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FOUNDATIONS

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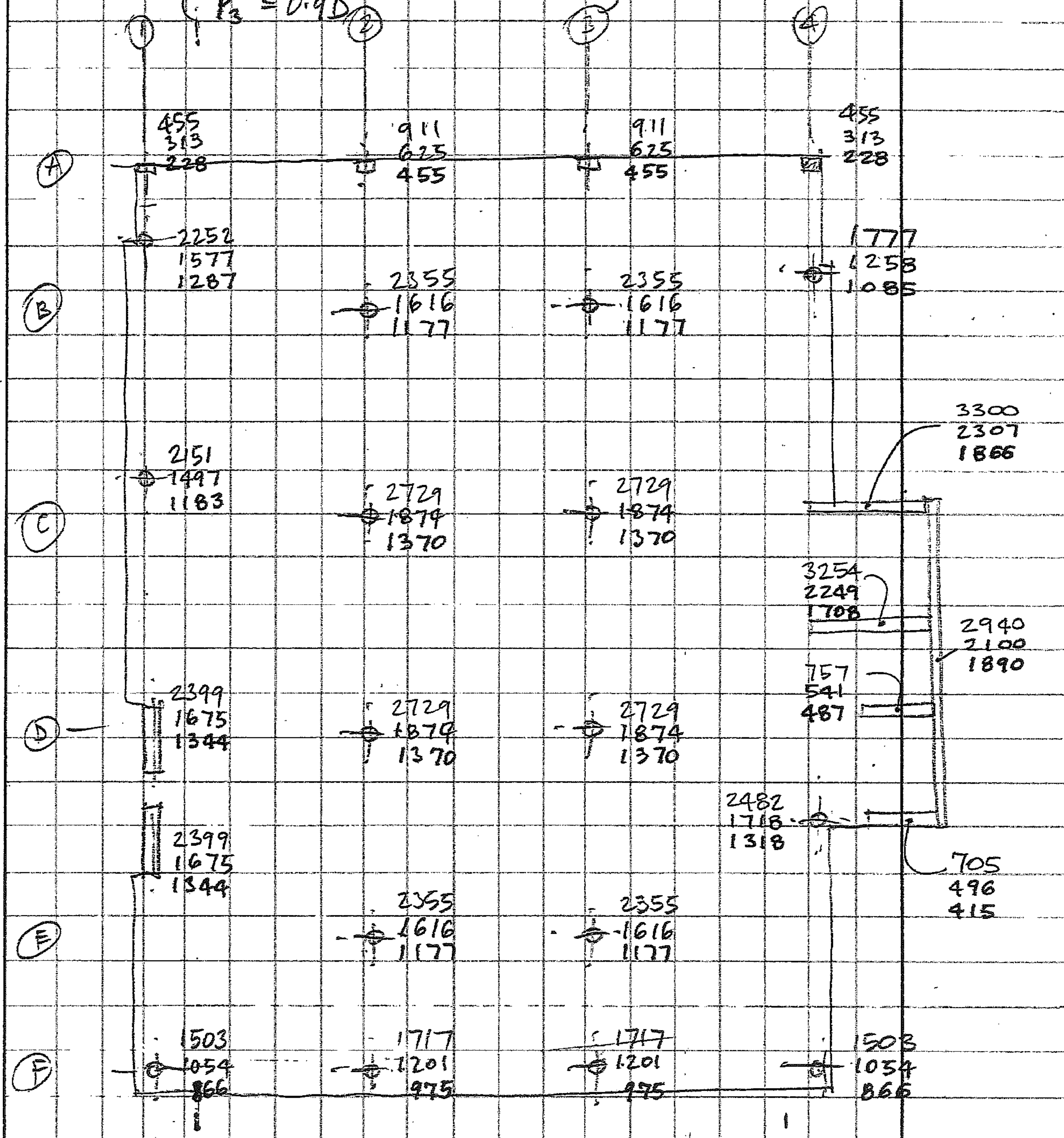
SECT
FILE
DATE 26/6/86

Summary of vertical loads

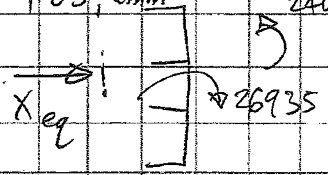
$P_1 = 1.4D + 1.7L_R$
$P_2 = 1.0D + 1.0L_R$
$P_3 = 0.9D$

from pp G36, G40, G41, G42, S23-26



Overall actions on lift block

for X_{eq} 1.0S, e_{min}

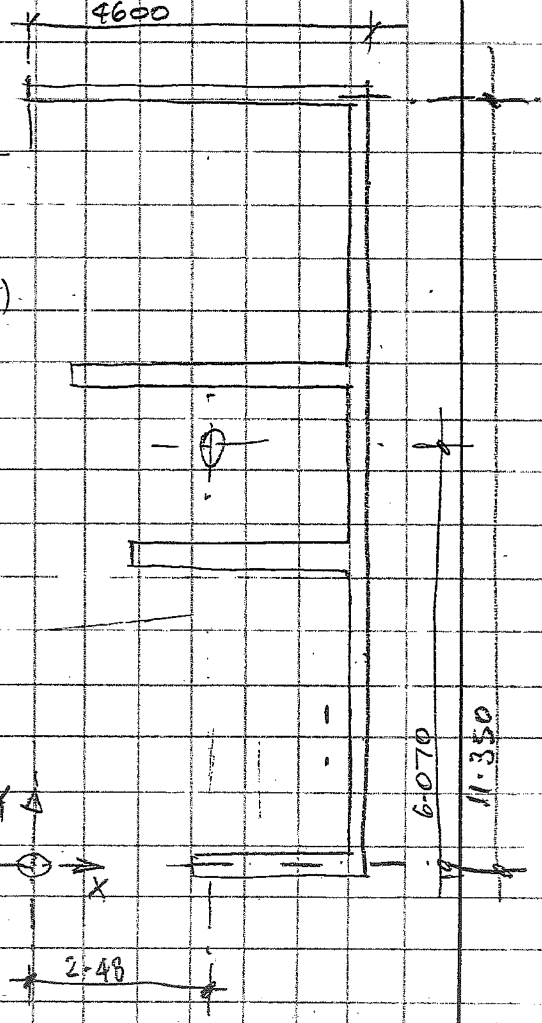


$X_{dir}{}^{n}$; for 1.0S $M = 47254$ kNm
for 0.57S $M = 0.57 \times 47254$
$= 26935$ kNm (design case) (from pp 18-20)

$Y_{dir}{}^{n}$, $M = 0.57 \times 4223 = 2407$ kNm

vertical loads from F1, 0.9D $= 1866+1708+487+1890+415+1318 =$ 7684 kN
1.0D+1.0L $= 2307+2249+541+2100+496+1718 =$ 9411
1.4D+1.7L $= 3300+3254+2940+757+2482+705 =$ 13438

Consider a raft footing under lift block:



local co-ordinates:

$\bar{X} = \dfrac{(415 \times 3.215) + (6061 \times 2.3) + (1890 \times 4.45)}{7684}$

$= 2.48$

$\bar{Y} = \{(487 \times 4.7) + (1708 \times 7.3) + (1866 \times 11.35)$
$+ (1890 \times 5.675)\} \div 7684$

$= 6.07$

$e_x = \dfrac{Pe_x}{P} = \dfrac{26935 \text{ kNm}}{7684 \text{ kN}}$

$= 3.50$ m

$e_y = \dfrac{Pe_y}{P} = \dfrac{2407 \text{ kNm}}{7684 \text{ kN}}$

$= 0.31$ m

if a raft is built under block, 11·500 × 4·600.
total area = 52·9 m^2

av pressures, 0·9D = $\frac{7684^{kN}}{52\cdot 9\,m^2}$ = 145 kPa

1·0D+1·0L = $\frac{9411}{52\cdot 9}$ = 178 kPa

1·4D+1·7L = $\frac{13438}{52\cdot 9}$ = 254 kPa

First Consider footing sizes reqd. for DL+Long term LL for soil pressures to limit settlements to 25 mm. & keep below allow. bearing

(1) Columns C2C3, D2, D3

$P_D = \frac{1370}{0\cdot 9} = 1522^{kN}$, $P_{D+L} = 1874$ — C2C3 D2D3 4·0×4·0.

over gravel, L/B = 1, B = 2·0 allow press = 160 kPa
→ reqd. area = 3·6 m × 3·6 m
for 4·0m × 4·0m pad, $\frac{D}{B} = \frac{1}{4} = 0\cdot 25$

allow bearing press = 120 kPa × 16 = 1920 > 1874 ok
allow settlement o/gravel = 115 kPa × 16 = 1840 > 1522 ok

av press P_D = $\frac{1522}{16}$ = 95 kPa

(2) Columns B2 B3 E2 E3

$P_D = \frac{1177}{\cdot 9} = 1307$ $P_{D+L} = 1616$

over gravel, L/B = 1   B = 1
for 3·5m pad — D/B = $\frac{1}{3\cdot 5}$ = 0·29

allow bearing = 110
settlement o/gravel = 120
settlement silt = 72 kPa

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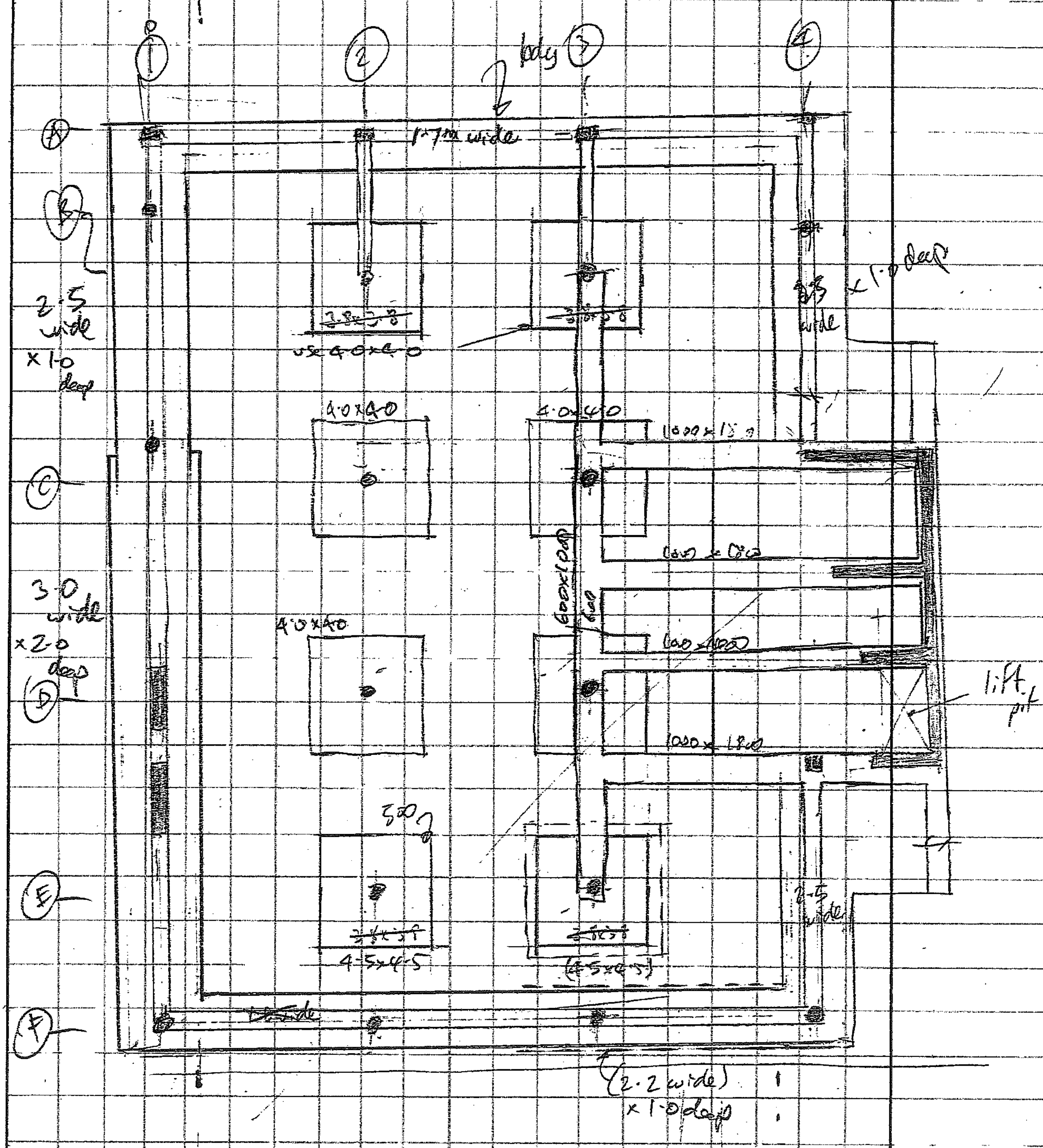
Summary of pad sizes



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3.5 sq pad, press $P_D = \frac{1307}{3.5^2} = 106$ kPa < 120 ok

$P_{DE} = \frac{1616}{3.5^2} = 132$ kPa > 110 NG.

3.8 sq pad. allow. bearing = 115 kPa — B2 B3 E2 E3

allow. settlement o/g = 115

silt = 70

$P_D = \frac{1307}{14.44} = 90 < 115$ — o/gravel 3.8 sq.

$P_{DE} = \frac{1616}{14.44} = 112 < 115$ ok — o/silt 4.5 sq.

4.0 pad allow settlement o/silt = 65

$P_D = \frac{1307}{16} = 81$ high

4.5 sq allow o/silt = 62

$P_D = \frac{1307}{4.5^2} = 64$ kPa ok

Front wall, col. line F1 F2 F3 F4

$P_D \;\; \frac{866}{.9} + \frac{975}{.9} + \frac{975}{.9} + \frac{866}{.9} = 962 + 1083 + 1083 + 962 = 4090$

$P_{DE} = 1054 + 1201 + 1201 + 1054 = 4501$

for $L/B = 5$ o/gravel 2.0^m wide, allow. settlement press = 125 — Front wall 1.8 strip o/gravel

$D/B = 1/2 = .5$ allow bearing press = 84

o/silt allow settlement press = 65

strip footing 2.0 wide, contrib. area = $2.0 \times (22.5 + 4 + 3)$ — 2.2 strip o/silt.

$= 59\ m^2$

P_D press $= \frac{4090}{59} = 69 \simeq 65$ ok.

$P_{DE} = \frac{4501}{59} = 76 < 84$ ok.

for footing o/gravel use $1.8^m \rightarrow P_D = 77 < 125$

$P_{DE} = 84 = 84$ allow.

o/silt use 2.2^m $P_D = 62 \simeq$ allow 62 ok

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Side wall A1 B1 C1 D1 E1.

under B1 & C1 $P_D = \frac{1287+1183}{.9} = 2744$

$P_{DL} = 1577 + 1497 = 3074$

effective length of footing = 13 m
for gravel 1·8 wide allow settlement = 135
D/B = 1/1·8 = ·55 bearing = 80 kPa.
av press @ 9D = $\frac{2744}{1.8 \times 13}$ = 117 < 135 OK.

1·0D+1·0L = $\frac{3074}{1.8 \times 13}$ = 131 > 80 NG

for 2·0 wide allow bearing = 90
actual = $\frac{3074}{26}$ = 118 NG

2·2 wide allow = 95
actual = 107

2·5 wide bearing allow = 105 OK
actual = 95

under wall, D1 E1

$P_D = \frac{2 \times 1344}{.9} = 2987$ kN.

$P_{DL} = 2 \times 1675 = 3350$ kN

effective length of footing = 12·5 m
For 2·5 wide allow. bearing = 105
actual = $\frac{3350}{2.5 \times 12.5}$ = 107 OK.

allow. settlement of gravel = 118
actual = $\frac{2987}{2.5 \times 12.5}$ = 96 < 118 OK.

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Rear wall A1 A2 A3 A4

for A2 & A3, each $P_D = \frac{455}{0.9} = 505$

$P_{D+L} = 625$

length available $= 7.5^m$

for 1.0^m wide footing, allow settlement of silt $= 175$

$D/B = 1.0$ allow bearing $= 76$

P_D press $= \frac{505}{7.5} = 67 < 175$ ok

P_{D+L} press $= \frac{625}{7.5} = 83 > 76$ just.

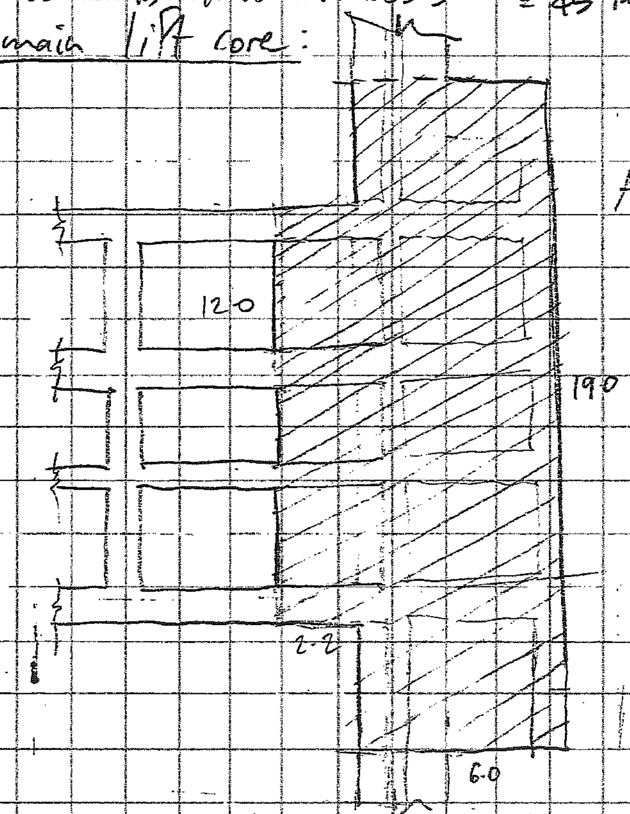
use 1.1 wide footing, allow bearing $= 84$

$P_{DL} = \frac{625}{1.1} = 75 < 84$ ok

back wall 1.1 wide +0.6 1.7^m wide

extra for 200 blockwalls $= 10 \times 18^{kN} \times 19^m \times 23.5 \approx 45^{kN/m} = 0.6^m$ extra wide

Under main lift core:



$A = (19 \times 6)$
$+ (12 \times 2.2)$
$= 114 + 26$
$= 140^{m^2}$

For increased area as above

if $A = 140^{m^2}$

$P_D = \frac{7684}{.9} = 8538^{kN}$ press $= \frac{8538}{140} = 61$ kPa for 1·0D + 1·0L P_{tot} = 9411 kN

press = $\frac{9411}{140}$ = 67 kPa

allowable: for footing over silt, settlement

$\frac{L}{B} = \frac{19}{8 \cdot 2} = 2 \cdot 3$ allow = 60 kPa

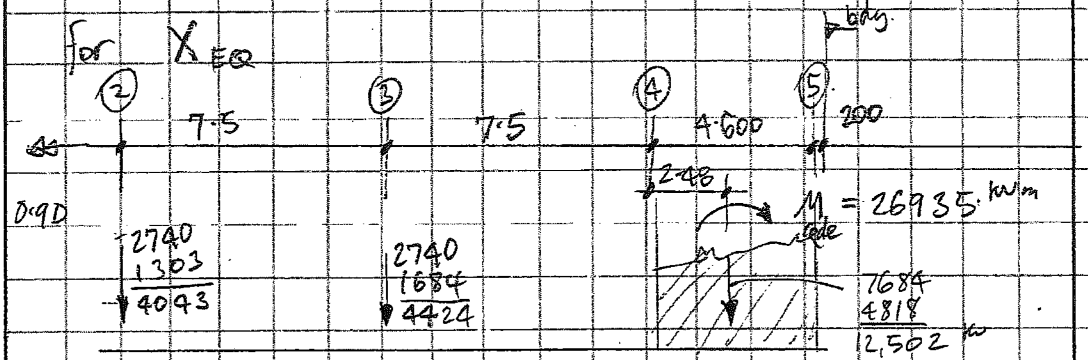
for footing over gravel, settlement allow = 105 kPa

bearing, $D/B = \frac{1 \cdot 0}{8 \cdot 2}$ 0·12

allow ≃ 150 kPa.

