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STRUCTURAL CALCULATIONS

Job No:	17799		
Date:	12/05/2016	Designer:	TH
Revision A:	09/06/2016	Designer:	TH
Revision B:	25/07/2016	Designer:	TH
Revision C:	30/03/2017	Designer:	TH
Revision D:	07/04/2017	Designer:	TH
Revision E:	27/09/2017	Designer:	TH
Revision F:	09/04/2018	Designer:	TH
Client:	Bert Farina Constructions		
Site Address:	No. 147 Marion Road, RICHMOND		

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SA1

Our Ref: 17799

7th June 2016

Bert Farina Contructions
1185 South Road
ST MARYS SA 5042

Attn: Anthony Farina

RE: STRUCTURAL ADEQUACY REPORT FOR UPPER STOREY ADDITION
AT: No. 147 MARION ROAD, RICHMOND
FOR: FARINA INVESTMENT TRUST

Dear Anthony,

As requested a Structural Engineer from this office inspected the above property on Wednesday 13th August 2014. The purpose of the inspection was to assess the suitability of the existing dwelling to support a light weight upper storey addition.

The existing dwelling consists of the following construction:-

- Single storey
- Non-articulated solid brick external walls
- Brick on flat load bearing internal walls
- Conventionally framed timber roof system
- Sheet roofing
- Strip footing construction (timber floors)

It is proposed to construct a lightweight upper storey addition consisting of :-

- timber framed floors
- timber framed walls
- trussed framed roof
- sheet roof

No significant cracking or signs of movement were noted during the inspection. As a result it is concluded that the existing footing structure is performing adequately and is suitable to support the additional load of the light weight addition described above.

All point loads must be distributed down to existing footings either through the existing masonry walls via stub columns or new full height columns down to footing level (as shown on attached structural drawings).

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The increased wind load from the upper storey addition (increase in overall area of elevation) will be resisted by a combination of the existing brick walls to the lower level, new precast panel walls over new footings and a new steel sway frame in the western elevation.

The owner is to be made aware that due to the additional weight of the proposed structure and redistribution of the loads some minor cracking and movement in the dwelling is to be expected. It is the recommendation of this office that any painting and redecorating of the lower level be delayed as long as possible after the construction of the addition.

We trust that this report is sufficient for your present requirements. If you have any further queries please do not hesitate to contact this office.

Yours sincerely,

David Angeloni, BE (Civil) Hons, MIE Aust
RCI Consulting Engineers

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Wind Site Assessment

Region A1 / Building Importance Level II

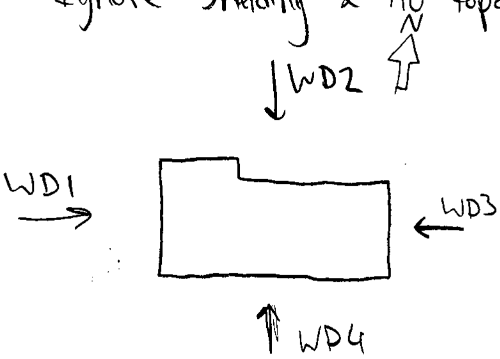
↳ 500 year reccurrance interval for ultimate design and 25 year for serviceability

$V_{500} = 45$ m/s

$V_{25} = 37$ m/s

Building Height 8.5m / Terrain Cat 3 all directions $M_{z,cat} = 0.83$

Ignore shielding & no topographic effects $M_s = M_t = 1.0$



WD1 → $M_d = 1.0$ (West)

$q_z = 0.6 \frac{(0.83 \times 45)^2}{1000} = \underline{0.837 \text{ kPa}}$

WD2 → $M_d = 0.95$ (North-West)

$q_z = 0.6 \frac{(0.83 \times 0.95 \times 45)^2}{1000} = \underline{0.755 \text{ kPa}}$

WD3 → $M_d = 0.8$ (east)

$q_z = 0.6 \frac{(0.83 \times 0.8 \times 45)^2}{1000} = \underline{0.536 \text{ kPa}}$

WD4 → $M_d = 0.85$ (South)

$q_z = 0.6 \frac{(0.83 \times 0.85 \times 45)^2}{1000} = 0.605 \text{ kPa}$

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Rev A: 6/4/17

Mezzanine Floor Framing

Apartments 1 & 2 are mirror image → same framing

MFB1 Span 5.4 m *** Re-Design MFB1 → Refer S86 ***

FLW = 1.4 m plus 1.0 kN/m to allow for balustrade above, bulkhead below and bearer self weight

$W_{DL} = (1.4 \times 0.5) + 1.0$
$= 1.7$ kN/m

$W^* = 5.19$ kN/m

$W_{LL} = (1.4 \times 1.5)$
$= 2.1$ kN/m

$M^* = 18.9$ kNm

Select 400×63 LVL 13 → $I_x = 336 \times 10^6\ mm^4$ $E = 13200$ MPa

$\Delta_{DL} = 2\ \frac{5 \times 1.7 \times 5400^4}{384 \times 13200 \times 336 \times 10^6}$

$= 8.49$ mm < 12 mm

$\Delta_{LL} = \frac{5 \times 2.1 \times 5400^4}{384 \times 13200 \times 336 \times 10^6}$

$= 5.24$ mm < 9 mm

→ Acceptable

Bearer restrained @ 450 crs by floor trusses
↳ by inspection/experience 400×63 LVL13 ok for Strength!!

