CALCULATIONS

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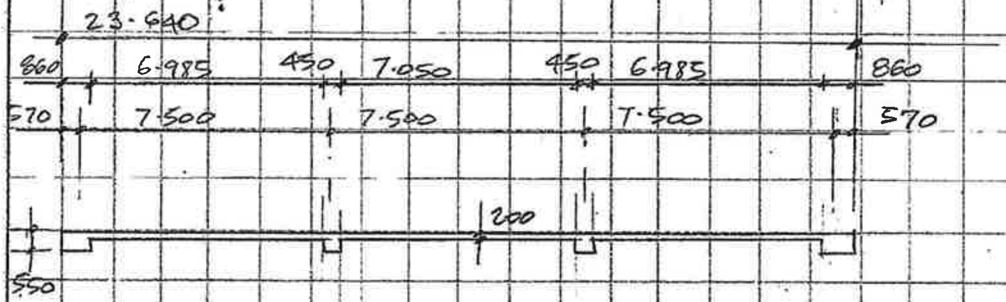
NEW OFFICE BUILDING -MADRAS ST for WILLIAMS DEVELOPMENT

PAGE: INDEX
SECT:
FILE: 2503
DATE:

Floor Slab

Dimensions:



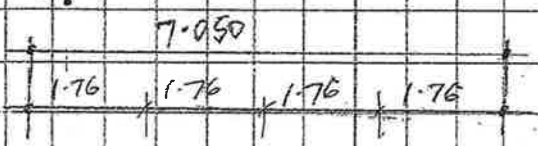
Loads:

DL	200mm o/A slab on Hi-bond,	=	4·0 kPa
	Superimposed DL		0·5 kPa
LL	offices, item 6·11		2·5 kPa.

(a) stresses due to wet concrete.

DL of slab	=	4·0 kPa
Construction LL	=	2·0 kPa
		6·0 kPa

for 3 rows of props @ 1·76 m crs.



BM coeffts

·107 ·072
·077 ·036
·121 ·107
·099 ·081

DL 4·0 kPa
LL 2·0 kPa.

max neg BM over support

$= 0.107 \times 4.0 \times 1.76^2 = 1.33$

$+ 0.121 \times 2.0 \times 1.76^2 = .75$

2.08 kNm.

max. pos. BM. at midspan

$= 0.077 \times 4 \times 1.76^2 = 0.95$

$+ .099 \times 2 \times 1.76^2 = 0.61$

1.56 kNm

max steel stress at support $= \dfrac{2080}{14.0} = 148$ N/mm²

$Z_p(-) = 14.0$ cm³/m

max steel stress at midspan $= \dfrac{1560}{12.9} = 121$ N/mm²

$Z(+) = 12.9$

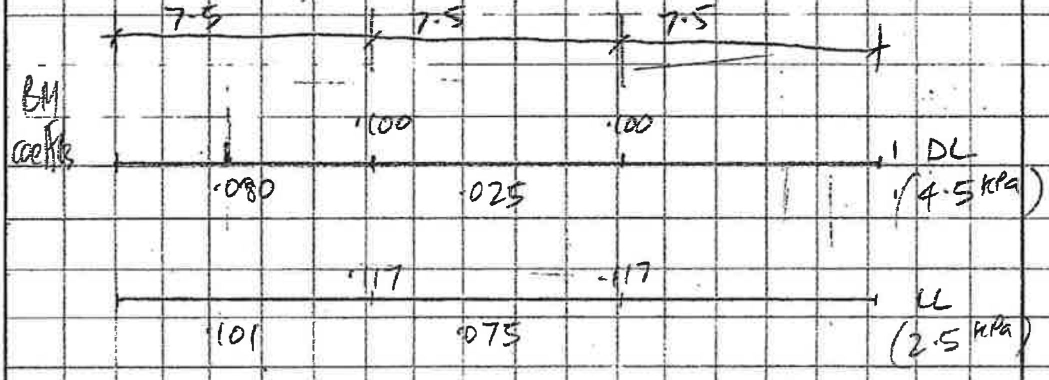
both < 300 N/mm² OK.

(b) deflection due to wet concrete

max defl. $= \dfrac{0.0069 \times 4.0 \times 1.76^4 \times 10^3}{200 \times 0.452} = 2.9$ mm.

< 3 mm OK.

(c) stresses in composite slab:



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max pos BM = $0.080 \times 4.5 \times 7.5^2 = 20.25$
$+ \; 0.10 \times 2.5 \times 7.5^2 = 14.20$
34.45 kNm

steel Z'_{cb} for 20 MPa conc = 140 cm^3/m

$\therefore f_{bt} = \frac{34450}{140}$

$= 246 \; N/mm^2$

$p_{bt} = \frac{500}{1.5(2)} = 250 \; N/mm^2$ OK.

concrete $f_{bc} = \frac{34,450}{4000} = 8.61 \; N/mm^2$

$p_{bc} = 20 \; N/mm^2 \times .45 = 9.0 \; N/mm^2$

OK

(d) deflection of composite slab:

for simply supported slab, 20 MPa concrete
deflection on de-propping

$\delta_{DL} = \frac{5}{384} \times \left[\frac{4.0 \times 7.05^4 \times 10^3}{200 \times 21.1}\right] = 30.5$ mm

$= 0.0130 \times [2341.6]$

I of transformed section in steel from Dimond H'book = 21.1

deflection corresponds OK with Dimond H'book

for 3 span continuous slab, 20 MPa concrete
deflection on de-propping

$\delta_{DL} = 0.0069 \times 2341.6 = 16.2$ mm

$\delta_{SDL} = 16.2 \times 0.5/4.0 = 2.0$

$\delta_{LL} = 16.2 \times 2.5/4.0 = 10.1$

28.3

'00

CALCULATIONS

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PAGE 94
SECT
FILE
DATE

Creep : by NZS 3101 4.4.1.3 (a)(2)

$$K_{cp} = \left[2 - 1.2\left(\frac{A_s'}{A_s}\right)\right] > 0.6$$

A_s' = 662 M = 260 mm²/m

A_s = 1058 mm²/m

$$\therefore K_{cp} = 2 - 1.2 \times \frac{260}{1058}$$

$$= 2 - .29$$

$$= 1.70$$

∴ additional creep deflection = 1.70 × 16.2 = 27.7 mm

Total deflection = 28.3 + 27.7 = 56.0 mm

If camber before depropping = $\frac{span}{360}$ = $\frac{7500}{360}$ = 20 mm

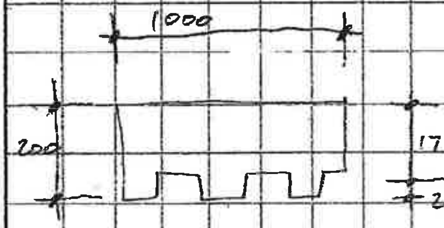
nett deflection after creep = 36.0 mm

Check composite transformed section properties

(a) before creep, E_s = 200 kN/mm²

$E_c = 4750\sqrt{25}$ = 23.7 kN/mm²

$$n = \frac{E_s}{E_c} = \frac{200}{23.7} = 8.44$$



A_c = 1000 × 173 = 173,000 mm²

A_s = 1058 mm²

cracked section : for A_s = 1058 mm²

$$a = \frac{1058 \times 500}{.85 \times 25 \times 1000} = 25 \text{ mm}$$

c = 25/.85 = 30 mm

∴ A_c = 1000 × 30 = 30000 mm²

transformed to steel = $\frac{30000}{8.44}$ = 3550 mm²

CALCULATIONS

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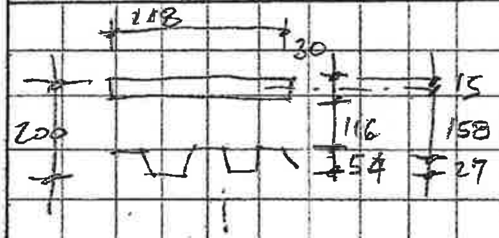
PAGE G5.
SECT
FILE
DATE



A = 3550

A = 1058

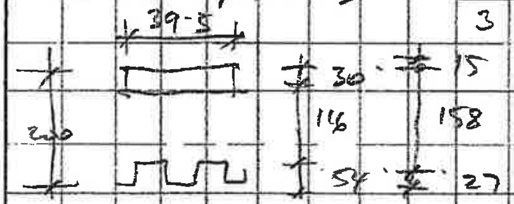
4608

$\bar{x} = \frac{(3550 \times 15) + (1058 \times 173)}{4608} = 51$ mm

$I = \frac{118 \times 30^3}{12} + (3550 \times 36^2) + 452{,}000 + (1058 \times 122^2)$

$= \underline{21.1 \times 10^6 \text{ mm}^4}$ — same as diamond?!!

With creep, $E_s = \frac{23.7}{3}$, $n = 25.3$



A = 1185

A = 1058

2243

$\bar{x} = \frac{(1185 \times 15) + (1058 \times 173)}{2243}$

$= 89.5$

$I = \frac{39.5 \times 30^3}{12} + (1185 \times 74.5^2) + 452{,}000 + (1058 \times 83.5^2)$

$= \underline{14.5 \times 10^6 \text{ mm}^4}$

≈ 68% of initial I.

TL ℄ deflection incl. creep

$= 18.2^{mm} \times \frac{21.1}{14.5} = 26.5^{mm}$

LL deflection = 10.1^{mm}

→ precamber floor by 20^{mm} at midspan
12^{mm} at ¼ points.

Floor camber on props and on topping level

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Negative reinforcement at beams

$w_u = (1.4 \times 4.5) + (1.7 \times 2.5)$
$= 6.3 + 4.25$
$= 10.55$ kN/m²

664 M in slab.

interior support

neg BM $= 0.100 \times 6.3 \times 7.5^2 = 35.4$
$+ 0.117 \times 4.25 \times 7.5^2 = 28.0$
63.4 kNm.

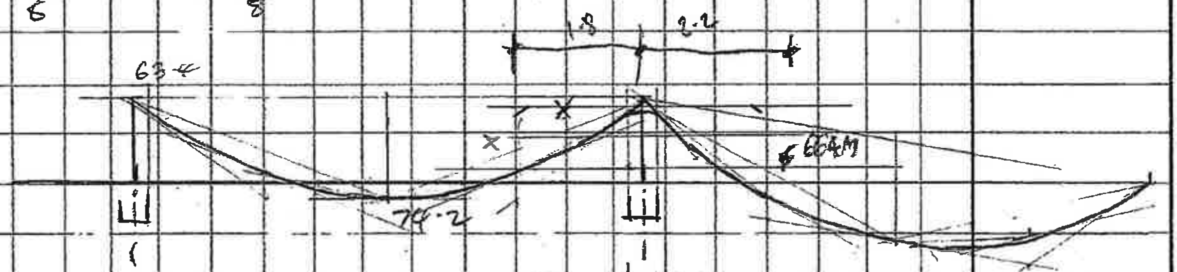
for 200 mm slab: $d = 180 - 6 = 174$ mm
$A_{s\,reqd} = 1600$ D
1157 H

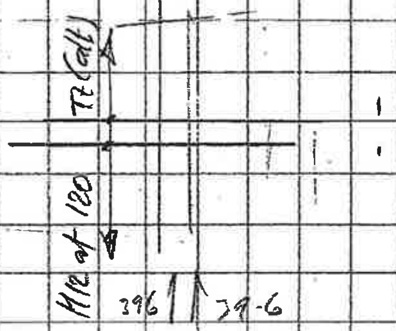
for H12 at 120, $A_s = 942$ mm²/m
664 M, $A_s = 186$
1128

$a = \frac{1128 \times 380}{.85 \times 25 \times 500} = 40$

$jd = 200 - (20 + 6 + 20) = 154$
$M_u = .9 \times 1128 \times 380 \times 154 = 59.4$ kNm

$\frac{w_u \ell^2}{8} = \frac{10.55 \times 7.5^2}{8} = 74.2$





H12–120 top steel at interior beams.

for 10.55 kN/m

39.6 ↑ 39.6 | 39.6 ↑ 39.6 | 39.6 ↑
8.5 | 8.5 ↓
Reactions | 87.7 (8.31 m) | 31.1

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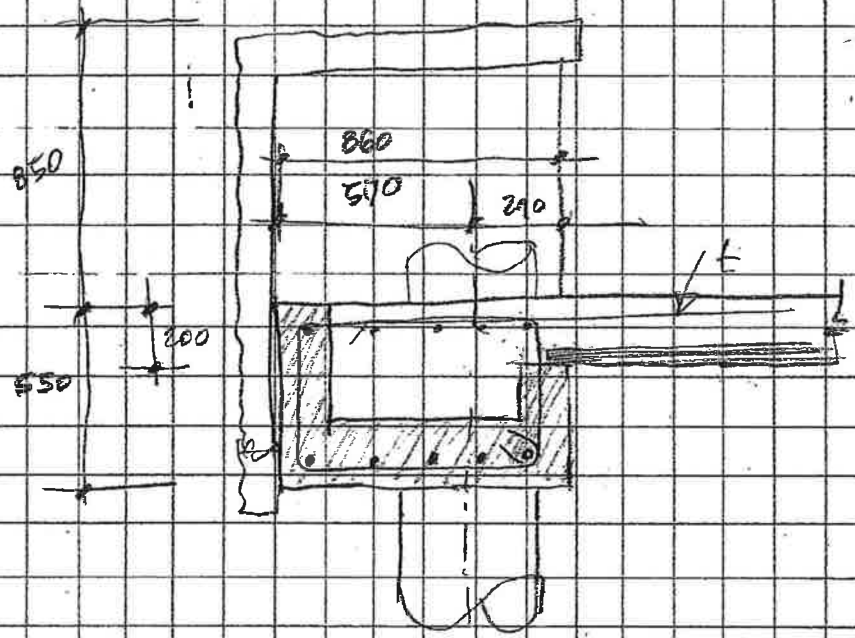
exterior support:



BM from load outside ℄ of column:

DC : precast panel = 2·26 x ·110 x 23·5 = 5·84 kN/m
beam = 0·55 x 0·86 x 23·5 = 11·11 kN/m
16·95
LL = 0·97 m x 0·50 kPa = ·49 kN/m

w_u = (1·4 x 16·95) + (1·7 x ·49)
= 23·7 + ·8
= 24·5 kN/m
M_u = 24·5 kN/m x 0·30 m
= 7·35 kN m/m

for 664 M A_s = 186 mm²/m a = 6·6 jd = 176 M_u = 11·2

Cantilever at stair & lift well

cantilever = 2·200
CLM = 10·55 x 2·2²/2 = 25·53 kNm
for H12 at 200 T A_s = 566 , a = 20 jd = 164
M_u = 31·7 kNm > 25·53 OK.

664 M Top steel at edge beams line 1

H12 at 200 T + 664 M at lever line 4

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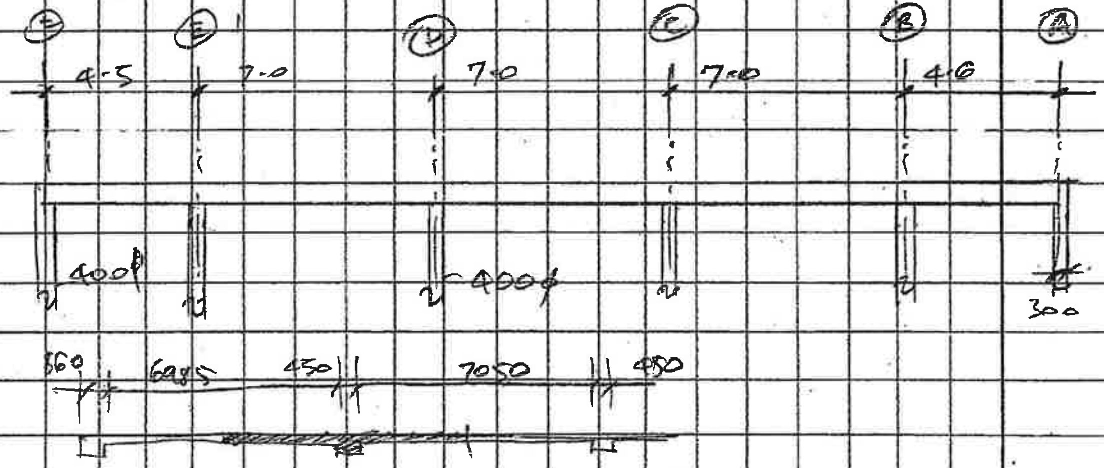
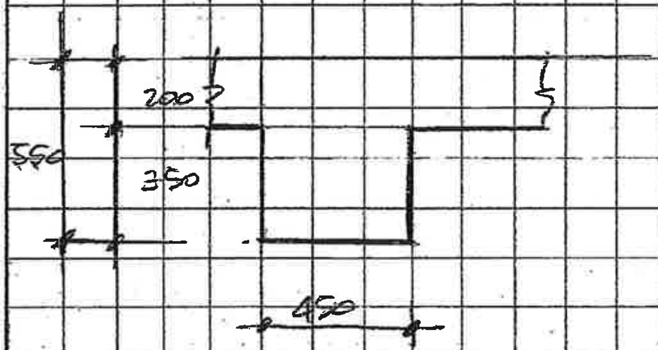
Floor beams:

(A) Central Beams, lines 2 & 3

Dimensions:





Loads:

$w_D = (4 \cdot 5 \times 8 \cdot 31) + (\cdot 45 \times \cdot 35 \times 23 \cdot 5) = 41 \cdot 1$ kN/m

$w_L = 2 \cdot 5 \times 8 \cdot 31 \qquad 20 \cdot 8$

$w_u = (1 \cdot 4 \times 41 \cdot 1) + (1 \cdot 7 \times 20 \cdot 8)$

$= 57 \cdot 5 + 35 \cdot 4$

$= 92 \cdot 9$ kN/m

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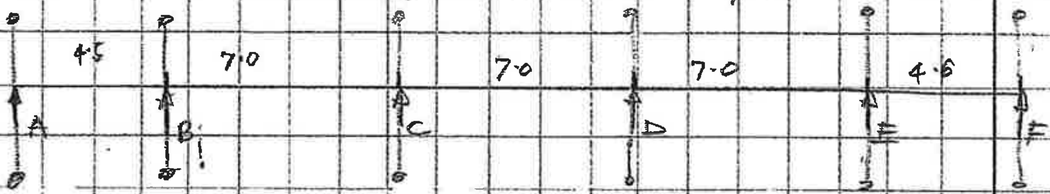
(a) distribution for DL + full LL all spans.



if assume columns have no stiffness:

$S_{BA} = \frac{3.10}{4.5} = 6.67$ $\qquad D_{BA} = 0.54$

$S_{BC} = \frac{4.10}{7.0} = \frac{5.71}{12.38}$ $\qquad D_{BC} = 0.46$

$S_{CB} = 5.71$ $\qquad D_{CB} = .67$

$S_{CD} = \frac{2.10}{7} = \frac{2.86}{8.57}$ $\qquad D_{CD} = .33$

$M^F_{BA} = \frac{wL^2}{8} = \frac{92.9 \times 4.5^2}{8} = 235$

$M^F_{BC} = \frac{w_u L^2}{12} = \frac{92.9 \times 7.0^2}{12} = 379$

SSM BA = 235 kNm

SSM BC = 379 x 1.5 = 569 kNm

CALCULATIONS

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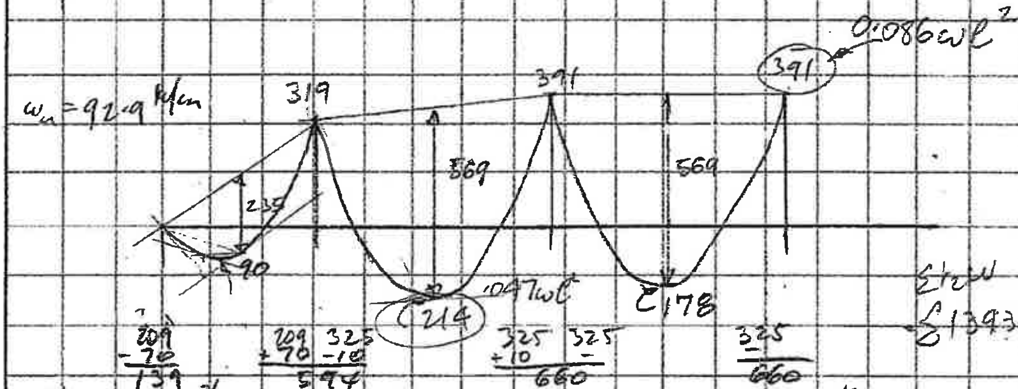
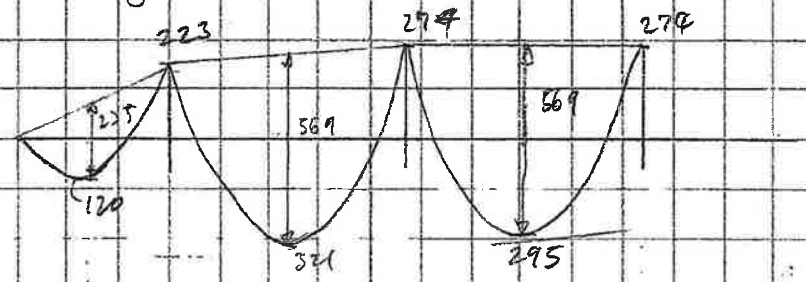
PAGE G10.
SECT
FILE
DATE

BA	BC	CB	CD
0.54	0.46	.67	.33
235	-379	379	-379
78	66	0	0
	0	33	
0	0	-22	-11
	-11	0	
6	5	0	0
0	0	3	
0	0	-2	-1
319	-319	391	-391



with 30% redistribution. 0.3 × 391 = 117 kNm

neg BM = 391 - 117 = 274



209	209	325	325	325	325
-50	+50	-16	+16	-	-
159	259	309	341	325	

159 568 666 Σ1393 ok.

CALCULATIONS

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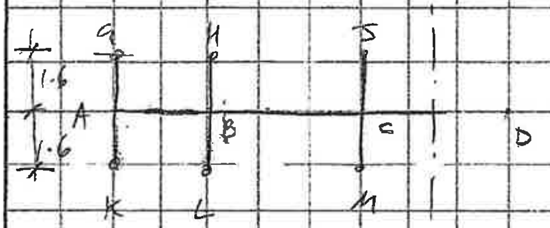
PAGE G 11
SECT
FILE
DATE

Check effect of columns (full LL all spans)



I_{beam} 450 × 550 $= \frac{450 \times 550^3}{12} = 6 \cdot 24 \times 10^9$ mm^4

I_{col} 400 ⌀ $= \frac{\pi . 400^4}{64} = 1 \cdot 26 \times 10^9$ mm^4

$S_{AB} = \frac{4 . 6 \cdot 24}{4 \cdot 5} = 5 \cdot 55$ $D_{AB} = 0 \cdot 54$

$S_{AG} = \frac{3 . 1 \cdot 26}{1 \cdot 6} = 2 \cdot 36$ $D_{AG} = 0 \cdot 23$

$S_{AK} = S_{AG} = \frac{2 \cdot 36}{10 \cdot 27}$ $D_{AK} = 0 \cdot 23$

$S_{BA} = \frac{4 . 6 \cdot 24}{4 \cdot 5} = 5 \cdot 55$ $D_{BA} = 0 \cdot 40$

$S_{BH} = S_{AG} = 2 \cdot 36$ $D_{BH} = 0 \cdot 17$

S_{BL} = " $2 \cdot 36$ $D_{BL} = 0 \cdot 17$

$S_{BC} = \frac{4 . 6 \cdot 24}{7 \cdot 0} = 3 \cdot 57$ $D_{BC} = 0 \cdot 26$

$13 \cdot 84$

$S_{CB} = S_{BC} = 3 \cdot 57$ $D_{CB} = 0 \cdot 36$

S_{CJ} = " $= 2 \cdot 36$ $D_{CJ} = 0 \cdot 23$

S_{CM} = " $= 2 \cdot 36$ $D_{CM} = 0 \cdot 23$

$S_{CD} = \frac{2 . 6 \cdot 24}{7 \cdot 0} = 1 \cdot 78$ $D_{CD} = 0 \cdot 18$

$10 \cdot 07$

$M^F_{AB} = \frac{92 \cdot 9 \times 4 \cdot 5^2}{12} = 157$

CALCULATIONS

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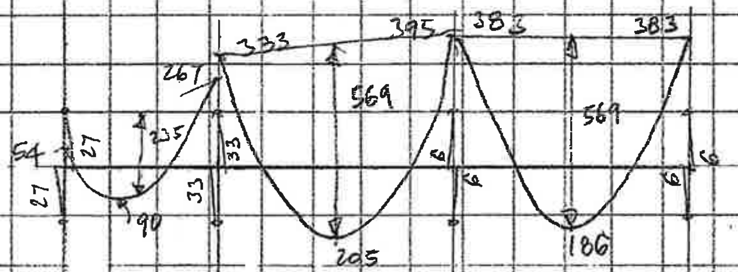
PAGE G12.
SECT
FILE
DATE

AG	AK	AB	BA	BH	BL	BC	CB	CS	CM	CD
·23	·23	·54	·40	·17	·17	·26	·36	·23	·23	·18
		−157	157			−379	379			−379
36	36	85	88	38	38	58	0	0	0	0
		44	43			0	29			
−10	−10	−24	−17	−7	−7	−12	−10	−7	−7	−5
		−9	−12			−5	−6			
2	2	5	7	3	3	4	3	1	1	1
		4	3			2	2			
−1	−1	−2	−2	−1	−1	−1	−2	0	0	0
27	27	−54	267	33	33	−333	395	−6	−6	−383

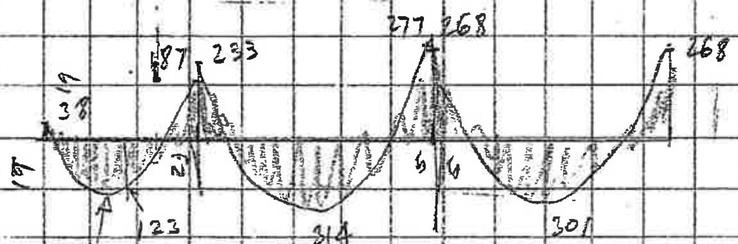


(1)

209 209 325 325 325 325
−47 +47 −9 +9 −
162 256 316 334 325 325

162 572 659 659 Σ 1393

30% redistribution.