Check loads for seismic actions:

for X_{EQ}



column lines 2 & 3, 0·9D each = 2×1370 = 2740
walls, lines 4 to 5 0·9D total = (From F2) = 7684

est. weight of footings

line 2 pads = 2×4·0m×4·0m × 0·8 × 23·5 = 602 kN
beams = 1·5×1·0 × 24m × 23·5 = 846
1448
×0·9 = 1303 kN

line 3 pads = 602, beams 36m = 1269, tot = 1871×·9 = 1684

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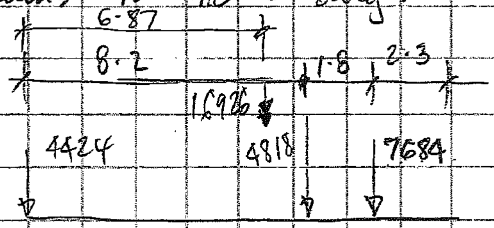
lines 4 to 5

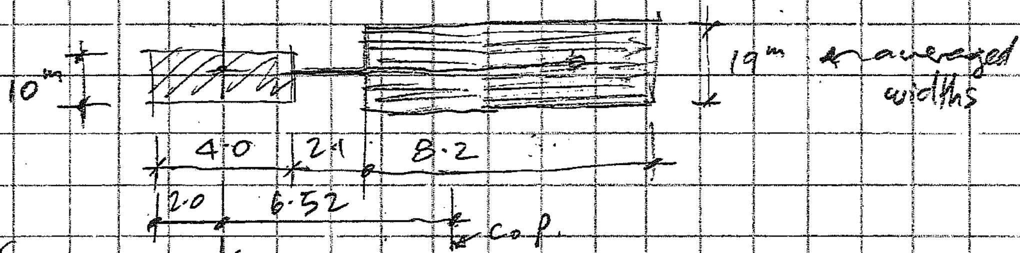
pads = $140^{kN/m^3} \times 0.8^m \times 23.5 = 2632^{kN}$

beams $1.0 \times 1.5 \times 62^{kN} \times 23.5 = 2186$

4818^{kN}

try for beams to line 3 only:





for 0.9D loads as above.

tot load = 16926^{kN}

area = $40 + 156 = 196^{m^2}$

av press = $\frac{16926}{196} = 86$ kPa

$\bar{x}$ for loads $= \frac{(4424 \times 0) + (4818 \times 8.2) + (7684 \times 10)}{16926}$

$= 6.87$

for idealised footing.

$A_1 = 4 \times 10 = 40$ $\quad x_1 = 0$ $\quad A_1 x_1 = 0$

$A_2 = 19 \times 8.2 = 156$ $\quad x_2 = 8.2$ $\quad A_2 x_2 = 1279$

$\Sigma A = 196$ $\quad \Sigma A_x = 1279$

$\bar{x} = \frac{\Sigma A x}{\Sigma A} = \frac{1279}{196} = 6.52^m$

∴ v. close to 6.87 ⇒ av press consistent over complete footing (see F10)

with OTM = 26935^{kNm}, $e = \frac{Pe}{P} = \frac{26935^{kNm}}{16926^{kN}} = 1.59^m$

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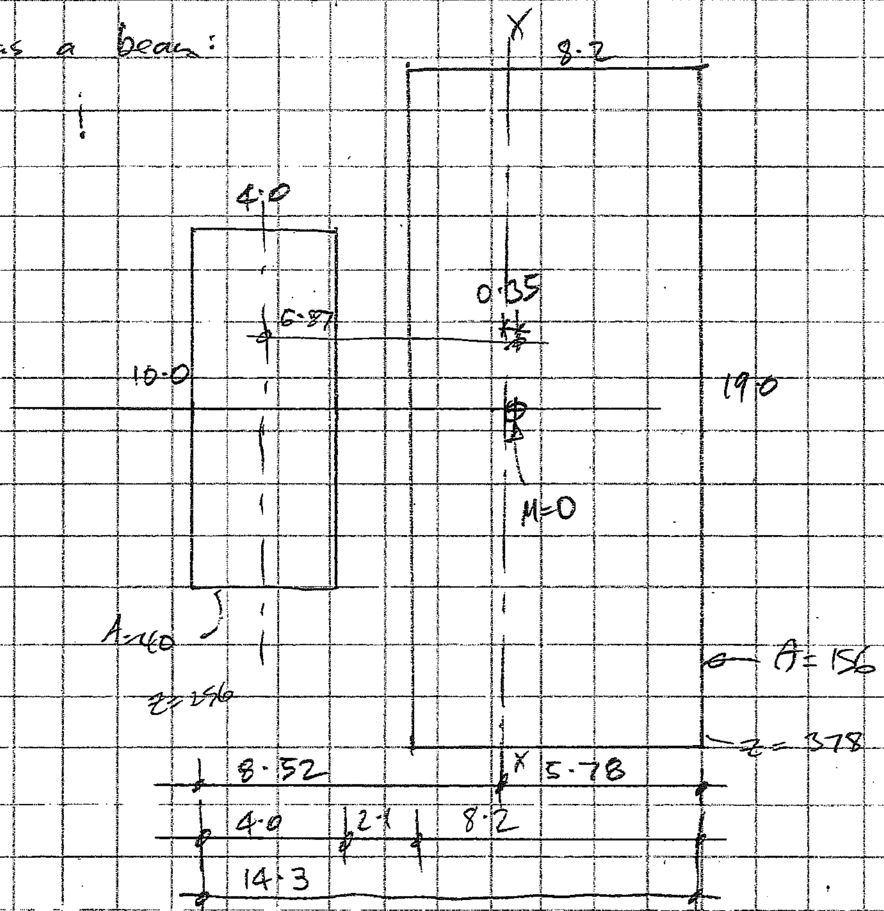
Hence calculate max pressures:

Treat as a beam:



Calculate $I_{xx} = \frac{4^3 \times 10}{12} + \frac{19 \times 8.2^3}{12} + (40 \times 4.52^2) + (156 \times 1.68^2)$

$= 2184 \text{ m}^3$

$Z_{bot} = \frac{2184}{8.52} = 256$

$Z_{top} = \frac{2184}{5.78} = 378$

ie for No eq $e = 6.87 - 6.52 = 0.35 \text{ m}$

$BM = 16926 \times .35 = 5924 \text{ kNm}$

$\sigma_{max} = 86 + \frac{5924}{378} = 86 + 16 = 102 \text{ kPa}$ less

$\sigma_{min} = 86 - 16 = 70 \text{ kPa}$ less.

For eq, additional $e = 1.59 \text{ m}$ ie $e = 1.59 + .35 = 1.94$

$BM = 1.94 \times 16926 = 32836 \text{ kNm}$

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$$\sigma_{max} = 86 + \frac{32836}{378} = 86 + 86 = 172 \text{ kPa}$$

$$\sigma_{min} = 86 - \frac{32836}{256} = 86 - 128 = -42 \text{ kPa}$$

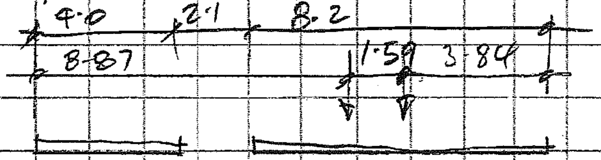
not possible

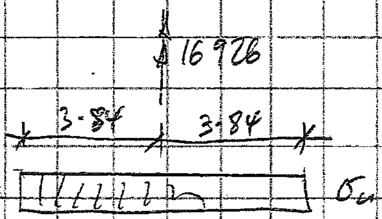
!

other way, $e = 1.59 - .35 = 1.24$ $M_u = 20988$

$\sigma_{max} = 86 + 55 = 141$ OK

compare "plastic" design. min $= 86 - 81 = 5$ OK.





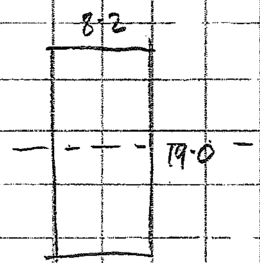
$\sigma_u \times 2 \times 3.84 \times 19.0 = 16926$

$\sigma_u = \underline{116}$ kPa

max σ_u for EQ = 116 kPa.

Compare Y direction for X_{EQ} - $M = 4223$ kNm

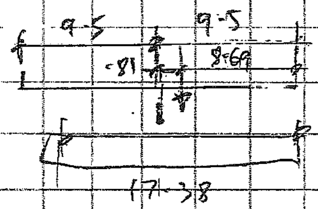
p522 Y_{EQ}, $M = 10,151$ kNm.



ie $P = 7684 + 4818 = 12,502$ kN

$Pe = M = 10,151$ kNm

$e = \frac{Pe}{P} = \frac{10151}{12502} = 0.81$ m



$\sigma_u \times 8.2 \times 17.38 = 12502$

$\sigma_u = \underline{88}$ kN/m²

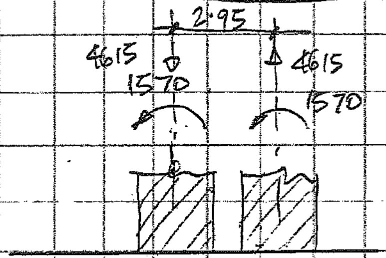
ie less than X_{EQ} figure - not critical.

allow bearing under footing 8.0 m wide:

$L/B = 18/8 = 2.25$ $D/B = 1/8 = .125$

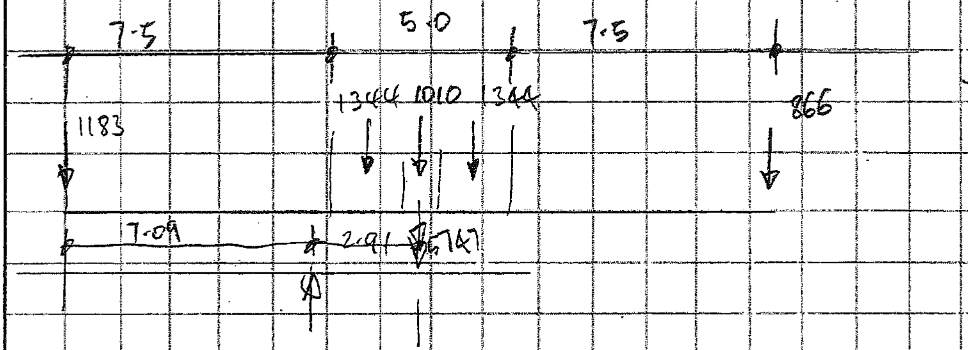
allow bearing $= 200 \text{ kPa} \times \left[0.5 + 0.5 \times \frac{2.8}{1+8}\right] = 200 \times .656 = 131$ kPa > 110 OK

Shear Wall on line 1



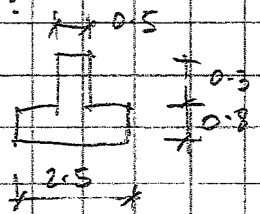
max. actions pp S19 & S20
for 0.57 S
Y eq, e_{min}

Total OTM = $(4615 \times 2.95) + (1570 \times 2)$
= 16,754 kNm



0.9D : From col C1 = 1183
walls = 1344

Foundation beam :



vol = $2.15\,m^2 \times 20\,m$
= $43\,m^3$
wt = 43×23.5
= 1010 kN

$\Sigma Wt = 1183 + 1344 + 1010 + 1344 + 866$
P = 5747 kN

Pe = OTM = 16,754 kNm

$\therefore e = \frac{Pe}{P} = \frac{16754}{5747} = 2.91\,m$

$\sigma_u \times 2.5\,m \times 2 \times 7.09\,m = 5747\,kN$
$\sigma_u = 162\,kPa$

allow bearing @ 2.5 wide $Fd^k = 105\,kPa + 18 = 123$

for $3.0\,m$ wide Fd^k ~~σ_u~~ = 135 & 123

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Structural Design of footings:

Pads to columns C2, D2, C3, D3

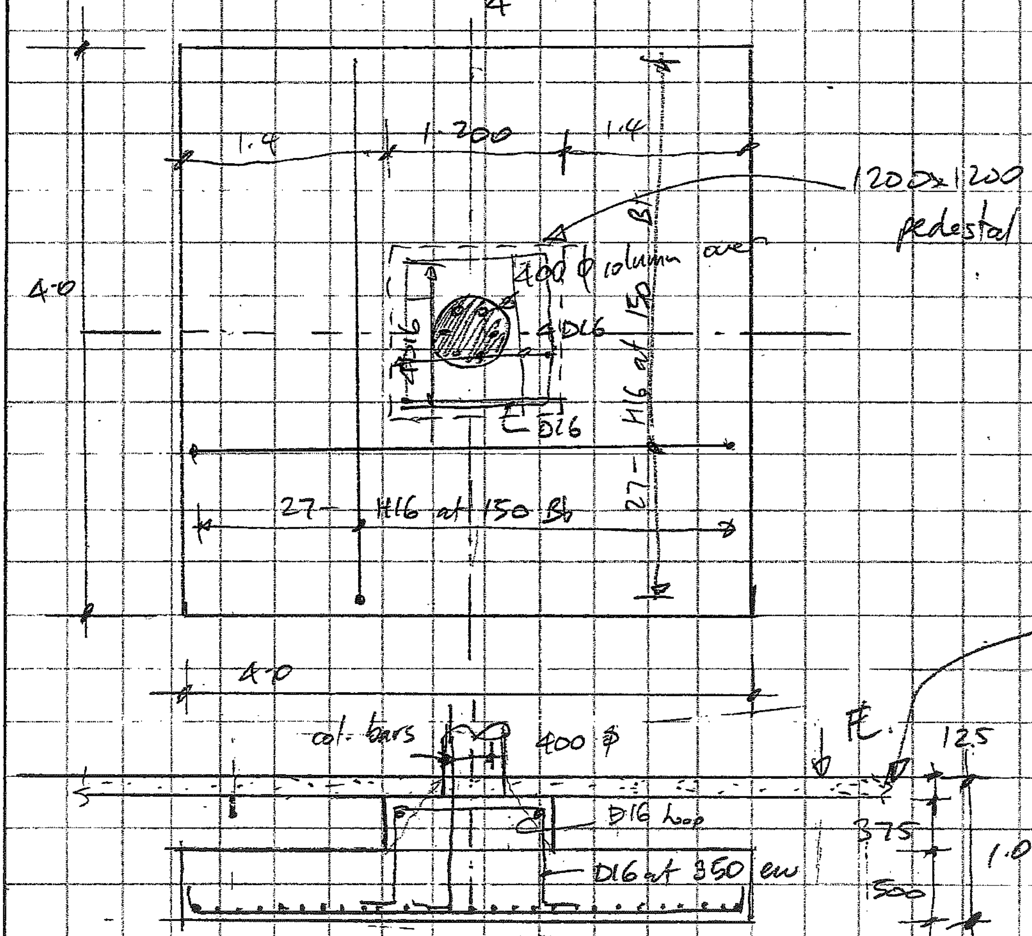
$P_{1.4D+1.7L} = 2729$ kN

From P. F3, use 4.0×4.0 footing.

footing wt. = 4.0×4.0×0.5×23.5 = 188 kN
×1.4 = 263

∴ P_u = 2729 + 263
= 2992 kN

$\sigma_u = \frac{2992}{4^2} = 187$ kN/m²



Floorslab
use 125 conc
for car park
unreinf.

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punching shear:
for 500 deep pad.

$P_u = 2992^{kN} - \left[187^{kN/m^2} \times \frac{\pi \times .92^2}{4}\right]$

$= 2992 - 124$

$= 2868^{kN}$

$v_i = \frac{2868{,}000^{N}}{\pi \times 920 \times 420 \times .85}$

$= 2.77 \ N/mm^2$

$v_c = 0.17\,(1 + 2.1)\sqrt{20}$

$= .51\sqrt{20} \ \ngtr .33\sqrt{20}$

$.33\sqrt{20} = 1.47 \ N/mm^2 \ < 2.77$ NG

consider using $1.2^m \times 1.2^m$ pedestal
then either for 500 deep pad

$P_u = 2992 - \left[187 \times 1.62^2\right]$

$= 2501$

$v_i = \frac{2501000}{.85 \times 4 \times 1620 \times 420} = 1.08 < 1.47$ OK

or for 875 deep pad.

$P_u = 2992 - \left[187 \times 1.2^2\right]$

$= 2722$

$v_i = \frac{2722{,}000}{.85 \times 875 \times 4 \times 1000} = .76 < 1.47$ OK

Pad reinforcement:

(a) about face of pedestal.

$M_u = 187 \times \frac{1.4^2}{2} = 184 \ kNm/m$

$d = 420, \quad A_s/m = 1800 \ mm^2/m$ D
1302 H

for H16 at 150, $A_s/m = 1340$, $a = 30$, $d_{min} = 386$

$M_u = .9 \times 1340 \times 380 \times 386 = 177 \ kNm/m$.

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(b) abt. face of column

$M_u = \frac{187 \times 1.8^2}{2} = 303$ kNm/m

Tot. $M_u = 4 \times 303 = 1211$ kNm.

for 27 - H16, $A_s = 5427$

$a = \frac{5427 \times 380}{.85 \times 20 \times 1200} = 101$

$jd = 875 - (75 + 16 + 8 + 51)$

$= 725$

$M_u = .9 \times 5427 \times 380 \times 725$

$= 1346$ kNm > 1211 kNm OK.

Pads for columns B2, B3, E2, E3

$P_{1.6D+1.7L} = 2355$ kN.