* Support each end on 3/90×35 MGP10 studs (nail lam) *

MFB2 Span = 5450 mm

Supporting LB external upper wall (2.6m high → take 0.7 kPa)
(conservative as large opening/windows over)

RLW = 6.0m
FLW = 1.5m
Balcony FLW = 1.2m

$W_{DL} = (2.6 \times 0.7) + (6 \times 0.4) + (2.7 \times 0.5) + 0.4$
$= 5.97$ kN/m

$W^* = 16.4$ kN/m

$W_{LL} = (6 \times 0.25) + (1.5 \times 1.5) + (1.2 \times 2)$
$= 6.15$ kN/m

$M^* = 60.9$ kNm

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Limit Δ_{LL} to max 9mm

$$I_x \geq \frac{5 \times 6.15 \times 5450^4}{384 \times 200 \times 10^3 \times 9}$$

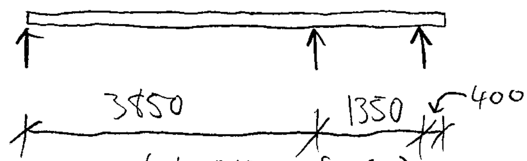
$$\geq 39.25 \times 10^6 \ mm^4$$

Adopt 310 UB 32 $\rightarrow$ $I_x = 63.2 \times 10^6 \ mm^4$

$\phi M_b = \phi M_s = 134$ kNm (FLR – Floor joists/trusses)

$\boxed{\text{MFB3}}$ critical case under load bearing external wall (Similar loading to MFB2)

continuous over 2 spans with small cantilever at eastern end



(only 0.15 kN/m for S.W)

$W_{DL} = 5.72$ kN/m
$W_{LL} = 6.15$ kN/m
$W^* = 16.4$ kN/m

ignore small overhang $\rightarrow$ $M^*_{-ve} = \frac{16.4 \times 3.85^3 + 16.4 \times 1.35^3}{8 \times 5.2}$

$= 23.5$ kNm

Select 300×63 LVL 13

$S_1 < 10$ by inspection
↳ restrained by joists

$$\phi M_b = \phi k_1 k_4 k_6 k_9 k_{12} f'_b Z$$

$k_1 = 0.8$
$k_4 = k_6 = k_9 = k_{12} = 1.0$
$f'_b = 41.9$ MPa

$$\phi M_b = 0.9 \times 0.8 \times 41.9 \times \frac{63 \times 300^2}{6} \times 10^{-6}$$

$= 28.5$ kNm $> M^* \Rightarrow$ ok

$$I_x = \frac{63 \times 300^3}{12} = 141.8 \times 10^6 \ mm^4$$

$$\Delta_{DL} = \left[2 \times \frac{5 \times 5.72 \times 3850}{384 \times 13200 \times 141.8 \times 10^6}\right] \times \frac{1}{2}$$

formula for single span
↳ reduce by 50% due to cont. span

$= 8.74$ mm $\approx$ span/441 $\Rightarrow$ Acceptable

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MFB4 Cantilevered steel bearer 3500mm backspan / 1850mm

UDL from balcony floor load & balustrade wall

$$W_{DL} = (1.1 \times 0.8) + (1.0 \times 0.8) + 0.3$$
$$= 1.98 \text{ kN/m}$$

$$W^* = 5.68 \text{ kN/m}$$

$$W_{LL} = (1.1 \times 2)$$
$$= 2.2 \text{ kN/m}$$

Point load @ free end from timber bearer MFB5 → $4.5\,m^2$ of external wall

$$P_{DL} = 4.5 \times 1.0$$
$$= 4.5 \text{ kN}$$

Select 230 PFC → $I_x = 26.4 \times 10^6 \text{ mm}^4$

Deflection under DL

$$\Delta_{DL} = \frac{1.98 \times 1850\left[(3 \times 1850^3) + (4 \times 1850^2 \times 3500) - (3500^3)\right]}{24 \times 2 \times 10^5 \times 26.4 \times 10^6} + \frac{4500 \times 1850^2 \times 5350}{3 \times 200 \times 10^3 \times 26.4 \times 10^6}$$

$$= 5.9\text{mm} \approx \frac{\text{cant}}{314} \Rightarrow \text{Acceptable!!}$$

$$\Delta_{LL} = \frac{2.2 \times 1850^3 \times 19550}{24 \times 2 \times 10^5 \times 26.4 \times 10^6}$$ (load over cantilever only)

$$= 2.1\text{ mm} \approx \frac{\text{cant}}{881} \Rightarrow \text{Acceptable}$$

Negative bending over support

$$M^*_{-ve} = \frac{5.68 \times 1.85^2}{2} + 1.2(4.5 \times 1.8)$$
$$= 19.7 \text{ kNm}$$

Steel PFC member restrained by floor framing

↳ $\phi M_b = 67.0$ kNm ($l_e = 1.0$m)

MFB5 Supporting wall only → Adopt 300x63 LVL 13

↳ ok by inspection!!