? 140 ?
209 209 325 325 325 325
−33 33 −6 6 0
176 242 319 331 325 325

Σ 1393

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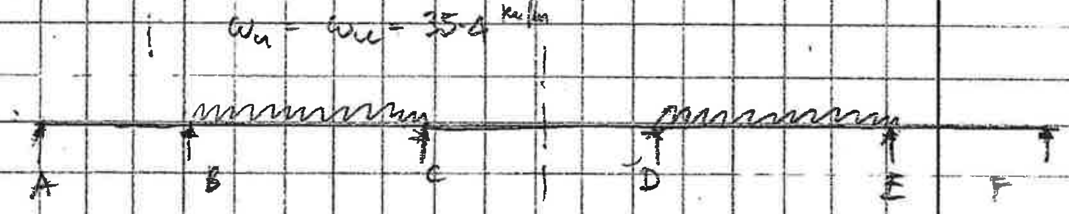
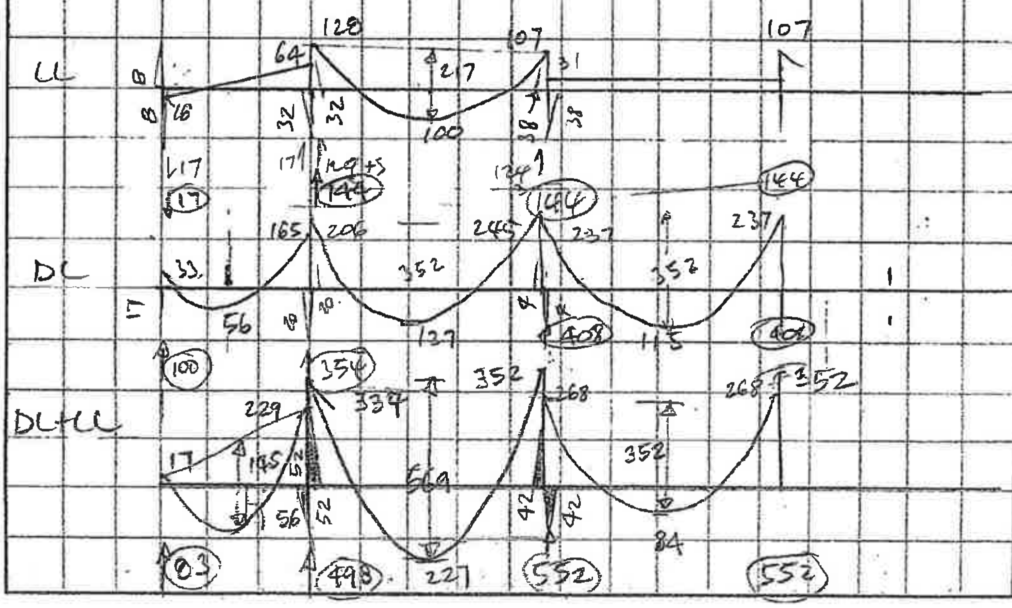
(b) distribution with alternate LL

for



SSM = $\frac{35.4 \times 7^2}{8}$ = 217   FEM = $\frac{wL^2}{12}$ = 145

AG	AK	AB	BA	BM	BL	BC	CB	CJ	CM	CD
·23	·23	·54	·40	·17	·17	·26	·36	·23	·23	·18
						−145	145			
			57	25	25	38	−53	−33	−33	−26
		29				−27	19			
−7	−7	−15	10	5	5	7	−7	−4	−4	−4
		5	−8			−4	4			
−1	−	−3	5	2	2	3	−1	−1	−1	−1
−8	−8	16	64	32	32	−128	107	−38	−38	−31

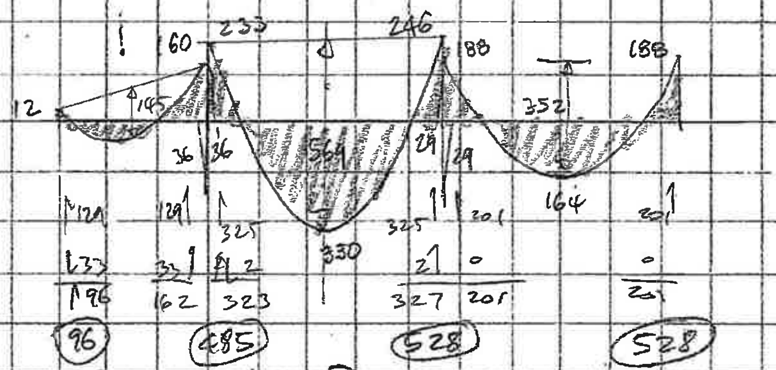


LL
From above
(35.4 kN/m)

DL From
p. G12.(1)
$\frac{\times 57.5}{92.4} = .6189$

CALCULATIONS		PAGE	914.
ALAN M. REAY CONSULTING ENGINEER CHRISTCHURCH		SECT	
		FILE	
		DATE	

DL+LL with 30% redistribution:



Beam Reinforcement : Flexural

positive bending :

for 7·0^m span : max T-beam width $= \ell/4 = \frac{7000}{4} = 1750$

$=(8\times150)+450 = 1650$

$b \qquad = 3000.$

∴ use $b = 1650^{mm}$.

for $M_u = 330$ ^{kNm}

$m_u/m = \frac{330}{1\cdot65} = 200$ $^{kNm/m}$ $\qquad d \simeq 450$

$A_s/m \simeq 2000$ D

1447 H

$A_s = 3300$ D

2388 H

for 4H28, $A_s = 2460$

steel	A_s	a	d	jd	M_u		
4H28	2460	26	486	473	398	kNm	
3H28	1850	20	486	476	301	kNm	550 o/A depth
2H28	1230	13	486	479	201	kNm	
2H28	1230	13	286	279	117	k	350 o/A depth
4H24	1809	20	488	478	296		550 depth

CALCULATIONS

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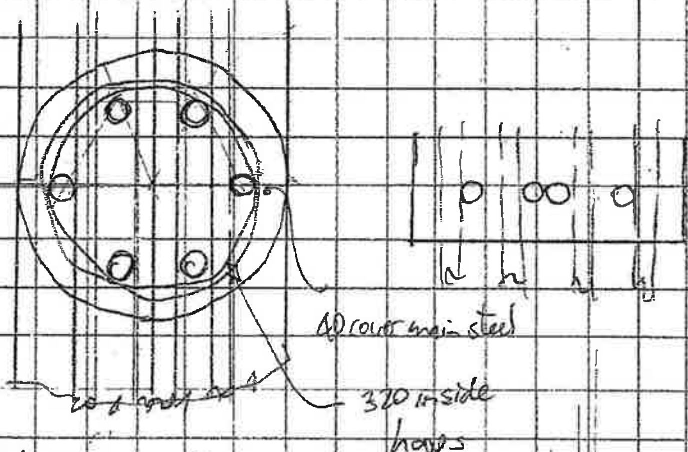
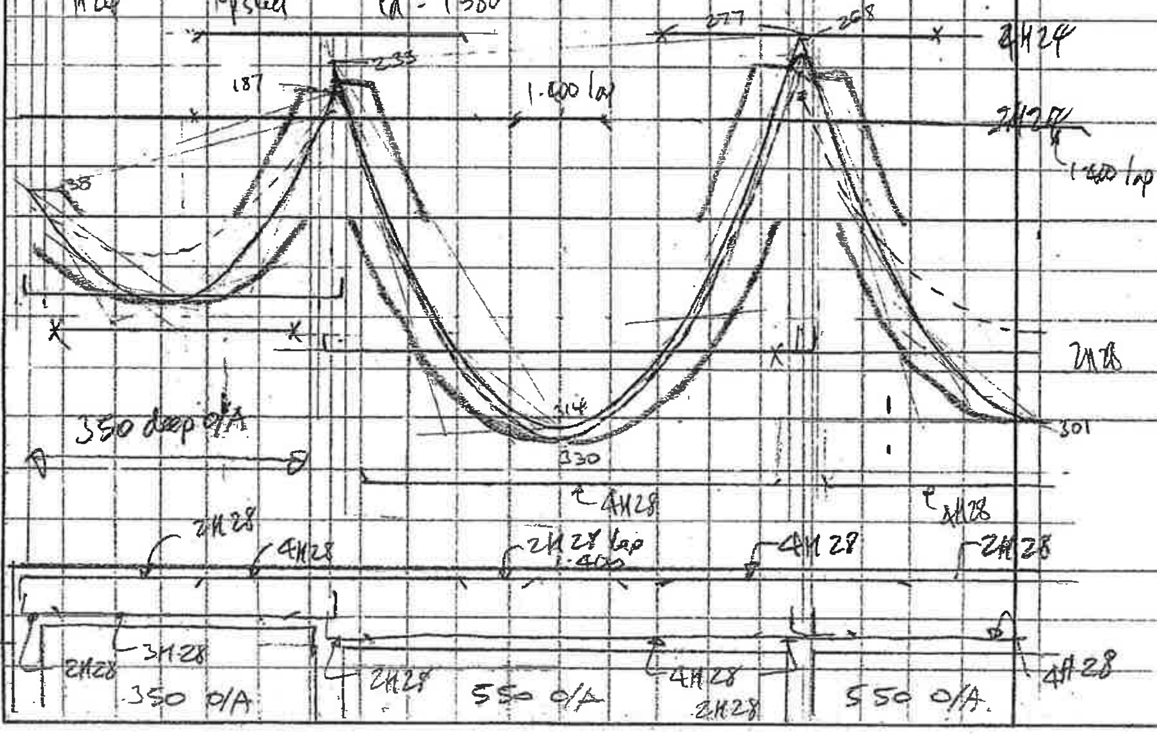
PAGE G15.
SECT
FILE
DATE

Negative Bending

	Steel	A_s	a	d	jd	M_u
b = 450	4H28	2460	98	486	437	367
	3H28	1850	75	486	448	283
	2H28	1230	49	486	461	194
b 400	4H24	1809	81	488	448	277
	3H24	1357	60	488	458	212
	2H24	905	40	488	468	145



for H28, bot steel 90mm c-c
f'c = 25 ld = 1440
H24 top steel ld = 1380



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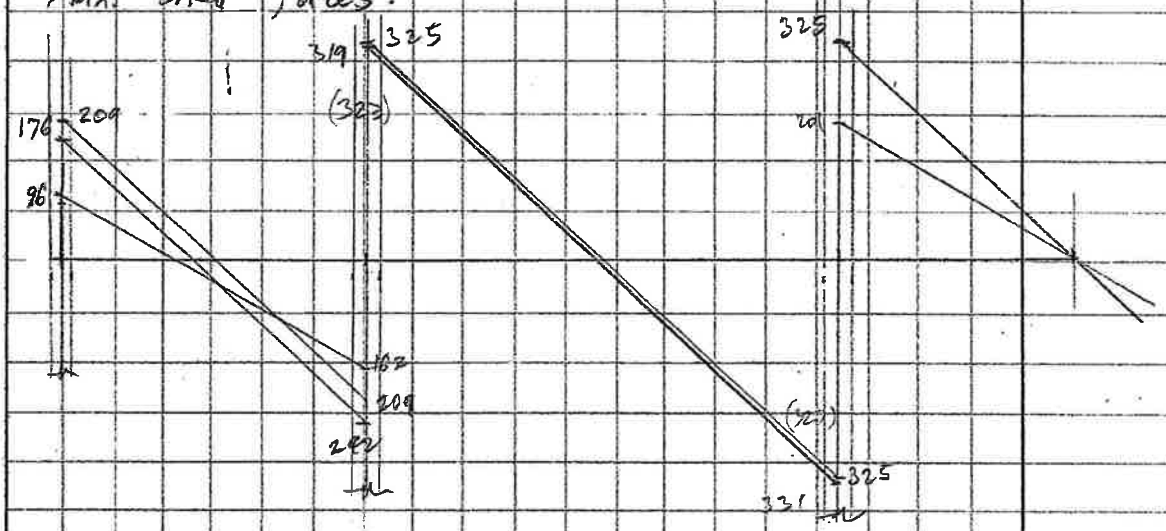
Shear Reinforcement:

Max. Shear Forces:



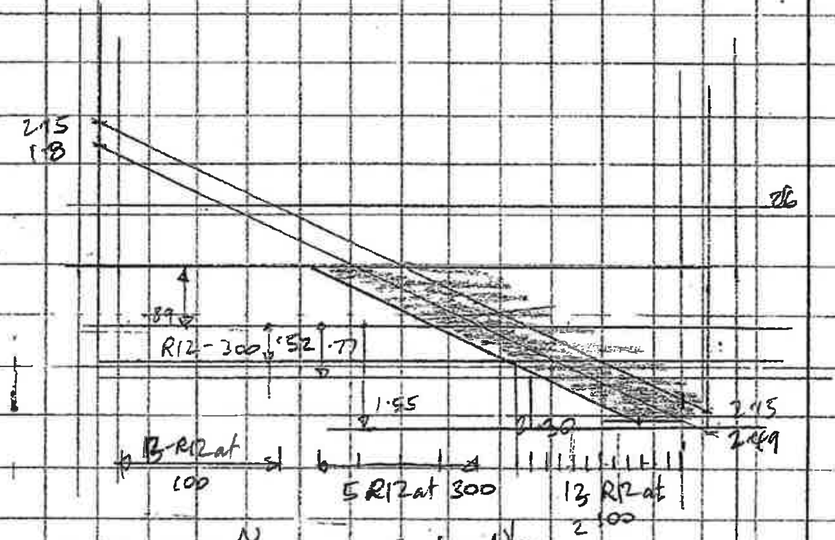
b = beam width = 400 mm

O/A depth, 7.0 span = 550 mm, d = 550 − (50 + 14)

= 486

4.5 span = 350, d = 286

(a) For 4.5 span beam:



v_u max = $\frac{242,000^{N}}{.85 \times 400 \times 286}$ = 2.49 N/mm²

$p = \frac{1230}{400 \times 286}$ = .0108 v_c = .177 $\sqrt{25}$ = 0.89 N/mm²

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CALCULATIONS

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PAGE G17
SECT
FILE
DATE

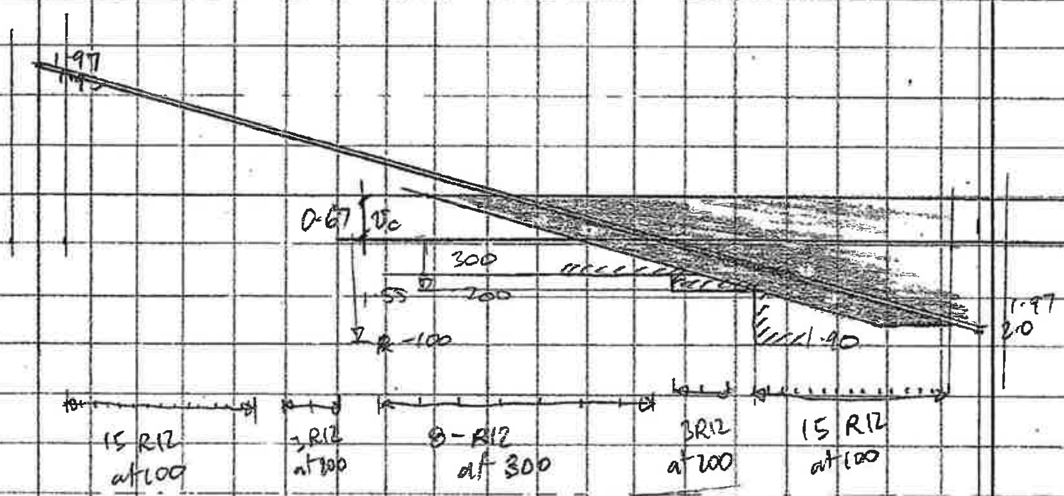
$v_{s\,max} = 2.30 - 0.89 = 1.41$

$A_s = \frac{1.41 \times 400 \times 1000}{275} = 2051$ mm²/m

for R12 at 100, $A_v = 2260$

stirrups	A_v	v_s
R12 - 100	2260	1.55
- 200	1130	0.77
- 300	753	0.52
- 150		
- 250		

(b) For 7.0m span beams:



$v_{a\,max} = \frac{331,000}{.85 \times 400 \times 486} = 2.00$ N/mm²

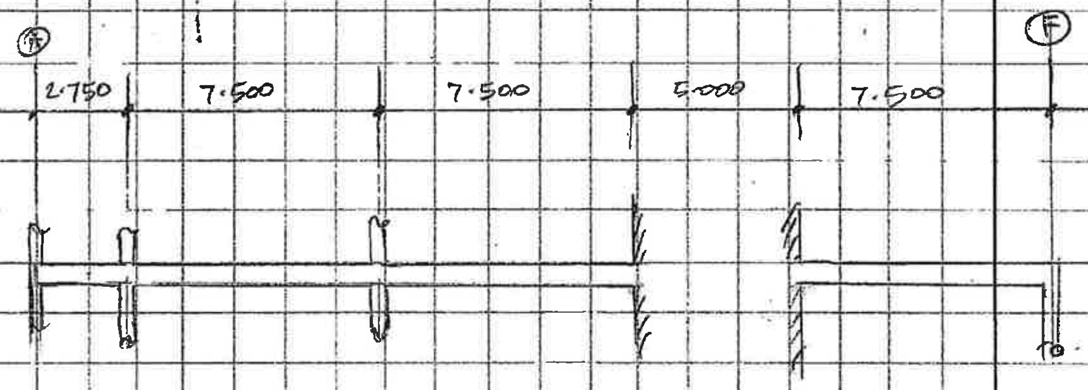
$\rho = \frac{1230}{400 \times 486} = .0063$, $v_c = .13\sqrt{25} = 0.67$ N/mm²

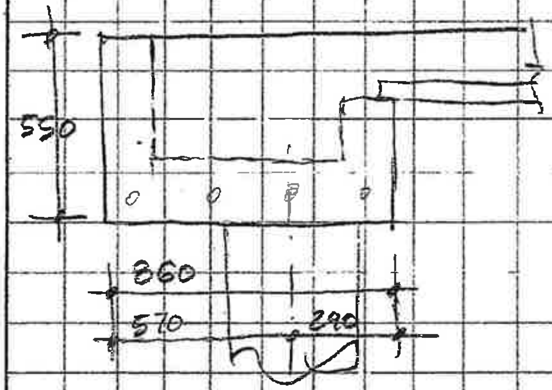
$v_{s\,max} = 1.90 - 0.67 = 1.23$ N/mm²

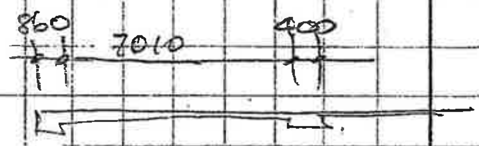
$A_{v\,max} = \frac{1.23 \times 400 \times 1000}{275} = 1790$ mm²/m

(B) Outer beam line 1

Dimensions:







Loads:

w_D : precast panels, from p. G7 = 5.84 kN/m

self wt beam = 11.11

slab wt = 4.5 kPa × 3.505 = 15.77

32.72

w_L : (2.5 × 3.505) + (0.5 × .86) = 9.19 kN/m

w_u = (1.4 × 32.72) + (1.7 × 9.19)

= 45.8 + 15.6

= 61.4 kN/m

CALCULATIONS

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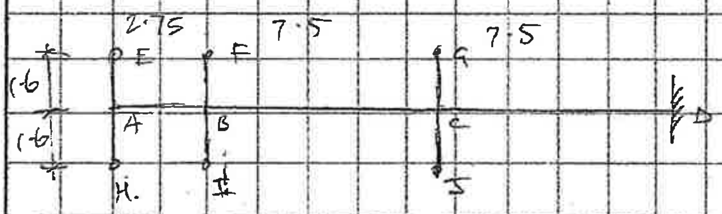


For DL+LL on all spans:

860 × 550 $\quad I_g = \frac{860 \times 550^3}{12} = 11.92 \times 10^9 \text{ mm}^4$

$\bigcirc$ 400 $\quad I_g = \frac{\pi \times 400^4}{64} = 1.26 \times 10^9 \text{ mm}^4$

$S_{AE} = \frac{3.1.26}{1.6} = 2.36 \qquad D_{AE} = 0.11$

$S_{AH} = S_{AE} = 2.36 \qquad D_{AH} = 0.11$

$S_{AB} = \frac{4.11.92}{2.75} = 17.34 \qquad D_{AB} = 0.78$

$= 22.06$

$S_{BA} = S_{AB} = 17.34 \qquad D_{BA} = .62$

$S_{BF} = 2.36 \qquad D_{BF} = .08$

$S_{BI} = 2.36 \qquad D_{BI} = .08$

$S_{BC} = \frac{4.11.92}{7.5} = 6.36 \qquad D_{BC} = .22$

$= 28.42$

$S_{CB} = 6.36 \qquad D_{CB} = 0.36$

$S_{CG} \quad 2.36 \qquad D_{CG} \quad 0.14$

$_{CJ} \quad 2.36 \qquad D_{CJ} \quad 0.14$

$_{CD} = 6.36 \qquad D_{CD} = 0.36$

17.44

AB: $SSM = \frac{61.4 \times 2.75^2}{8} = 58 \quad , FEM = 39$

BC, CD: $SSM = \frac{61.4 \times 7.5^2}{8} = 432 \quad FEM = 288.$

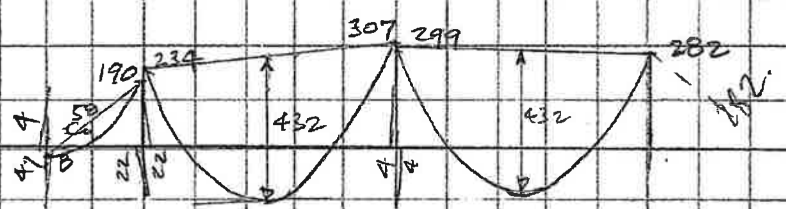
BUI.MAD249.0273.21

CALCULATIONS	PAGE	G 20
ALAN M. REAY CONSULTING ENGINEER CHRISTCHURCH	SECT	
	FILE	
	DATE	

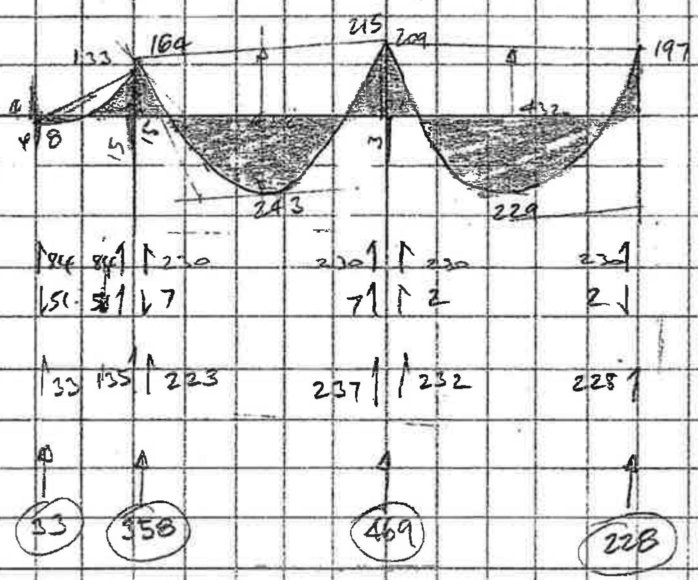
AE	AH	AB	BA	BF	BI	BC	CB	CG	CJ	CD	DC
·11	·11	·78	·62	·08	·08	·22	·36	·14	·14	·36	
		-39	39			-288	288			-288	288
4	4	31	154	20	20	55	0	0	0	0	0
		77	16				28			0	0
-8	-8	-61	-10	-1	-1	-4	-10	-4	-4	-10	
		-5	-30			-5	-2				-5
1	1	3	21	3	3	8	1	0	0	1	
		10					4				0
-1	-1	-8					-2			-2	-1
-4	-4	8	190	22	22	-234	307	-4	-4	299	282