From p. F5
pad wt. for 3.8×3.8, $= 170$ kN

$\times 1.4 = 238$

$\therefore P_u = 2355 + 238 = 2593$ kN

$\sigma_u = \frac{2593}{3.8^2} = 180$ kN/m²

Punching shear: OK by inspection (keep 1.2 sq pedestal)

Bending moment:

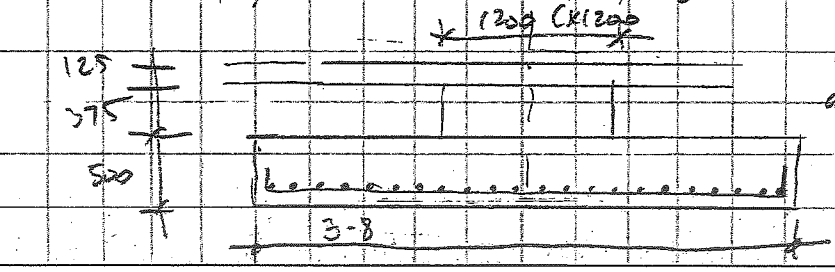
$M_u = \frac{180 \times 1.3^2}{2} = 152$ kNm/m

$d = 420$, $A_s/m = 1500$ mm²/m D

$= 1085$ M

for H16 at 175, $A_s = 1148$

B2 ~~B3~~
normal areas
use same as C2 etc.



other details as for C2, D2 etc.

22-H16 at 175 ew B

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For soft areas, 4.5 x 4.5 pad.

pad wt = 333

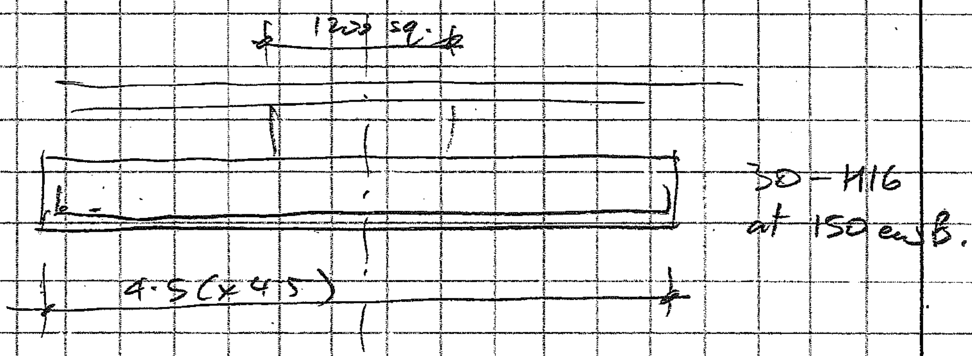
P_u = 2688

σ_u = 133 kPa

E2 E3 soft area

$M_u = \frac{133 \times 1.65^2}{2} = 181$ kNm/m.

⇒ use H16 at 150 ew as for C2



Foundation Line A

Loads: from columns.

$P_{DL} = \frac{625 \text{ kN}}{7.5 \text{ m}} = 83$ kN/m

from blockwalls 45 kN/m

128 kN/m

1.7 wide ⇒ 75 kPa from p. F7.

UDL on wall: $w_u = \frac{911}{7.5} + 1.4 \times 45$

(neglect selfwt of fdn as it causes no BM)

= 121 kN/m + 63 kN/m

= 184 kN/m ÷ 1.7 m = 108 kN/m²

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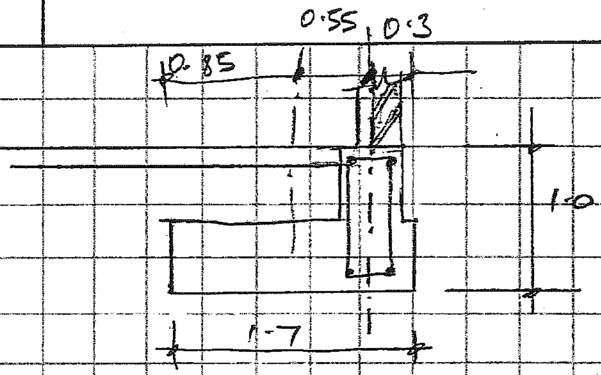
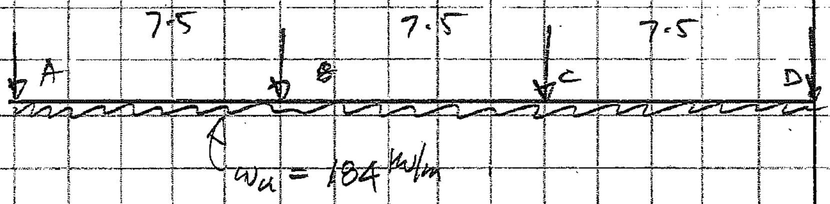
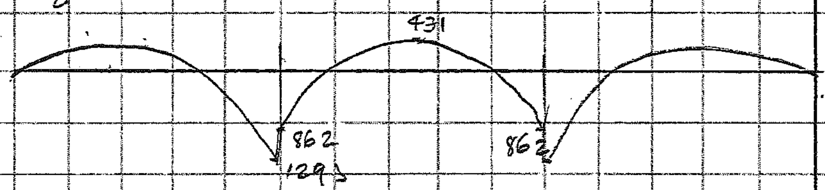


vertical BM =

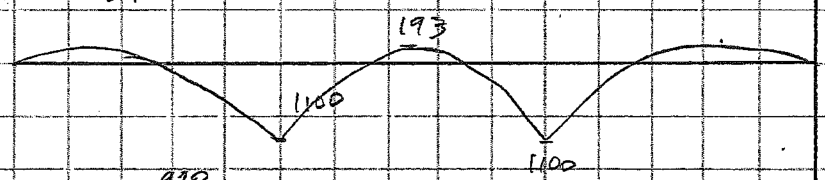


$$SSM = \frac{184 \times 7.5^2}{8} = 1293 \text{ kNm}$$

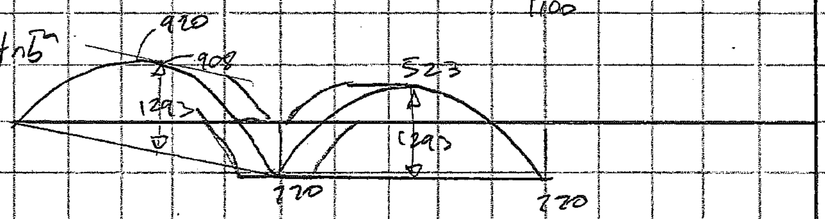
FEM's



$$\frac{D_{BA}}{D_{BC}} = \frac{3}{4} = \frac{.43}{.57}$$



with 30% redistribn



for M = 770 kNm, b = 500, M_u/m = 1560
d = 900, A_s/m = 7600 D
5355 H
A_s = 2677 H

A_s		a	d	rd	M_u
4H32	3220	143	909	838	922
2H32	1610	71	909	874	481
2H32 & 2H24	2515	111	909	854	735
3H32					721

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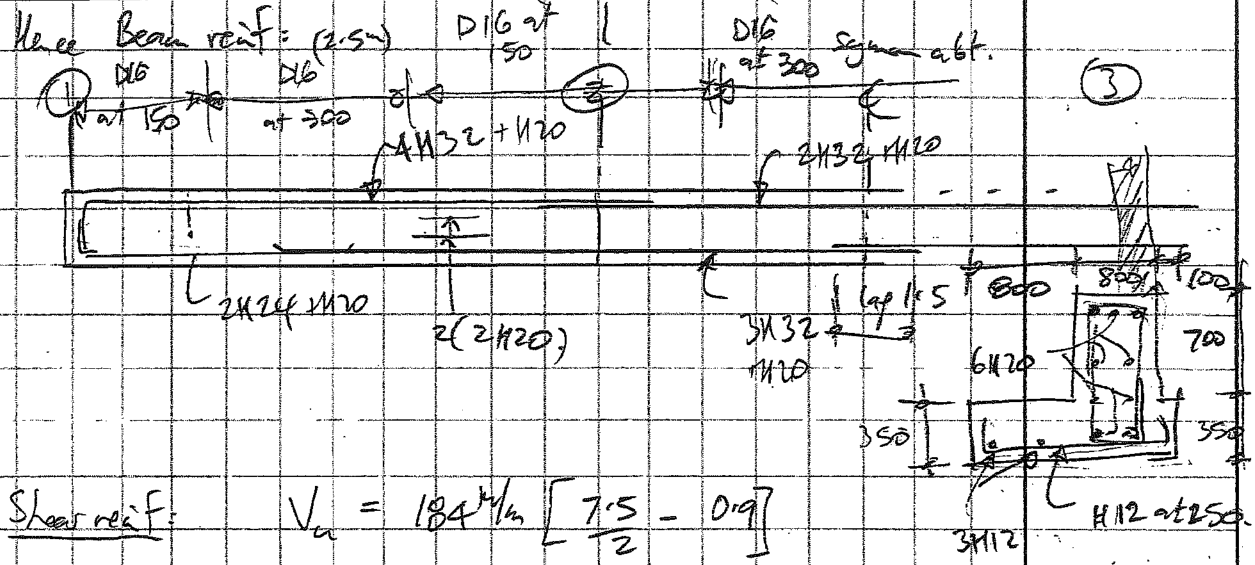
Hence Beam reinf: (2.5m) (5.0m) (2.5m) sym about ③



Shear reinf: $V_u = 184^{kN/m}\left[\frac{7.5}{2} - 0.9\right]$

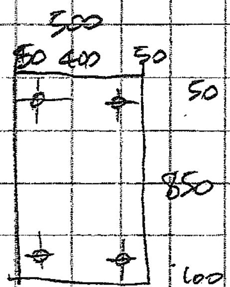
$= 184 \times 2.85$

$= 524^{kN}$

$v_i = \frac{524{,}000^{N}}{.85 \times 500 \times 920} = 1.34^{N/mm^2}$

Torsion $T_u = 524^{kN} \times 0.55^{m}$

$= 288^{kNm}$



$A_o = 400 \times 850$

$= 340{,}000^{mm^2}$

$A_c = 500 \times 1000$

$= 500{,}000$

$p_c = 3000$

$p_o = 2500$

$t_c = .75 \times \frac{500{,}000}{3000} = 125$

$t_o = .75 \times \frac{340{,}000}{2500} = 102$

$v_{ti} = \frac{288{,}000{,}000^{N\text{-}mm}}{.85 \times 2 \times 500{,}000^{mm^2} \times 102^{mm}}$

$= 3.32\ N/mm^2$

$\therefore v_i + v_{ti} = 1.34 + 3.32$

$= 4.66$

max allow $v_i + v_t = 0.2 f'_c = 0.2 \times 20 = 4.0\ N/mm^2$

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if beam is 600 wide:

$v_i = \frac{524,000}{.85 \times 600 \times 920} = 1.12$

$A_o = 425,000$

$A_{co} = 600,000$

$p_c = 3200$

$p_o = 2700$

$t_c = 141$

$t_o = 118.$

$v_{ti} = \frac{288,000,000}{.85 \times 2 \times 600,000 \times 118} = 2.39$

$v_i + v_{ti} = 3.51 < 4.0$ OK

Reinf: for shear: $v_b = \left(0.07 + 10 \cdot \frac{1616}{600 \times 920}\right)\sqrt{20}$

$= 0.44$ N/mm² $< .17\sqrt{f'_c}$

$\therefore v_s = 1.12 - 0.44$

$= 0.68$ N/mm²

$A_v = \frac{.68 \cdot 600 \cdot 1000}{275}$

$= 1483$ mm²/m

torsion: $A_t = \frac{v_{ti}\, t_o\, s}{f_y}$

$= \frac{2.39 \text{ N/mm}^2 \times 118 \text{ mm} \times 1000 \text{ mm}}{275 \text{ N/mm}^2}$

$= 1026$ mm²/m.

$\therefore$ min vert steel $= 1026 + \frac{1483}{2} = 1767$ mm²/m each leg.

for D16 at 110 crs high!

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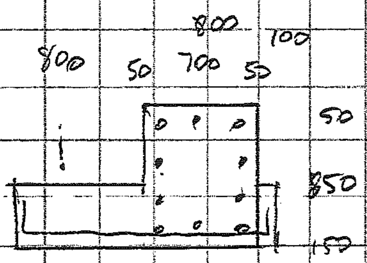
Make beam 800 wide:



extra 6 H20 horiz bars for torsion.

$v_i = \dfrac{524{,}000}{.85 \times 800 \times 920} = 0.84$

$A_o = 700 \times 850 = 595{,}000$

$A_{co} = 800 \times 1000 = 800{,}000$

$p_c = 3600$

$p_o = 3100$

$t_c = \dfrac{.75 \times 800{,}000}{3600} = 167$

$t_o = \dfrac{.75 \times 595{,}000}{3100} = 144$

stirrup: $v_s = .84 - .44 = 0.40$

$A_v = \dfrac{.40 \times 800 \times 1000}{275} = 1164$

$v_t = \dfrac{288{,}000{,}000}{.85 \times 2 \times 800{,}000 \times 144} = 1.47$

$A_t = \dfrac{1.47 \times 144 \times 1000}{275} = 770$ mm²/m

total each leg $= 770 + \dfrac{1164}{2} = 1352$ mm²/m

for D16 at 150, $A_v = 1340$ mm²/m.

$A_l = \dfrac{v_{ti}\, t_o p_o}{f_y} = \dfrac{1.47 \times 144 \times 3100}{380} = 1726$ mm²

6 H 20 = 1884 mm² = OK.

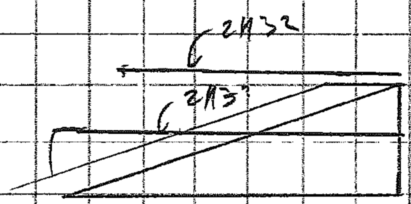
for middle 1/3 of beams.
torsion reif is 1/2
shear reif is less than 1/2
⇒ use D16 at 300 ctrs.

Cross beams to take torsion:
$BM = 184^{kN/m} \times 7.5^m \times 0.55^m$
$= 759^{kNm}$

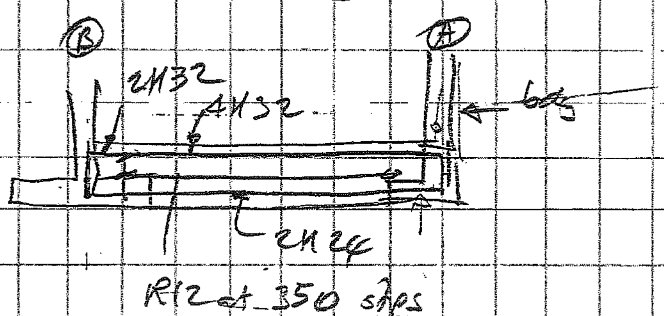
uplift on cross column $= \frac{759^{kNm}}{4.6^m} = 165^{kN}$

P_u drag $= 1177 > 165$ OK
4H32, $jd = 800$ $M_u = 886 > 759$

BMD:



Shear: $V_a = \frac{165{,}000^N}{.85 \times 500 \times 920} = 0.42$
V_c $= 0.4$
$V_s = V_{s\,min} = \frac{.345 \cdot 500 \cdot 000}{275} = 627^{mm^2/m}$ v_s 600 width
R16 - 450
For R12 at 350 $A_v = 2 \times 323$ $v_s = .41$
$= 626$. p. F22



H12 at 250

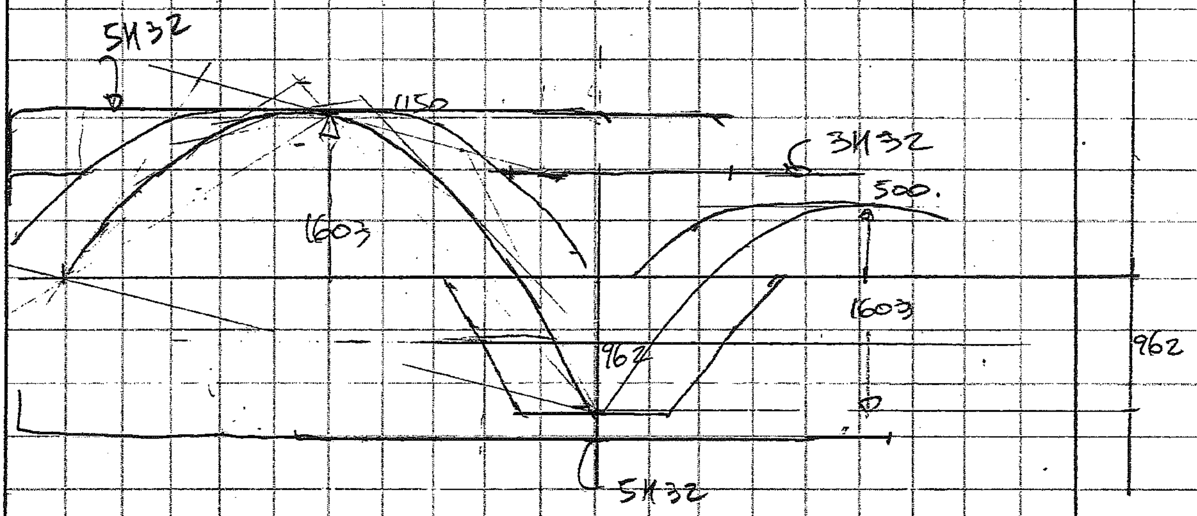
outstand footing:
$M_u = 108 \times .8^2 = 35^{kNm/m}$
3500/A $d = 267$ $A_s = H16$ at $450 = 447$ $M_u = .9 \times 447 \times 380 \times 262 = 40.0$
3500/A $d = 210$, $A_s = H12$ at $250 = 452$ $M_u = .9 \times 452 \times 380 \times 264 = 40.8$

Front Wall, line F

p_u = nett load on footings
$= \frac{1717 \text{ kN}}{7.5 \text{ m}} = 229$ kN/m

av. pressure $= \frac{229}{2.2} = 104$ kN/m² ult.

compare with line A, $w_u = 184$ kN/m
scale up BMD on F17 $\times \frac{229}{184} = 1.24$



Shear: max $V_u = 1.24 \times 524$ kN
$= 650$ kN

$v_i = \frac{650{,}000 \text{ N}}{.85 \times 600 \times 920} = 1.39$ N/mm²

$v_c = \left(.07 + 10 . \frac{4020}{600 \times 920}\right)\sqrt{20} = 0.64$

$v_s = 1.39 - 0.64 = 0.75$

$A_v = \frac{.75 \times 600 \times 1000}{275} = 1636$ mm²/m
$= 818$ mm²/m each leg.