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Column C3

Supporting end of MFB2 steel bearer

$P_{DL} = 16.3$ kN $\quad N_c^* = 44.8$ kN
$P_{LL} = 16.8$ kN

Assume 125 eccentricity $\Rightarrow M^* = 0.125 \times 44.8$
≈ 5.6 kNm

conservative

Select 89x3.5 SHS $\rightarrow \phi N_c = 211$ kN $(l_e = 3.0\text{m})$

$\phi M_s = 11.5$ kNm

$\phi M_i = \phi M_o = 11.5\left(1 - \frac{44.8}{211}\right) = 9.06$ kNm $> M^*$

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First Floor Floor Framing - Apartments

$\boxed{\text{FB14 \& FB16}}$ Floor bearer supporting LBW above (Apartments 1 & 2)

Span = 5700mm

2.5m T/F wall above
Mez FLW = 2.9m (Timber truss, structaflor, ceiling)

$W_{DL} = (2.5 \times 0.5) + (2.9 \times 0.5) + 0.3$
$= 3.0$ kN/m

$W_{LL} = 2.9 \times 1.5$
$= 4.35$ kN/m

$W^* = 10.1$ kN/m

$M^* = 41.0$ kNm

Both FB14 & FB16 are supported each end by other steel bearers so adopt strict deflection limits to ensure global/overall deflection is not excessive.

Limit Δ_{DL} to 5.0mm Max!

$I_x \geq \frac{5 \times 3 \times 5700^4}{384 \times 200 \times 10^3 \times 5}$

$\geq 41.2 \times 10^6$ mm⁴

Adopt 310 UB 32

$I_x = 63.2 \times 10^6$ mm⁴
$\phi M_b = 43$ kNm ($l_e = 5.7$m)

FB19 Similar to bearers above but in apartment 3
↳ reduced span of only 5200mm

By inspection/comparison to above adopt 310 UB 32

FB18 Span = 5.2

Point load: 1.1m from eastern end 4m² of mez floor above

$P_{DL} = 4 \times 0.5 = 2.0$ kN

$P_{LL} = 4 \times 1.5 = 6.0$ kN

Also assume 600mm FLW (conservative)

$W_{DL} = 0.6$ kN/m
$W_{LL} = 0.9$ kN/m

Change to 200 UB 18 due to steel column over!

Adopt either 2/360x45 LVL13 or 400x63 LVL13
Depending on selected floor depth.

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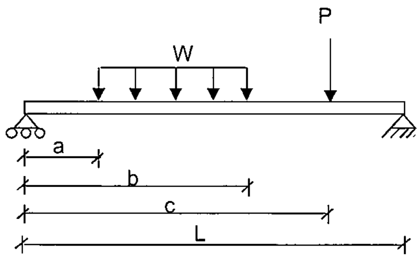
Floor Bearer FB18 Design

Length = 5.2 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	1.32E+007	kPa
Ix	336.00	X10^6 mm4
EI	4.44E+03	kN.m2

Dead Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	0.6	0	0	0	0
a_i	0	0	0	0	0
b_i	5.2	0	0	0	0

	1	2	3	4	5
P_i	2	0	0	0	0
c_i	1.1	0	0	0	0

Live Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	0.9	0	0	0	0
a_i	0	0	0	0	0
b_i	5.2	0	0	0	0

	1	2	3	4	5
P_i	6	0	0	0	0
c_i	1.1	0	0	0	0

M_{DL} = 3.3 kNm M_{LL} = 7.2 kNm

M* = 1.2 DL + 1.5 LL = 14.8 kNm

OR 1.35 DL = 4.4 kNm

$R1_{DL}$ = 3.1 kN $R2_{DL}$ = 2.0 kN

$R1_{LL}$ = 7.1 kN $R2_{LL}$ = 3.6 kN

MAX DL DEFLECTION = 2.08 mm

MAX LL DEFLECTION = 4.31 mm

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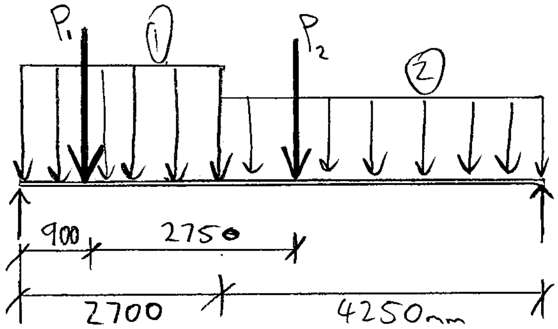
$\boxed{FB1}$ Span = 6950mm

Southern end of bearer will have precast panel over



UDL 1 FLW = 3.6m 6.5m high 150 thick panel wall

$W_{DL} = (3.6 \times 1.0) + (6.5 \times 0.15 \times 25) + 1.0$
$= 29.0$ kN/m

$W_{LL} = (3.6 \times 1.5)$
$= 5.4$ kN/m

UDL 2 FLW = 3.8m joists/trusses spanning between FB1 members will support 6.2m high party wall → half to each FB1 member