$w_u = 61{\cdot}4^{kN/m}$

Σ 1090



redistribute 30%



Σ 1088

CALCULATIONS

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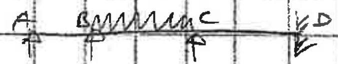
CHRISTCHURCH

PAGE 621
SECT
FILE
DATE

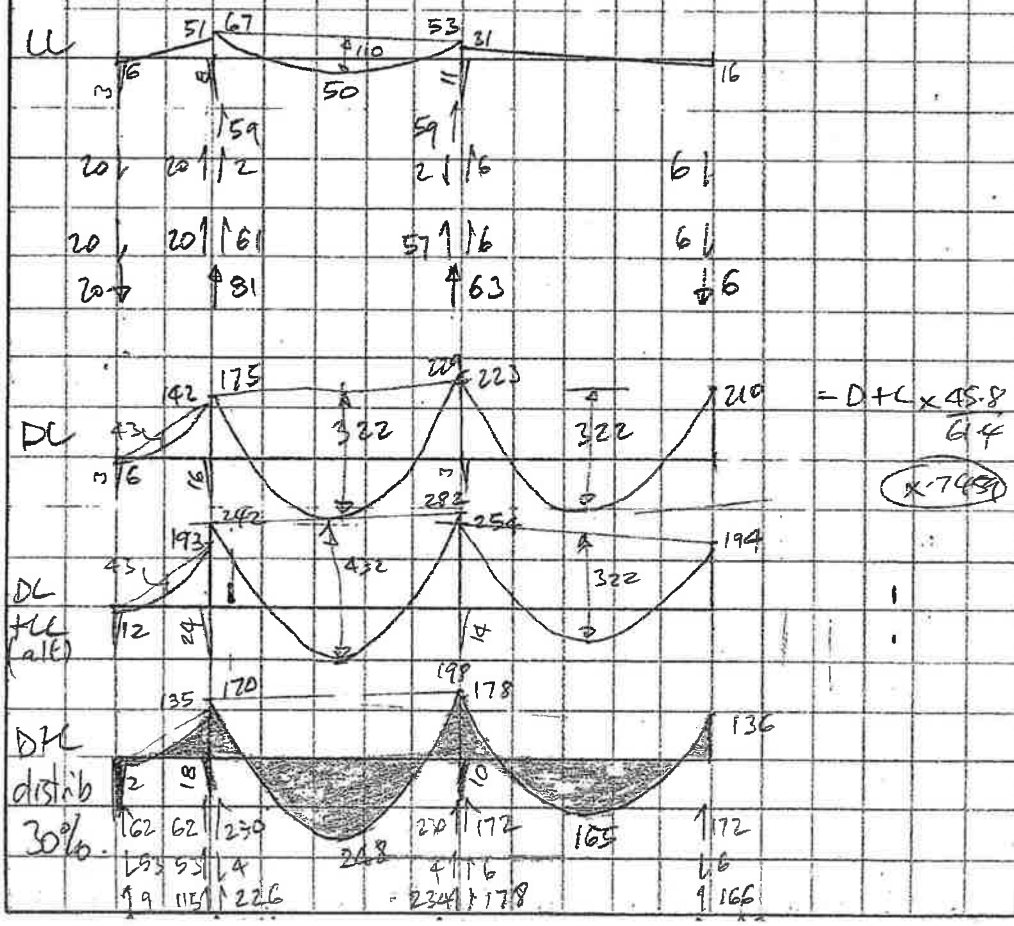
$w_u = 15.6$ kN/m

for LL on middle span only



AE	AH	AB	BA	BF	BI	BC	CB	CG	CJ	CD	DC
·11	·11	·78	·62	·08	·08	·22	·36	·14	·14	·36	
						−73	73				
			45	6	6	16	−26	−10	−10	−27	
		23				−13	8				−14
−3	−3	−17	8	1	1	3	−3	−1	−1	−3	
		4	−9			−2	2				−2
		−4	7	1	1	2	−1			−1	
−3	−3	6	51	8	8	−67	53	−11	−11	−31	−16

$SSM = \frac{15.6 \times 7.5^2}{8} = 110$ FEM = 73



CALCULATIONS

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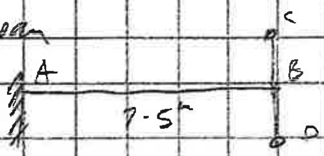
CHRISTCHURCH

PAGE G22.
SECT
FILE
DATE

shortspan



$S_{BA} = \frac{4 \cdot 11.92}{7.5} = 17.34$ $D_{BA} = .78$

$S_{BC} = 2.36$ $D_{BC} = .11$

$S_{BD} = 2.36$ $D_{BD} = .11$

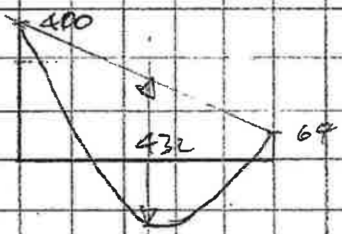
22.06

$w_u = 61.4$, SSM = 432   FEM = 288.

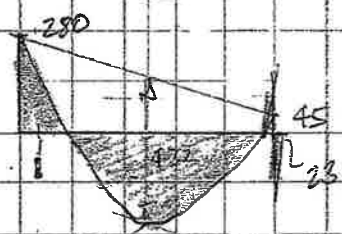
$\Sigma W = 460.$

AB	BA	BC	BD
	.78	.11	.11
−288	288		
	−224	−32	−32
−112			
400	64	−32	−32

DL+LL



distrib
30%



↑230   230↑
↑31   31↓
↑261   199↑

$\Sigma 460$

CALCULATIONS

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CHRISTCHURCH

PAGE G23
SECT
FILE
DATE

Beam Reinforcement : Flexural :

L-beam flange width $\leq \ell/4 = \frac{7500}{4} = 1875$

$860 + \frac{7500}{12} = 1485$ ←

$860 + 6 \times 250 = 1760$

$860 + 7050/2 = 4388$

min reinf $= \frac{1.4}{380} = 0.0037$

$A_s = .0037 \times 860 \times 486 = 1540 \text{ mm}^2$

max. reinf for $M_u = 270$ pos BM

& $M_u = 280$ neg BM

pos BM, $\frac{M_u}{m} = \frac{270}{1.485} = 182 \text{ kNm/m}$

$A_s/m = 1700 \times \frac{275}{380} = 1230$

$A_s = 1.485 \times 1230 = 1827 \text{ mm}^2$

neg BM, $\frac{M_u}{m} = \frac{280}{.86} = 326$

$A_s/m = 2800$ D

2026 H

$A_s = 1742$ H

Pos. BM.

Reinft	A_s	a	d	jd	M_u
3H24	1357	16	488	480	223
4H24	1809	22	"	477	295 ← min.
5H24	2262	27	"	474	367

CALCULATIONS

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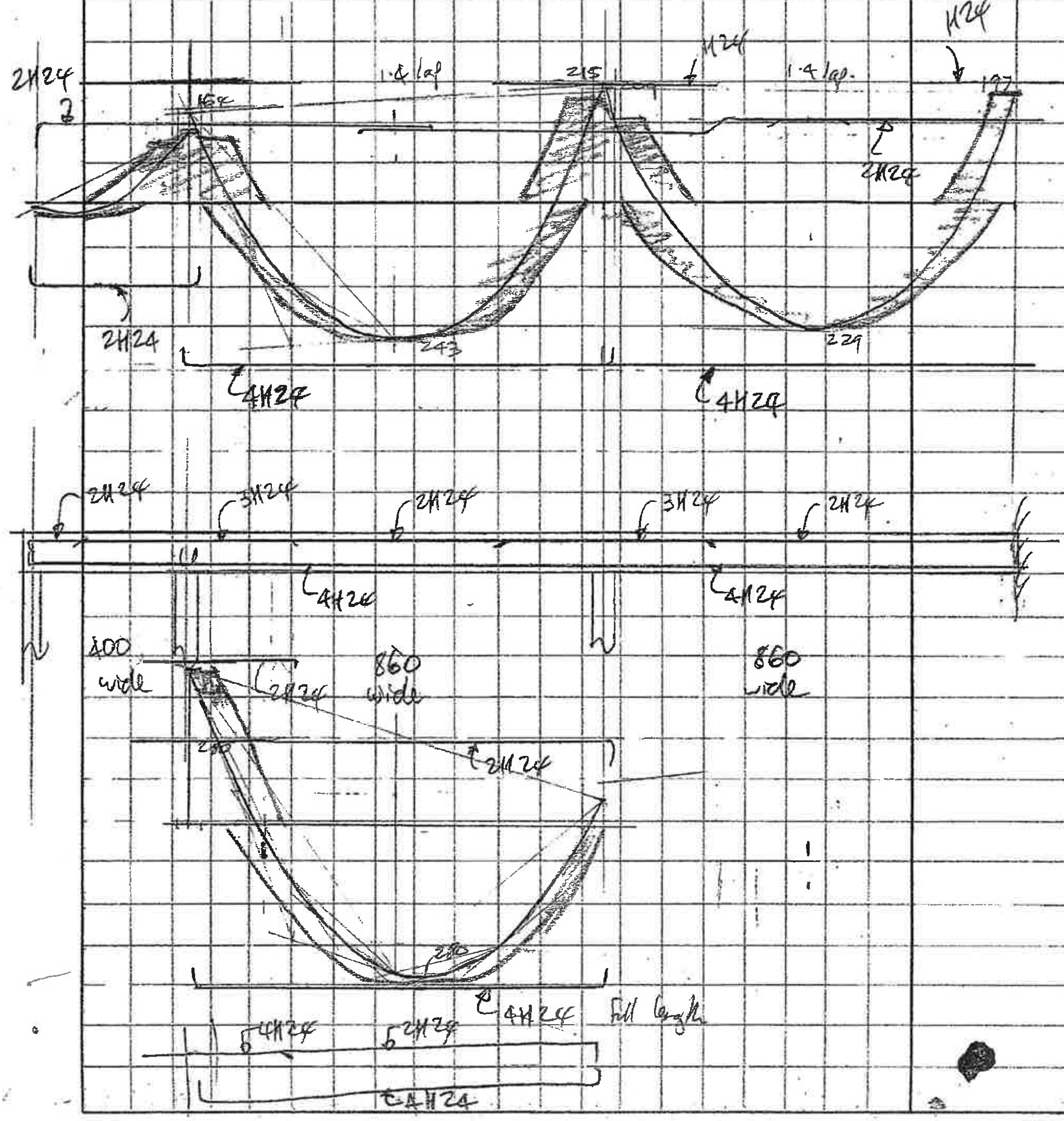
PAGE 624
SECT
FILE
DATE

Neg. BM

Reinf	A_s	a	d	jd	M_u
3 H24	1357	28	488	474	220
4 H24	1809	38	"	469	290
5 H24	2262	48	"	464	359
2H24	905	19	"	478	148



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CALCULATIONS

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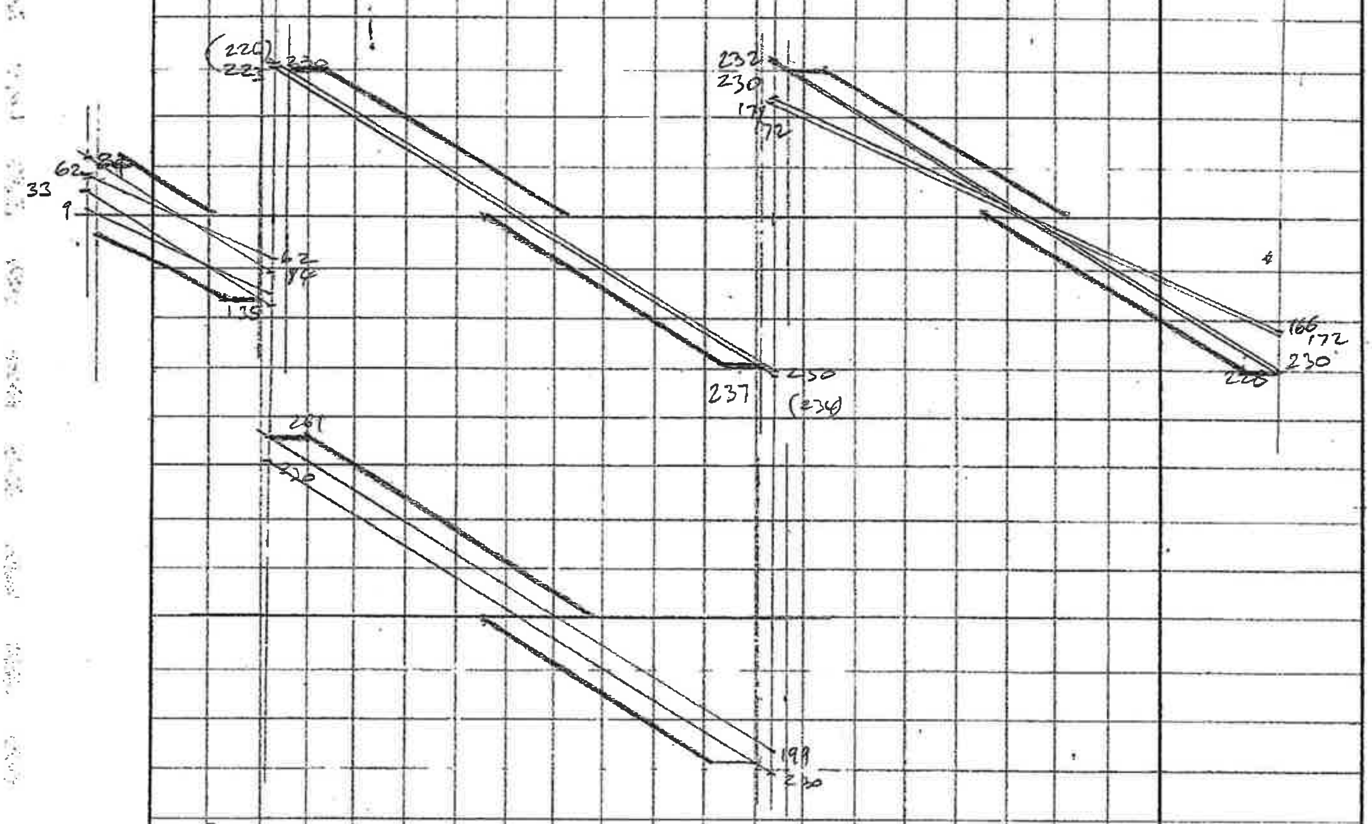
PAGE G25.
SECT
FILE
DATE

Shear Reinforcement.

Max. Shear Forces:



for 400 beam width

$$\max v_u = \frac{125{,}000}{.85 \times 400 \times 486} = 0.76$$

$v_b = 0.60$

$v_{s\,min}$ (R12 @ 300) $= \underline{.52}$

$1.12 > .76$

∴ use R12 @ 300 full length

400 wide beam
use R12@ 300 full length

for 860 beam width

$$\max v_u = \frac{261{,}000}{.85 \times 860 \times 486} = 0.73 \text{ N/mm}^2$$

$$p = \frac{1809}{860 \times 486} = .0043 \quad v_b = 0.57 \text{ N/mm}^2$$

$v_{s\,reqd} = 0.16$

$$A_v\ min = \frac{.345 \times 860 \times 1000}{275} = 1079 \text{ mm}^2/\text{m}$$

860 wide beam
use R12 at 200 full length.

for R12 at 200 $A_v = 1132$, $v_s = .36 > .16$.

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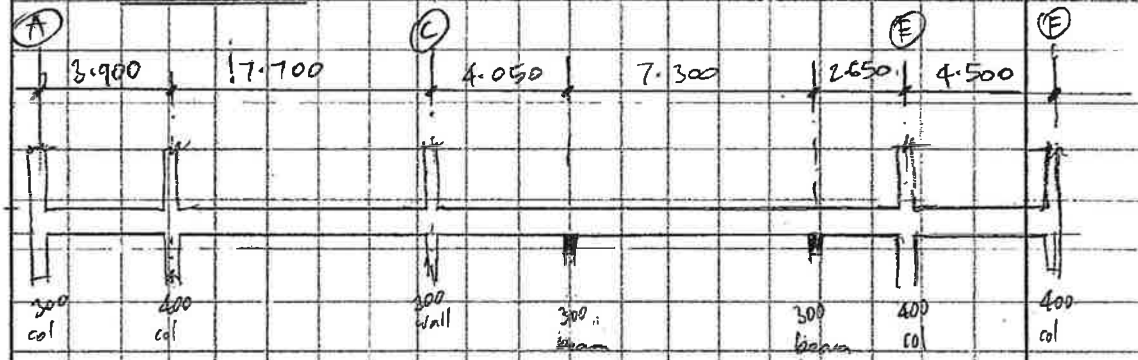
(C) Beam line 4

Dimensions:



width

= 400 | 860 | 400 | 400 | 860 | 860

Loads:

3·900 span : w_D = floor = 4·5 × 3·505 = 15·8
beam ·4 × ·55 = 5·2
21·0
w_L = 2·5 × 3·6 9·0
w_u = (1·4 × 21·0) + (1·7 × 9·0)
= 29·4 + 15·3 = 44·7 kN/m

7·700 span: w_D = Floor 15·8
beam ·86 × ·55 = 11·1
p/c panels 5·8
32·7
w_L = 9·2
w_u = 45·8 + 15·6 = 61·4 kN/m

4·05 & 7·30 } spans: w_D floor = 4·5 × 5·7^m = 25·7 kN/m

beam 5·2

30·9

w_L floor = 2·5 × 6·1 = 15·3

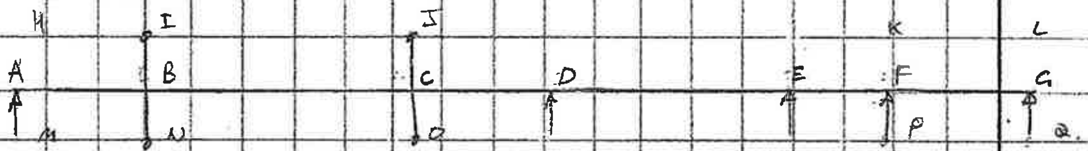
w_u = (1·4 × 30·9) + (1·7 × 15·3)

= 43·3 + 26·0

= 69·3 kN/m

2·6 & 4·5 } spans as for 7·7 span w_u = 61·4 kN/m

Summary



span	L	3·9	7·7	4·05	7·3	2·65	4·500
width	B	400	860	400	400	860	860
w_u		44·7	61·4	69·3	69·3	61·4	61·4
W_u		174	472	280	506	162	276
SSM	WL/8	85	455	142	462	54	155
FEM	WL/12	57	303	95	308	36	104

ΣW = 1870

550 / 860: I_g = 11·92 × 10^9 mm^4

○ 400: I_g = 1·26

400 / 300: $I_g = \frac{400 \times 300^3}{12}$ = 0·90

400 / 550: I_g = 5·50

S_{AH} = $\frac{3.0 \cdot 9}{3 \cdot 2}$ = ·84   0·06 Σ 0

S_{AB} = $\frac{4.11 \cdot 92}{3 \cdot 9}$ 12·22   ·88 Σ 1·0

S_{AM} ·84 / 13·9   ·06 Σ 0.

CALCULATIONS

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PAGE G 28.
SECT
FILE
DATE

$S_{BA} = \frac{3.5.5}{3.9}$ 4.23 $D_{BA} = 0.28$

$S_{BF} = \frac{3.1.26}{1.6}$ 2.36 $D_{BF} = 0.16$

S_{BN} 2.36 $D_{BN} = 0.16$

$S_{BC} = \frac{4.11.92}{7.7}$ 6.19 $D_{BC} = 0.40$

15.14

$S_{CB} =$ 6.19 $D_{CB} = 0.39$

S_{CJ} 2.36 $D_{CJ} = 0.14$

S_{CO} 2.36 $D_{CO} = 0.14$

$S_{CD} = \frac{4.5.5}{4.05}$ 5.43 $D_{CD} = 0.33$

16.34

$S_{DC} =$ 5.43 $D_{DC} = 0.64$

$S_{DE} = \frac{4.5.5}{7.3}$ 3.01 $D_{DE} = 0.36$

8.44

$S_{ED} =$ 3.01 $D_{ED} = 0.14$

$S_{EF} = \frac{4.11.92}{2.65}$ 17.99 $D_{EF} = 0.86$

21.00

$S_{FE} =$ 17.99 $D_{FE} = 0.69$

$S_{FG} = \frac{3.1.92}{4.5}$ 7.95 $D_{FG} = 0.31$

25.94

BUI.MAD249.0273.30

CALCULATIONS

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PAGE G 28.
SECT
FILE
DATE

$S_{BA} = \frac{3.5.5}{3.9}$ = 4.23 D_{BA} = 0.28

$S_{BF} = \frac{3.1.26}{1.6}$ 2.36 D_{BF} = 0.16

S_{BN} 2.36 D_{BN} = 0.16

$S_{BC} = \frac{4.11.92}{7.7}$ 6.19 D_{BC} = 0.40

13.14

S_{CB} = 6.19 D_{CB} = 0.39

S_{CJ} 2.36 D_{CJ} = 0.14

S_{CO} 2.36 D_{CO} = 0.14

$S_{CD} = \frac{4.5.5}{4.05}$ 5.43 D_{CD} = 0.33

16.34

S_{DC} = 5.43 D_{DC} = 0.64

$S_{DE} = \frac{4.5.5}{7.3}$ 3.01 D_{DE} = 0.36

8.44

S_{ED} = 3.01 D_{ED} = 0.14

$S_{EF} = \frac{4.11.92}{2.65}$ 17.99 D_{EF} = 0.86

21.00

S_{FE} = 17.99 D_{FE} = 0.69

$S_{FG} = \frac{3.1.92}{4.5}$ 7.95 D_{FG} = 0.31

25.94

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CALCULATIONS

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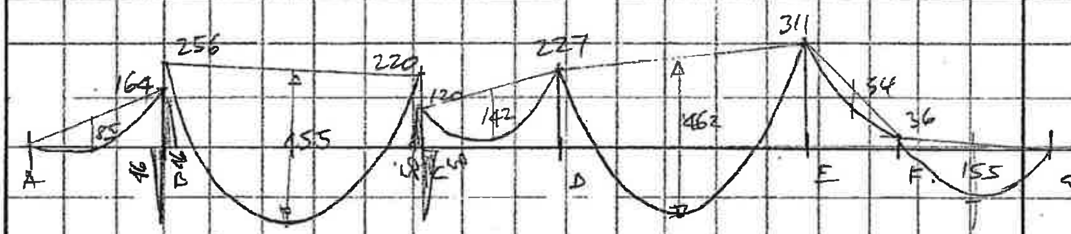
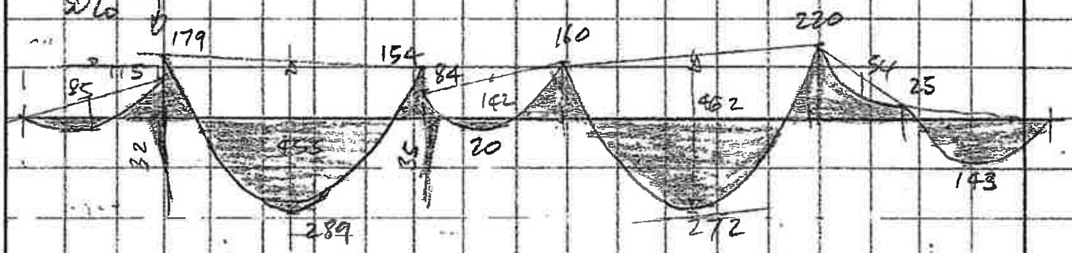
PAGE G 29
SECT
FILE
DATE

for Full DL+LL

AB	BA	BI	BU	BC	CB	CJ	CO	CD	DC	DE	ED	EF	FE	FG
	·28	·16	·16	·40	·39	·14	·14	·33	·64	·36	·14	·86	·69	·31
	85			-303	303			-95	95	-308	308	-36	36	
	61	35	35	87	-81	-29	-29	-69	136	77	-38	-234	-25	-11
				-40	44			68	-35	-19	-39	-12	-117	
	11	6	6	17	-44	-16	-16	-36	35	19	-4	-23	81	36
				-22	9			18	-18	-2	10	40	-12	
	6	4	4	8	-10	-4	-4	-9	13	7	-7	-43	8	4
				-5	4			7	-5	-4	4	4	-22	
	1	1	1	2	-5	-1	-1	-4	6	3	-1	-7	15	7
	164	46	46	-256	220	-50	-50	-120	227	-227	311	-311	-36	36



redistribute
-30%



↑87	87 ↑ ↑236	236 ↑ ↑140	140 ↑ ↑253	253 ↑ ↑81	81 ↑ ↑138	↑138
↓29	29 ↑ ↑3	3 ↓ ↓19	19 ↑ ↓8	8 ↑ ↑73	73 ↑ ↑6	6
↑58	116 ↑ ↑239	233 ↑ ↑121	159 ↑ ↑245	261 ↑ ↑154	8 ↑ ↑144	↑132
↑58	↑355	↑354	↑404	↑415	↑152	↑132