for D16 at 200 $A_v = 2010$ $v_s = 0.92$ N/mm²

$A_{v\,min} = .345 \times \;\; = 753$

D16 at 400 $A_v = 1005 > 753$ $v_s = .46$

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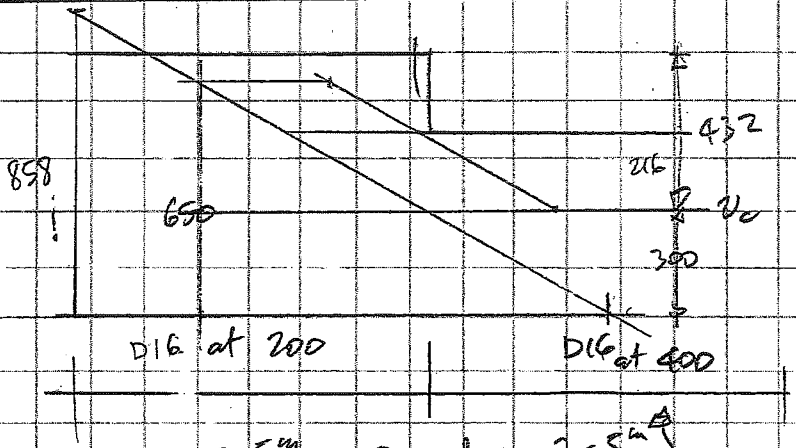
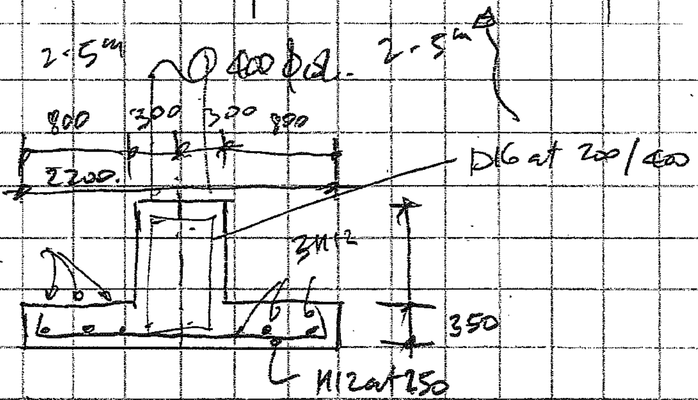


Spread Footings:



$CLM = 104 \text{ kN/m} \times \frac{.8^2}{2} = 33.3 \text{ kNm/m}$

use H12 at 250 as on p. F21

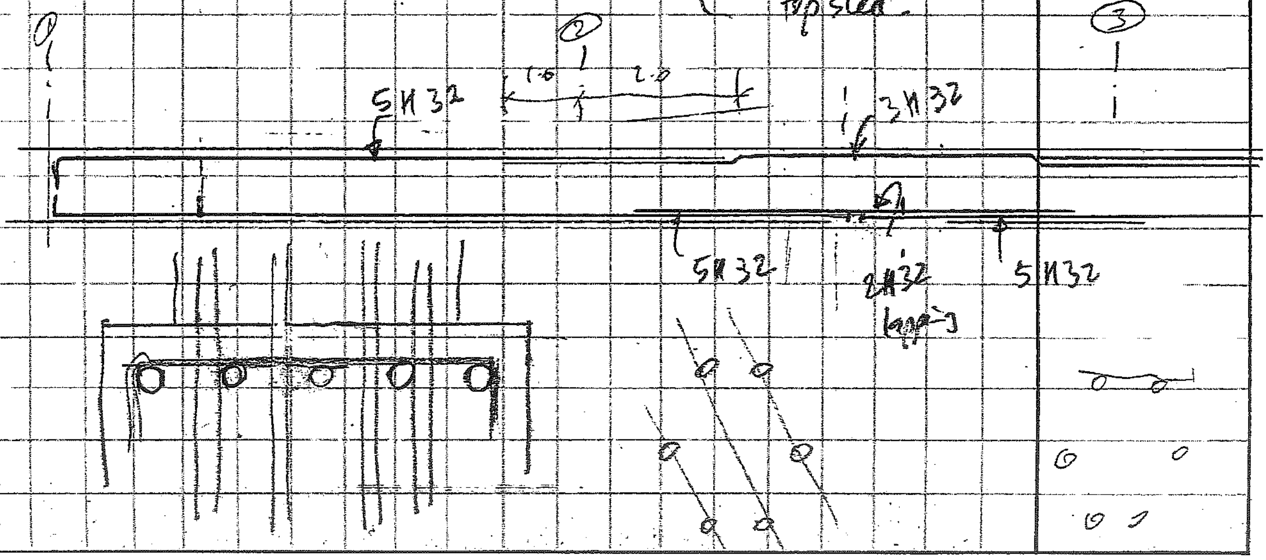
Horiz reinf = $.0018 \times 269 \times 1000 = 484 \text{ mm}^2/\text{m}$

$d = 269$ = H12 at 250 (452)

Assume 200 (565)

Main reinf:

l_d for H32 = 3.07 m
$f'_c = 20$
spacing = 80 bars
top steel



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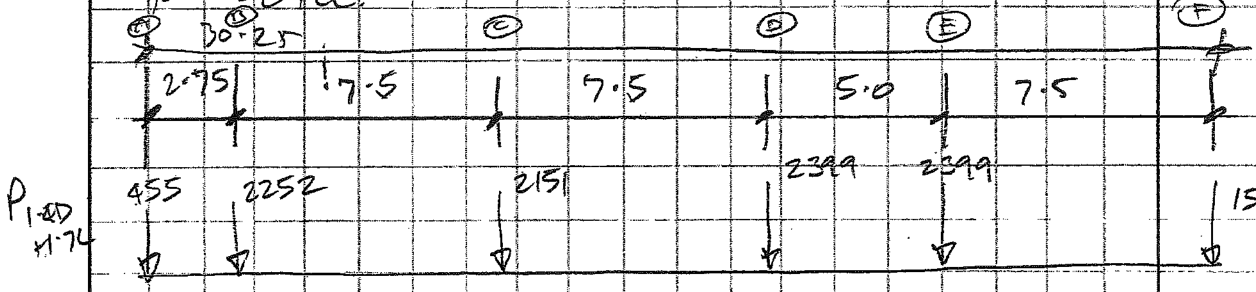
Line 1 footing

for DL+LL



$\Sigma P = 11{,}159$ kN

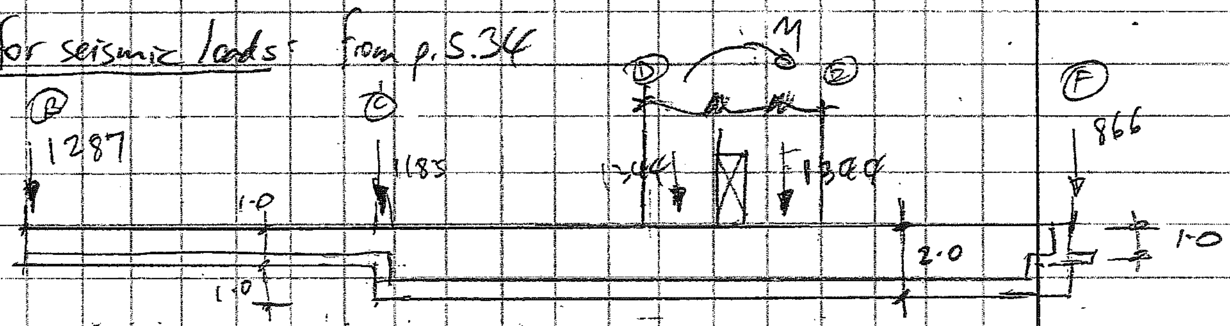
average $w_u = \frac{11{,}159}{30.25} = 369$ kN/m

For 3·0 m width, $w_u = 123$ kN/m²

max BM $\approx \frac{w_u l^2}{10} = \frac{369 \times 7.5^2}{10} = 2077$ kNm

SSM $= \frac{w_u l^2}{8}$. 2595 kNm.

for seismic loads: from p. S.34



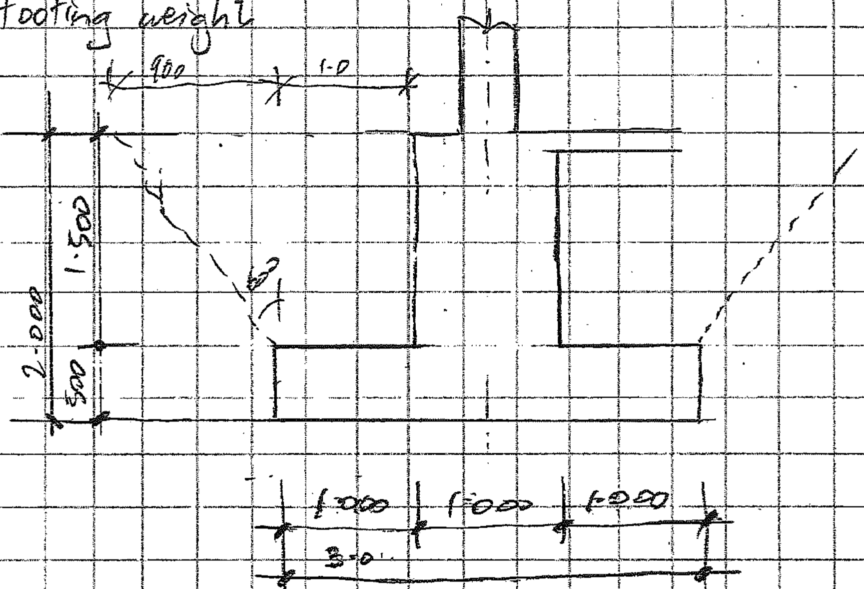
$M_{code} = 16{,}754$ kNm, SM $= 0.8 \times 0.8 = 0.64$

$M_o = 28{,}205$ kNm equiv. to SM $= \frac{.64 \times 28205}{16754}$

$= 1.08 < 1.6$

∴ use $M_o = 28205$ kNm, with 0·9D vertically as shown on p. F12.

Footing weight



$wt = (3 \times \cdot 5) + (1\cdot 5 \times 1\cdot 0) \times 23\cdot 5 = \underline{70\cdot 5}$ kN/m

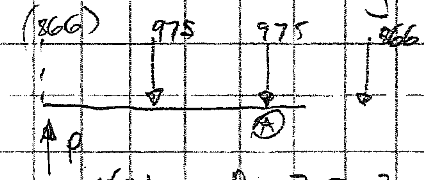
wt of soil above for uplift

$= 1\cdot 5 \times (1\cdot 0 + 1\cdot 0 + 0\cdot 9) \times 20\cdot 0 = \underline{87\cdot 0}$ kN/m

wt available at F.

0·9D on column at F = $\underline{866}$ kN

load on cross footing line F



M(A) $P \times 7\cdot 5 \times 2 = 975 \times 7\cdot 5$

$P = 488$ kN.

BM on footing $= 488 \times 7\cdot 5 = 3660$ kNm

for SM 32 B, from page F22, $M_u = 1150$ kNm

∴ available $P = \frac{1150}{7\cdot 5} = \underline{153}$ kN.

footing wt, line F $= (2\cdot 2 \times \cdot 3) + (0\cdot 7 \times \cdot 6) \times 23\cdot 5$

$= 25\cdot 4$ kN/m $\times 3\cdot 75 = \underline{95}$ kN.

soil over line F footing $= 0\cdot 7(\cdot 8 + \cdot 8 + \cdot 7) \times 20$

$= 32$ kN/m $\times 3\cdot 75 = \underline{120}$ kN

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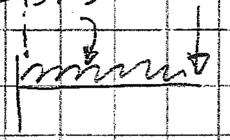
hence total R.M. (E) to (F)

$w = 70.5 + 87 = 157.5$ kN/m

$P = 866 + 153 + 95 + 120 = 1234$ kN



$\therefore RM = (1234 \times 7.5) + (157.5 \times 7.5^2/2)$
$= 9255 + 4430$
$= 13,685$ kNm.

RM from C to D is greater, by inspection.

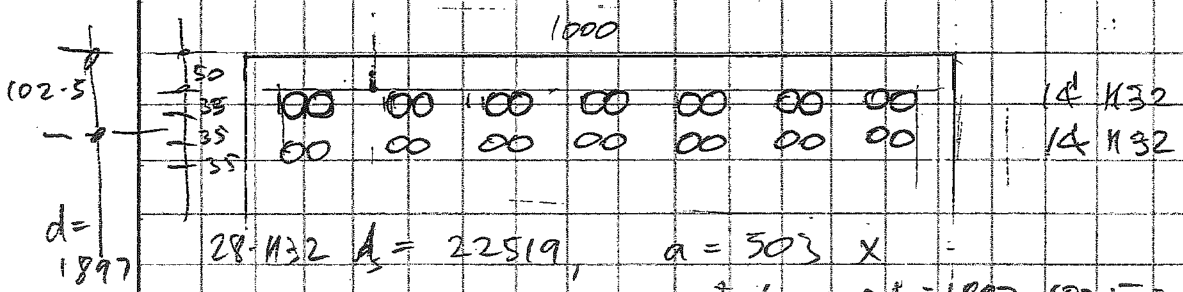
RM required each way $= \frac{28,205}{2} = 14102$ kNm at overstress $\frac{}{8}$

or $= \frac{16754}{2} = 8377$ code load.

For $M_u = 14102$ kNm, 1.0 wide, $\frac{M_u}{m} = 14102$ kNm.

For 2.0 m deep footing, $d = 1900$
ie $14102 = .9 \times A_s \times 380 \times 1800$
$A_s = 22907$
for 29-H32, $A_s = 23323$
$p = \frac{23323}{1000 \times 1900} = 0.0123$

$p_{max} = \frac{7}{380} = 0.0184 > .0123$ OK.



28-H32 $A_s = 22519$, $a = 503$ X

unescamped steel, $\Rightarrow jd = 1897 - 102.5 = 1795$

$M_u = 1.0 \times 22519 \times 382 \times 1795 = 15,360$ kNm
> 14102 OK.

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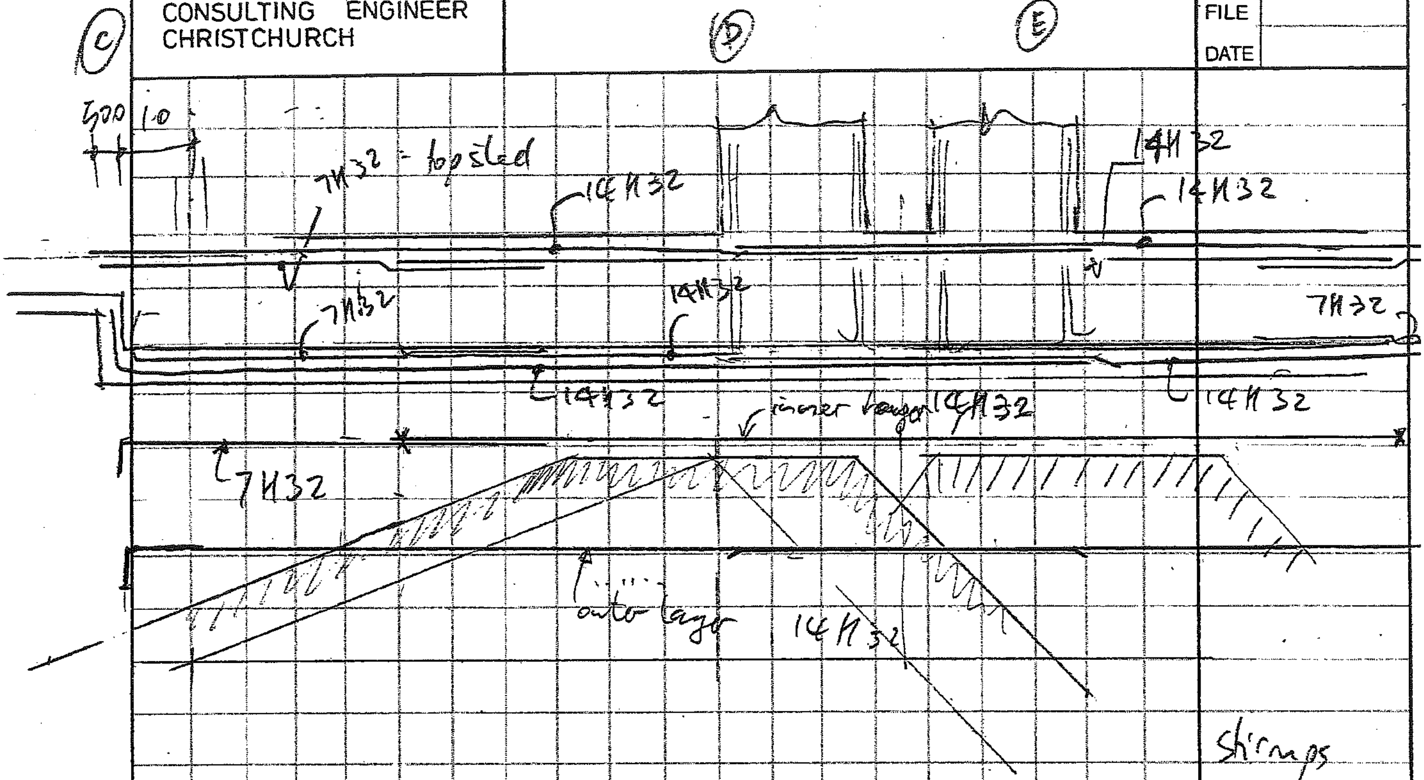

Stirrups D16 at 250 Full length

Shear: max $V_{code} \approx \frac{8377\,\text{kNm}}{7.5\,\text{m}} = 1117\,\text{kN}$

max $V_o \approx 1234 + (157.5 \times 7.5) = 2415\,\text{kN}$

$v_i = \frac{2415000\,\text{N}}{1.0 \times 1000 \times 1900} = 1.27\,\text{N/mm}^2$

$v_b = (.07 + 16 \times .0123)\sqrt{20} = 0.86\,\text{N/mm}^2$

$v_s = 1.27 - 0.86 = 0.41\,\text{N/mm}^2$

$A_v = \frac{.41 \times 1000 \times 1000}{275} = 1490\,\text{mm}^2/\text{m}$

745 each leg

for D16 at 250 $A_v = 804 \times 2$
$= 1608\,\text{mm}^2/\text{m}$

Joint shear:
max tension from wall steel = 5H28
$= 3080\,\text{mm}^2 \times .38$
$= 1170\,\text{kN}$

i.e. same steps OK.

