Client	D & K Bryars
Project	Horseshoe Lane Chadlington New House

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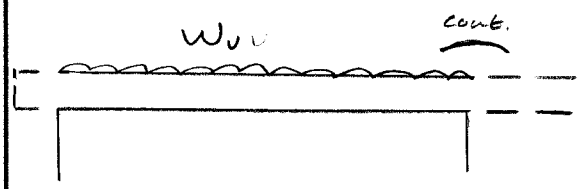
Section	By	Date	Ckd	Job No	Sheet Ref
Terrace Support – RC Beam	KPB	Dec.17	chb	7011	12

MAXIMUM SPAN of REAR LINTEL BEAM = 3.775m
@ W12 & W13
(combined)

MAXIMUM UNIT SPAN ONTO LINTEL BEAM = 5.530m



UDL – W_V

DL – $6.4 \times \frac{5.53}{2} \times 3.775$ = 66.8
LL – 1.5 = 15.7
DL – $3.35 \times 0.66 \times 3.775$ = 8.4
DL – $24 \times 0.15 \times 0.35 \times 3.775$ = 4.8

E_k = 95.7 kN.
E_V = 137.2kN.

BENDING

$M_V \Rightarrow 137.2 \times \frac{3.775}{8} = 64.9$ kNm (Simple Span Moment)

$\frac{M}{bd^2} = \frac{64.9 \times 10^6}{150 \times 279^2} = 5.56 N/mm^2$

$f_{cu} = 35 N/mm^2 \rightarrow K = 0.159$

$\therefore z \Rightarrow 0.775d.$

$A_s \Leftarrow \frac{64.9 \times 10^6}{0.95 \times 460 \times 0.775 \times 279} = 687 mm^2$

2.B25. B
($A_s = 981 mm^2$)
+
2.B16. T

SHEAR

$V = \frac{137.2}{2} = 68.6$ kN. $\therefore v = \frac{V}{bd} = 1.64 N/mm^2$

For A_s prov $v_c = 1.06 N/mm^2$ for $s_v = 150$ mm (ave) } $A_{sv} \Leftarrow 150^2 (1.64 - 1.06) / 0.95 \times 460$
$= 30 mm^2$

B8.150
($A_{sv} = 101 mm^2$ 2 LEGS)

DEFLECTION CHECK.

For A_s Provided (2.B25) $f_s = \frac{2}{3} \cdot 460 \cdot \frac{687}{981} = 215 N/mm^2$

$M_E \Leftarrow 1.2 \Rightarrow L_{max} \Leftarrow 1.2 \times 20 \times 279 = 6.700$m.

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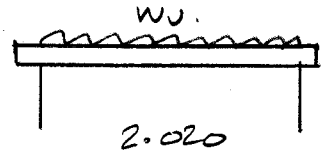
Terrace Internal Support – Angle Lintels	KPB	Dec.17	chb	7011	13

LIMITING LOAD CASE; LONGEST + 2 LEVEL SPANNING ON D4 OVER.



(MAXIMUM SPAN)

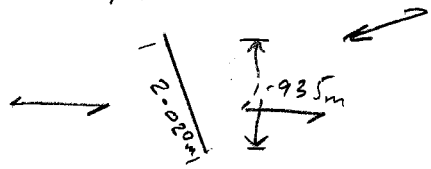
UDL – W_U

Terrace	DL – 6.4 × * 1.935 × 2.77	=	34.3
	LL – 1.5	=	8.1
Floor	DL – 5.14 × * 1.935 × 2.81	=	27.9
	LL – 1.5	=	8.2
	DL – 2.35 × 0.4 × 2.02	=	1.9
	DL – 0.95 × 0.3 × 2.02	=	0.6

F_{1k} = 81 kN

[F_{1U} = 116.7 kN]

* ANGLE SKEWED TO FLOOR SPANS



$M_{MAX} = \frac{81 \times 2.02}{8} = 20.5$ kNm

$Z_{xx} \geq \frac{20.5 \times 10^3}{180} = 114\ cm^3$ (ANGLE FULLY RESTRAINED BY FLOOR CONSTRUCTION)

$\frac{I_{xx}}{360} \geq 2.29 \times 81 \times 2.02^2 = 756\ cm^4$ (5.6 mm)

PROVIDE 200 × 200 × 16 ORSA — Z_x = 164 cm4, I_x = 2370 cm4, Δ = 1.8 mm

AT WALL SUPPORT.

$V_U = \frac{116.7}{2} = 58.4$ kN.

LDS (190 block) $\geq \frac{2.38}{3.1} \times 1.25 = 0.95\ N/mm^2$

ANGLE BEARING $\geq \frac{3100 - 2020}{2} = 540$ mm

CAPACITY 0.95 × 190 × 540 = 97.5 kN > V_U ∴ OK.

∴

Client D & K Bryars					
Project Horseshoe Lane Chadlington New House					
Section Rear Upper Roof Support – RC Beam	By KPB	Date Dec.17	Ckd chb	Job No 7011	Sheet Ref 14

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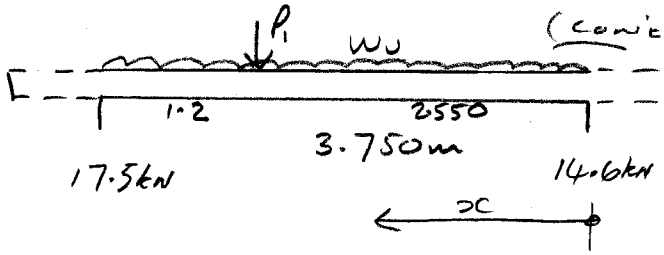
MAXIMUM SPAN = 3.750m



(con't)

UDL – W_u Roof.