Σ 1870 ok

CALCULATIONS

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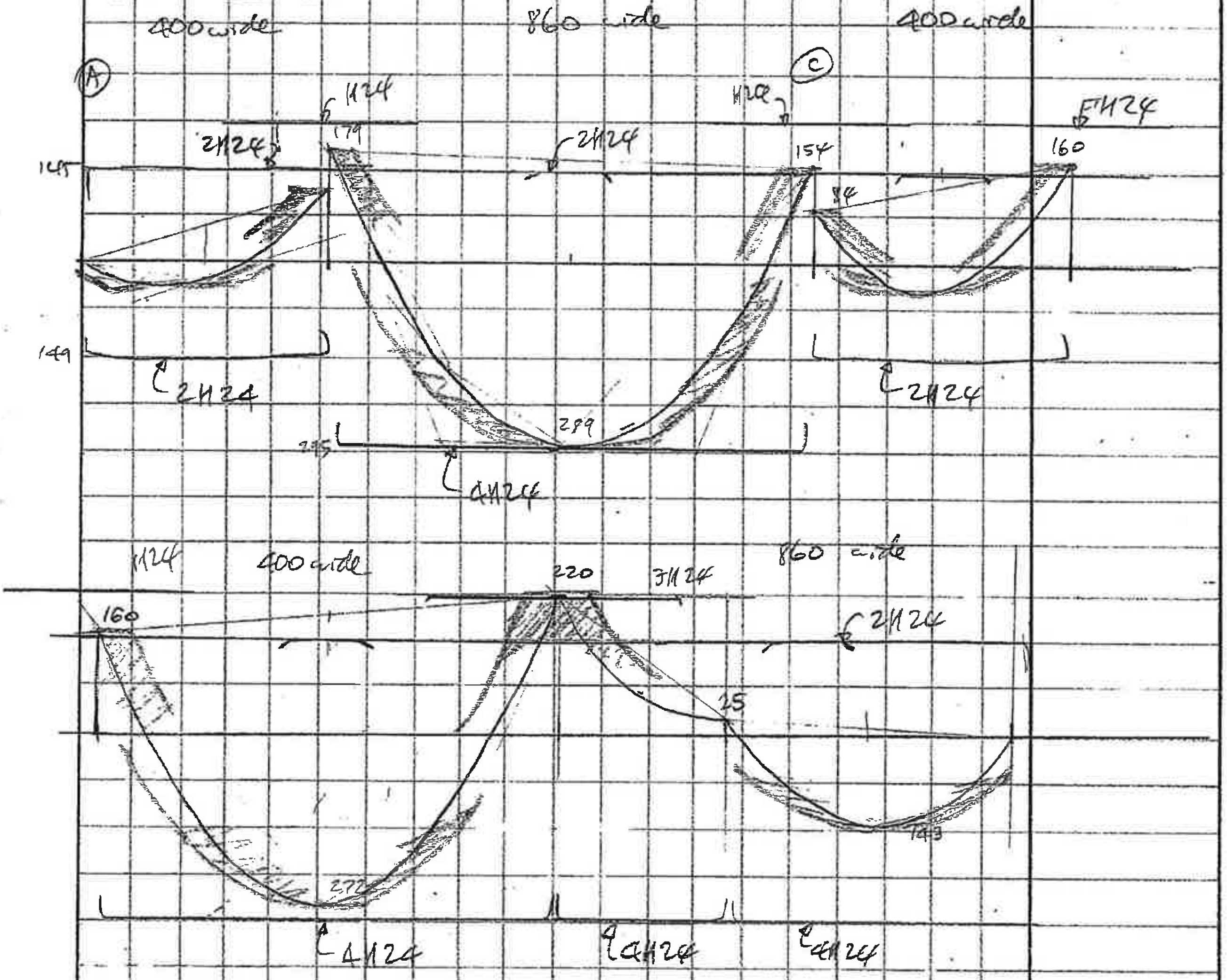
PAGE G30
SECT
FILE
DATE

Flexural reinforcement:



CALCULATIONS

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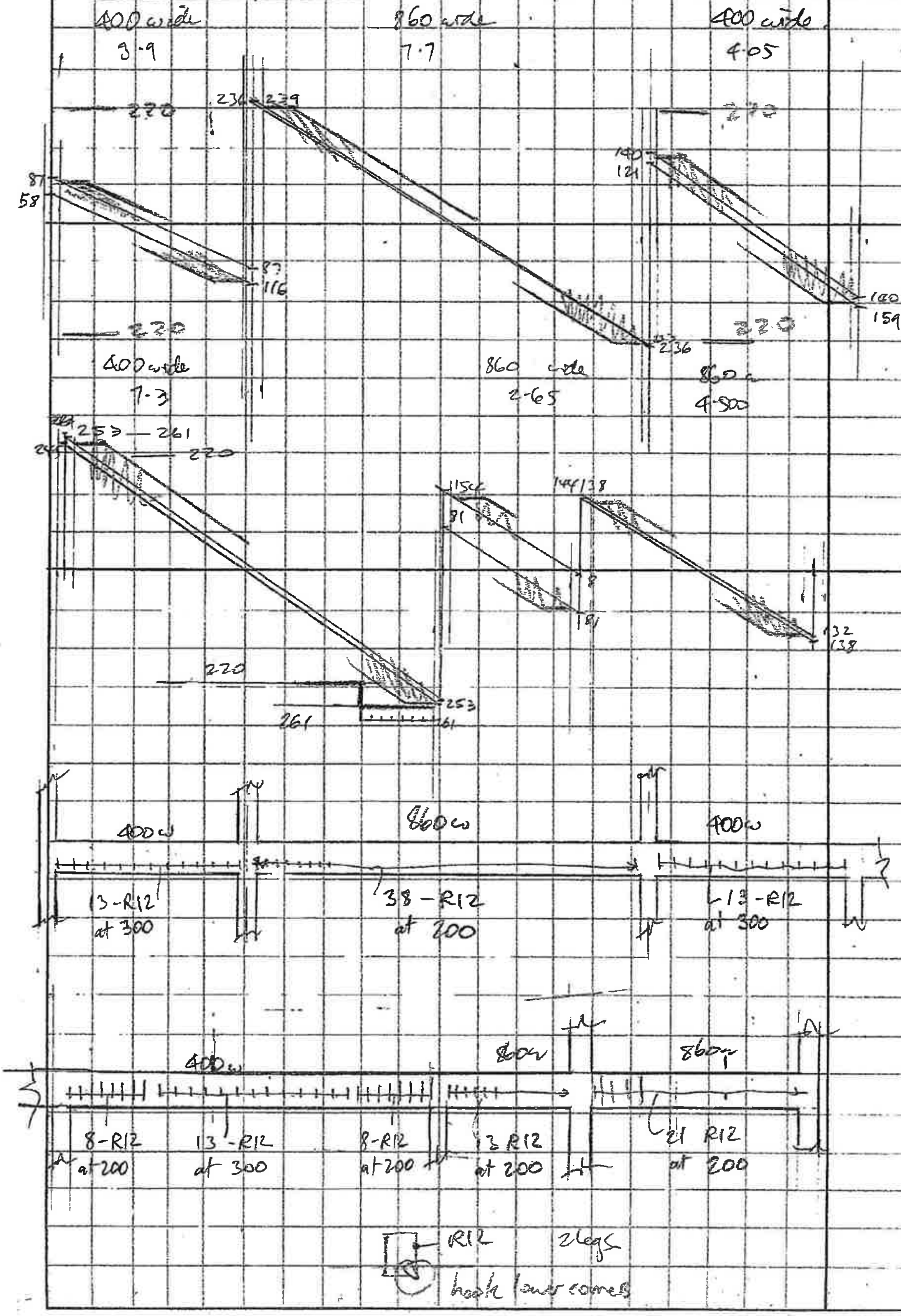
PAGE 431
SECT
FILE
DATE

Shear Reinforcement



CALCULATIONS
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400 wide beams

$V_u\ max = 254^{kN}$

$v_u = \frac{254,000}{.85 \times 400 \times 486} = 1.54\ ^{Mpa}$

$\rho = \frac{1809}{400 \times 486} = .0093,\ v_c = .81,\ V_c = 134^{kN}$

$v_{s\,max} = 1.54 - .81 = 0.73\ ^{N/mm^2}$

For R12 - 200 $v_s = 0.77$ $V_s = 127$, V_{s} = 261
R12 - 300 $v_s = 0.52$ $V_s = 86$ = 220

860 wide beams

$V_u\ max = 234^{kN}$

$v_u = \frac{234,000}{0.85 \times 860 \times 486} = 0.65$

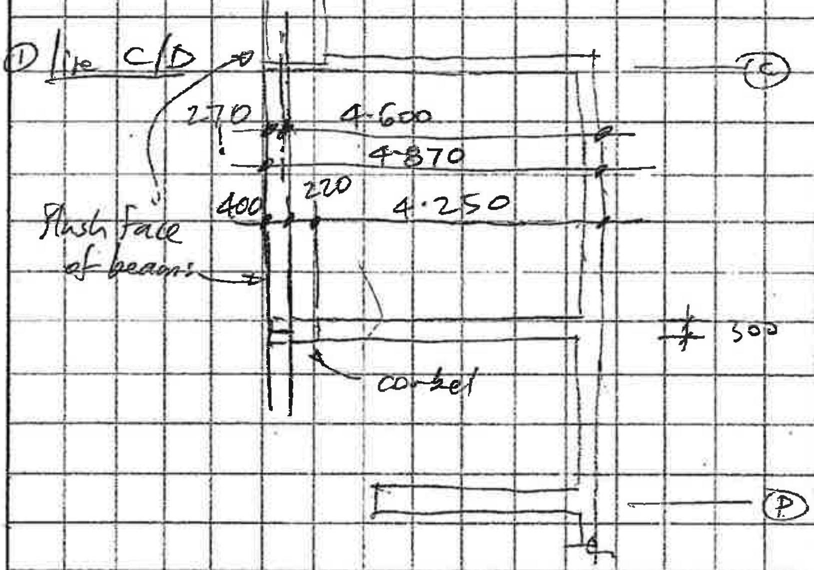
$v_c = 0.57$

$v_{s\,min}$, R12 at 200 $= 0.36$

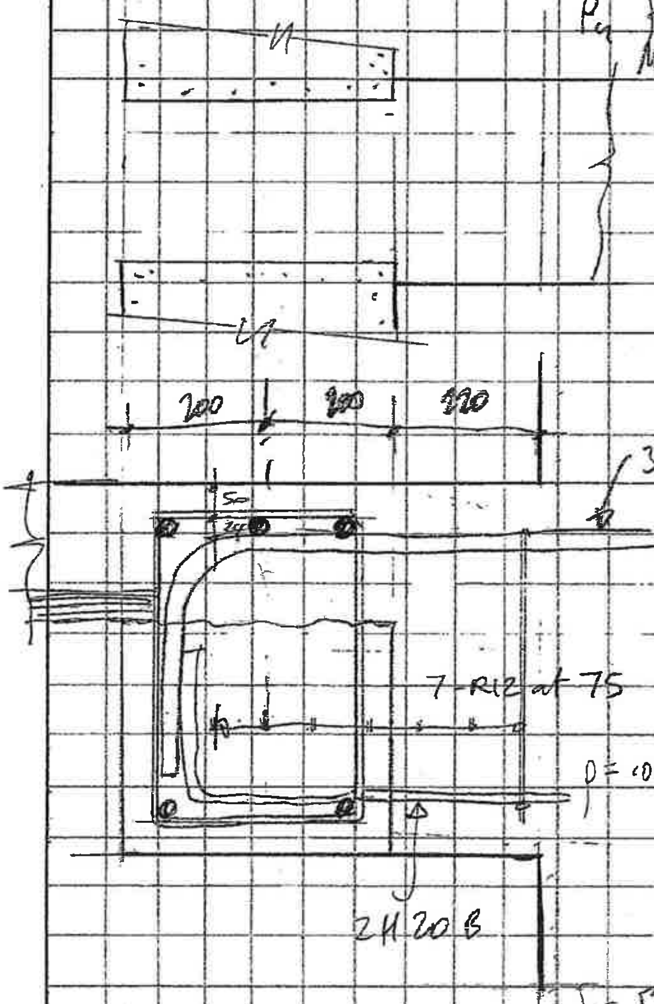
$.93 > .65$

Use R12 at 200 throughout.

Corbels to support beams-



PLAN



P_u from p. G29 = 404 kN

M_u = 404 × .42 = 170 kNm

3H24 T A_s = 1356

b = 300

a = 80

d = 550-(74+12)

= 464

jd = 424

M_u = 197 kNm

> 170.

$v_i = \frac{404,000}{.85 \times 300 \times 464}$

= 3.41 N/mm²

< 5.0 ok

ρ = .0097 v_b = 0.83

v_s = 3.41 - .83

= 2.58 N/mm²

$A_v = \frac{2.58 \times 300 \times 1000}{275}$ = 2815

for [] R12 at 75 A_v = 3013

CALCULATIONS

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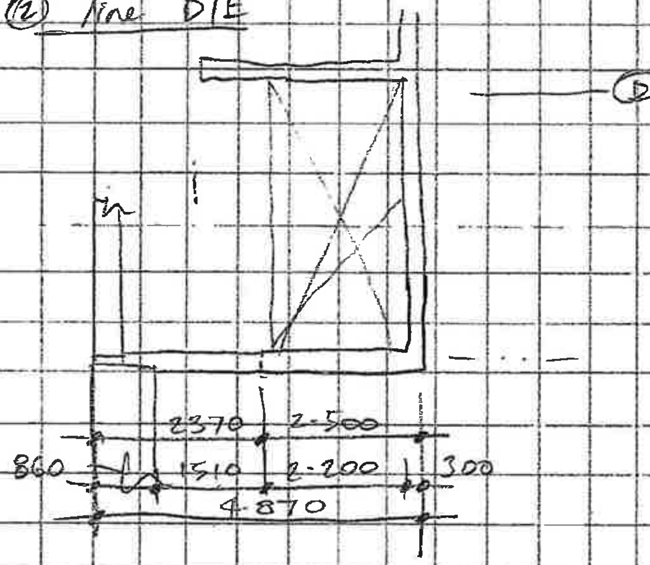
(12) line D/E



P_u from p. G29 = 415 kN.

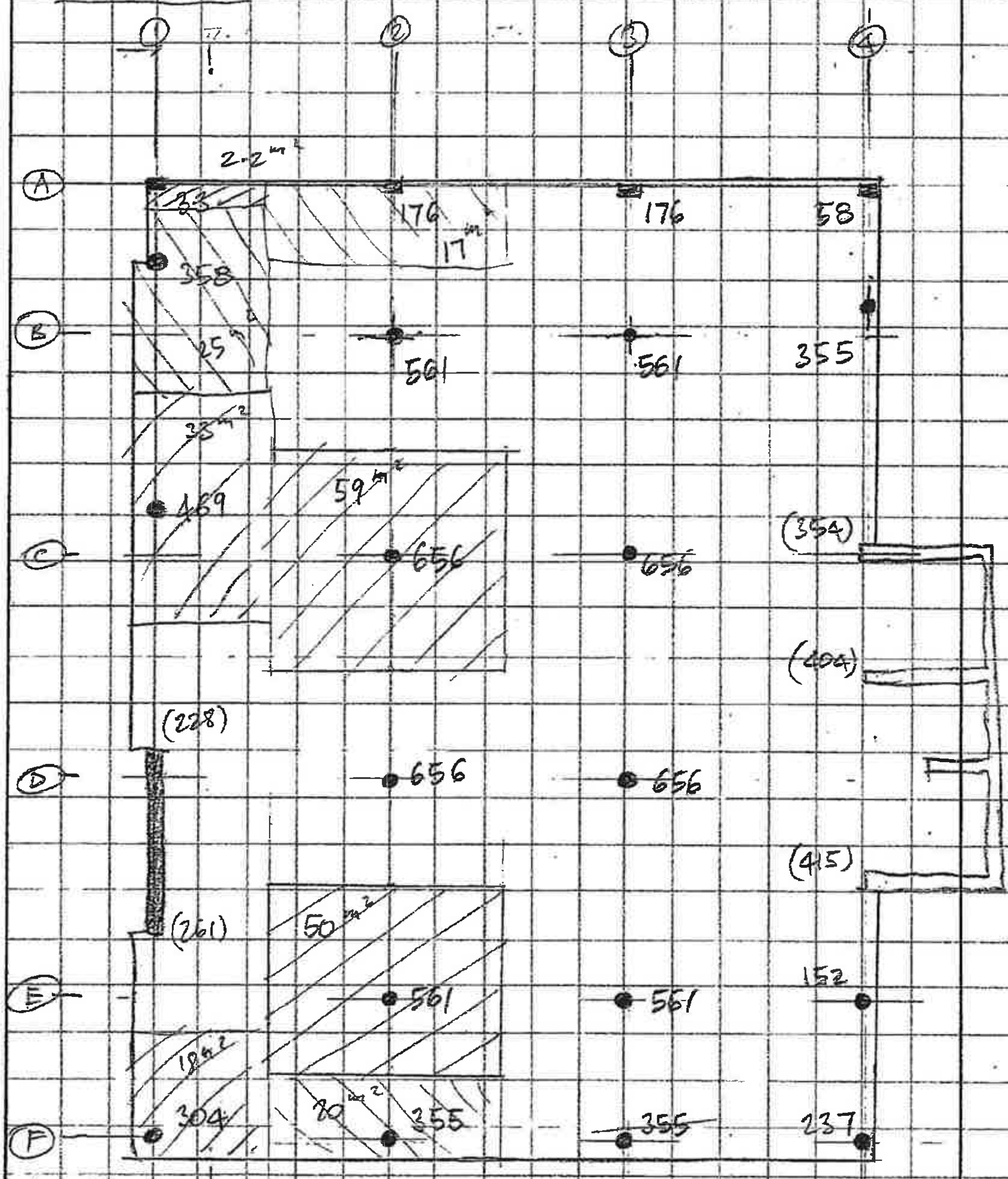
e = 2370 − 200 = 2170 m

CLM = M_n = 415 × 2.17 = 900 kNm high.

use column below

Columns:

Dimensions:



Loads:

P_u From typical floor slabs as shown'
maxima, summarised from pages G12, G20, G29

column	F1	$P_u = 199 + (23.8 \times 4.4)$ =	304
column	F2 & F3,	$P_u = 176 + (23.8 \times 7.5)$ =	355
column	F4	$P_u = 132 + (23.8 \times 4.4)$ =	237

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Elevation:

R — 20·600
5 — 16·200
4 — 12·960
3 — 9·720
2 — 6·480
1 — 3·240
G

Columns C2, C3, D2, D3

Level supported	P_D	P_L	ΣA m^2	R	P_{LR}	P_D incl col wt	$P_D + P_{LR}$	P_u = $1.4 P_D + 1.7 P_{LR}$	design f'_c
R	24	30	59	0·7	21	32	53	80	25
5	314	177	118	·56	99	330	429	630	25
4	604	324	177	·52	168	628	796	1165	25
3	894	471	236	·49	231	926	1157	1689	30
2	1184	618	295	·47	290	1224	1514	2207	35
1	1474	765	354	·46	352	1522	1874	2729	40

roof, $P_D = 59 m^2 \times .40 = 24$, $P_L = .5 \times 60 = 30^{kN}$

each floor $P_D = 290 \times 1.4 = 406$
$P_L = 147 \times 1.7 = 250$
437 656 kN

area supported $= \frac{656}{92.9} \times 8.31 m^2 = 59 m^2$

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Column braced against sidesway, $k = 1.0$

NZS 3101, (6.4.11.2)

$r = 0.25 \times 400 = 100^{mm}$

$l_u = 3240 - 550 = 2690$

$\therefore \frac{k l_u}{r} = \frac{1.0 \times 2690}{100} = 26.90$

$M_1 = -M_2 = 5^{kNm}$ from p. G12

$34 - 12\frac{M_1}{M_2} = 34 + 12 = 46$

ie $\frac{k l_u}{r} = 26.90 < 34 - 12\frac{M_1}{M_2} = 46$

$\therefore$ neglect effects of slenderness

$e_{min} = 15 + .03 \times 400 = 27^{mm}$

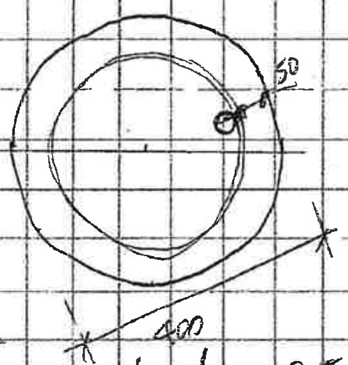
design for $\phi = 0.7$

$D = 400$

$gD = 400 - (80 + 28)$

$= 292$

$g = 0.73$ use 0.70



for chart C5.6, 380/0.7

at Ground to 1st floor

$P_u = 2729^{kN}$, $\frac{P_u}{f'_c D^2} = \frac{2729000^{N}}{25 \times 400^2} = 0.68$