$W_{DL} = (3.8 \times 1.0) + (6.2 \times 0.4) + 1.0$
$= 7.28$ kN/m

$W_{LL} = 3.8 \times 1.5$
$= 5.7$ kN/m

Point Load P_1 From FB14/16

$P_{DL} = \frac{5.7}{2} \times 3 = 8.55$ kN

$P_{LL} = \frac{5.7}{2} \times 4.35 = 12.4$ kN

Point Load P_2 From concentrated load studs supporting MFB1 Above

$P_{DL} = \frac{5.4}{2} \times 1.7 = 4.59$ kN

$P_{LL} = \frac{5.4}{2} \times 2.1 = 5.67$ kN

Select 310 UC 118 & enter into beam design spreadsheet
↳ refer next page

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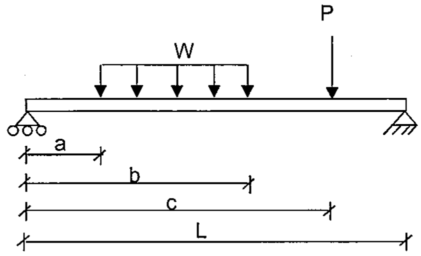
Floor Bearer FB1 Design

Length = 6.95 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	277.00	X10^6 mm4
EI	5.54E+04	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	29	7.28	0	0	0
a_i	0	2.7	0	0	0
b_i	2.7	6.95	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	8.55	4.59	0	0	0
c_i	0.9	3.65	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	5.4	5.7	0	0	0
a_i	0	2.7	0	0	0
b_i	2.7	6.95	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	12.4	5.67	0	0	0
c_i	0.9	3.65	0	0	0

M_{DL} = 101.1 kNm M_{LL} = 48.8 kNm

M* = 1.2 DL + 1.5 LL = 194.6 kNm

OR 1.35 DL = 136.5 kNm

$R1_{DL}$ = 82.2 kN $R2_{DL}$ = 40.2 kN

$R1_{LL}$ = 32.6 kN $R2_{LL}$ = 24.2 kN

MAX DL DEFLECTION = 8.87 mm

MAX LL DEFLECTION = 4.37 mm

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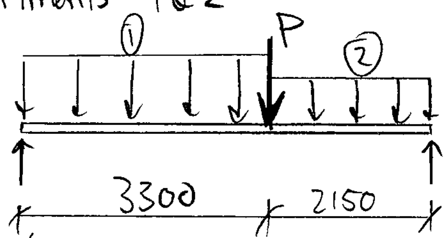
FB15 & FB17

Apartments 1 & 2

Span = 5.45 m



UDL 1

2.5m Non-LB wall over (T/F with Fyrchek & top hat + Fibre cement sheet → 0.7 kPa)
0.7m Balcony FLW (Tiled → 0.8 kPa)

$W_{DL} = (2.5 \times 0.7) + (0.7 \times 0.8) + 0.3$
$= 2.61$

$W_{LL} = 0.7 \times 2$
$= 1.4$ kN/m

UDL 2

$W_{DL} = (2.5 \times 0.4 + 0.3$
$= 1.3$ kN/m

2/400×45
3/360×45
200 PFC

Point Load from TB1

1.8m² external wall → 0.7 kPa
1.0m² floor

$P_{DL} = (1.8 \times 0.7) + (1 \times 0.5)$
$= 1.76$ kN

$P_{LL} = 1 \times 1.5$
$= 1.5$ kN

FB2

Bearer below party wall b/n apartments 2 & 3

Span = 3550 mm

FLW = 4.6m
6.5m double stud party wall
Point load 1300 from Southern end (FB17) & Point load 2300mm from Southern end (Column C3/Studs over)

$W_{DL} = (4.6 \times 1.0) + (6.5 \times 1.0) + 0.3$
$= 11.4$ kN/m

$W_{LL} = (4.6 \times 1.5)$
$= 6.9$ kN/m

$P_{DL} = 7.3$ kN
$P_{LL} = 3.8$ kN

$P_{DL} = 27.8$ kN
$P_{LL} = 28.6$ kN

Enter loads into beam design spreadsheet
↳ Select 310 UB 32

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Sheet : S11

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Floor Bearer FB15/FB17 Design

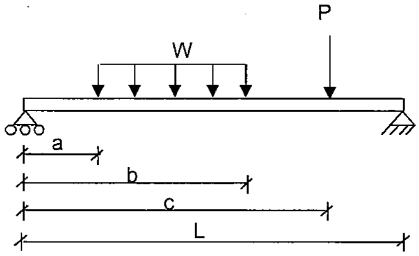
Length = 5.45 m

Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35



Sevriceability Limit State

DL	1
LL	0.7

E	1.32E+007	kPa
Ix	524.90	X10^6 mm4
EI	6.93E+03	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	2.61	1.3	0	0	0
a_i	0	3.3	0	0	0
b_i	3.3	5.45	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.76	0	0	0	0
c_i	3.3	0	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	1.4	0	0	0	0
a_i	0	0	0	0	0
b_i	3.3	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.5	0	0	0	0
c_i	3.3	0	0	0	0