DL – $0.65 \times \frac{3}{2} \times 3.750$ = 3.7
LL – 0.75 = 4.2
DL – $24 \times 0.15 \times 0.35 \times 3.750$ = 4.7
DL – $2.35 \times 0.45 \times 3.750$ = 4.0

ΣF_k = 16.6kN
[ΣF_u = 24.1kN]

$V = 0$ when $x = \frac{14.6 \times 3.75}{24.1} = 2.272$m

$M_u = \frac{14.6 \times 2.272}{2} = 16.6$kNm

P_1 – STEEL

DL – $0.65 \times \frac{4.8}{2} \times \frac{5.85}{2} \times \frac{3}{5.6}$ = 2.5
LL – 0.75 = 2.8

ΣF_k = 5.3kN
[ΣF_u = 8.0kN.]

BENDING

$\frac{M}{bd^2} = \frac{16.6 \times 10^6}{150 \times 279^2} = 1.42$ N/mm²

$f_{cu} = 35$ N/mm² $\Rightarrow K = 0.04$

$\therefore z = 0.95d$ (limit)

$A_s \geq \frac{16.6 \times 10^6}{0.95 \times 460 \times 0.95 \times 279} = 143.4$ mm² → 2.B16.B (A_s = 402mm²)

$A_{s\,min} = 0.13\% bh = 68$mm² + 2.B12.T

SHEAR

$\frac{V}{bd} \geq \frac{17.5 \times 10^3}{150 \times 279} = 0.42$ N/mm² = v < $v_c + 0.4$

∴ Min^m Links.

for A_s prov. $v_c = 0.68$ N/mm² for S_v = 200mm

$A_{sv} \geq 28$mm²

B8. 200 LINKS (A_{sv} = 101/mm) 2 LEGS

DEFLECTION

$f_s = \frac{2}{3} \cdot 460 \times \frac{144}{402} = 110$ N/mm² ∴ $M_t \geq 1.8$

$L_{max} = 1.8 \times 20 \times 279 = 10$m.

Client	D & K Bryars
Project	Horseshoe Lane Chadlington New House
Section	Terrace Handrail – Anchorage Structure

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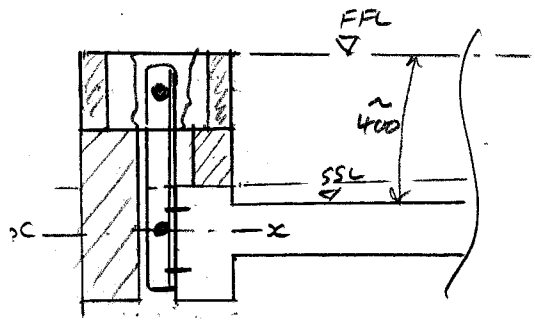
By	Date	Ckd	Job No	Sheet Ref
KPB	Jul.18	chb	7011	15



HANDRAIL LOAD = 0.74 kN/m @ 1.1 above FFL

$\Rightarrow M_{@x} = 0.74 \times 1.5 = 1.11$ kNm/m

FOR 60×60×8 RSA $M_{max} \star 180 \times 7 \times 10^{-3} = 1.26$ kNm per angle

SPACING $\star \frac{1.26}{1.11} = 1.130$ m.

LET ANCHOR BOLT VERT SPACING = 250 mm (min)

$T_{max} = \frac{1.260 \times 10^3}{250} = 5.04$ kN.

$\Rightarrow$ USE M10 'EXCALIBUR' SCREWBOLTS 50 mm embedment

(SL = 6.3 kN)

RAILING DESIGN BY OTHERS.

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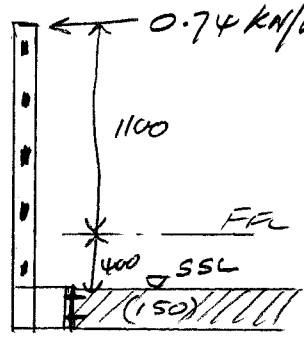
Section	By	Date	Ckd	Job No	Sheet Ref
Terrace Sidewalls – Guarding Structure	KPB	Jul.18	chb	7011	16



$M_{max} = 0.74 \times 1.5 = 1.11$ kNm/m

FOR 80x8 FLT (STAINLESS) BUILT INTO WALL

$M_{cap} = 0.7 \times 180 \times \frac{80^2 \times 8}{6} \times 10^{-6} = 1.07$ kNm

$= \text{SPACING} \times \frac{1.07}{1.11} = 968$

$\Rightarrow$ CO-ORD WITH BLOCKS = 900 c/c

$M_{max} = 1.11 \times 0.9 = 1$ kNm.

FIXING CENTRES = $150 - 2 \times 35 = 80$ mm.

$T_{max} = \frac{1 \times 10^3 \times 105}{25^2 + 105^2} = 9$ kN $\rightarrow$ PAIR BOLTS

$T = 4.5$ kN.

USE M10 'EXCALIBUR' BOLTS 50mm Embedment.
(SLC = 6.3kN)

$\Delta = \frac{0.74 \times 0.9 \times 1500^3 \times 12}{205 \times 80^3 \times 8} = 10.7$mm ∴ OK.

(Consider 100 PCT; $\Delta = 5.5$mm
(Blockwork = 140))