$M_u = P_u e_{min}$

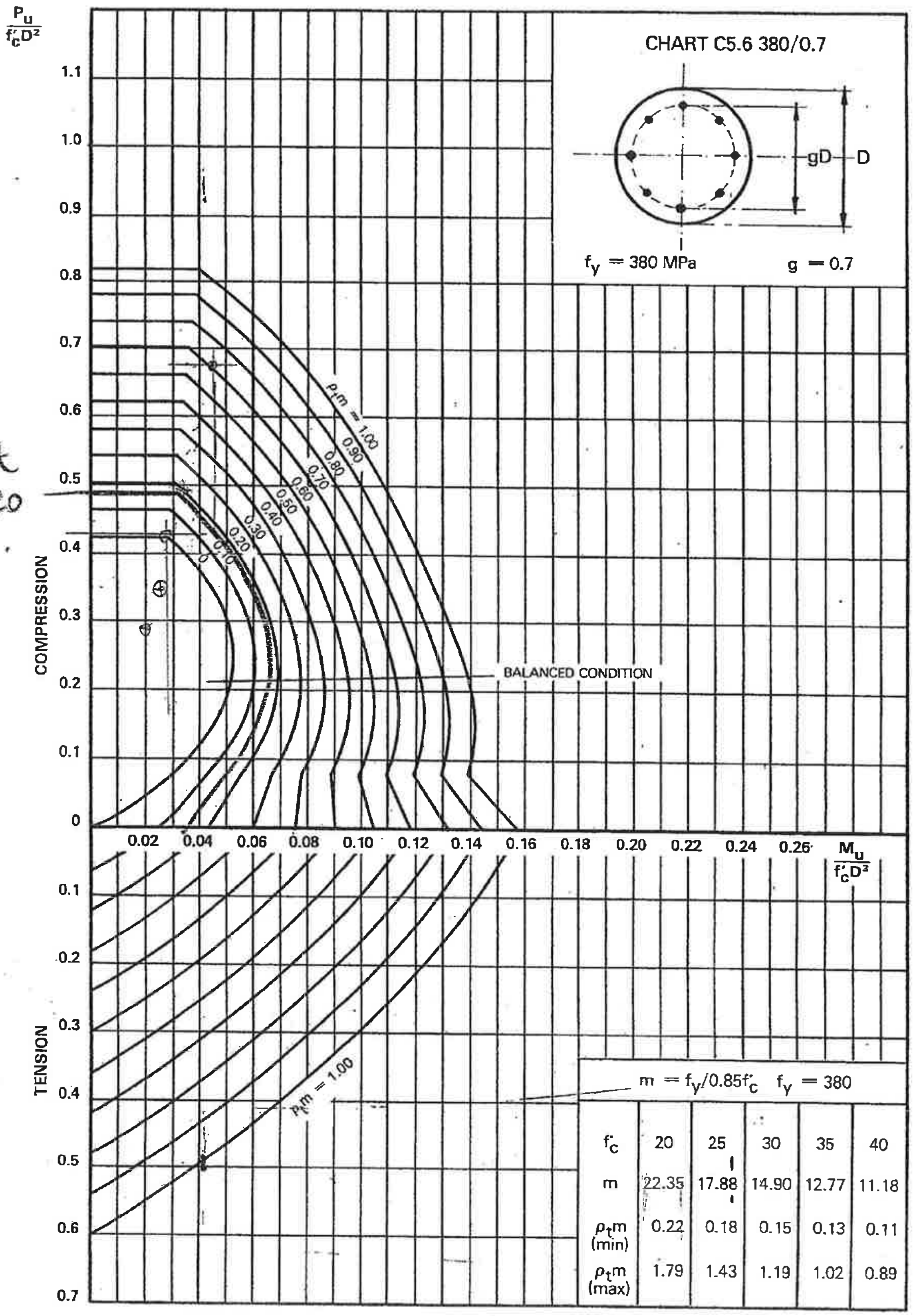
$= 2729 \times .027$

$= 74^{kNm}$ $\frac{M_u}{f'_c D^3} = \frac{74,000,000}{25 \times 400^3} = 0.046$

$\Rightarrow p_t m = 0.70$ $m = 17.88$

938

C5.6 COLUMN DESIGN CHART



$m = f_y/0.85f'_c$ f_y = 380

f'_c	20	25	30	35	40
m	22.35	17.88	14.90	12.77	11.18
$\rho_t m$ (min)	0.22	0.18	0.15	0.13	0.11
$\rho_t m$ (max)	1.79	1.43	1.19	1.02	0.89

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PAGE G39
SECT
FILE
DATE

$pt = \frac{0.70}{17.88} = 0.039$

$A_s = 400^2 \times \frac{\pi}{4} \times .039$

$= 4900\ mm^2$

6H32 $= 4830\ mm^2$

Compare using 40 MPa concrete 35 MPa

$\frac{P_u}{f'_c D^2} = \frac{2729000}{40 \times 400^2} = 0.43$ ·49 G to 1st f'_c 40 MPa

$\frac{M_u}{f'_c D^3} = \frac{74 \times 10^6}{40 \times 400^3} = 0.029$ ·033 6H20.

$\Rightarrow ptm = 0.09 < ptm$ min. 1st & 2nd

$m = 11.18$ $f'_c = 35$

$pt = .008 < 0.01$ 6H20.

$A_s = 0.01 \times 125,663$
$= 1257\ mm^2$

for 6H20, $A_s = 1880\ mm^2$

$pt = 0.015$

$ptm = .015 \times 11.18$
$= 0.167$.

1st to 2nd Fl

use 35 MPa concrete

$\frac{P_u}{f'_c D^2} = \frac{2207000}{35 \times 400^2} = 0.39$

$\frac{M_u}{f'_c D^3} = \frac{2207 \times 27,000}{35 \times 400^3} = 0.027$

$\Rightarrow ptm < 0.0$

$pt = 0.010$ use 6H20.

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PAGE G 40
SECT
FILE
DATE

2nd to 3rd fl

30 MPa concrete

$\frac{P_u}{f'_c D^2} = \frac{1,689\,000}{30 \times 400^2} = 0.35$

$M_u = 1689 \times .027$

$= 45.6$ kNm

$\frac{M_u}{f'_c D^3} = \frac{45.6 \times 10^6}{30 \times 400^3} = 0.024$

p_{t_m} 20.0 use 6H20.

3rd to 4th

25 MPa concrete

$\frac{P_u}{f'_c D^2} = \frac{1,165,000}{25 \times 400^2} = 0.29$

$\frac{M_u}{f'_c D^3} = \frac{31.45 \times 10^6}{25 \times 400^3} = 0.020$

$< p_{t_{min}}$ use 6H20.

Columns B2, B3, E2, E3

P_u from typ floor slab = 561 kN

Floor area supported $= \frac{561\text{ kN}}{92.9\text{ kN/m}} \times 8.31 = 50$ m²

typical floor $P_D = 248 \times 1.4 = 347$

$P_L = \frac{126}{374} \times 1.7 = \frac{214}{561}$

at ground floor $\Sigma A = 6 \times 50 = 300$, $R = .47$

$P_D = (248 \times 5) + (50 \times .4) + (6 \times 8) = 1308$

$P_{LR} = 0.47\left[(5 \times 50) + (126 \times 5)\right] = \frac{308}{1616}$ kN

B2, etc same col as C2 etc

$P_u = (1.4 \times 1308) + (1.7 \times 308)$

$= 2355$ kN

M_u from page G12 = 23 kNm

$P_u e = 2355 \times 0.027 = 64$ kNm $> M_u$.

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PAGE G41

SECT

FILE

DATE

→ column design as for C2 OK by inspection

Columns B1, C1, F1, F2, F3 F4 E4, B4

Column No.	P_L typ. floor	P_D typ fl	P_L typ fl.	A m^2	ΣA m^2	R	P_D grd fl incl cols	P_{CR}	P_D +P_{CR}	P_u =1.4D +1.7L	M_u kNm	P_{ue}
B1	358	190	53	25	150	·53	1430	147	1577	2252	15	61
C1	469	250	70	33	198	·50	1314	183	1497	2151	3	58
F1	304	181	30	18	108	·58	962	92	1054	1503	23	40
F2 F3	355	205	40	20	120	·56	1083	118	1201	1717	19	46
F4	237	145	20	18	109	·58	782	63	845	1202	–	32
E4	152	84	20	18	108	·58	477	63	540	775	–	21
B4	355	229	20	25	150	·53	1205	53	1258	1777	32	48

Column No	$\frac{P_u}{25 \times 400^2}$	$\frac{P_{ue}}{25 \times 400^3}$	p_{tm}	p_t	$\frac{P_u}{35 \times 400^2}$	p_{tm}	p_t
B1	·563	·038	·40	·022	·402	0.0	
C1	·54	·036	·34	·019	·39	0.0	
F1	·38	·025	0				
F2, F3	·43	·029	0·03	·0017	·307	0·0	
F4	·30	·020	0				
E4	·19	·013	0				
B4	·44	·030	0·05	·003	0·31	0·0	

grd fl G-1
35 MPa

next 1-2
30

others 2-R
25 MPa.

p_t min = 0·01

6H20, p_t = 0·015

insufficient

∴ Use f'_c = 35 MPa & As min ie 6H20.

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PAGE G41A.
SECT
FILE
DATE

Column hoops :

min bar dia 5.3.29.2 = 6 mm

all loops R6 at 250. spiral.

ϕ = 0.7

non-seismic loading case 6.4.7.1

max spacing = 400 mm

16 x 20 = 320.

48 x 6 = 288

use R6 at 250.

compare seismic loading case in potential p'hinge regions.

$0.45\left(\frac{A_g}{A_c}-1\right) = 0.45\left(\frac{400^2}{300^2}-1\right) = 0.35 > .12$

f'c = 35

$\rho_s = 0.35 . \frac{35}{275}\left(0.5+1.25.0\right) = 0.0223$

$\therefore A_{st} = .0223 \times \frac{\pi . 300^2}{4} = 1576 \ mm^2/m$

R10 at 100 A_{st} = 1570

for O R6 at 80 A_{st} = 1413

max spacing s/f = 400/5 = 80

(ii) 6 x 20 = 120

(iii) = 200

- these do not apply as columns are non-seismic.

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Columns A1, A2, A3, A4

For columns A2 & A3

Level Supported	P_D	P_L	ΣA m^2	R	P_{LR}	P_D incl wall & col wt	$P_D + P_{LR}$	$P_u = 1.4P_D + 1.7P_{LR}$
R	7	9	17	1.0	9	25	34	50
5	85	48	34	·82	39	121	160	236
4	163	87	51	·72	63	217	280	411
3	241	126	68	·65	82	313	395	578
2	319	165	85	·62	102	409	511	746
1	397	204	102	·59	120	505	625	911

roof, $P_D = 17 \times ·4 = 7$ $P_L = 17 \times ·5 = 9$

floor $P_D = 78 \times 1·4 = 109$

$P_L = 39 \times 1·7 = 67$

117 176

wall wt. $1·0$ kN/m $\times 8·5^c = 9·0$ kN

col wt $= ·4 \times ·3 \times 3·2 \times 23·5 = 9·0$

18·0 kN.

Column size: 400 × 300.

78
39

braced against sidesway $k = 1·0$

$r = 0·3 \times 300 = 90$

$l_u = 3240 - 350 = 2990$

$\therefore \frac{kl_u}{r} = \frac{1·0 \times 2990}{90} = 33·2$

$M_1 = -M_2 = 19$ kNm From p. G12

$34 - 12\frac{M_1}{M_2} = 34 + 12 = 46$

$\therefore \frac{kl_u}{r} = 33·2 < 34 - 12\frac{M_1}{M_2} = 46$

∴ neglect effects of slenderness.

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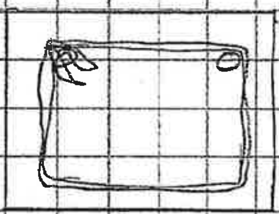
PAGE 443
SECT
FILE
DATE

$e_{min} = 15 + \cdot 03 \times 300$
$= 15 + 9$
$= 24$

$M_u \doteq P_u e_{min}$ at

grd Fl $= 1770^{kN} \times \cdot 024$
$= 42\cdot5^{kNm} > 19^{kNm}$ - governs.



50
12
62
176
300
62

$D = 300$
$gD = 176$
$g = \cdot 58$
$\approx \cdot 60$

$\phi = 0{\cdot}70$
for chart C5·3 380/0·6

$\frac{P_u}{f'_c bh} = \frac{1770{,}000^{N}}{40 \times 300 \times 400} = 0{\cdot}37$

$\frac{M_u}{f'_c bh^2} = \frac{42{,}500{,}000}{40 \times 400 \times 300^2} = 0{\cdot}030$

A1, A2, A3, A4

ie $pt_m = 0$
$pt_m\ m_i = 0{\cdot}11$, $m = 11$, $pt_{min} = 0{\cdot}01$

400x300

$A_{s\,min} = 300 \times 400 \times \cdot 01 = 1200^{mm^2}$

$4H20$, $A_s = 4 \times 314 = 1260^{mm^2}$

4 H20 rods

for $f'_c = 30$ $\quad \frac{P_u}{f'_c bh} = 0{\cdot}49$

grd Fl $f'_c = 30$

$\frac{M_u}{} \quad 0{\cdot}04$

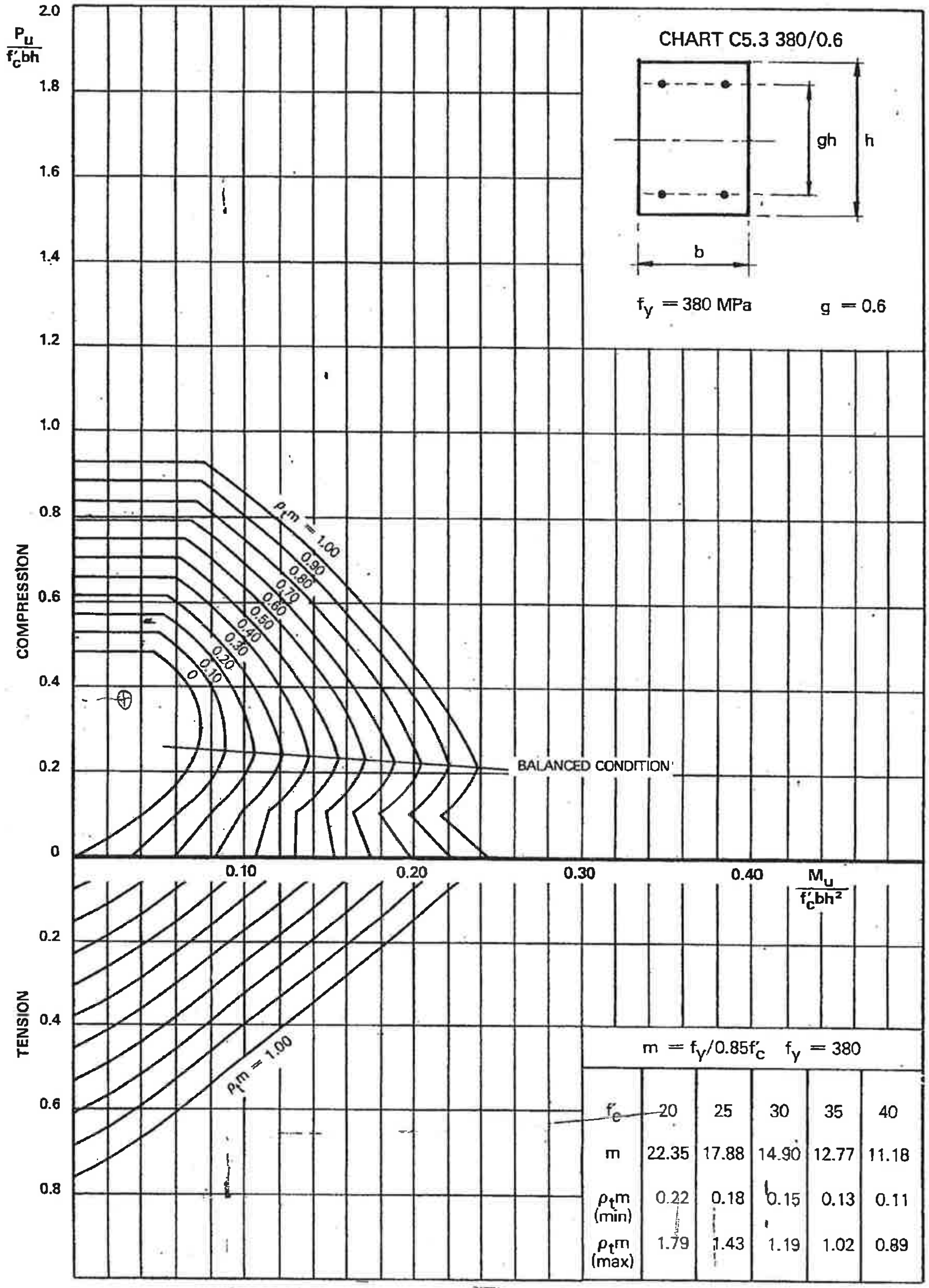
1st 2nd above $f'_c = 25$.

$pt_m = 0 \Rightarrow 4H20$ still

1st Fl $\quad f'_c = 25$, $\frac{P_u}{f'_c bh} = \frac{1441000}{25 \times 400 \times 300} = \cdot 48$ OK

944

C5.3 COLUMN DESIGN CHART



$m = f_y/0.85f'_c$ $f_y = 380$

f'_c	20	25	30	35	40
m	22.35	17.88	14.90	12.77	11.18
$\rho_t m$ (min)	0.22	0.18	0.15	0.13	0.11
$\rho_t m$ (max)	1.79	1.43	1.19	1.02	0.89

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Roof level.

if $P_u = 0$.

BM capacity of section : $A_s = 2H20 = 628$

$a = 28$

$jd = 300 - (14 + 62)$

$= 224$

$M_u = .9 \times 628 \times 380 \times 224$

$= 48.1^{kNm} > 19^{kNm}$ ok.

Ties :

max spacing $= b = 300$

$16 \times 20 = 320$

$48 \times 6 = 288$ $\quad$ $48 \times 10 = 480$

use ~~R6 at 250~~.

max tie size for 20^{ϕ} rods $= 10^{mm}$

use R10 at 300.

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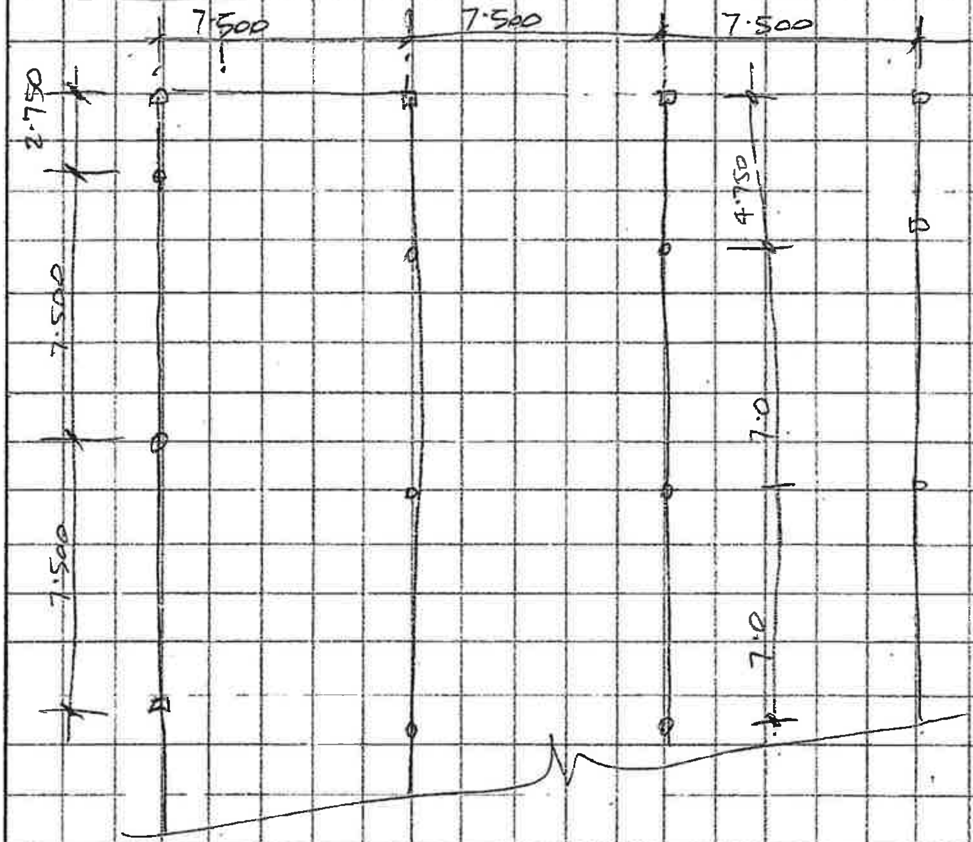
PAGE G46
SECT
FILE
DATE 16

Roof Framing.

Dimensions:



Loads:

DL 0·40 mm Trimdek = 0·050
BP & netting = 0·010
BP 200/19 at 1200 ·055/1·2 = 0·050
ceiling pier 0·050
insulation 0·010
0·170

SL Z4, Cr = 1·0 h < 100 m 0·480
WL V = 40 m/s
S = 1·0
GR 3, B, 20 m S_2 = ·90
V_s = 36 m/s q = ·794

max uplift = 1·2q = = 0·953 kPa

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PAGE	G47
SECT	
FILE	
DATE	

Purlins: spacing = 1·200 span = 7·500 − ·082 = 7418.

inward load

w_D = 0.17 x 1.2 = ·204

w_S = ·48 x 1·2 = ·576

w_{D+S} = ·78 kN/m ↓

outward load

w_w = ·953 x 1·2 = 1·14 kN/m

$w_{·7D-w}$ = 0.14 − 1·14 = −1·00 kN/m ↑

for BP 200/19 7500 span

inward cap, fully restrained = 1·57 > ·78

2 rows braces = 1·24 > ·78

outward capacity, 2 braces = 1·34 > 1·0.
(x 1·33)

deflection @ ·006 = 0·90

$defl_w = \frac{5}{384} \times \frac{1·14 \times 7·418^4 \times 10^3}{200 \times 4·373}$ 51·4 mm

·006 x 7500 = 45 mm.

for BP 200/25 7500 span.

inward cap fully restrained = 2·19

1 row braces = 0·82 > 0·78

outward cap 1 row braces = 1·05 x 1·33 > 1·0.

deflection = $\frac{51·4 \times 4·373}{5·800}$ = 38·8 mm < 45 mm

= ·0052 ℓ ok

Purlins
BP 200/25
1 row braces.

@ 1200 crs

Rafters

(A) Central rafters: max span = 7·000

spacing = 7·500.

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$w_D = (0.170 \times 7.5) + .25 = 1.53$ kN/m
$w_S = 0.48 \times 7.5 \qquad 3.60$

$w_{D+S} = 5.13$ kN/m ↓

$w_W = 0.953 \times 7.5 = 7.15$
$w_{.7D-W} = 1.07 - 7.15 = 6.08$ kN/m
$\div 1.33$ for equiv. stresses $= 4.56$ kN/m ↑

Design for continuity to keep deflections down.

max. SSM $= 0.125\, w l^2$
$= 0.125 \times 5.13 \times 7^2$
$= .125 \times 251$ kNm
$= 31.4$ kNm

max BM if continuous, neg BM $= 0.086 \times w l^2$
from p. G10 $= .086 \times 251$
$= 21.6$ kNm

$Z_{min} = \frac{21600}{162} = 133$ cm^3

strength method: (1) $w_u = 1.4(D+S)$
$= 1.4 \times 5.13 = 7.18$

(2) $w_u = 0.9D + 1.3W$
$= 1.38 - 9.30 = 7.92$ kN/m

$M_u = \frac{w_u l^2}{16} = \frac{7.92 \times 7^2}{16} = 24.3$ kNm

$S_{min} = 24.3 / .25 = 97$ cm^3

for $178 \times 89 \times 22$ kg/m RSJ $Z = 168$
$S =$

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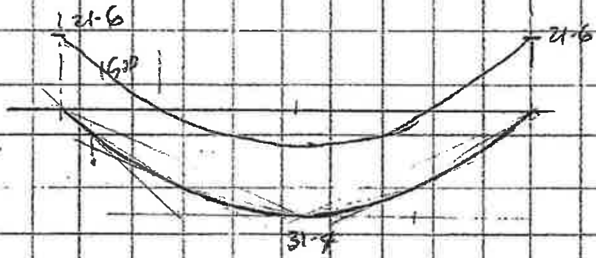
PAGE 649
SECT
FILE
DATE



max neg BM ∴ $f_b = \frac{21600}{168} = 128$ N/mm^2

p_b: $l/r = \frac{0.85 \times 1500}{18.8} = 68$ } $p_b = 160$ OK.