M_{DL} = 10.1 kNm M_{LL} = 5.2 kNm

M* = 1.2 DL + 1.5 LL = 19.9 kNm

OR 1.35 DL = 13.6 kNm

$R1_{DL}$ = 7.3 kN $R2_{DL}$ = 5.9 kN

$R1_{LL}$ = 3.8 kN $R2_{LL}$ = 2.3 kN

MAX DL DEFLECTION = 4.40 mm

MAX LL DEFLECTION = 2.22 mm

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Floor Bearer FB15/FB17 Design

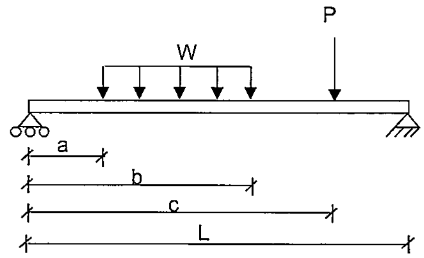
Length = 5.45 m

Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35



Sevriceability Limit State

DL	1
LL	0.7

E	1.32E+007	kPa
Ix	480.00	X10^6 mm4
EI	6.34E+03	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	2.61	1.3	0	0	0
a_i	0	3.3	0	0	0
b_i	3.3	5.45	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.76	0	0	0	0
c_i	3.3	0	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	1.4	0	0	0	0
a_i	0	0	0	0	0
b_i	3.3	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.5	0	0	0	0
c_i	3.3	0	0	0	0

M_{DL} = 10.1 kNm M_{LL} = 5.2 kNm

M* = 1.2 DL + 1.5 LL = 19.9 kNm

OR 1.35 DL = 13.6 kNm

$R1_{DL}$ = 7.3 kN $R2_{DL}$ = 5.9 kN

$R1_{LL}$ = 3.8 kN $R2_{LL}$ = 2.3 kN

MAX DL DEFLECTION = 4.81 mm

MAX LL DEFLECTION = 2.43 mm

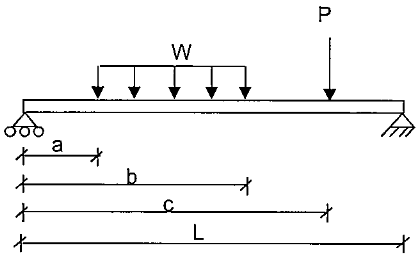
RCI CONSULTING ENGINEERS

Sheet : S13

Job No : **17799**

Floor Bearer FB15/FB17 Design

Length = 5.45 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	19.10	X10^6 mm4
EI	3.82E+03	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	2.61	1.3	0	0	0
a_i	0	3.3	0	0	0
b_i	3.3	5.45	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.76	0	0	0	0
c_i	3.3	0	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	1.4	0	0	0	0
a_i	0	0	0	0	0
b_i	3.3	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.5	0	0	0	0
c_i	3.3	0	0	0	0

M_{DL} = 10.1 kNm M_{LL} = 5.2 kNm

M* = 1.2 DL + 1.5 LL = 19.9 kNm

OR 1.35 DL = 13.6 kNm

$R1_{DL}$ = 7.3 kN $R2_{DL}$ = 5.9 kN

$R1_{LL}$ = 3.8 kN $R2_{LL}$ = 2.3 kN

MAX DL DEFLECTION = 7.97 mm

MAX LL DEFLECTION = 4.04 mm

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Sheet : S/14

Job No : **17799**

Floor Bearer FB2 Design

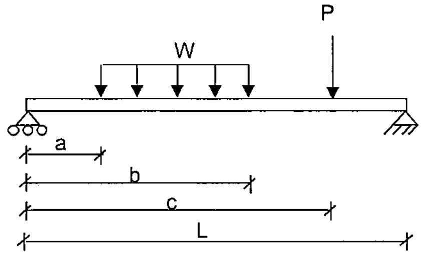
Length = 3.55 m

Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35



Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	63.20	X10^6 mm4
EI	1.26E+04	kN.m2

Dead Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	11.4	0	0	0	0
a_i	0	0	0	0	0
b_i	3.55	0	0	0	0

	1	2	3	4	5
P_i	7.3	27.8	0	0	0
c_i	1.3	2.3	0	0	0

Live Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	6.9	0	0	0	0
a_i	0	0	0	0	0
b_i	3.55	0	0	0	0

	1	2	3	4	5
P_i	3.8	28.6	0	0	0
c_i	1.3	2.3	0	0	0

M_{DL} = 42.2 kNm M_{LL} = 34.5 kNm

M^* = 1.2 DL + 1.5 LL = 102.4 kNm

OR 1.35 DL = 56.9 kNm

$R1_{DL}$ = 34.7 kN $R2_{DL}$ = 40.9 kN

$R1_{LL}$ = 24.7 kN $R2_{LL}$ = 32.2 kN

MAX DL DEFLECTION = 4.15 mm

MAX LL DEFLECTION = 3.24 mm

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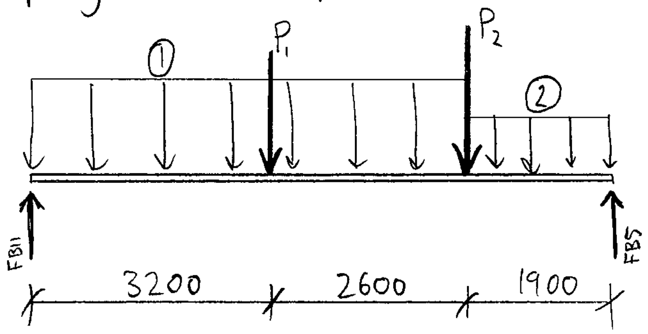
$\boxed{FB3}$ Steel bearer below party wall b/n Apartments 2 & 3