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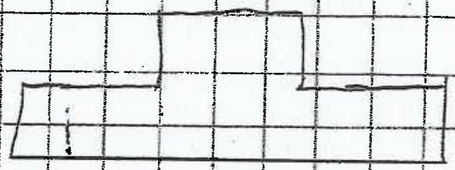
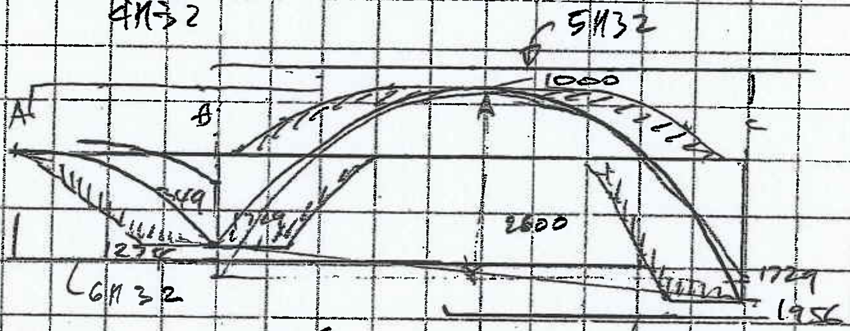
steel in beam beyond 2.0m deep region:

$\sigma_a = 147$ MPa.
$w_u = 369$ kN/m





due to DL/LL $\frac{369 \times 7.5^2}{12} = 1729$

$\frac{369 \times 2.75^2}{8} = 349$

$\frac{D_{BA}}{D_{BC}} = \frac{3}{2.75} \times \frac{7.5}{4} = \frac{2.046}{1} = \frac{0.67}{0.33}$

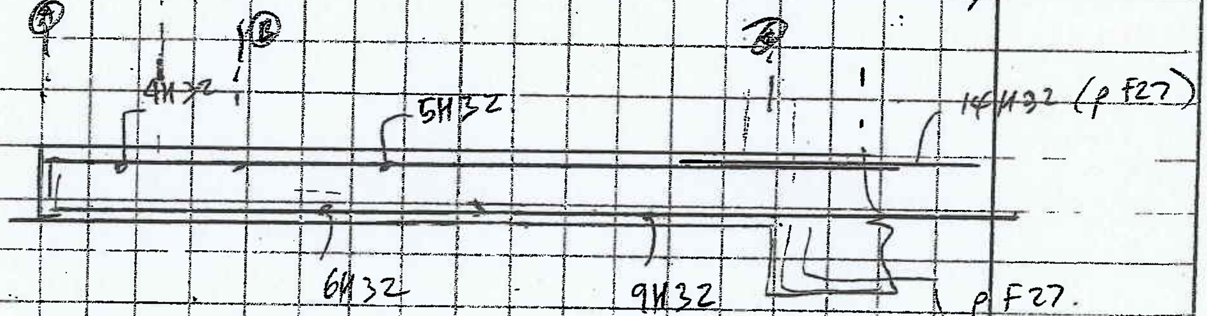
for $\frac{M_u}{m} = 1956$ $\frac{A_s}{m}$ reqd = 9500 D
6875 N

for 9-H32 $A_s = 7240$

$M_u = .9 \times 7240 \times 380 \times 870$
$= 2154$

for 1274 → 6 H32 (1450 kNm)
1000 → 5 H32 (1207)

$A_{s\,min} = \left(\frac{1.4}{f_y}\right) .0037 \times 1000 \times 890 = 3205$ mm²
→ 4 H32 (960)



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Shear: $V_u = 369 \text{ kN/m} (3.75 - 0.9)$
$= 1052 \text{ kN}$

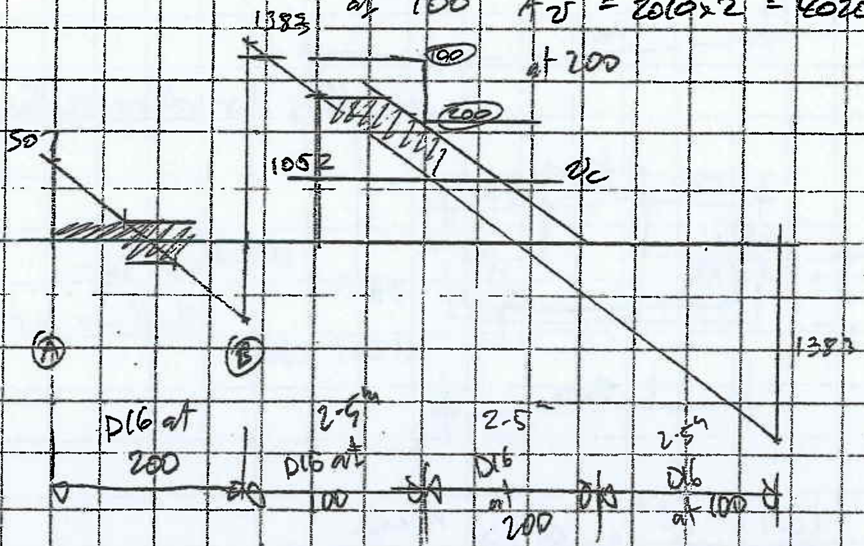
$v_i = \frac{1052,000}{.85 \times 1000 \times 870}$
$= 1.42 \text{ N/mm}^2$

$v_b = (.07 + 10 \cdot .0037)\sqrt{20}$
$= 0.48$

$v_s = 1.42 - .48$
$= 0.94 \text{ N/mm}^2$

$A_v = \frac{.94 \cdot 1000 \cdot 1000}{275}$
$= 3418 \text{ mm}^2/\text{m} \div 2 = 1709$

for D16 at 125 $A_v = 1608 \times 2 = 3216$
at 100 $A_v = 2010 \times 2 = 4020 \Rightarrow 819^{kN}$
at 200 410^{kN}



– same reinforcement applies on line 4

outstanding footing
1.4D + 1.7L, $\omega_u = 123 \text{ kN/m}^2$ from p. F24.
.9D + E, $\omega_u = 135 \text{ kN/m}^2$ from p. F12.

design for $\omega_u = 160 \text{ kN/m}^2$ to allow for some possible "rocking" of foundn?

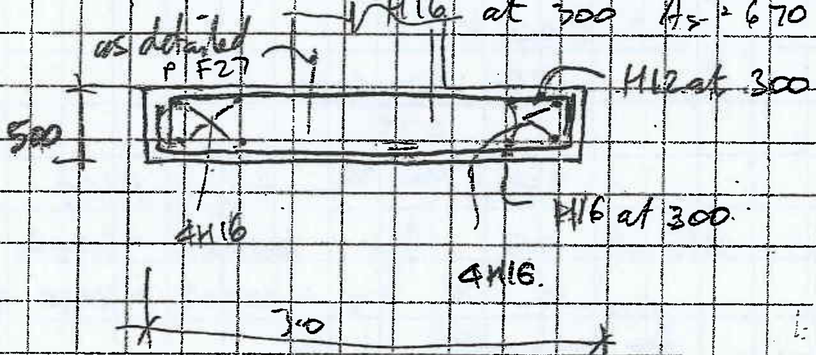
down load from uplift case
1.5 of soil + 0.5 of concrete
$\omega_u = (1.5 \times 20 \times 1.4) + (0.5 \times 23.5 \times 1.4) = 58 \text{ kN/m}^2$

max Pos BM $= \frac{160 \times 1.0^2}{2} = 80$ kNm/m

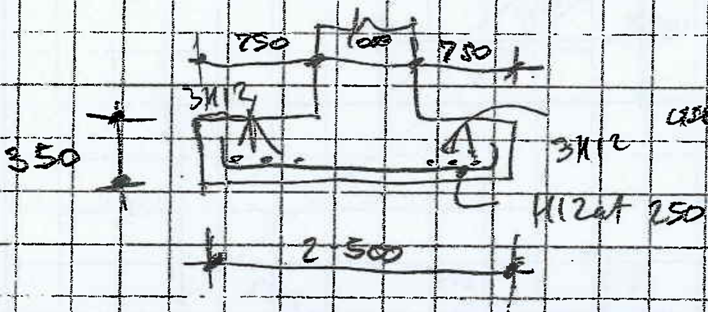
req $= 58/2 = 29$ kNm/m

for 500 mm footing, $A_{s\,min} = .0018 \times 420{,}000$
$= 756$ mm²/m $\div 2 = 378$

for H16 at 450, $A_s = 447$ $d = 412$ $M_u = 63$
H12 at 300: $A_s = 377$ $M_u = 53 > 29$
H16 at 300 $A_s = 670$ $M_u = 94 > 80$



use this for deep section



use this for shallow section

$M_u = \frac{147 \times .75^2}{2} = 41.3$ kNm/m

with H12 at 250, $M_u = 40.8$ kNm/m - OK.

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Footing to Wall 3

from page S37, $M_{code} = 1646$ kNm.
$SM = 1.0 \times 0.8 = 0.8$.

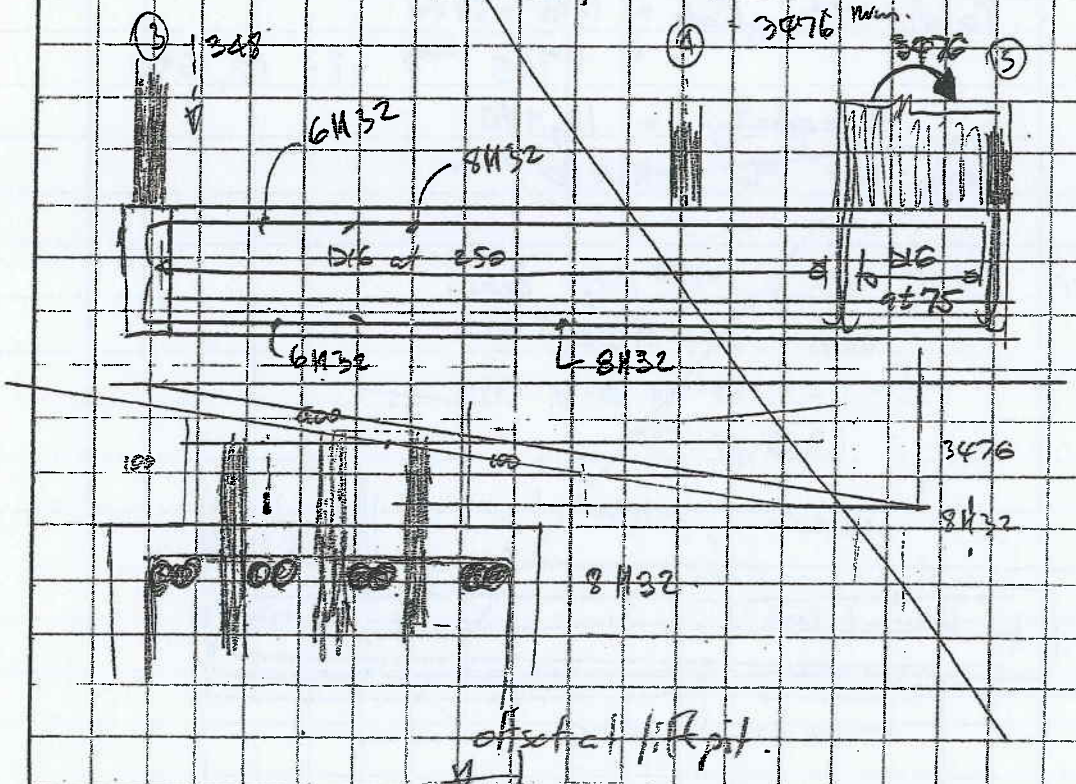
From p. S39, $M_o = 12,980$ kNm
equiv to $SM = \frac{.8 \times 12980}{1646}$
$= 6.30$

use $SM = 1.6$, by 3.5.12.4.
∴ $M_{design} = 1646 \times 2$
$= 3292$ kNm.

for 1800 deep x 1000 wide beam
$M_u/m = 3292$
$d = 1700$ reqd $A_s/m = 8800$ D
6078 H

for 8 - H32, $A_s = 6434$ mm²

use 600 wide beam, $M_u = .9 \times 6434 \times 380 \times 1580$
$= 3476$ kNm.



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Shear: $V_u = \frac{3476 \text{ kN}}{10} = 348$ kN.

$v_i = \frac{348,000}{.85 \times 600 \times 1700} = 0.40$

use $m = s_{bs} = \frac{.345 \cdot 600 / 1000}{275} = 752$

for D16 at 250 $A_v = 2 \times 804$
$= 1608$ OK.

Joint: max force from 12 H28 = 7380 × .38
= 2804 kN

$v_i = 3.23$
$v_c = .84$
$v_s = 2.39$
$A_v = 5223$ ⇒ D16 at 75
= 5360

→ Consider wall to take extra BM due to axial load couple, = 4161 kN × $\left(\frac{2.175}{2} + .15\right)$

= 5149 kNm.

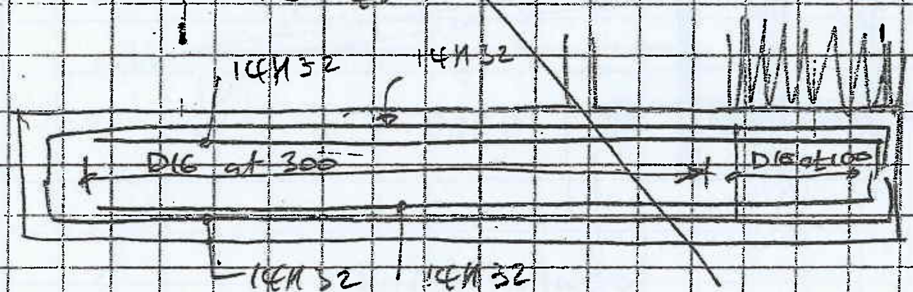
∴ Total OTM applied = 1646 + 5149
= 6795 kNm × 2 = 13,590.

M_o capacity = 12,980

∴ design for M_o = 12980 kNm

for 1.000 wide × 1.800 deep beam
with 28 - H32 T&B.

M_u = 1.0 × 22519 × 380 × 1595
= 13648 kNm



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Shear :

$V_u = \frac{12980}{10 \cdot 0} = 1298$

$v_i = \frac{1298000}{\cdot 85 \times 1000 \times 1700} = 0 \cdot 89$ N/mm²

$v_b = \cdot 84$

use $v_{s\,min} = \frac{0 \cdot 395 . 1000 . 1000}{275} = 1254$

D16 at 300 = 2×670 = 1340

Joint : from 12/28, $P_u = 2804$ kN

$v_i = 1 \cdot 94$

$v_c = \cdot 84$

$v_s = 1 \cdot 10$

$A_v = 3998$

fr - D16 at 100, = 4020.

Revise again - design as a ductile beam with plastic hinge possible inboard of the cross-beam on line 4 :

ie design BM = M_u = 6795 kNm

for 1·0 wide × 1·800 deep beam, 14 - H32 T & B

A_s = 11,260 , a = 251 jd = 1585

M_u = ·9 × 11260 × 380 × 1585 = 6096

use 16 - H32, M_u = 6967 > 6795.

Shear :

beam o'strength = 1·4 × 6967 = 9754 kNm

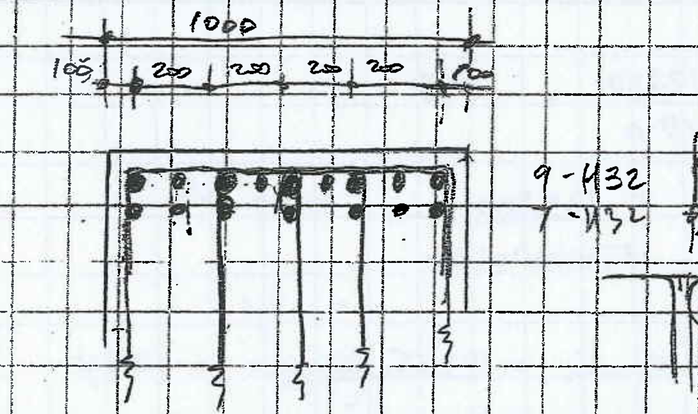
max shear / uplift at end = $\frac{9754 \text{ kNm}}{7 \cdot 5 \text{ m}}$ = 1300 kN

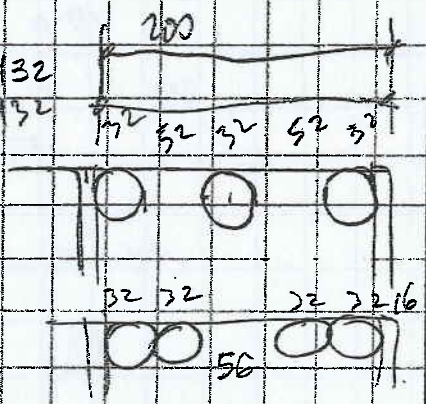
$\therefore v_i = \frac{1300,000}{\cdot 85 \times 1000 \times 1700} = 0 \cdot 90$ N/mm²

at p. hinge, $v_c = 0$

$\therefore v_s = 0 \cdot 90$

$A_v = \frac{0 \cdot 90 \times 1000 \times 1000}{275} = 3273$ mm²/m





12 desirable?

16 max

Check antibuckling requirements

min barsize = $A_{te} = \dfrac{\sum A_b f_y}{16 f_{yt}} \dfrac{s}{100}$

$A_{te} = \dfrac{3 \times 804 \times 380}{16 \times 275} \times \dfrac{150}{100} = 312 \text{ mm}^2$

max spacing $= d/4 = \dfrac{1700}{4} = 425$

$6 \times 32 = 192$

150

for 16 mm bars max $s = \dfrac{201}{312} \times 150 = 97$ mm

for 5 leg D16 at 100, $A_v = \dfrac{1005}{.1} = 10050$ mm²/m

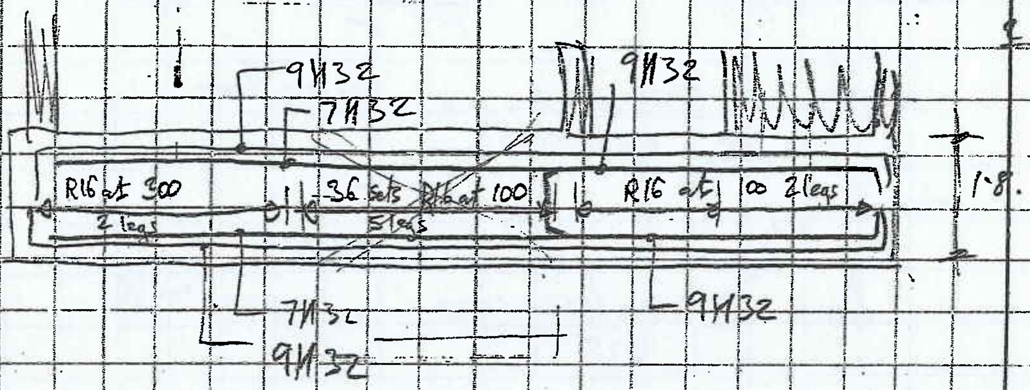
> 3273 reqd.

beyond p. hinge $v_c = \left(0.07 + 10 \cdot \dfrac{1260}{1000 \times 1700}\right)\sqrt{20} = 0.60$

$\therefore v_s = 0.90 - 0.60 = 0.30$

use min stps = D16 at 300

Beam 3
line D



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Wall 5

ρ_c $M_{code} = 4267 + \left(5082 \times \frac{3.125}{2} + .15\right)$

$= 4267 + 8702$

$= 12970^{kNm} \times 2 = 25\,940$

overstrength capacity of wall from p 547 = 22669 kNm

design beam for p'hinge. design M = 12970 kNm

Beam 1000 × 1800 deep

A_s reqd. ≈ 36 H32 = 28944 mm^2

$\rho = .017 < .0184$

12 H32 o o — 75
12 H32 o o — 64 — 139
7 H32 o o — 64 — 1661

$jd = 1522$

$M_n = 15066$ for 31 – H32

$M_n \geq 12973$ OK.

Shear $M_o = 1.4 \times 12973 = 18162$

max shear at end $= \frac{18162}{7.5} = 2421$ kN.

$v_i = \frac{2421{,}000}{.85 \times 1000 \times 1700} = 1.67$

at hinge v_c = 0

$\therefore v_s$ = 1.67

A_v reqd = 6072

antibuckling steel use R16 at 80 crs. 5 legs

$A_v = 12562$ mm^2/m > 6072 OK.

beyond hinge v_c = 0.60

v_s = 1.07

$A_v = \frac{1.07 \times 1600 \times 1000}{275} = 3890$ mm^2/m

use 4 leg D16 at 200.