D/T = 17·6

pos BM OK restrained by purlins.

deflection: DL $\delta_{ss} = \frac{5}{384} \cdot \frac{1.53 \cdot 7^4 \cdot 10^3}{200 \times 14.9} = 16.0$

SL 3·6 = 38·0

WL 7·15 = 75·0 mm high

·006 ℓ = 6 × 7·0 = 42 mm

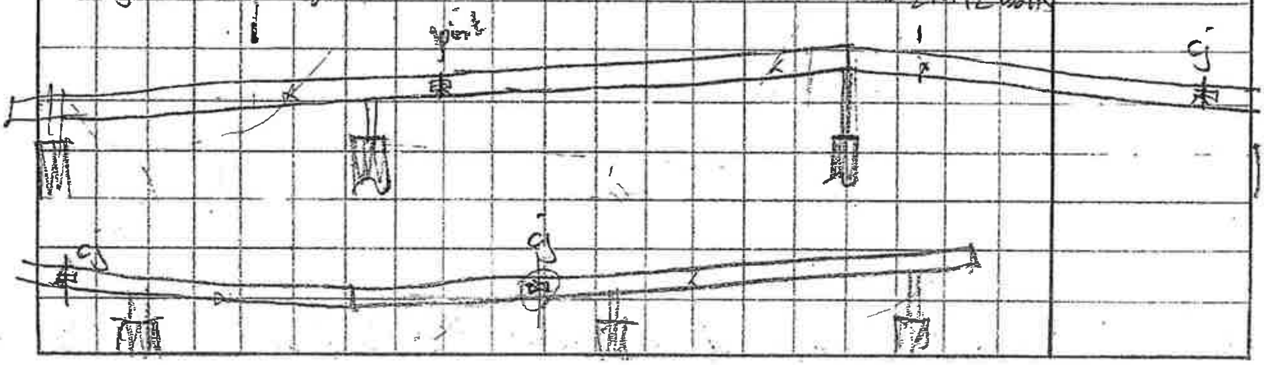
If continuous δ = $\frac{75}{5}$ = 15·0 mm OK

if SS use 200UB25, I = 23·6 δ_{DL} = 10·1

δ_{SL} = 23·8

δ_{WL} = 47·2 mm

joint at p of C's: shear = 5 kN/m × 3 = 15 kN
→ 2M12 bolts

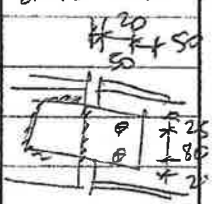


Rafter
178×89
×22 kg/m
RSS.
continuous

or

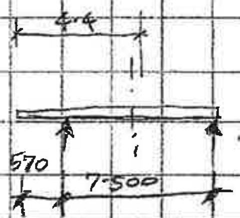
~~200UB 25 SS at columns.~~

130×10 Plate
2M12 bolts.



(B) Perimeter rafters

max span = 7.0 m
width supp : 4.4 m



Loads : DL self wt rafter 0.22
roof purlins etc = .17 x 4.4 = 0.75
10 mm fibrolite .153 x 4.0 m = 0.61
framing 0.3 kPa x 3.6 1.08

2.66

LL 0.48 x 4.4 2.11

DL+LL 4.77 kN/m

cf 5.13 kN/m for internal frames - use the same size.

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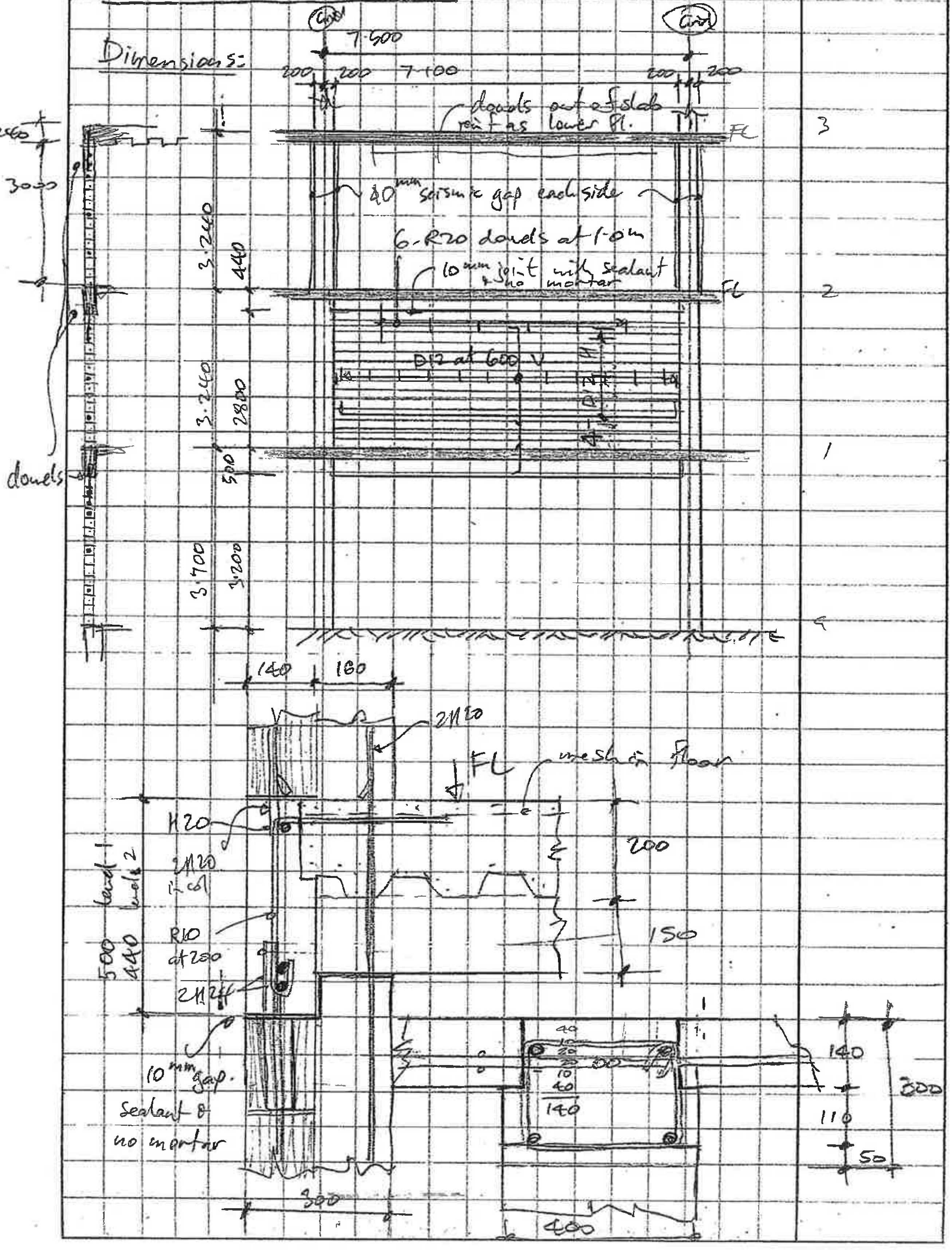
Blockwalls on line A

Dimensions:



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Loads:

face loads: Wind : $V = 40$
$S_1 = 1.0$
GR3, B, 10^m $S_2 = .74$
$V_s = 29.6$ m/s
$q = 0.537$
$w = 1.3q = 0.700$ kPa
$w_u = 1.3w = 0.91$ kPa

EQ : 140 wall filled solid
$w = .14 \times 23.5 = 3.29$ kPa
$S_p = 1$ $C_{pmax} = 0.5 \times 5/6$
$= 0.42$
ie $w_u = .42 \times 3.29$
$= 1.38$ kPa

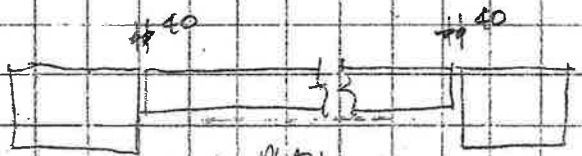
Simply supported between floors.
$SSM = \frac{1.38 \times 3.04^2}{8} = 1.59$ kNm

140^mm wall with D12 at 800
$A_s = 141$ mm²/m , $a = 5.7$ $d = 70$ $jd = 67$
$M_u = .85 \times 141 \times 275 \times 67$
$= 2.21$ kNm/m > 1.59 ok

Seismic clearances:
interstorey deflection

2 to 3	= 10.8	× K/SM (= 1.78)	= 19.2	× 2	= 38.4
1 to 2	= 8.0		= 14.2		= 28.4
G to 1	= 4.3		= 7.7		= 15.4
		(3.8.1)		(3.8.4.2a)	

design all walls for 40^mm clearance



PLAN

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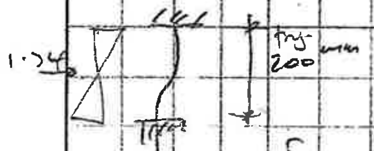
PAGE G53
SECT
FILE
DATE

check loads on D12 dowel, with shear load from wall: $P_u = 1.38^{kPa} \times 0.6^{m} \times \frac{3.24}{2}$

$= 1.34^{kN}$



$BM = 1.34^{kN} \times 0.1^{m}$
$= 0.134^{kNm}$

for D12, $Z = \frac{\pi . 12^3}{32} = 170^{mm^3}$

$S \simeq 1.2Z = 203^{mm^3}$

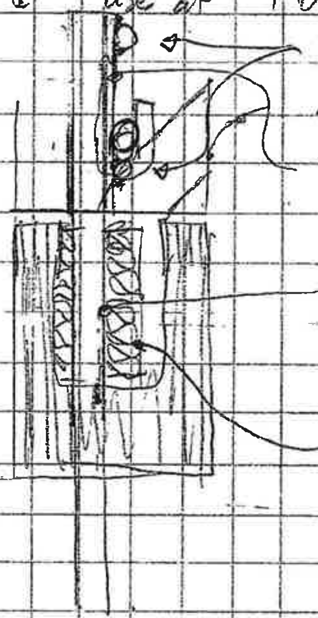
$SF_y = .25 \times .203 = 0.05^{kNm}$

i.e. max allow c'lever $= \frac{50^{kN\text{-}mm}}{1.34^{kN}} = 37^{mm}$
$\times 2 = 75^{mm}$

i.e. specify top of concrete in bbeam left 75 down from U/S of beam, or

for D20 $Z = 785$
$S \simeq 942$
$SF_y = 0.23^{kNm}$

i.e. use at 1.0^{m} crs, 200^{mm} free.



D12 x 800 long, SF bar to take ducts to fill blockwork.

20 NB Med. galv tube, 150 long.

20 NB galv rods at 1200 crs.
750 long

Unistrut UN63 foamglass ferrule, 130^{mm} long trimmed to clear block shell.

21 I

UN63-21

21 ID
102 OD.

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PAGE 954
SECT
FILE
DATE

Precast beam:

$w_D = 0.14 \times 3.24 \times 23.5 = 10.66$ kN/m
$w_u = 1.4 w_D = 14.92$ kN/m

$SSM. = \frac{14.92 \times 7.1^2}{8} = 94.0$ kNm

if $d = 380$ A_s reqd $= 785$ mm², $\underline{2H24} = 905$.
check : $b = 7100/12 = 591$
$= 6 \times 200 = 1200$
$a = \frac{905 \times 380}{.85 \times 20 \times 591} = 34$
$jd = 440 - (50 + 24 + 12 + 17)$
$= 337$
$M_u = .9 \times 905 \times 380 \times 337$
$= 104$ kNm > 94 OK.

Shear: $V_u = 14.92 \left[\frac{7100}{2} - 380 \right)$

$= 47.3$ kN
$v_i = \frac{47300}{.85 \times 140 \times 354} = 1.12$ N/mm²
$v_c = \left(0.07 + 10 \times \frac{905}{140 \cdot 354} \right) \sqrt{20}$
$= 0.2 \sqrt{20} = 0.89$
$v_s = 1.12 - .89 = 0.23$
$v_{s\,min} = .35$
$A_{v\,min} = \frac{.35 \times 140 \times 1000}{275} = 178$ mm²/m

for $\underline{\text{R10 at } 200}$, $A_v = 392$ mm²/m

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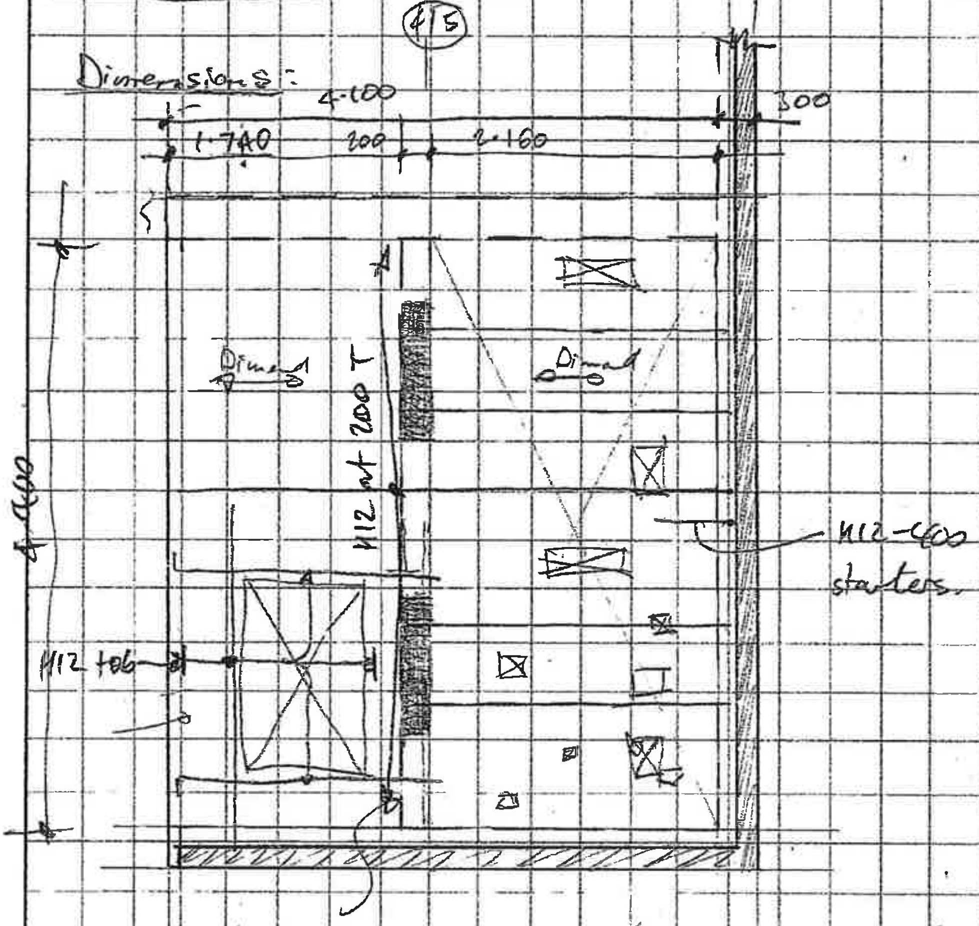
Lift machine Rm Floor Slab.



page G58

Loads: 150 o/A = 2·90
DL (solid 150 mm floor slab = 3·53 kPa)
LL Power lift rules cl 7·8 = 5·00 kPa

span between 4/5 and 5

stresses due to wet concrete
DL of slab 2·9
Constn LL 1·0
3·9 kPa

$SSM = \frac{3·9 \times 2·16^2}{8} = 2·27$ kNm.

max steel stress in steel ($Z_{min} = 12·9$) = $\frac{2270}{12·9}$ = 176 N/mm²

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CALCULATIONS

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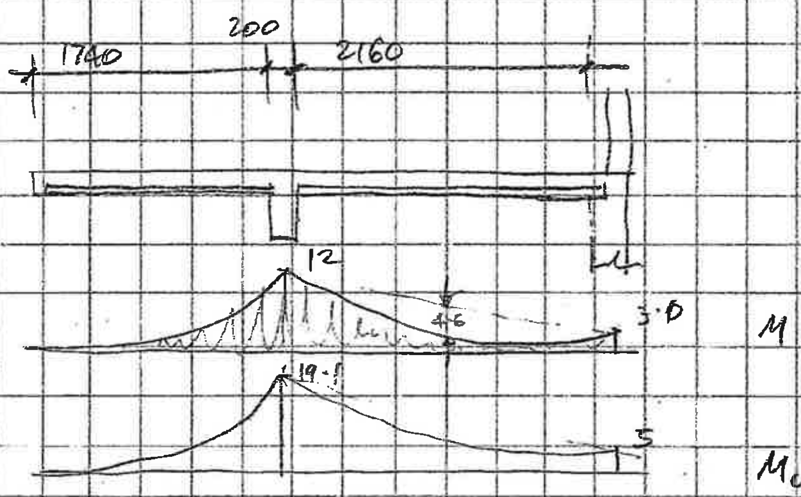
PAGE G56.
SECT
FILE
DATE

(b) deflection $= \frac{5}{384} \cdot \frac{3.9 \cdot 2.16^4 \cdot 10^3}{200 \times .452} = 12.2$ mm

DL floor alone! for 150 mm total floor thickness,
$w_{DL} = 2.9$ kPa $\Rightarrow \delta = 9.1$ mm
$.004 l = 8.64$ mm $-$ OK?

(c) composite slab stresses:



max CCM $= 7.9 \text{ kN/m}^2 \times \frac{1.74^2}{2} = 12$ kNm

SSM $= 7.9 \times \frac{2.16^2}{8} = 4.6$ kNm

ie no stresses in Dimond
$w_u = (1.4 \times 2.9) + (1.7 \times 5.0)$
$= 4.1 + 8.5$
$= 12.6 \text{ kN/m}^2 = 1.59 \times M$

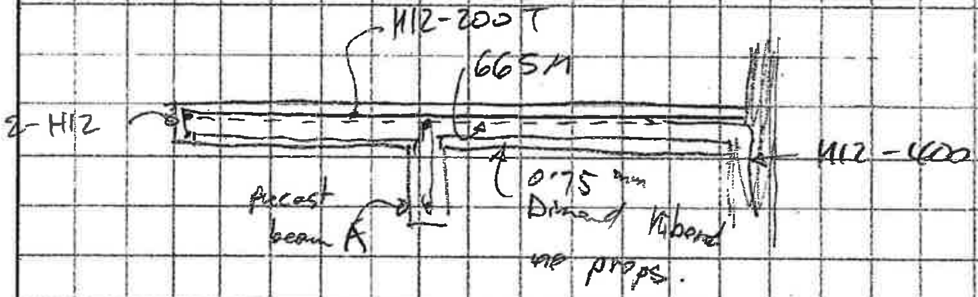
for $M_u = 12.6 \times \frac{1.74^2}{2} = 19.1$ kNm.

$d = 150 - (20 + 6) = 124$
for H12 at 250 T, $A_s = 452 \text{ mm}^2/\text{m}$
$a = \frac{452 \times 380}{.85 \times 20 \times 1000} = 24$

$jd = 124 - 12 = 112$

$M_u = \cdot 9 \times 452 \times 380 \times 112 = 17 \cdot 3$ kNm/m /m.

H12 at 200 T, $A_s = 566$ $M_u = 21 \cdot 3$ kNm/m OK.



Trimming around opening as sheet G55

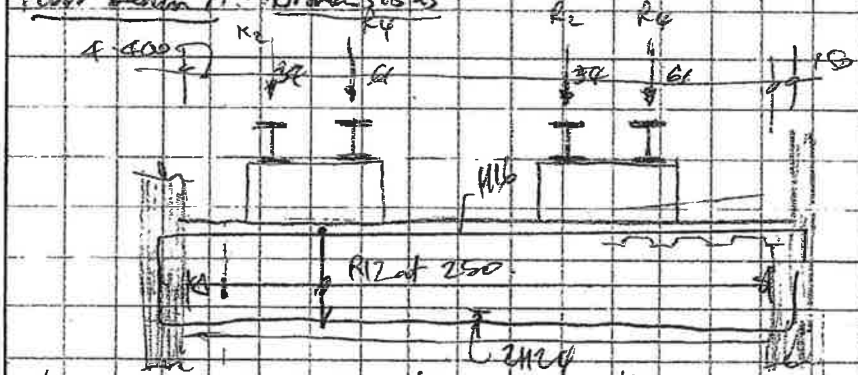
check $SSM = \frac{12 \cdot 6 \times 1 \cdot 3^2}{8} = 2 \cdot 66$ kNm

for H12 t or b $M_u = \cdot 9 \times 113 \times 380 \times 130$
$= 5 \cdot 0$ kNm $> 2 \cdot 66$ OK.

$w_u = 13 \cdot 44$ $SSM = 13 \cdot 44 \times 2.175^2/8 = 7 \cdot 95$ kNm

H12 at 350 $A_s = 323$, $M_u = 13 \cdot 25$ kNm OK.

__Floor Beam A: Dimensions__



loads: $R_2 = 2005$ kg $= 20$ kN $\times 1 \cdot 7 = 34$ kN P_u
$R_4 = 3575$ kg $= 36$ kN $\times 1 \cdot 7 = 61$ kN P_u

total load $= 68 + 122 = 190$ kN.

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UDL:

$w_D = (2.9^{kPa} \times 4.0^{m}) + (.2 \times .9 \times 23.5) = 15.8$ kN/m

$w_L = 5.0^{kPa} \times 4 = 20.0$ kN/m

$w_u = (1.4 \times 15.8) + (1.7 \times 20) = 56.1$ kN/m

$\therefore$ total load $W = (56.1 \times 4.4) + 190$

$= 247 + 190$

$= 437$ kN

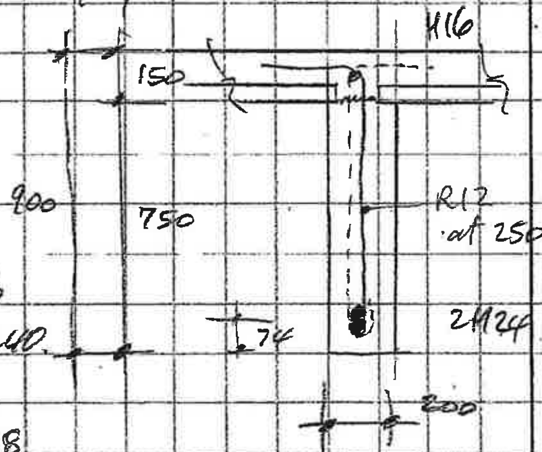
$SSM = M_u = \frac{437 \times 4.4}{8} = 240$ kNm

if 200 wide, $M_u/b = 1201$

$d = 800$ $\quad As/bd = 6500$ D

$A_s = 940$ mm² H.

For 2-H24 $\quad A_s = 904$



Beam A

check: $b = 8 \times 100 = 800$

$a = 20$

$d = 826$

$jd = 816$

$M_u = .9 \times 904 \times 380 \times 816$

$= 252$ kNm > 240.