Span = 7700mm



UDL 1

FLW = 5.4 m
6.2 m double stud party wall

$W_{DL} = (5.4 \times 1.0) + (6.2 \times 1.0) + 1.0$
$= 12.6$ kN/m

$W_{LL} = 5.4 \times 1.5$
$= 8.1$ kN/m

UDL 2

FLW = 4.8 m
6.4 m double stud party wall

$W_{DL} = (4.8 \times 1.0) + (6.4 \times 1.0) + 1.0$
$= 12.2$ kN/m

$W_{LL} = 4.8 \times 1.5$
$= 7.2$ kN/m

Point Load P_1 from FB18 & concentrated load Stds

6.5m² of Mez floor + FB18 reaction

$P_{DL} = (6.5 \times 0.5) + 2$
$= 5.25$ kN

$P_{LL} = (6.5 \times 1.5) + 3.6$
$= 13.4$ kN

Point Load P_2 from FB16 & FB19

$P_{DL} = \frac{10.9}{2} \times 3 = 16.4$ kN

$P_{LL} = \frac{10.9}{2} \times 4.35 = 23.7$ kN

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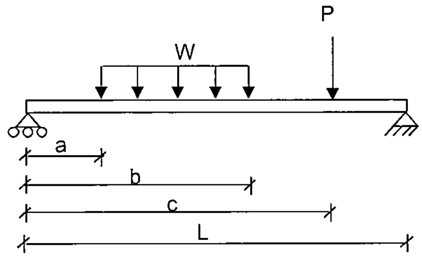
Sheet : S16

Job No : **17799**

Floor Bearer FB3 Design

Length = 7.7 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	329.00	X10^6 mm4
EI	6.58E+04	kN.m2

310 UC 137

Dead Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	12.6	12.2	0	0	0
a_i	0	5.8	0	0	0
b_i	5.8	7.7	0	0	0

	1	2	3	4	5
P_i	5.25	16.4	0	0	0
c_i	3.2	5.8	0	0	0

Live Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	8.1	7.2	0	0	0
a_i	0	5.8	0	0	0
b_i	5.8	7.7	0	0	0

	1	2	3	4	5
P_i	13.4	23.7	0	0	0
c_i	3.2	5.8	0	0	0

M_{DL} = 117.0 kNm M_{LL} = 103.2 kNm

M^* = 1.2 DL + 1.5 LL = 295.2 kNm

OR 1.35 DL = 158.0 kNm

$R1_{DL}$ = 55.5 kN $R2_{DL}$ = 62.4 kN

$R1_{LL}$ = 44.7 kN $R2_{LL}$ = 53.1 kN

MAX DL DEFLECTION = 11.05 mm

MAX LL DEFLECTION = 9.72 mm

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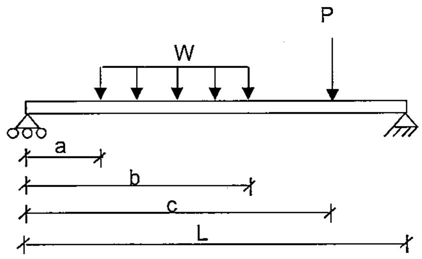
Sheet : S17

Job No : **17799**

Floor Bearer FB3 Design

Length = 7.7 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	277.00	X10^6 mm4
EI	5.54E+04	kN.m2

310 UC 118

Dead Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	12.6	12.2	0	0	0
a_i	0	5.8	0	0	0
b_i	5.8	7.7	0	0	0

	1	2	3	4	5
P_i	5.25	16.4	0	0	0
c_i	3.2	5.8	0	0	0

Live Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	8.1	7.2	0	0	0
a_i	0	5.8	0	0	0
b_i	5.8	7.7	0	0	0

	1	2	3	4	5
P_i	13.4	23.7	0	0	0
c_i	3.2	5.8	0	0	0

M_{DL} = 117.0 kNm　　　　M_{LL} = 103.2 kNm

M^* = 1.2 DL + 1.5 LL = 295.2 kNm

OR 1.35 DL = 158.0 kNm

$R1_{DL}$ = 55.5 kN　　　　$R2_{DL}$ = 62.4 kN

$R1_{LL}$ = 44.7 kN　　　　$R2_{LL}$ = 53.1 kN

MAX DL DEFLECTION = 13.12 mm

MAX LL DEFLECTION = 11.54 mm

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FB4 Steel bearer supporting eastern external wall