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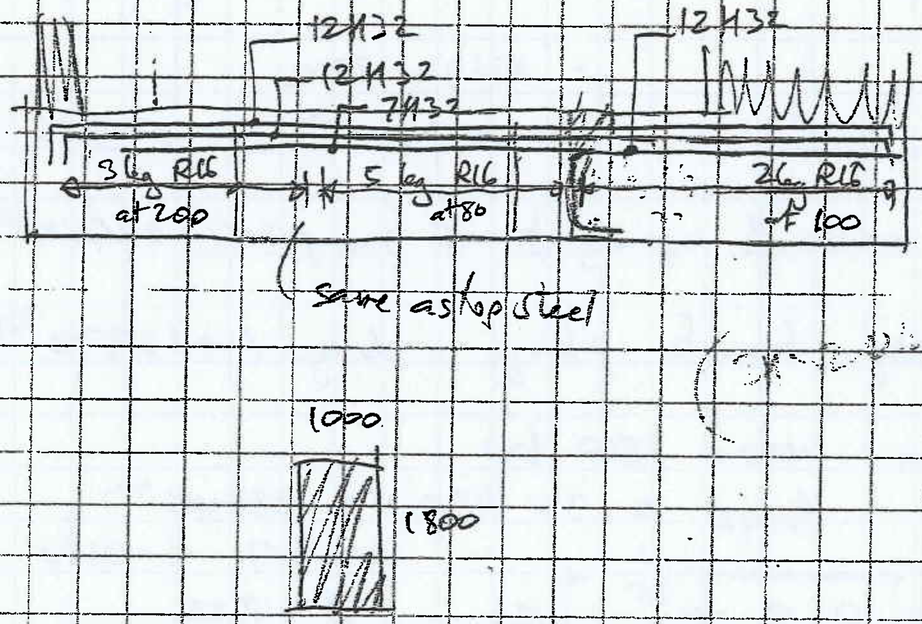
Beam 5



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Wall 7

From p. S51 $M_{code} = 3995$ kNm
$M_o = 5025$ kNm
~~design for $M_o = 5025$ kNm~~

design for phase 8
$M = 3995$

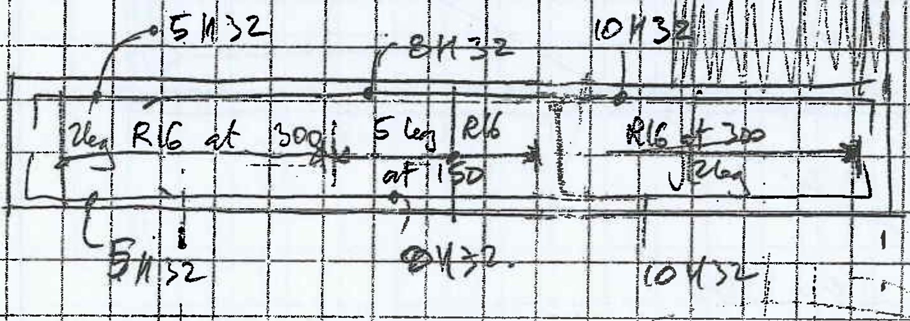
for 10 wide × 1·8 deep
10 - H32 bars, $A_s = 8040$ 8H32 = 6434
$a = 180$ ~~jd = 1619~~ jd 1640
$M_u = 1{\cdot}0 \times 8040 \times$ ~~380 × 1619~~ $M_u = 4010$
= ~~4946 kNm. 40%~~

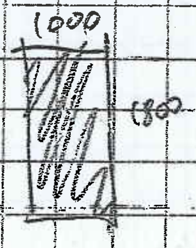
$V_{code} = \frac{3995}{10} = 400$ kN. $M_o = 1{\cdot}4 \times 4010 = 5614$
max $V_o = \frac{5614}{7{\cdot}5} = 748$

$V_{overstrength} = \frac{1{\cdot}4 \times 4946}{10} = 692$ kN
$v_i' = 0{\cdot}44$

min $A_{vj} = \frac{{\cdot}44 \cdot 1000 \cdot 1000}{275} = 1600$ mm²/m

min $A_{te} = 200$ mm² at, 5 legs at 150 crs
= 6700 mm²/m.





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Wall 9
p. 549

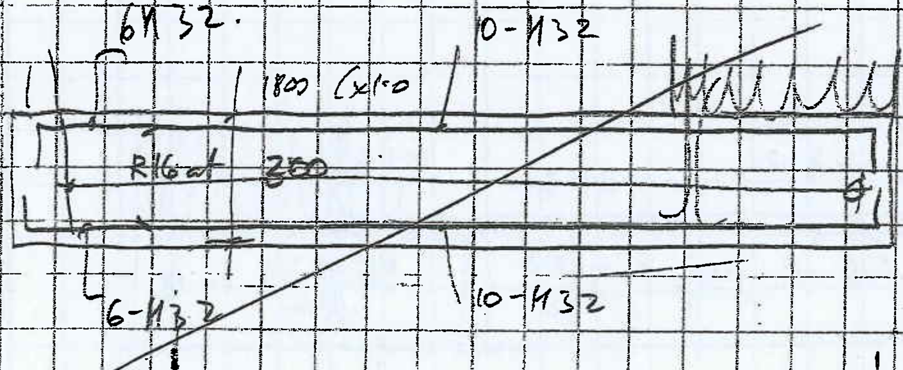
$M_{calc} = 4052^{kNm}$

$M_o = 5902^{kNm}$

design for $M = 4052^{kNm}$

1000×1800 ; $8H32 \Rightarrow M_u = 4010$ ·OK.

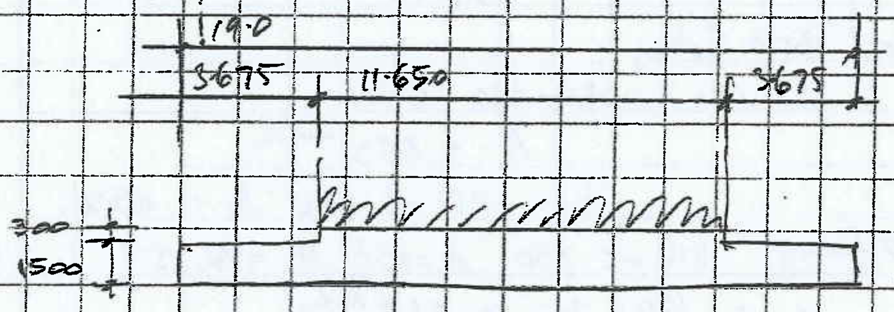
ie same reinf as wall 7



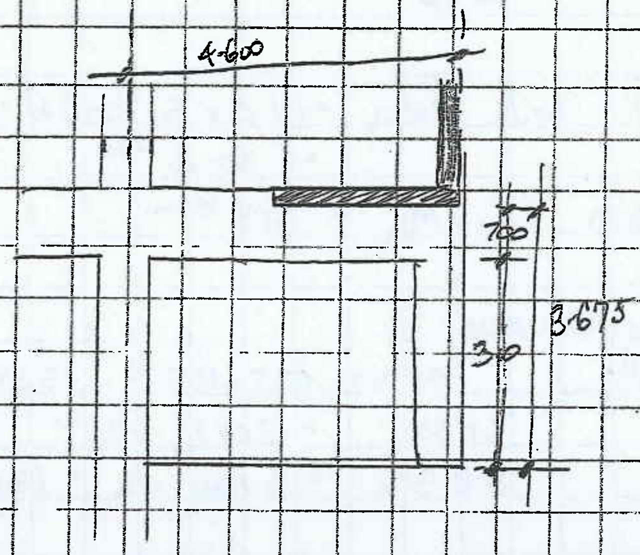
1·0 × 1800

Footing to Wall 4

footing extension from p. F17



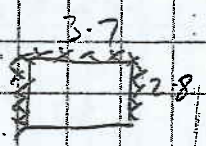
ELEVn.



max. press. under slab footing from p. F11 = 116 kPa $\bar{q}_n$

design for 40% over say 160 kPa to provide F.O. safety against "stamping" action.

for slab fixed on 3 sides;



$$a_r = \frac{a}{\sqrt{2}} = \frac{2.8}{\sqrt{2}} = 1.98$$

$$b_r = \frac{2 \times 3.7}{2\sqrt{2}} = 2.61$$

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$$m = \frac{q a_r b_r}{2\left(2 + \frac{b_r}{a_r} + \frac{4a_r}{b_r}\right)} = \frac{160 \times 1.98 \times 2.61}{2\left(2 + \frac{2.61}{1.98} + 4.\frac{1.98}{2.61}\right)} \text{ kNm/m}$$

$= 65.1$ kNm/m

For 500 footing

$.9 \times A_s \times 380 \times 420 = 65.1$

$A_s = 453$ mm²/m

For H12 at 250 $A_s = 452$. — 350 footn

for 350 footing H16 at 250, $A_s = 804$ $M_u = 66.5$ — H16-250 bot.

Beam : $w_u = 160 \times 2.3 = 368$ kN/m

$CLM = M_u = \frac{368 \times 3.0^2}{2} = 1656$ kNm/m.

pos. BM = 1656

uplift wt of soil + footing $= (1.4 \times .5 \times 23.5) + (1.4 \times .5 \times 20)$

$= 30.4$ kPa

neg BM $\therefore M_u = 315$ kNm.

for 475 wide x 1000 deep :

$A_{s\,min} = .0037 \times 475 \times 875 = 1538$

3H32 $= 2412$, $M_u = 721$

8H32 $= 6440$ $M_u = 1844$

> 1656.

Shear:

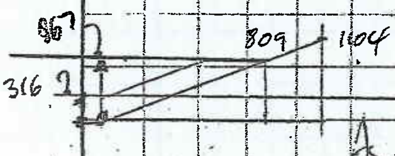
$V_u = 368 \times (3 - 0.8) = 809$ kN

$v_i = \frac{809{,}000}{.85 \times 475 \times 881} = 2.27$

$v_c = \left(.07 + 10 \times \frac{6440}{475 \times 881}\right)\sqrt{20} = 1.~~00~~\ 0.89$ — $2\sqrt{f'c}$

$v_s = 2.27 - 0.89 = 1.38$

$A_{v\,25} = \frac{1.38 \times 475 \times 1000}{275} = 2384 \div 2 = 1192$



D16 at 150 = 2680

$v_s = 1.55$ $V_s = 551$

$A_{s\,min} = .35 \times - = 604 \div 2 = 302 = $ D12 at 400.

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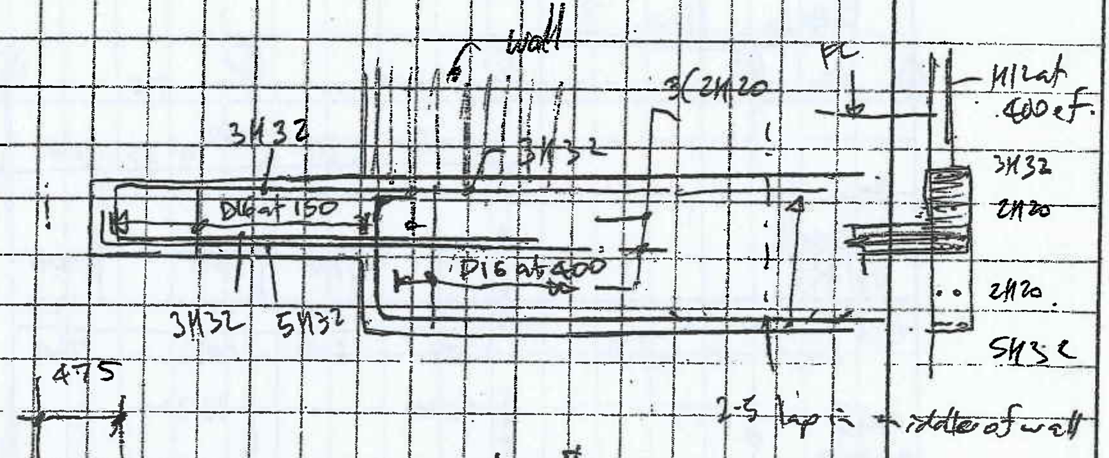
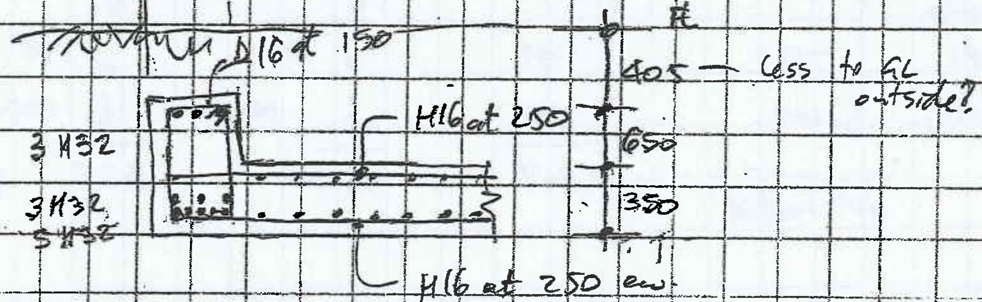




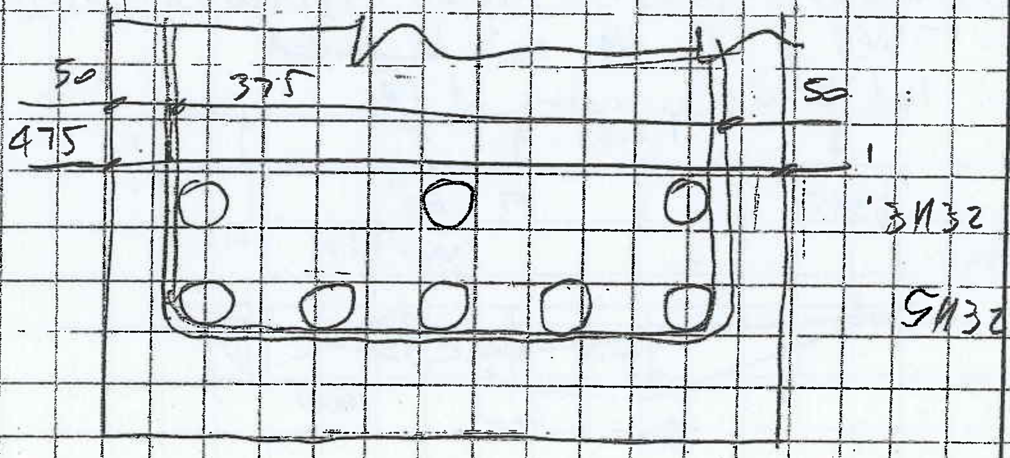
.0018 × 475 × 1000 = 855 ∴ 2H20 ef at 800 crs = 785.

Check total OTM capacity of beams

3, design strength = 6967 Overstrength = 9754
5, = 12973 = 18162
7, 4010 5614
9, 4010 5614

27,960 kNm × 1.4 = 39,144 kNm

cf. OTM on p. F2 = 26,935 kNm - OK.



Beam line 3

A 4.5 B 7.0 C 7.0 D 7.0 E 4.5 F

4.05 | 2.60 | 350 | 4.35 | 2.65

Beam Overstrength Uplifts.
2421 748 748 1300
140 140 140 140

less 1/2 beam wt max. nett uplift
2281 608 608 1160 Σ = 4657

DGD available
1177 1370 1370 1177

pads 348 348 348 348

foots. 100 100 100 100

not needed
1818 1818 1625 Σ = 5261

footing wt. 4.0×4.0×0.5, with 0.5 soil above = 348

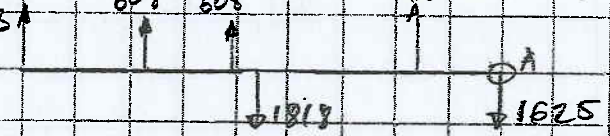
found beam 1.0×0.6 × 23.5 = 14.1 kN/m × 7 m = 100 kN

cross beam 1.0×1.7×23.5 = 40 kN/m × 7/2 = 140 k

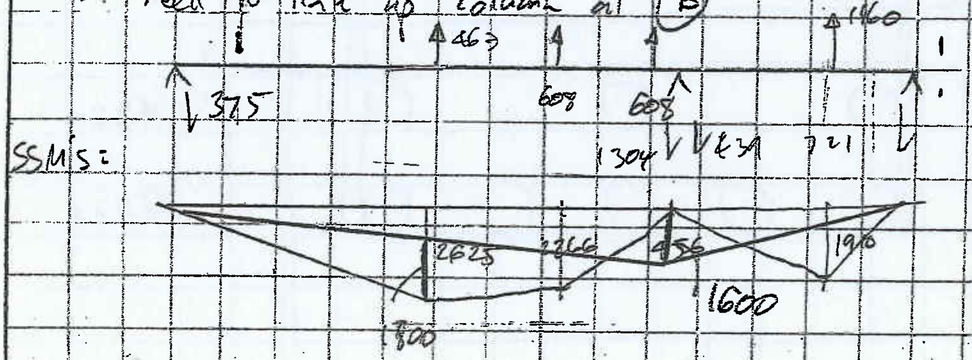
ie 4657 < 5261 no nett uplift.

hence beam loads: 2281 −1818 = 463

463 608 608 1160
1818 1625



M(A)
= (1160 × 2.65) + (608 × 7.35) + (608 × 9.95) + (463 × 14.0) − (1818 × 7)
= 7348 k nett BM can't be ~~resisted~~
∴ need to link up column at B