Shear:

$V_u = \frac{437}{2} - 56.1 \times .8$

$= 218 - 45$

$= 173$ kN

$v_u = \frac{173{,}000}{.85 \times 200 \times 826} = 1.23$ MPa

$v_c = \left(0.07 + 10 \times \frac{904}{200 \times 826}\right)\sqrt{25} =$

$= 0.125 \times 5.0 = 0.62$

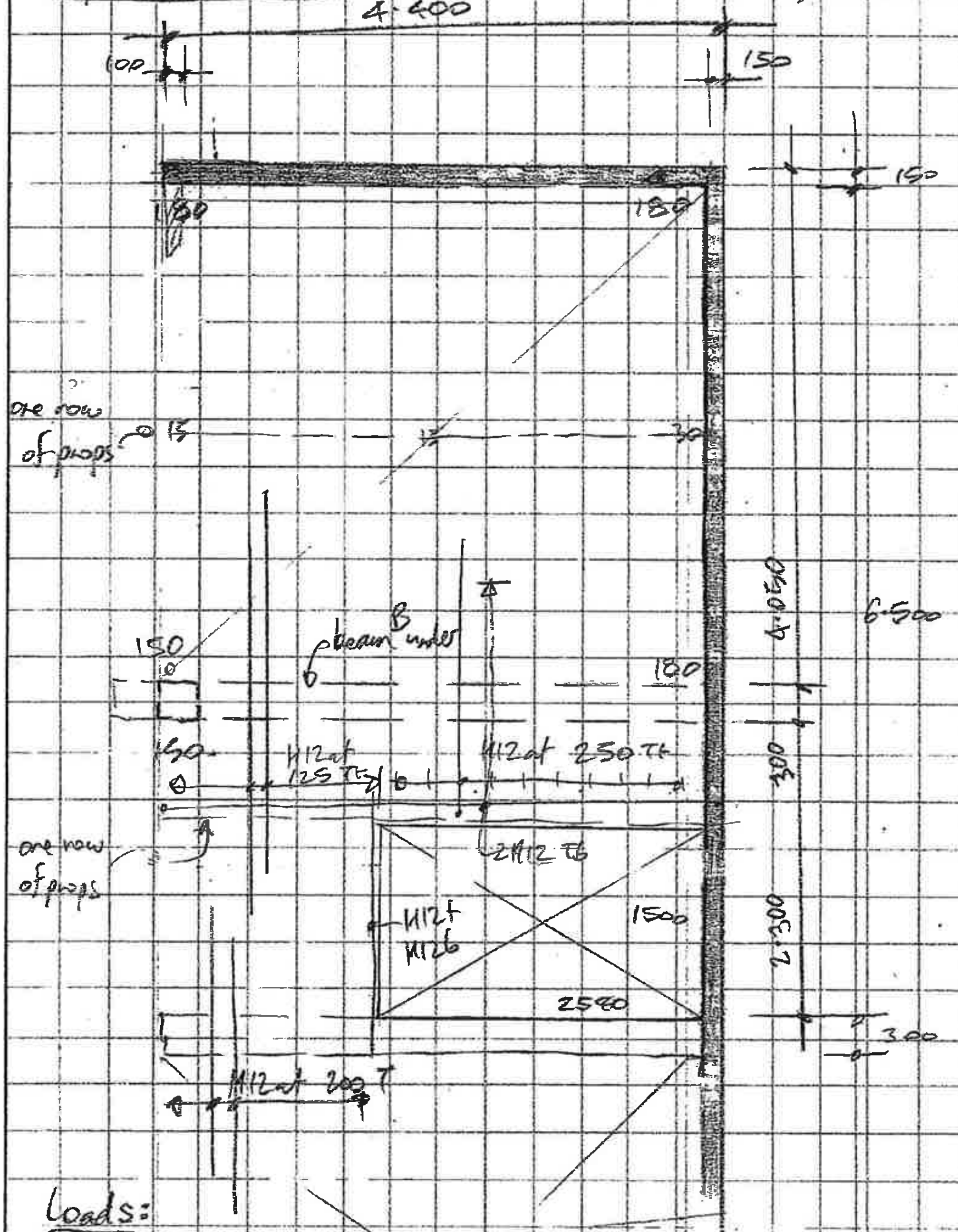
v_s max $= 1.23 - 0.62 = 0.61$

$A_v = \frac{0.61 \times 200 \times 1000}{275} = 443$ mm²/m

For R12 at 250, $A_v = 452$ mm²/m

CALCULATIONS		PAGE	G 59
ALAN M. REAY CONSULTING ENGINEER CHRISTCHURCH		SECT	
		FILE	
		DATE	

Exposed roof slab: (Plant Room similar)



Loads:

min depth at dp. = 150 mm
fall to cur = 50 mm
max depth = 200 mm

DL weight of slab (av 130 mm) = 3·5 kPa

LL water tanks 2000 litre = 20 kN

area = $\frac{\pi \cdot 1.35^2}{4}$ = 1·43 m²

av. press = 20/1·43 = 14·0 kN/m²

CALCULATIONS

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overall tanks, press $= \dfrac{3 \times 20 \text{ kN}}{1.5 \times 4.5} = 8.9$ kPa

design for LL = 12 kPa.

(a) construction loads, 1 row props.

max span between props $= \dfrac{4050}{2} = 2025$

DL wet concrete = 3.5
const LL = 1.0
4.5

max BM $= \dfrac{4.5 \times 2.025^2}{8} = 2.30$ kNm

tray only, Z min $\dfrac{2300}{12.9} = 179$ Nmm^2 < 300 OK.

(b) deflection wet concrete.

$\delta_{max} = \dfrac{1}{185} \times \dfrac{3.5 \times 2.025^4 \times 10^3}{200 \times 0.452} = 3.5$ mm OK.

(c) stresses in composite slab.

max pos. B.M. $= \dfrac{wL^2}{8} = (12 + 3.5) \times \dfrac{4.05^2}{8}$

$= 31.8$ kNm.

for composite slab,
20 MPa $Z_{ct} = 3300$, $F_{ct} = \dfrac{31800}{3300} = 9.6$ N/mm^2

allow $f_c = 20 \times .45 = 9.0$ N/mm^2

$Z_{cb} = 121$ $f_t = 262$ N/mm^2
> 250 NG.

for 25 MPa $Z_{ct} = 3060$ $F_{ct} = 10.39$
$f_c = 25 \times .45 = 11.25 > 10.39$ OK

$Z_{cb} = 123$ $f_t = 258$ still > 250 just
but OK will work with.

CALCULATIONS

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PAGE C61

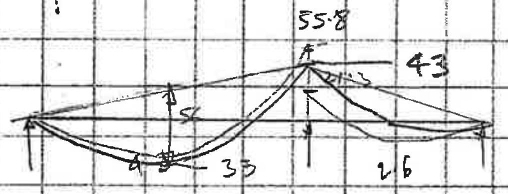
SECT

FILE

DATE

<u>negative steel over support</u>

$w_u = (1.4 \times 3.5) + (1.7 \times 12) = 25.3$ kPa



4.2 span, SSM $= \frac{25.3 \times 4.2^2}{8} = 55.8$

2.6 span $= \quad \times 2.6^2 = 21.3$

$\frac{D_{BA}}{D_{BC}} = \frac{2.6}{4.2} = \frac{0.38}{0.62}$

max neg. BM = 43 kNm. on d

redistribute $43 \times 0.8 = 34$ kNm

d = 120   As reqd = 868

for H12 at 125 $A_s = 904$ mm²/m

$f'_c = 25$,   a = 16

t=150   d = 150 − (30 + 8) = 112

$M_n = .9 \times 904 \times 380 \times 112 = 34.6$ kNm

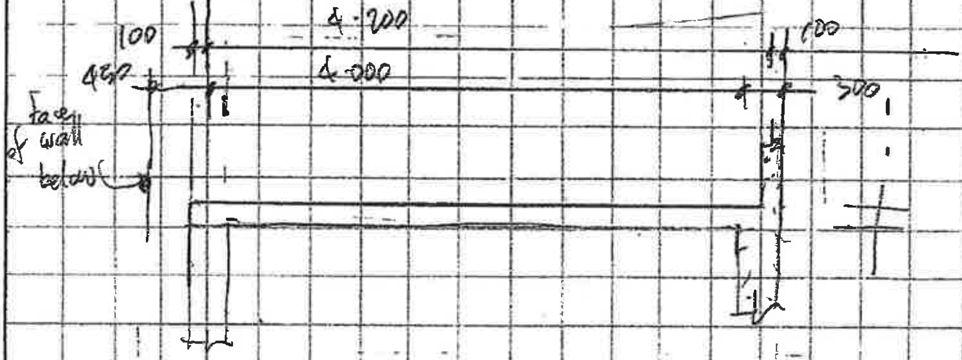
> 34

hole in slab
19/6/87
SSM = 25.3 × 2.3²/8
= 16.7
D6 − 150 M_n = 55 ×
· 25.3 × 3.9²/8 = 48 ×x6/2
2 D6 → M_n = 16.

CLM $= 25.3 \times 0.8^2/2 = 8.1$ kNm/m

H12 at 250 $M_n = 17$ kNm/m

<u>Floorbeam B</u>



CALCULATIONS

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PAGE G62
SECT
FILE
DATE

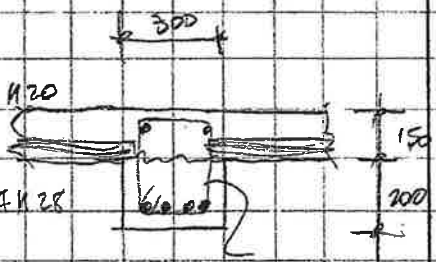
min depth $= \frac{3800}{17} = 223$ mm

$w_u = (25.3\ \text{kPa} \times 4.4^m) + (\cdot 3 \times \cdot 2 \times 23.5 \times 1.4)$

$= 114$ kN/m

SSM $= \frac{114 \times 3.8^2}{8} = 206$ kNm

$b = \frac{4000}{4} = 1000$



Beam B

R12 at 150.

for 4 H28 $A_s = 2460$

$a = 44$ $jd = 274$

$M_u = \cdot 9 \times 2460 \times 380 \times 274$

$= 230$ kNm > 206 OK.

Shear:

$V_u = 114$ kN/m $\times \left(\frac{3.8}{2} - \cdot 30\right)$ $d = \cdot 35$

$= 182$ kN   177

$v_i = \frac{182{,}000}{\cdot 85 \times 300 \times 296}$

$= 2.41$ N/mm²   2.34

$v_c = \cdot 2\sqrt{25} = 1.0$ N/mm² -1.0

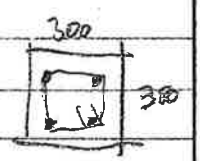
max $A_v/s = \frac{1.41 \times 300 \times 1000}{275}$ 1.34

$= 1538$ mm²/m

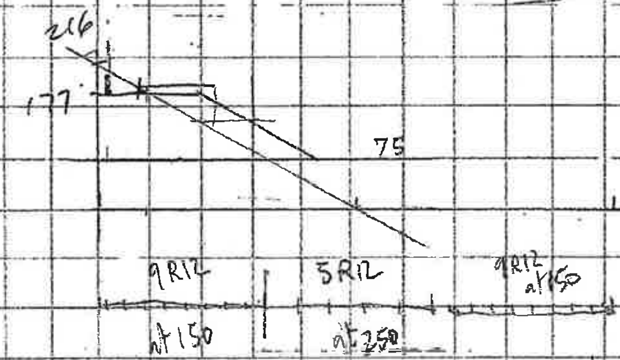
$\boxed{?}$ R12 at 150 $= 1507$ mm²/m $A_v =$ 1.38

R12 at ~~[illegible]~~ 300

Column



4D16
R6 at 200.

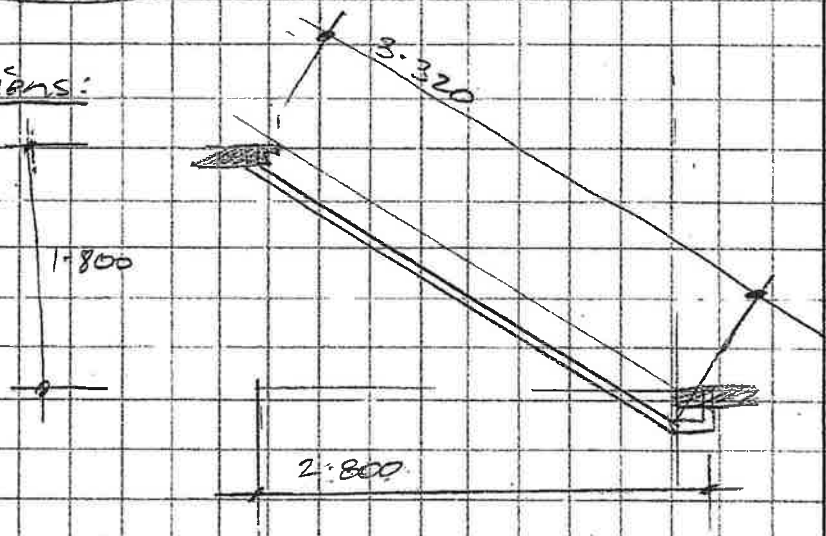


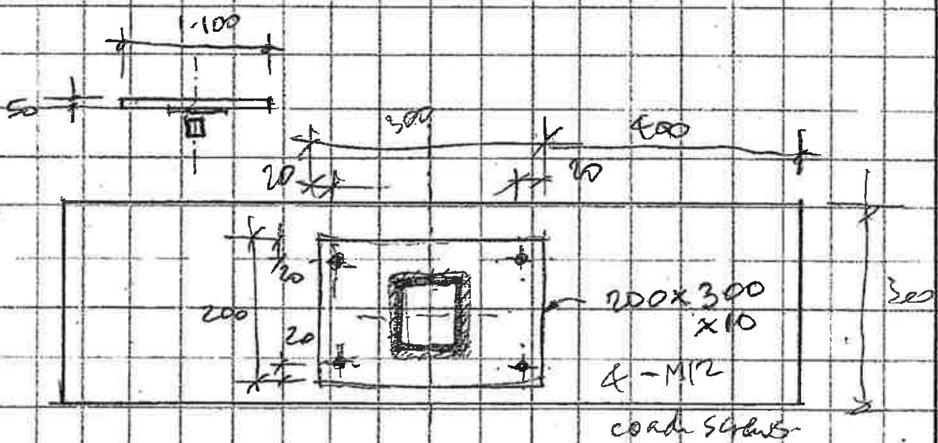
Column: as min $= 0.008 \times 300^2 = 720$ mm², 4D16 $= 804$

CALCULATIONS		PAGE	963
ALAN M. REAY CONSULTING ENGINEER CHRISTCHURCH		SECT	
		FILE	
		DATE	

Internal Stairs:

Dimensions:





Loads:

DL 50mm treads, 8 risers $w = \frac{.05 \times 1.1 \times 6.4 \times 25}{0.25} = 0.35$ kN/m

stringer, 102x102x4.0 0.14

cleats & brackets etc 0.14

0.63 kN/m

LL $LL = 2.5$ kPa $\times 1.1$m 2.75

D+L 3.38 kN/m

Stringer:

$SSM = \frac{3.38 \times 2.8^2}{8} = 3.35$ kNm

$Z_{min} = \frac{3310}{160} = 20.7$ cm^3

CALCULATIONS

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PAGE G64.
SECT
FILE
DATE

for 102x102x4·0 RHS, Z = 47.5
wt = 12.1 kg/m

$$\text{deflcn} = \frac{5}{384} \cdot \frac{2.75 \times 2.8^4 \times 10^3}{200 \times 2.41} = 4.56 \text{ mm}$$

.004 l = 4x2.8 = 11.2 mm > 4.56 OK

typical string

102x102 x4·0 RHS

check load ⊥ slope = 0.53 + 1.96 = 2.49 kN/m

1.8 / 3.32 / 32.5° / 2.8

$$SSM = \frac{2.49 \times 3.32^2}{8} = 3.43$$

$$\text{for } 102\times102, \ f_b = \frac{3430}{47.5} = 72.2 \text{ N/mm}^2 \text{ OK}$$

$$\text{defl} = \frac{5}{384} \cdot \frac{2.49 \times 3.32^4 \times 10^3}{200 \times 2.41} = 8.17 \text{ mm}$$

allow = .004 x 3320 = 13.3 mm OK

check torsion

on 127x76x4.9

$$w = \frac{2.75}{2} = 1.38 \text{ kN/m}$$

$$M = 1.38 \times \frac{1.1}{4} = 0.38 \text{ kNm/m}$$

$$\max M = 0.38 \times \frac{3.85}{2} = 0.73 \text{ kNm}$$

long str. grd floor use 127x76x6.3 RHS

$$\text{sh stress } f_q = \frac{T}{C} = \frac{0.73 \times 10^6 \text{ Nmm}}{75.4 \times 10^3 \text{ mm}^3}$$

$$= 9.7 \text{ N/mm}^2 < 115 \text{ N/mm}^2 \text{ OK}$$

~~long span grd floor~~

~~2.25 / 4.3 / 4.6~~

$$\text{max SSM} \perp \text{slope} = 3.06 \times \frac{4.3^2}{8}$$

$$= 7.07 \text{ kNm}$$

Z min = 44.4

for 127 x 76 x 4.9 Z = 63.4

$$\text{defl} = \frac{5}{384} \cdot \frac{3.06 \cdot 4.3^4 \times 10^3}{\ldots} = 16.9 \text{ ok? } .004l = 17.2$$

CALCULATIONS

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PAGE 465
SECT
FILE
DATE

Treads:

max BM = $10^{kN} \times 0.4^{m} = 0.4^{kNm}$

Z of tread = $\frac{280 \times 45^2}{6} = 94{,}500\ mm^3$

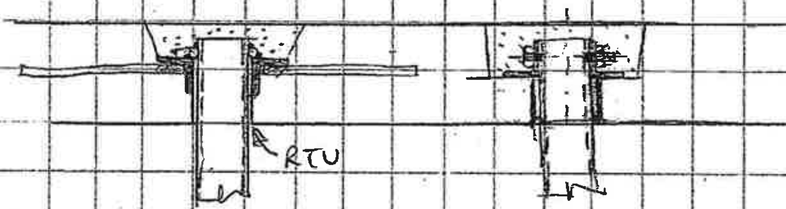
$f_b = \frac{400{,}000}{94{,}500} = 4.23\ N/mm^2$ OK.
< 6.0.

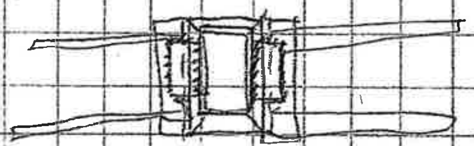
cleat BM = $1.0^{kN} \times 0.5^{m} = 0.5\ kNm$.

$Z = \frac{200 \times 10^2}{6} = 3333\ mm^3$

$f_b = \frac{500{,}000}{3333} = 150\ N/mm^2$ OK
< 162.

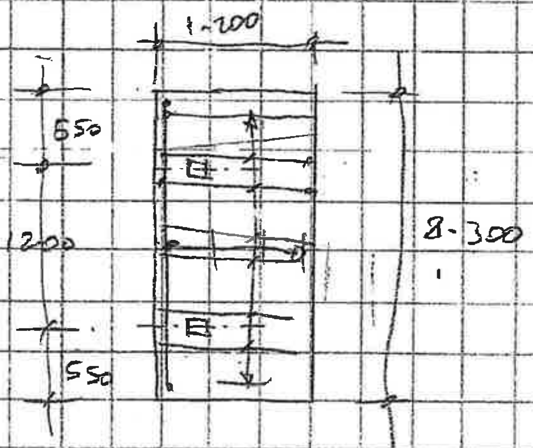
Fixing:





Stair landing:

Dimensions



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loads =

DL of slab = ·15 × 23.5 = 3.25

LL on slab = 2.50

pt load from string = 3.46 × 1.5 = 5.2 kN.

max $w_u = (1.4 \times 3.25) + (1.7 \times 2.5) = 8.8$ kN/m²

$P_u = 1.6 \times 5.2 = 8.3$ kN

max SSM $= \left(8.8 \times 1.2 \times \frac{2.3^2}{8}\right) + (8.3 \times .55)$

$= 6.98 + 4.57$

$= 11.55$ kNm.

For 150 slab, $A_{s\,min} = 0.0018 \times 150{,}000$

$= 270$ mm²/m

For H12 at 400 $A_s = 283$

$M_u = .9 \times 283 \times 380 \times 120$

$= 11.6$ kNm/m

landing

150 slab

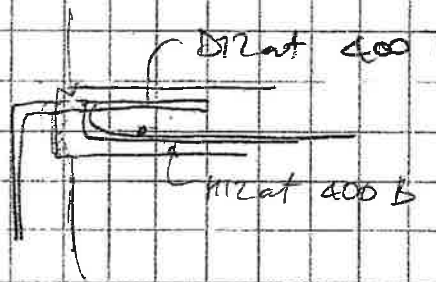
H12 @ 400 crs

bottom

use D12

at 400

starters



bend out of wall use p/s as temp. packing ⊂ wall

CALCULATIONS

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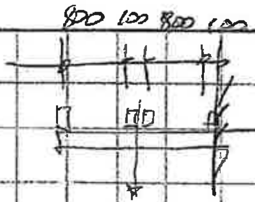
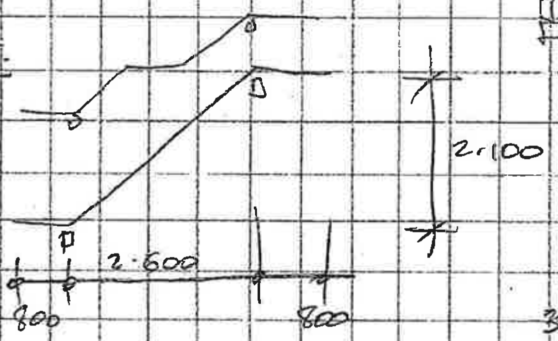
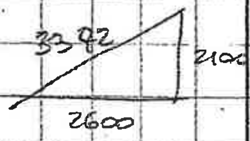
PAGE 667
SECT
FILE
DATE

External stair.

Dimensions:







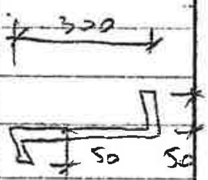
Loads:

DL : 5mm chequer plate treads = 0.39 kPa

LL 2.50 kPa

Treads

5mm chequer plate



strings:

$w_D = (0.39 \times 0.4) + .03 + .03 + .16 = 0.38$

$w_L = 2.50 \times .4 = 1.00$

wsr 1.38 kN/m

load ⊥ slope $= 1.38 \times 2.6/3.342 = 1.07$ kN/m

SSM ⊥ $= 1.07 \times \frac{3.342^2}{8} = 1.50$ kNm

CLM $= 1.38 \times .8^2/2 = 0.44$

Strings

$Z_{min} = 9.22$ cm³

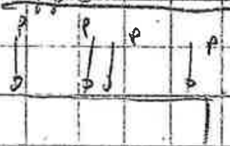
for 102x51x3.2, Z = 23.6

102x51 x3.2RHS

$defl_{D+L} = \frac{5}{384} \cdot \frac{1.07 \times 3.342^4 \times 10^3}{200 \times 1.20} = 7.2$ mm

$.004\ell = 13.4$ mm OK.

Support beams



$P = 1.38$ kN/m $\times 2.1 = 2.9$ kN

$M = 4 \times 2.9 \times 0.95 = 11.02$ kNm

$Z_{min} = 68$ cm³

for 152x76x8.9, Z = 83 cm³

check deflection : 152 × 76 × 4·9

equiv $w = \frac{2.9^{kN} \times 4}{1.8^{m}} = 6.44^{kN/m}$ — cantilever

$M = \frac{6.44 \times 1.8^2}{2} = 10.4$ — 152 × 76 × 6·3 RHS

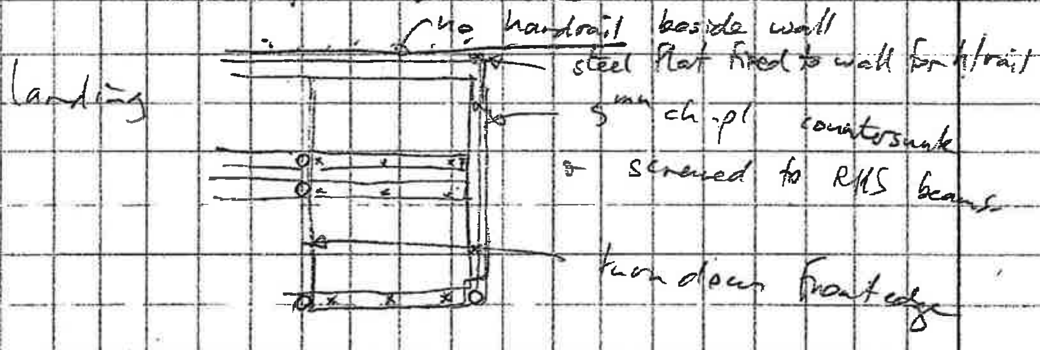
∴ use $w = 11.02/10.4 \times 6.44 = 6.8^{kN/m}$

∴ $defl = \frac{1}{8} \times \frac{6.8 \times 1.8^4 \times 10^3}{200 \times 6.33} = 7.0^{mm}$

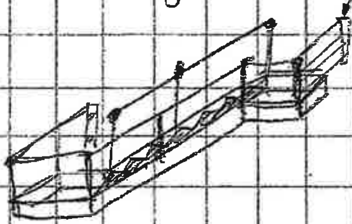
$l/180 = 10.0^{mm}$ – OK

for 6·3 RHS $I = 7.91$ $\delta = 5.6$ better

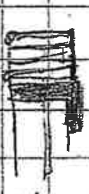
+ allow for base rotation

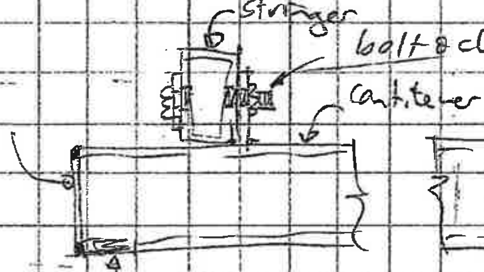


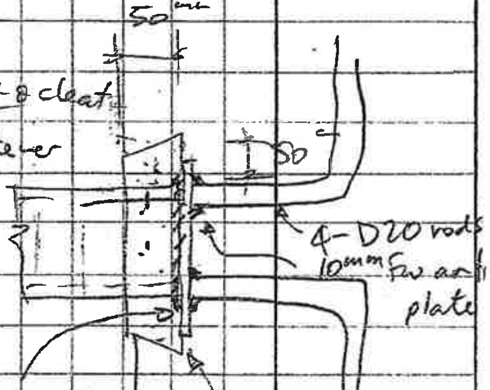
fabricate flights as:



fix landings on site by screwing & bolt fix. strings to cantilever arms







provide galv. vent (20 ∅) holes to underside at all member both ends

weld plate recessed into wall plaster up after welding

4–

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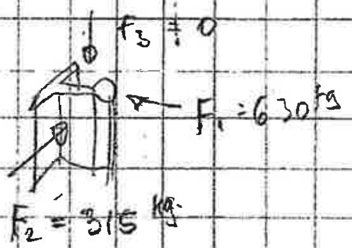
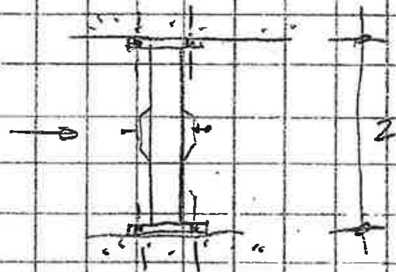
Lift shaft steelwork

Channel for guiderails





102×51 RSC horiz.

max horiz load each lift from OTIS dwg
for $C_{pmax} = 0.5g$ (Table 9 item 11 Zone B)

2 M16 bolts to concrete. 17.5 ϕ holes in plate (power lift rules 20.12.3)

Car $F_1 = 630$ kg $= 6.3$ kN.

