITY)

$E_h = 2500$

$E_t = 100$

$SR = 25$

$\beta = 0.38$

$SPA = 0.7 + (0.1 \times 850 \times 1.5) = 0.827$

CAPACITY = $\frac{0.38 \times 0.827 \times 100 \times 850 \times 3.5}{3.1} = \underline{\underline{30.1\text{KN WT}}}$ ($> 15\text{KN} \therefore \text{OK}$)

BEARINGS

ONTO 3.5 N/mm² BLOCKS ; $f_k = \frac{1.25 \times 3.5}{3.5} = \underline{\underline{1.25\text{N/mm}^2}}$

BEARING REQ. = $\frac{15.1 \times 10^3}{1.25 \times 89} = \underline{\underline{135\text{mm}}}$

USE 215x100x65dp ENGINEERING BRICK AS PAOSTONES

(NOTE: PARTY WALL NOTICE REQ'D - BEARING ONTO PARTY WALL)