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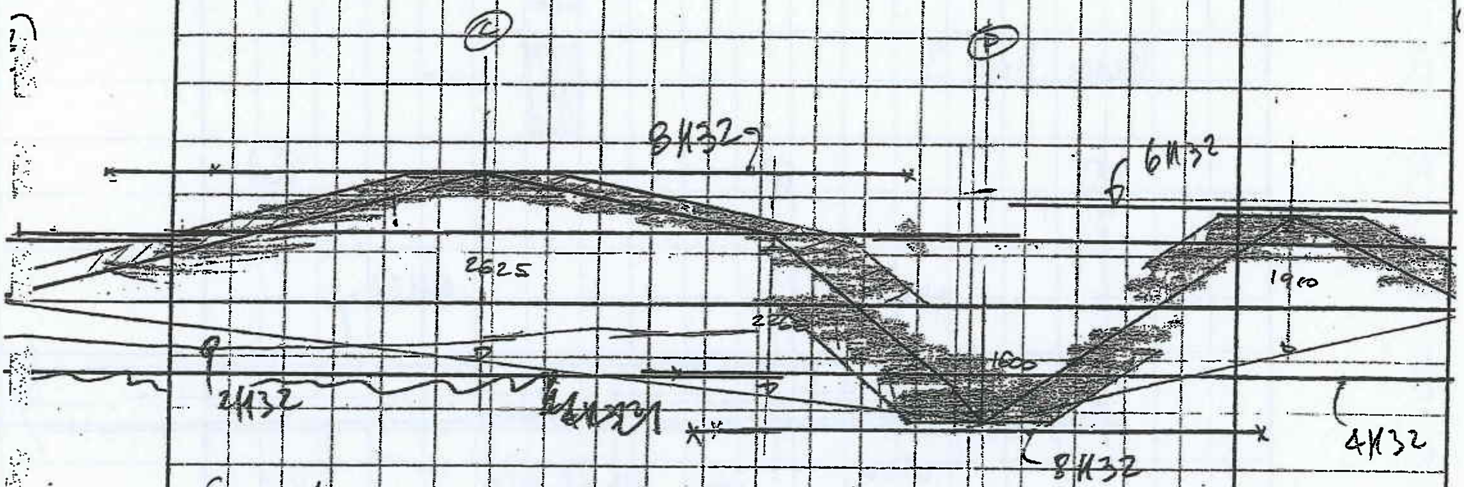
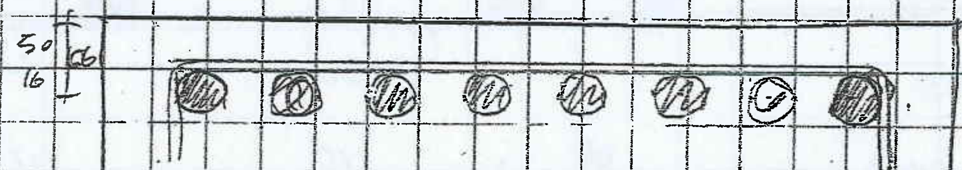


for b = 600, 1000 o/a, d = 920

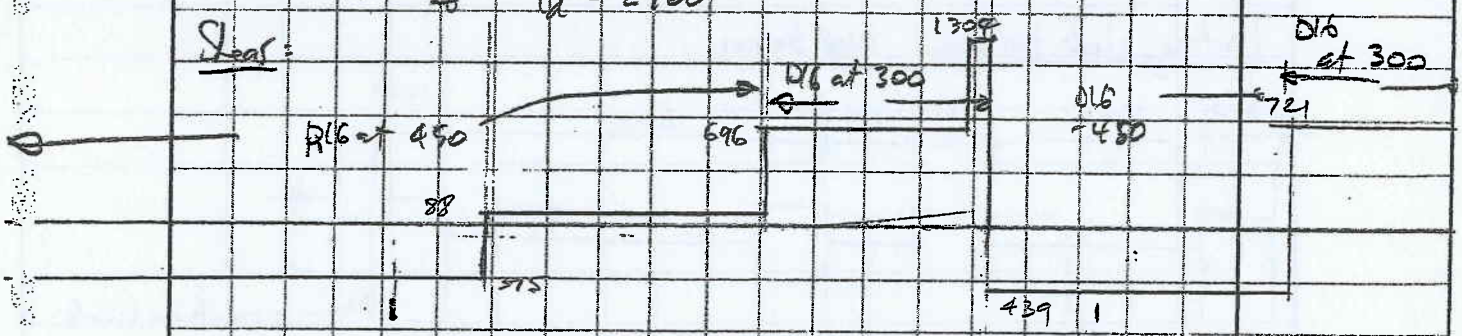
bars	A_s	b	a	d'	d	M'	M
8H32	6434	600	240	909	934	1736	1791
6H32	4826	600	180	909	934	1335	1376
4H32	3217	600	120	909	934	934	961



H32 l_d : for b = 70 f'_c = 20, f_y = 380

∴ l_d = 2700

Shear:



max shear = 721 kN. $v_i = \frac{721,000}{1.0 \times 600 \times 920} = 1.30 \,N/mm^2$

6H32 ρ = .0087 $v_c = (.07 + .0087)\sqrt{20} = 0.70$ 386 kN

∴ v_s = 1.3 − 0.7 = 0.60 339 kN

$A_v = \frac{0.60 \times 600 \times 1000}{275} = 1309 \, mm^2/m$ ∴ R16 at 300

= 1340.

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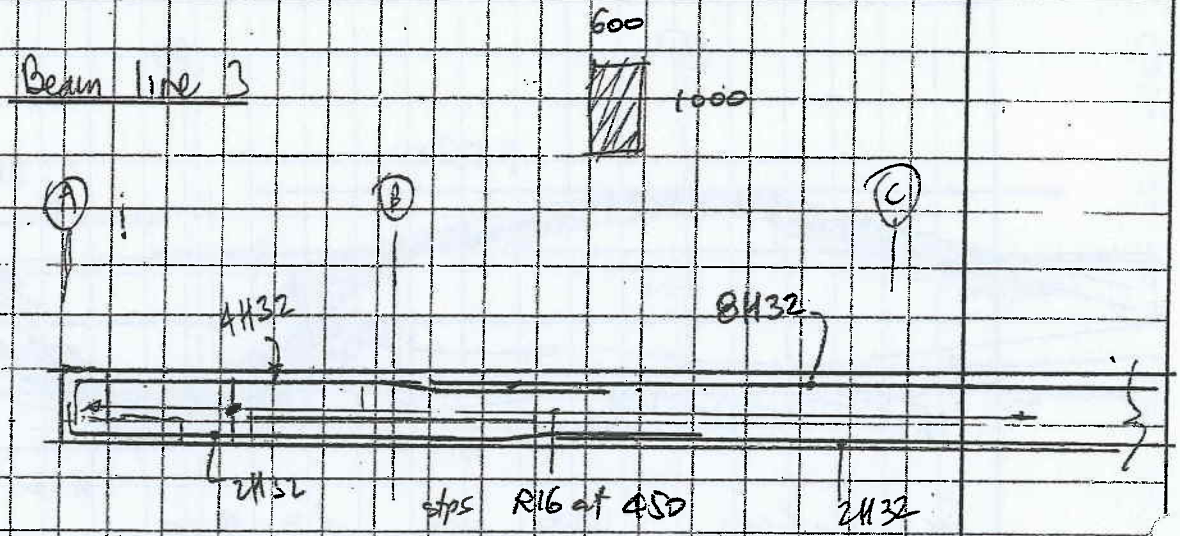
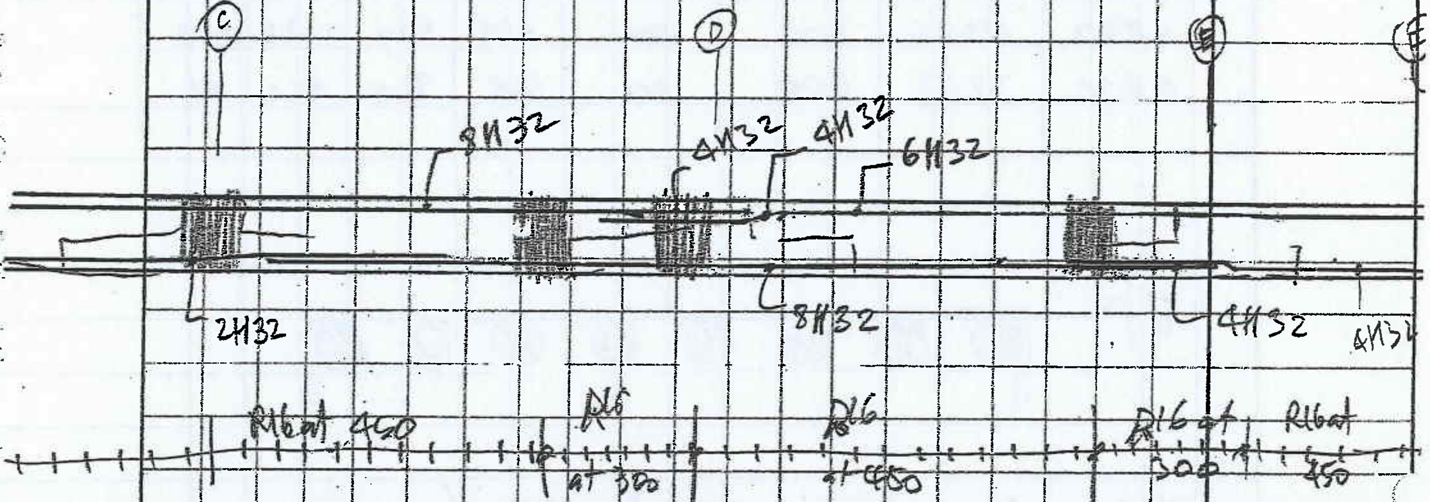
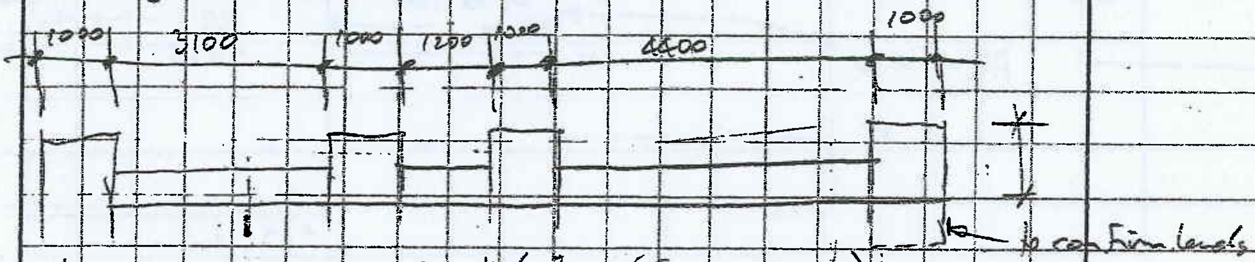
Beam line 3

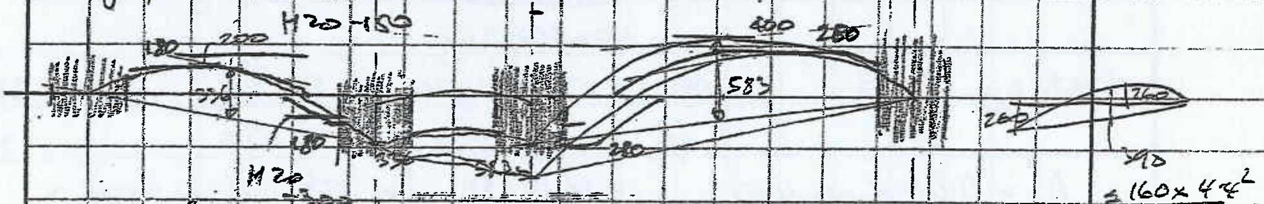




Footing slab between wall beams



design pressure σ_u = 160 kN/m² (from p. F37)



$\frac{160 \times 4.1^2}{8} = 336$

$\times \frac{2.2^2}{8} = 97$

$\frac{160 \times 5.4^2}{8} = 583$

$= \frac{160 \times 4.4^2}{8} = 387$

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for 500 deep footing

steel	A_s	a	d	jd	Mu	p
H12-250	452	10	418	415	64	·0011
H16-150	1340	30	417	402	184	·0032
H20-150	2090	47	415	391	279	·0050
H20-450	698				93	
- 300	1045				140	

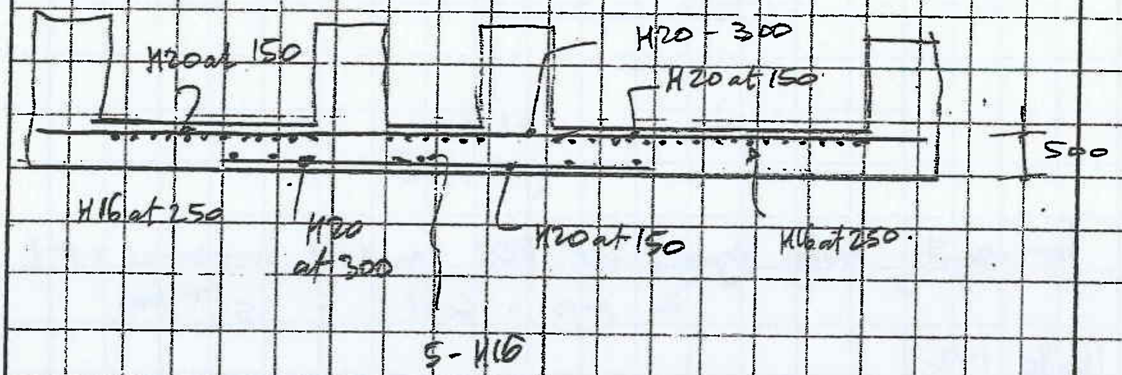
check shear : max shear = 160 kN/m^2 × (2·2 − 0·41)^m
= 286 kN/m

$v_i = \dfrac{286,000^N}{\cdot 85 \times 1000 \times 415} = 0\cdot 81\ N/mm^2$

max $v_c = \cdot 23\sqrt{20} = 1\cdot 47\ N/mm^2$

or $v_c = (0\cdot 07 + 10 \times \cdot 0032)\sqrt{20}$
$\nless 0\cdot 17\sqrt{f_c'} = \cdot 17\sqrt{20} = 0\cdot 76\ N/mm^2$

for 25 MPa concrete $\cdot 17\sqrt{f_c'} = 0\cdot 85\ N/mm^2$



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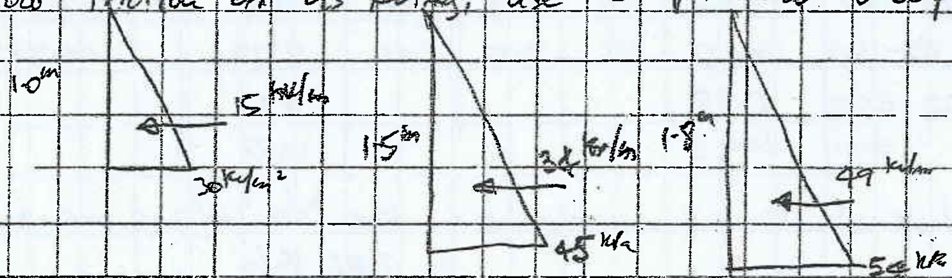
Lateral Seismic Loads on Foundations

$\phi = 30°$, $\omega = 20\ kN/m^2$

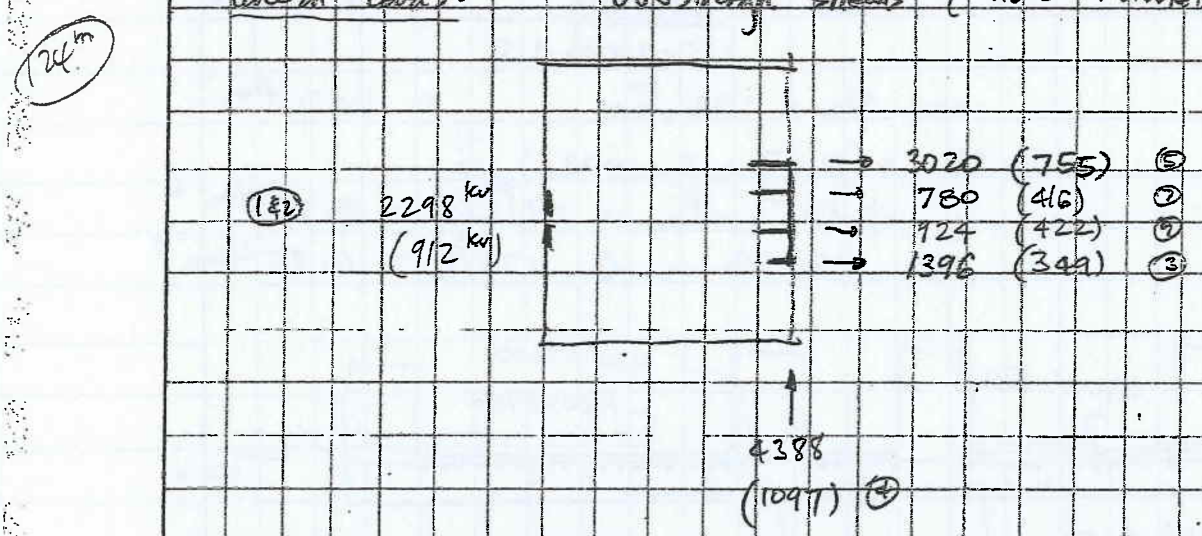
$\therefore K_p$ per 1^m depth $= 1.0 \times 20\ kN/m^2 \tan^2(45 + 30/2)$

$= 60\ kN/m^2$ with Fos $= 2$, $= 30\ kPa$.

allow friction on u/s footing, use $s = p \tan 20° = 0.36p$.



lateral Loads: Overstrength shears (Vcode in brackets)



for capacity design figures, use FOS on soil pressures = 1.1

ie press = 60/1.1 = 55 kPa/m

Walls 1&2

axial load = footing + 0.9D from building

$= (70.5\ kN/m \times 20^m) + 1185 + 1344 + 1344 + 866$

$= 6149\ kN$

friction on u/s $= 6149 \times 0.36$

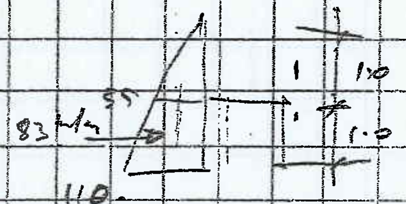
$= 2214\ kN$

press on end $= 83\ kN/m \times 3^m$

$= 249\ kN$

Total $= 2214 + 249 = 2463$

> 2298 OK.



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wall 3

footing wt = 42.3 x 11.5 = 486 kN.
0.9D wall = 496 kN
Peq = 4161 kN
5143

max overstrength shear is when Peq is compressive
ie soil shear force = 0.36 x 5143
= 1851 kN > 1396 OK

wall 4

footing wt = 19 x 20 = 380
0.9D wall = 1890 kN
Peq = 25 kN
2295 kN

friction on u/s footing = 2295 x .36 = 862 kN

lateral load on footings per m = 49 kN/m x 2.0 / 1.1 = 89 kN/m.

tot. length = 4 x 12 = 48 m x 89 k/m = 4272 kN.
+ 862
Total 51 44
> 4388 OK

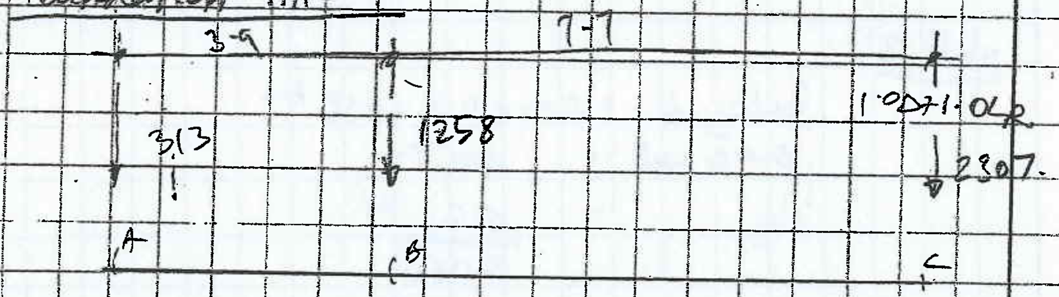
wall 5

footing wt = 486
0.9D wall = 1866
U overstrength critical Peq = 5794
8146
x 0.36 = 2932
passive press on wall = 49 kN/m x 4 m = 196
3028 > 3020 OK.

walls 7 & 9

footing wt = 486
0.9D = 1708
Peq c= 0
2194 x .36 = 790
passive press = 49 x 3 = 147
937 > 924 OK.