Span = 6850mm

FLW = 2.6m
6.0m external wall → 0.7 kPa

$W_{DL} = (2.6 \times 1.0) + (6 \times 0.7) + 0.6$
$= 7.4$ kN/m
$W_{LL} = 2.6 \times 1.5$
$= 3.9$ kN/m

Point load from FB18 (3.2m from northern end)
$P_{DL} = 3.1$ kN
$P_{LL} = 7.1$ kN

Point load from FB19 (5.7 from northern end)
↳ 5m² of mez
$P_{DL} = 5 \times 0.5$
$= 2.5$ kN

$P_{LL} = 5 \times 1.5$
$= 7.5$ kN

Select 380 PFC
↳ Enter into beam design spreadsheet

FB5 Floor Bearer built in floor space supporting both FB2 & FB3

Span = 6150mm   Point load only 450mm from end

$P_{DL} = 40.9 + 62.4 = 103.3$ kN

$P_{LL} = 32.2 + 53.1 = 85.3$ kN

Select 310UB40
↳ Enter loading into beam design spreadsheet

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Sheet : S19

Job No : **17799**

Floor Bearer FB4 Design

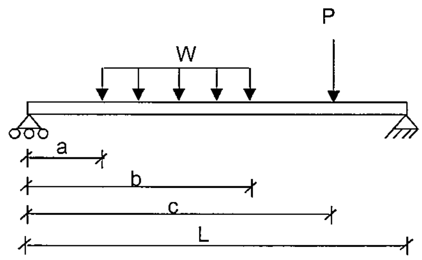
Length = 6.85 m

Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35



Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	152.00	X10^6 mm4
EI	3.04E+04	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	7.4	0	0	0	0
a_i	0	0	0	0	0
b_i	6.85	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	3.1	2.5	0	0	0
c_i	3.2	5.7	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	3.9	0	0	0	0
a_i	0	0	0	0	0
b_i	6.85	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	7.1	7.5	0	0	0
c_i	3.2	5.7	0	0	0

M_{DL} = 49.8 kNm M_{LL} = 38.9 kNm

M* = 1.2 DL + 1.5 LL = 118.1 kNm

OR 1.35 DL = 67.3 kNm

$R1_{DL}$ = 27.4 kN $R2_{DL}$ = 28.9 kN

$R1_{LL}$ = 18.4 kN $R2_{LL}$ = 22.9 kN

MAX DL DEFLECTION = 7.91 mm

MAX LL DEFLECTION = 6.02 mm

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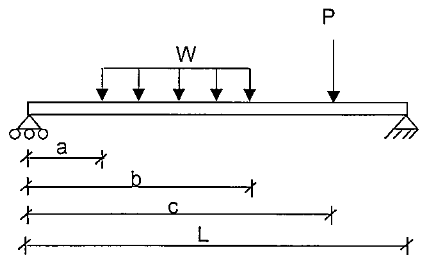
Sheet : S26

Job No : **17799**

Floor Bearer FB5 Design

Length = 6.15 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	86.40	X10^6 mm4
EI	1.73E+04	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	0.4	0	0	0	0
a_i	0	0	0	0	0
b_i	6.2	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	103.3	0	0	0	0
c_i	0.45	0	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	0	0	0	0	0
a_i	0	0	0	0	0
b_i	0	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	85.3	0	0	0	0
c_i	0.45	0	0	0	0

M_{DL} = 42.5 kNm M_{LL} = 34.5 kNm

M* = 1.2 DL + 1.5 LL = 102.8 kNm

OR 1.35 DL = 57.4 kNm

$R1_{DL}$ = 97.0 kN $R2_{DL}$ = 8.8 kN

$R1_{LL}$ = 79.1 kN $R2_{LL}$ = 6.2 kN

MAX DL DEFLECTION = 6.88 mm

MAX LL DEFLECTION = 5.34 mm

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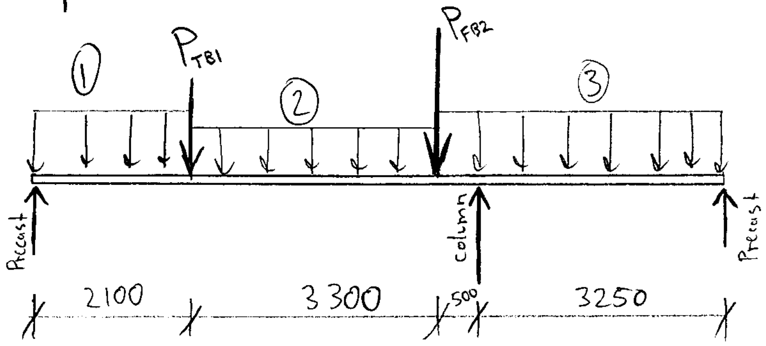
FB6 Supporting southern external wall of apartments 2 & 3 point load from bearer FB2 500mm from internal support

Spans → 5900 & 3250



UDL 1

Balcony FLW = 0·8m
Mez Balcony FLW = 1·1m
2·5m wall over (0·7)

$W_{DL} = (0{\cdot}8 \times 0{\cdot}8) + (1{\cdot}1 \times 0{\cdot}8) + (2{\cdot}5 \times 0{\cdot}7) + 0{\cdot}4$
$= 3{\cdot}67$ kN/m