$BM = M_u$ on rail $= \frac{6.3 \times 2.175}{4} = 3.43$ kNm

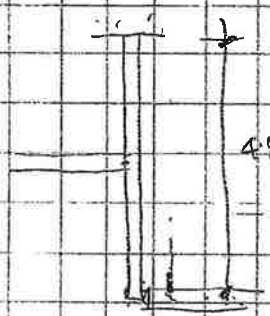
for 102×51 RSC, $f = \frac{3430}{40.9} = 83$ N/mm² $< .85 \times 250 = 212$

shear force $= 3.2$ kN

2 M12 bolts, capacity in concrete $= 2 \times 3.11 \times 1.5$ ok
bolt to steel $= 24.2$ kN(1.5) ok

$\text{defl} = \frac{1}{48} \times \frac{6.3 \times 2.175^3 \times 10^3}{200 \times 2.08} = 3.24$ mm < 6 mm (PLR 20.12.1(iii))

RHS post at g/1st fl to LMR.



max horiz load $= 3.2$ kN.

$M_u = BM = \frac{3.2 \times 4.85}{4}$

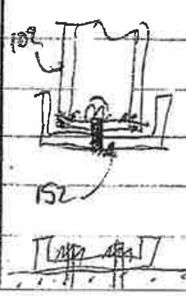
$= 3.88$ kNm

$Z_{min} = \frac{3880}{.85 \times 250} = 18.3$ cm³

vertical post

152×76 RSC 2-M16 bolts

for 76×76×3.2 RHS $Z \doteqdot 21.1$



deflection: $\delta = \frac{1}{48} \times \frac{3.2 \times 4.85^3 \times 10^3}{200 \times .805} = 47$ mm

allow defl $= 6$ mm ! $I_{min} = .805 \times \frac{47}{6} = 6.3$

⇒ 152×76 RSC (8.52) defl=

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Hoist Beam:

SWL = 1230 kg = 12.3 kN.

span = 4.200 m

design loads, vertically = 12.3 x 1.1 = 13.5 kN

4203: 22.3.11.(b)

horizontally = 0.05 x 12.3 = 0.62 kN

max SSM. = $\frac{13.5 \times 4.2}{4}$ = 14.2 kNm

Z_{min} = $\frac{14200}{162}$ = 87 cm^3

~~try 152x76x18 kg RSJ~~

~~Z = 115 fb = $\frac{14200}{115}$ = 123 N/mm^2~~

~~pb: l/r = $\frac{1.2 \times 4200}{16.3}$ = 309~~

~~D/T = 15.9~~

~~pb = 73 N/mm^2 < 123 NG.~~

for 178 x 89 x 22 kg RSJ

fb = $\frac{14200}{168}$ = 84 N/mm^2

pb: l/r = $\frac{1.2 \times 4200}{18.8}$ = 268

D/T = 17.6

pb = 76 N/mm^2 x 1.1 = 83.6 OK.

deflection

$\delta = \frac{1}{48} \times \frac{12.3 \times 4.2^3 \times 10^3}{200 \times 14.9}$ = 6.4 mm = 1.0015 l = l/656

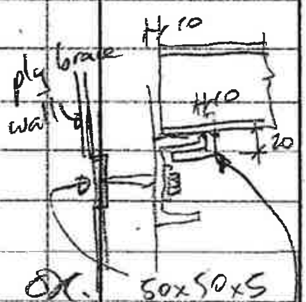
(.004 l = 16.8)

horiz. load: BM = $\frac{.62 \times 4.2}{4}$ = .65 kNm

fb = $\frac{650}{22.6}$ = 29 N/mm^2 OK. < 162

Hoist Beam
178 x 89
x 22 kg
RSJ

support ch.
102x51 RSC
bolt to all
studs



50x50x5
washer
checked
in.

ex. 40x40x6 a
x 400 long
notch over o/s
beam.

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Fixing:

capacity of M12 bolt ∥ grain = 3.1 kN
M16 bolt ∥ grain = 5.5 kN

require 3 studs to carry load: ↓ 13.5

↑ 4.5 ↑ 4.5 ↑ 4.5

BM on plate = 4.5 × .6 = 2.7 kNm
for 102 × 51 Bs = $\frac{2700}{40.9}$ = 66 N/mm² OK.

Studs:

load = 4.5 kN
$f_c = \frac{4500\ N}{94 \times 47} = 1.01$ N/mm²

length = 2880 − 1000 → 1880

$S = l/d = \frac{1850}{94} = 19.6$

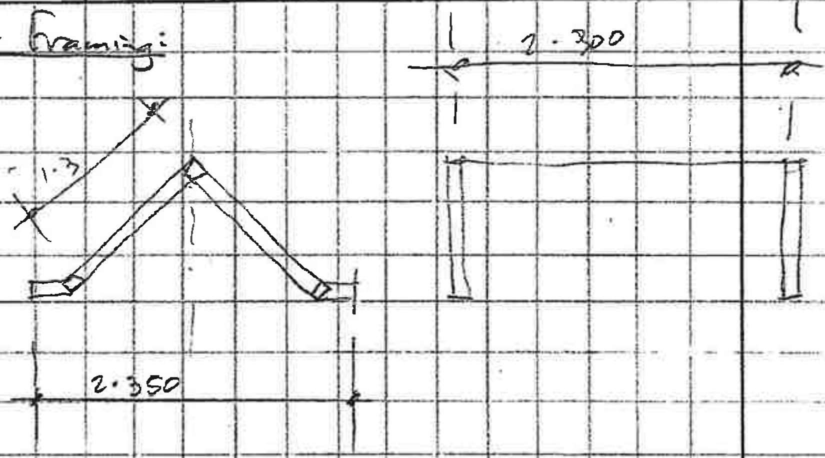
$p_c = 0.7 \times 7.1$ N/mm²
= 5.0 N/mm² > 1.01 OK.

For $l/d_n = \frac{1880}{47} = 39$, S = .25
$p_c = .25 \times 7.1$
= 1.77 > 1.01 still OK.

CALCULATIONS		PAGE	972
ALAN M. REAY CONSULTING ENGINEER CHRISTCHURCH		SECT	
		FILE	
		DATE	

Entry Foyer Framing:

Dimensions:



Loads:

DL al glazing bars = 0.190 kN/m²
SL 0.500 "

Ridgebeam DL = (.190 × 1.3) + .06 = 0.31
SL = 0.9 × 0.5 = 0.45
0.76

FEM SSM = $0.76 \times \frac{2.3^2}{12}$ = 0.34 kNm

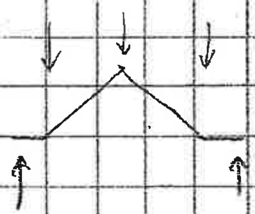
SSM = 0.50 kNm

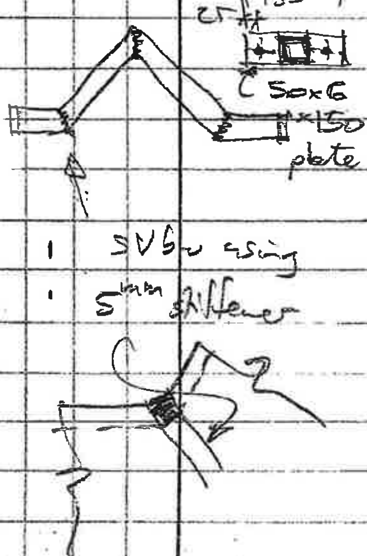
For 50×50 RHS × 3.2 Zx = 8.56

fb ≤ $\frac{500}{8.56}$ = 58 N/mm² low.

rafters
& frames
51×51×4.0
RHS.

Frames:





w = 0.84 kN/m² × 2.3 m = 1.94 kN/m

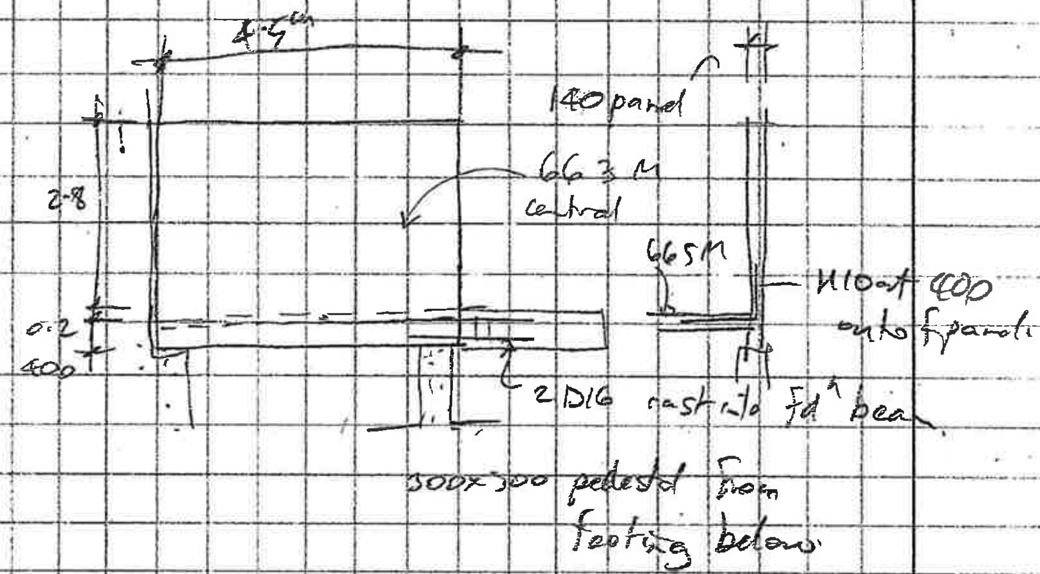
SSM = $1.94 \times \frac{2.35^2}{8}$ = 1.34 kNm

Zm = 1340/162 = 8.27 cm³

5VS ~ using
5mm stiffener

For 51×51×3.2 RHS Z = 8.56
51×51 × 4.0 RHS Z = 9.96 cm³

Precast Panel at entry



Face loads : $w_{WL} = 1.3 \times .414 = .54$ kPa

$w_{EQ} = .2 \times \frac{5}{6} \times .14 \times 2.5 = .55$ kPa

$CLM = \frac{.55 \times 3.0^2}{2} = 2.48$ kNm

min reinf $= .0018 \times 140{,}000 = 252$ mm²/m

for 663M. $A_s = 205$ mm²/m

$M_u = .9 \times 205 \times 380 \times 65 = 4.55$ kNm/m.

ties into slab:

$\sigma = \frac{3 \times 2.45}{.6 \times 1.0} = 20.4$ kPa ok.

horiz force $H = (.55 \times 3.0) + (.5 \times 20.4 \times .6)$

$= 165 + 610$

$= 7.65$ kN

A_s reqd $= 15.8$ mm²/m

H10 at 400 = 195 mm²/m

665M = 145 mm²/m

BUI.MAD249.0273.77

CALCULATIONS

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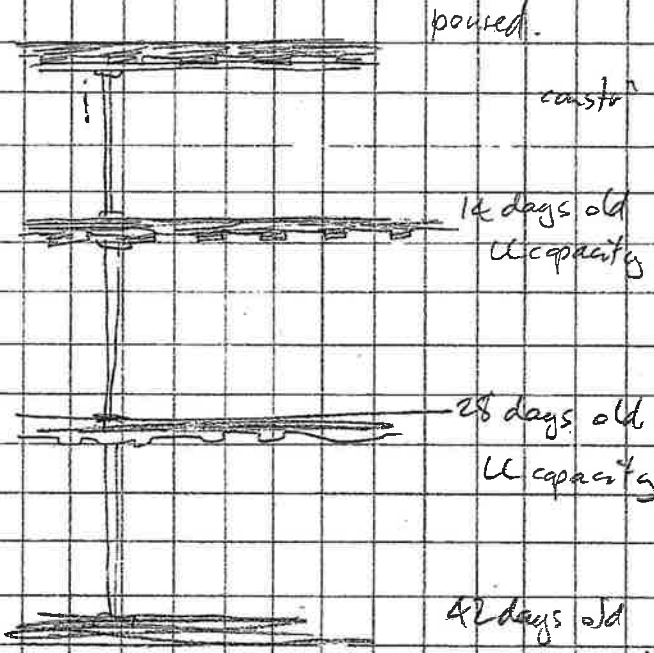
PAGE 674
SECT
FILE
DATE

Propping of Floors:



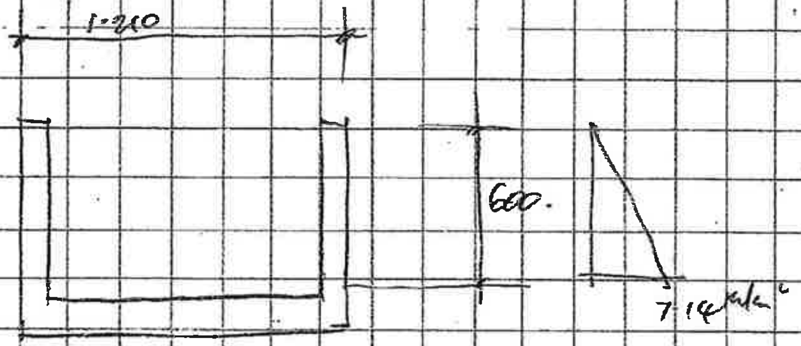
DL = 4.0 kPa
constn LL = 1.0
5.0 kPa

LL capacity = 1.0 kPa

LL capacity = 2.5 kPa

LL capacity = 2.5 kPa
6.0 kPa

Planter Boxes:



Loads: max press = 0.7 × 6.0 kPa = 4.2 kN/m²

D_{cl} = 1.7 = 7.14 kN/m²

w = 7.14 × 0.7 / 2 = 2.50 kN/m

M_u = 2.5 kN × 0.7/3 = 0.58 kNm/m

100 mm concrete

min reinf = .0018 × 100 = 180 mm²/m

for 664M A_s = 186 mm²/m

H10 at 400 A_s = 196 mm²/m

d = 4

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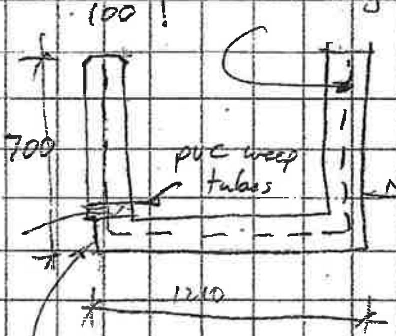
PAGE	G75
SECT	
FILE	
DATE	

M_u =

steel on ℄, d = 50

$M_u = 0.9 \times 186 \times 380 \times 45 = 2.86$ kNm/m

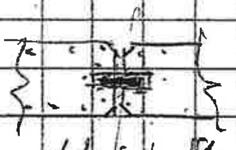
664 M galvanised mesh on ℄

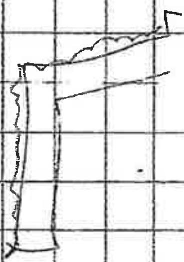


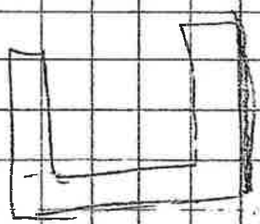
Area = $(2 \times .1 \times .7) + (.12 \times 1.01)$
$= 0.26\ m^2$

Wt = 0.26×2.45
$= 0.64^t$ per 1.0^m.

recess } as detailed on arch dwgs.
chamfers }
weep tubes }







CALCULATIONS

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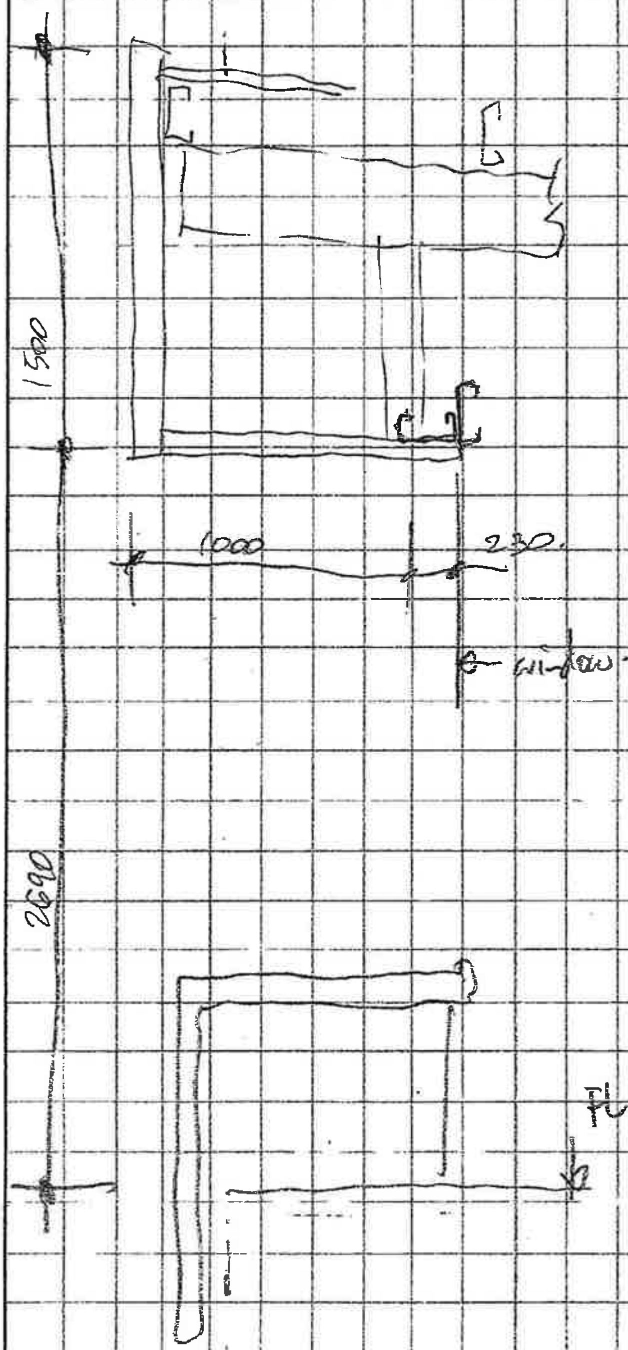
PAGE	976
SECT	
FILE	2503
DATE	20/8/86.

Purlin for Hardiflex Fascia

Dimensions



max
span between columns
= 7·500 m

Loads: DL of Fascia etc = 0·35 kPa × 0·7 m = 0·25
purlins 2/ BP 230 = 2×·08 = 0·16
0·41 kN/m

WL on window & fascia
= 0·794 kPa × 1·1 × 2·1 m = 1·83 kN/m

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PAGE 677
SECT
FILE
DATE

for BP 200/25

vertical loads capacity fully restrained ≈ 2.19 kN/m

deflection $\delta = \frac{5}{384} \cdot \frac{0.41 \times 7.5^4 \times 10^3}{200 \times 5.8} = 14.5^{mm}$

$.004 l = 4 \times 7.5 = 30^{mm}$ - ok.

horiz. loads

capacity fully restrained ≈ 2.19 > 1.83.

defl $\approx = \frac{5}{384} \cdot 1.83 \cdot 7.5^4 \ldots = 64.7^{mm}$

$.006 l = 45^{mm}$.

for combined section:

$A = 922$

203 76
107 102
22

$\bar{x} = \frac{(922 \times 102) + (922 \times 225)}{2 \times 922} = 164$

$I = 5{,}800{,}000 + 692{,}000 + (922 \times 62^2) + (922 \times 61^2)$
$= 13.47 \times 10^6 \ mm^4$

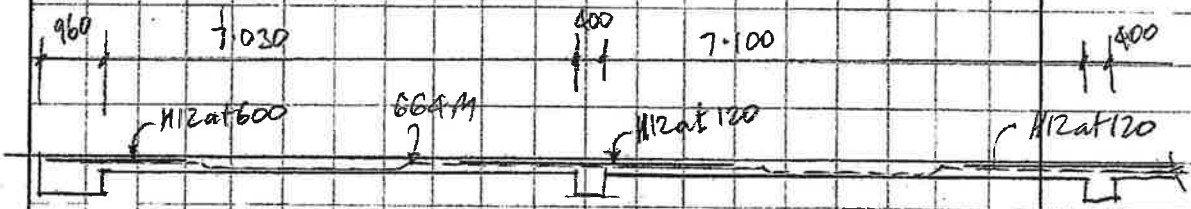
$\delta = 64.7 \times \frac{5.8}{13.47}$

$= 27.9^{mm} < 45^{mm}$ ok.

use 2/BP 200/25.

Hi-bond capacity in after-fire condition

Dimensions:



deck steel shown to support loads with no

Loads:

DL : Hibond = 4.0
LL Full code LL = 2.5

$w_u = (1.4 \times 4.0) + (1.7 \times 2.5)$
$= 5.6 + 4.25$
$= 9.85$ kN/m²

For after-fire condition, using FE reinf, a 25% increase in stesses is permitted. This effectively reduces design LL so that

design $w_u = \frac{9.85}{1.25} = 7.88$ kN/m²

for 7.03 span, SSM $= \frac{7.88 \times 7.03^2}{8} = 48.7$ kNm

for 7.10 span, SSM $= \frac{7.88 \times 7.1^2}{8} = 49.6$ kNm

Note: if LL is equiv to construction LL of 1.0 kPa

$w_u = (1.4 \times 4.0) + (1.7 \times 1.0)$
$= 5.6 + 1.7$
$= 7.3$ kPa < 7.88 so 7.88 is still conservative

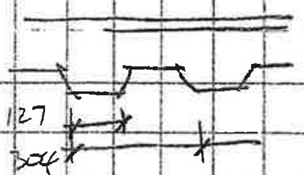
Slab capacities

(1) neg BM, exterior beam, line 1 & 4

664 Mesh, $A_s f_y = 186 \times .414 = 77.0$

H12-600 $A_s f_y = 188 \times .380 = 71.4$

148.4



$a = \frac{148.4 \times 10^3}{.85 \times 25 \times 1000} \times \frac{304}{127}$

$= 16.7$

$jd = 180 - 8 = 172$

$M_u = .9 \times 148.4 \times .172 = \underline{23.0}$ kNm

(2) neg. BM. interior beam lines 2 & 3

664M, $A_s f_y = 77.0$

H12-120 $= 942 \times .38 = 358$

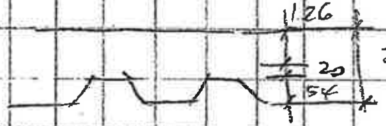
435

$a = \frac{435 \times 10^3}{.85 \times 25 \times 10^3} \times \frac{304}{127} = 49$

$jd = 180 - 25 = 155$

$M_u = .9 \times 435 \times .155 = 61.0$ kNm

(3) pos BM midspan



$A_s f_y = 77.0$

$a = \frac{77}{.85 \times 25} = 3.6$

$jd = 126 - 2 = 124$

$M_u = .9 \times 77 \times .124 = 8.6$ kNm

interior


capacity $= 61.0 + 8.6 = 69.6 > 49.6$ OK

exterior


capacity $= \frac{61+23}{2} + 8.6 = 50.6$

> 48.7

OK.