Foundation line 4



$w = \frac{1258^{kN}}{3\cdot9 + 7\cdot7} \times 2 = 217\ kN/m \rightarrow \frac{313 \times 2}{3\cdot9} = 160\ kN/m$

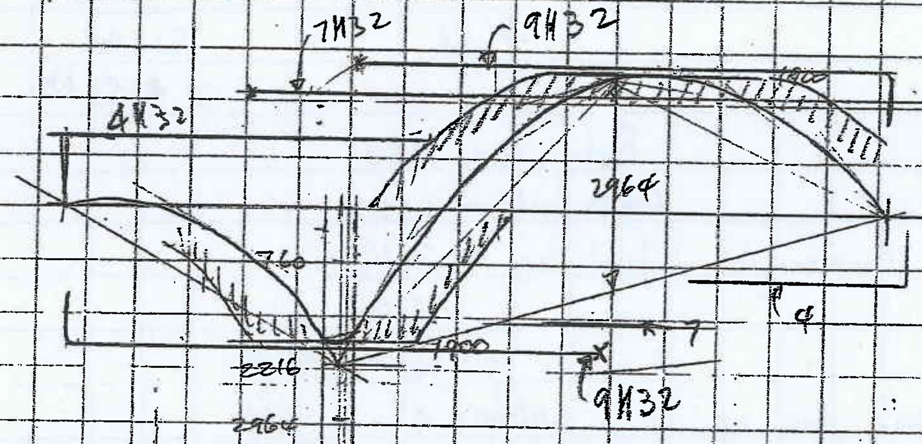
for 2·5 wide footing, $\sigma = 95\ kN/m^2$, $w = 2\cdot5 \times 95 = 238\ kN/m$

design for $\sigma_u = 1\cdot7 \times 95 = 160\ kN/m^2$
$w = 160 \times 2\cdot5 = 400\ kN/m$

for 3·9 spans, $SSM = \frac{400 \times 3\cdot9^2}{8} = 760\ kNm$

7·7 span $= \frac{400 \times 7\cdot7^2}{8} = 2964\ kNm.$

$\frac{D_{BA}}{D_{BC}} = \frac{S_{BA}}{S_{BC}} = \frac{7\cdot7}{3\cdot9} = \frac{0\cdot66}{0\cdot34}$



Steel	A_s	a	d	jd	M_u
4H32	3217	72	877	841	925
8 H32	6434	144	877	805	1771
12 H32	9651	216	877	769	2538
10 H32					2154
9 H32					1990

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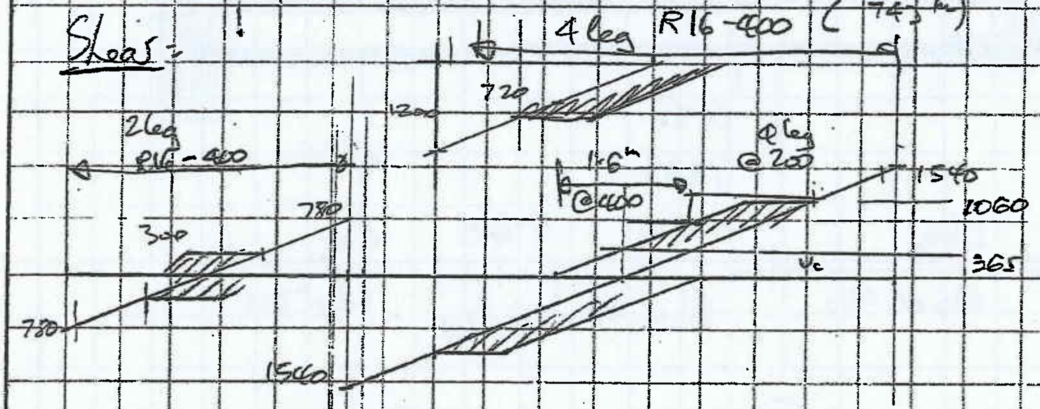
for 6.0m span

ssm = $\frac{400 \times 6.0^2}{8}$ = 1800 kNm.

use 8H32 T.

Shear:



v_i = $\frac{1060,000}{0.85 \times 600 \times 805}$ = 2.58 N/mm²

use 600 wide beam

v_b = .07 + 10 x .015) $\sqrt{20}$ = 0.89

v_s 1.69

A_v reqd = $\frac{1.69 \times 600 \times 1000}{1 \times 275}$ = 3687 mm²/m = $\boxed{4}$ R16 at 110

~~for 1000 wide footing, v_i = 1.54~~

~~v_b = .07 + .09   .72~~

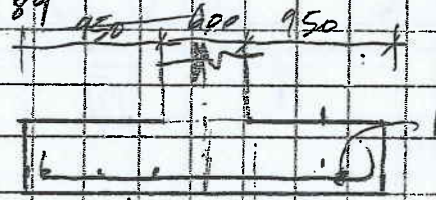
~~v_s .82~~

~~A_v = $\frac{82 \times 1000 \times 1000}{275}$ 2981 = $\boxed{4}$ R16 at 135~~

stirrups	A_v	v_s	V_u
4 leg R16-200	4020	1.84	755
4 leg R16-400	2010	0.92	378
2 leg R16-400	1005	0.46	189

V_c = 365

Footing outstand:



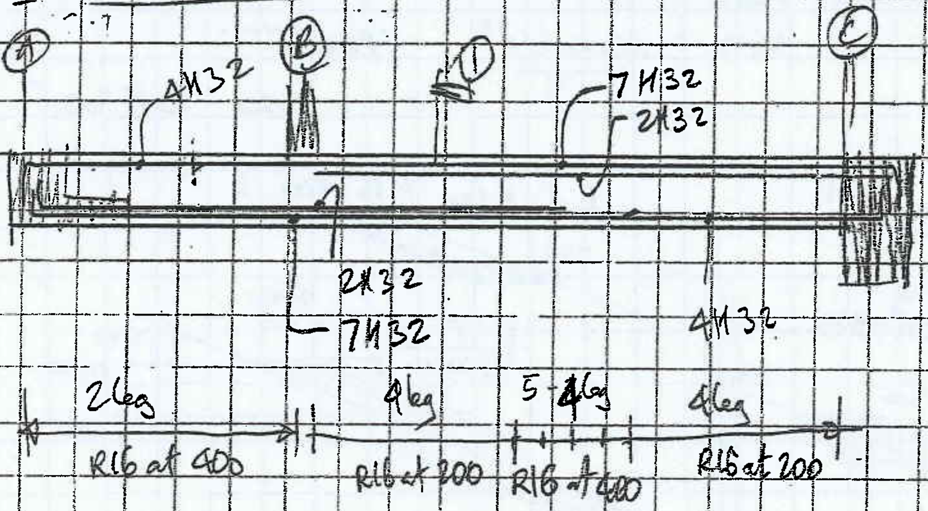
CLM = 160 x .95²/2 = 72.2 kNm/m

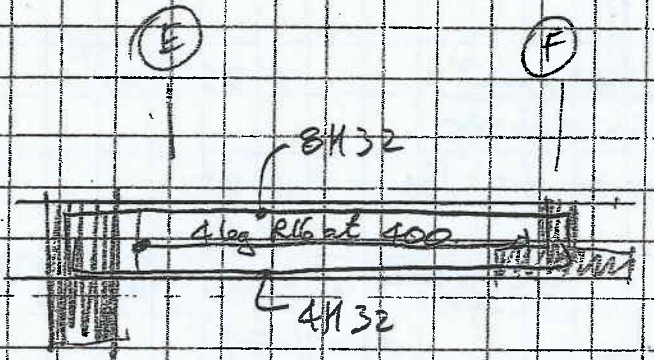
for 350 deep footing

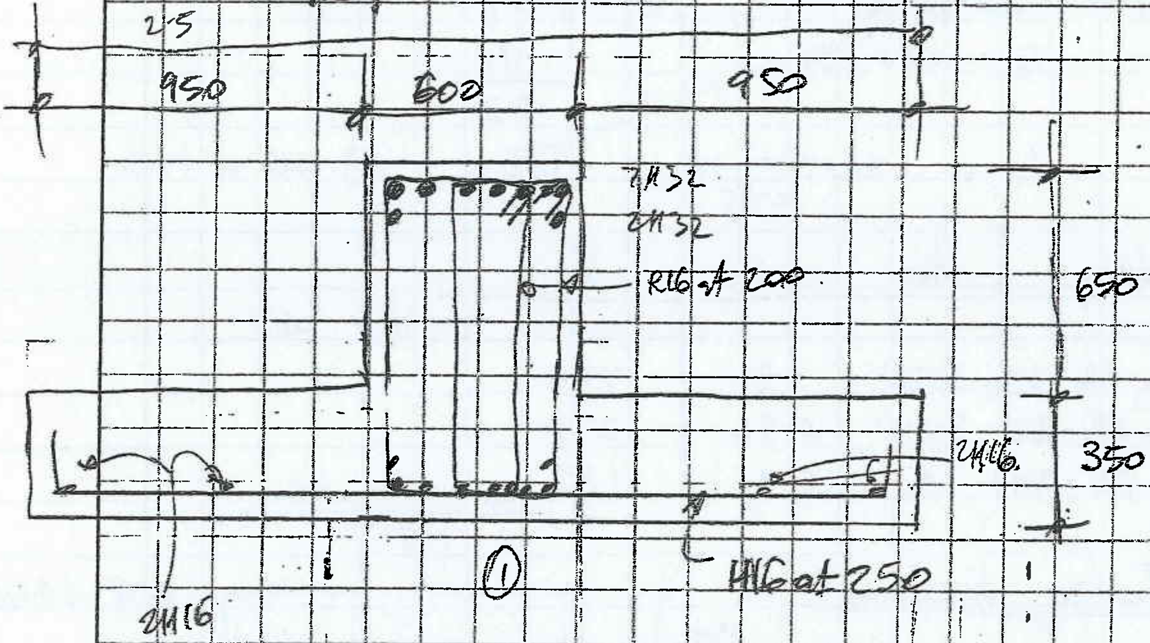
H16 at 250 A_s = 804, a = 18 d = 267 jd = 258 M_u = 71.0 kNm/m

OK.

line 4 beam 600x1000







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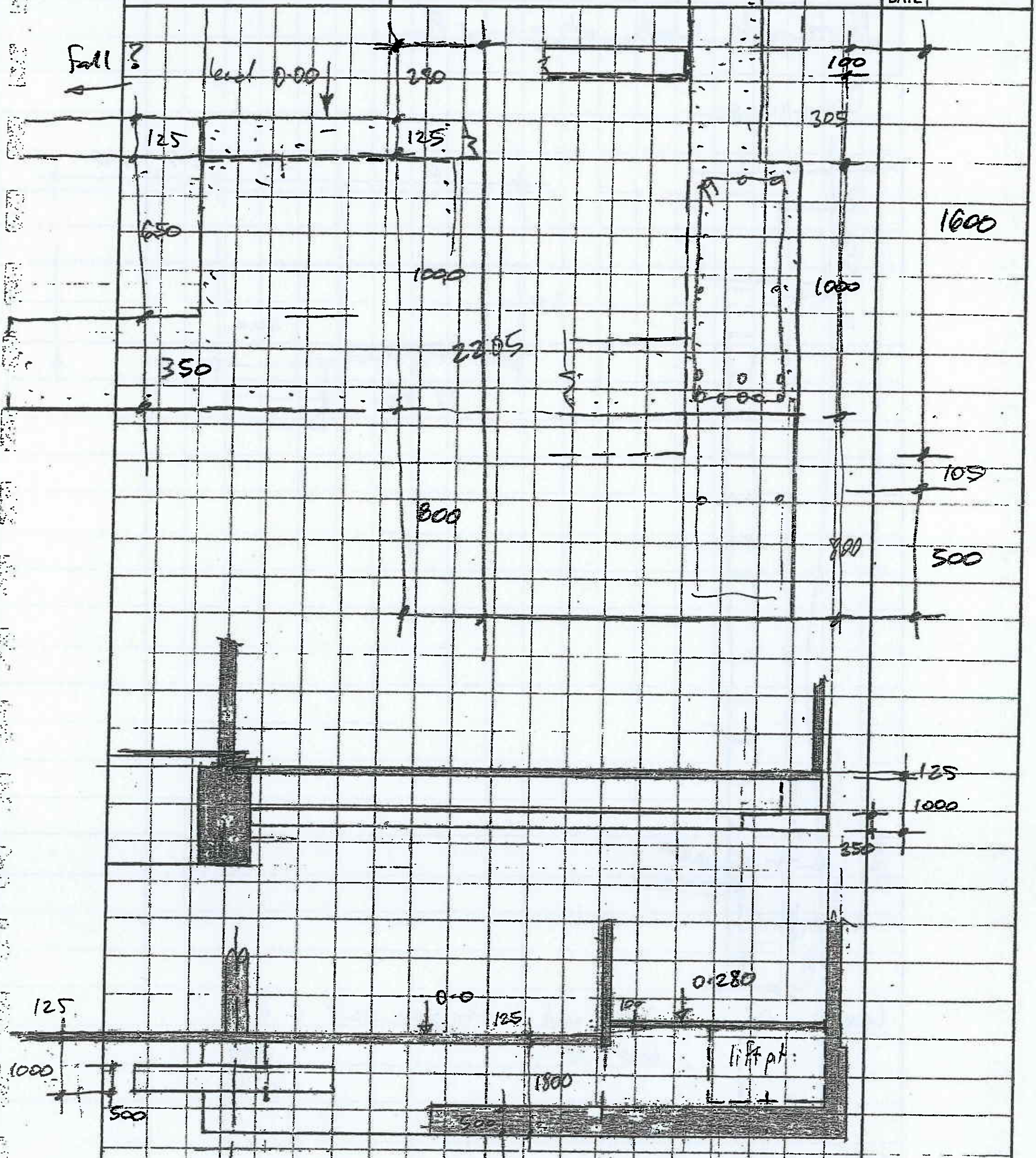

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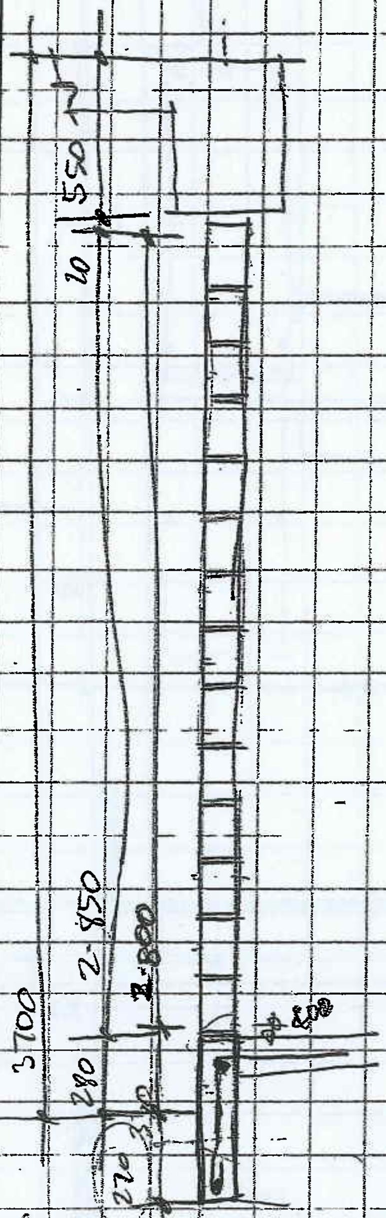
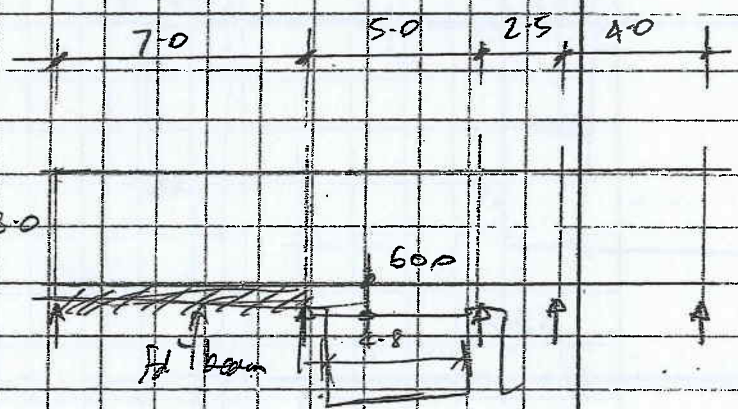
Blockwork walls at ground floor:

Dimensions:





Loads: DL = 3.0m wall = .14 x 3.0 x 23.5 = 9.9

600 Fdn = 2.0

11.9

$w_u = 1.4 \times 11.9 = 16.7$ kN/m

Fdn beam $M_u = \frac{16.7 \times 4.8^2}{12} = 32.0$

600 deep beam, $M_u/m = 229$ kNm/m

$d = 500$, $A_s/m = 2000$

$A_s = 201$ mm^2 ∴ use H16

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Graham

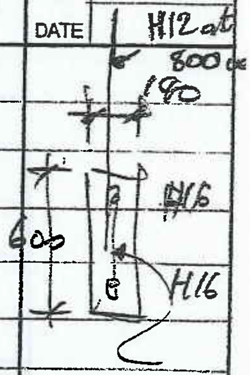
$V_u = 16.7 \times \left(\frac{4.8}{2} - .5\right) = 31.7^{kN}$

$v_u = \frac{31{,}700^N}{.85 \times 140 \times 500} = 0.53$

$v_b = .07 + .03\sqrt{20} = 0.45$

$A_{v\,min} = \frac{.35 \times 140 \times 1000}{275} = 178^{mm^2/m}$

for R10 at 250 $A_v = 312$ $A_v = .62$ OK



Face loads:

Wind: $w_u = 1.3 \times .414 = 0.54^{kPa}$

EQ $w_u = .2 \times \frac{5}{6} \times .14 \times 23.5 = 0.55^{kPa}$

$CLM = 0.55 \times \frac{3^2}{2} = 2.48^{kNm}$

D12 at 800 $A_s = 141^{mm^2/m}$

$M_u = \frac{.85 \times 141 \times 275 \times 67}{} = 2.20^{kNm/m}$

H12 at 800 $380 \times 60 = 2.73^{kNm}$

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Spandrel panels
100 thick

mi reinf (vert) = ·0018 × 100 × 1000 = 180 mm²/m

for 668 M, As = 186 mm²/m

check M_u cap = ·9 × 186 × 380 × 40
= 2·55 kNm.

max. BM = EQ. face load = ·5 × 1 × 2) ·5 = 1·2 kPa
CLM = $\frac{1·2 \times ·6^2}{2}$ = 0·22 kNm < 2·55 OK.

Use 668M = body & H10 at 400 in ends.

fixing: horiz load = 2·0 g
= 4·4 tonnes × 2
= 8·8 t
= 88 kN

load each fixing = $\frac{88}{4}$ = 22 kN × 0·8 = 17·6 kN.

shear capacity M20 bolt = 19·6 kN × 1·33
= 26·1 > 17·6 OK.

capacity TCM 20 insert
by test, ult load in 4000 psi conc
≥ 17000 lb
= 53 kN

∴ FOS = $\frac{53}{17·6}$ = 3·0 OK.