$W_{LL} = (1{\cdot}9 \times 2)$
$= 3{\cdot}8$ kN/m

UDL 2 Balcony FLW = 1·5m

$W_{DL} = (1{\cdot}5 \times 0{\cdot}8) + 0{\cdot}4$
$= 1{\cdot}6$ kN/m

$W_{LL} = 1{\cdot}5 \times 2$
$= 3{\cdot}0$ kN/m

UDL 3 → Same as 1 (Slightly conservative)

Point Load P_{TB1}

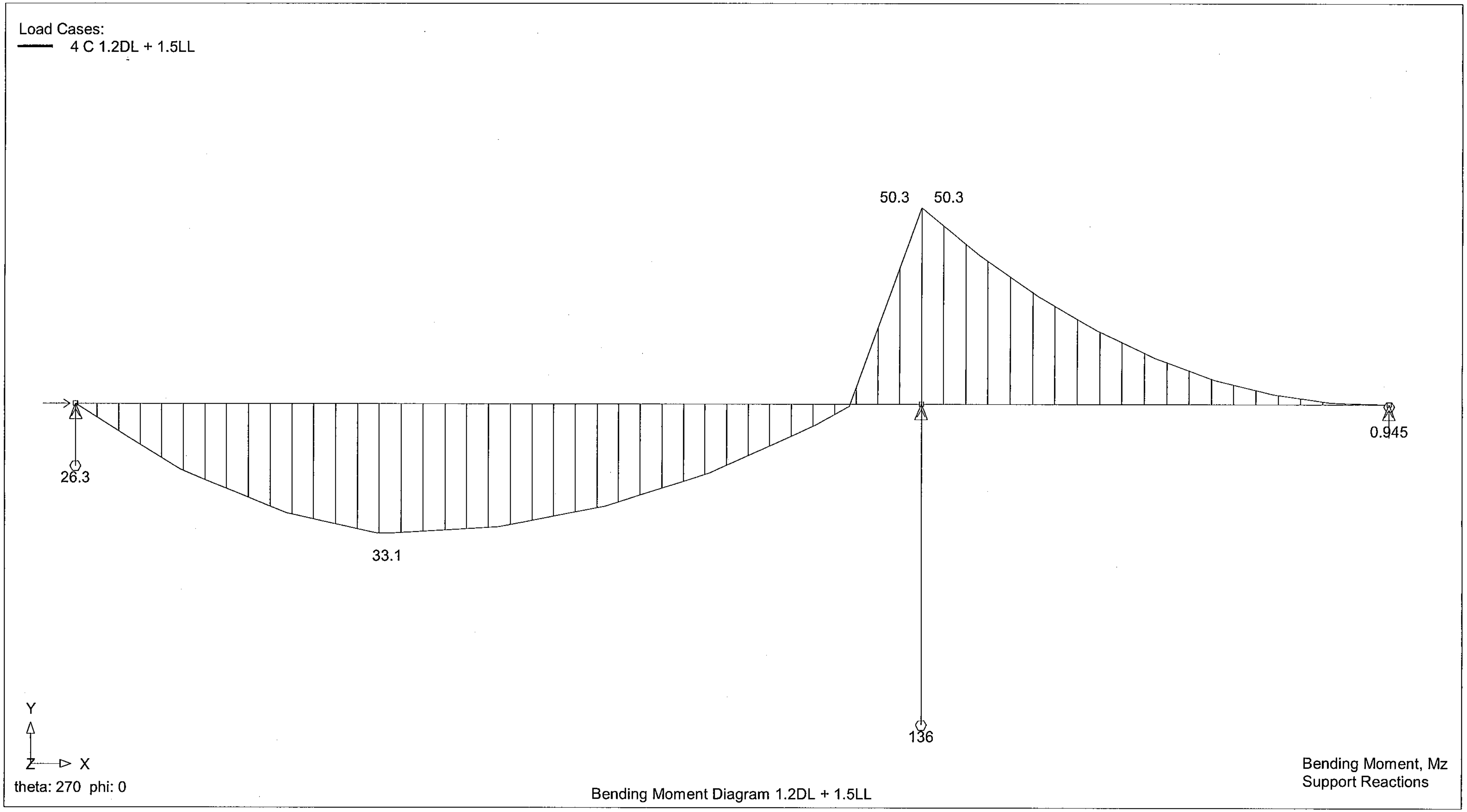
$P_{DL} = 1{\cdot}76$ kN
$P_{LL} = 1{\cdot}5$ kN
→ Refer FB17 Calc.

Point Load P_{FB2}

$P_{DL} = 34{\cdot}7$ kN
$P_{LL} = 24{\cdot}7$ kN

Enter loads into microstran model (refer following 3 pages)
↳ Select 310 UB 32 → $\phi M_b = \phi M_s = 134$ kNm

$\Delta_{DL} = 3{\cdot}0$mm
$\Delta_{LL} = 3{\cdot}7$mm
→ Acceptable



Bending Moment Diagram 1.2DL + 1.5LL

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Microstran V9
RCI Consulting Engineers
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16/02/2017
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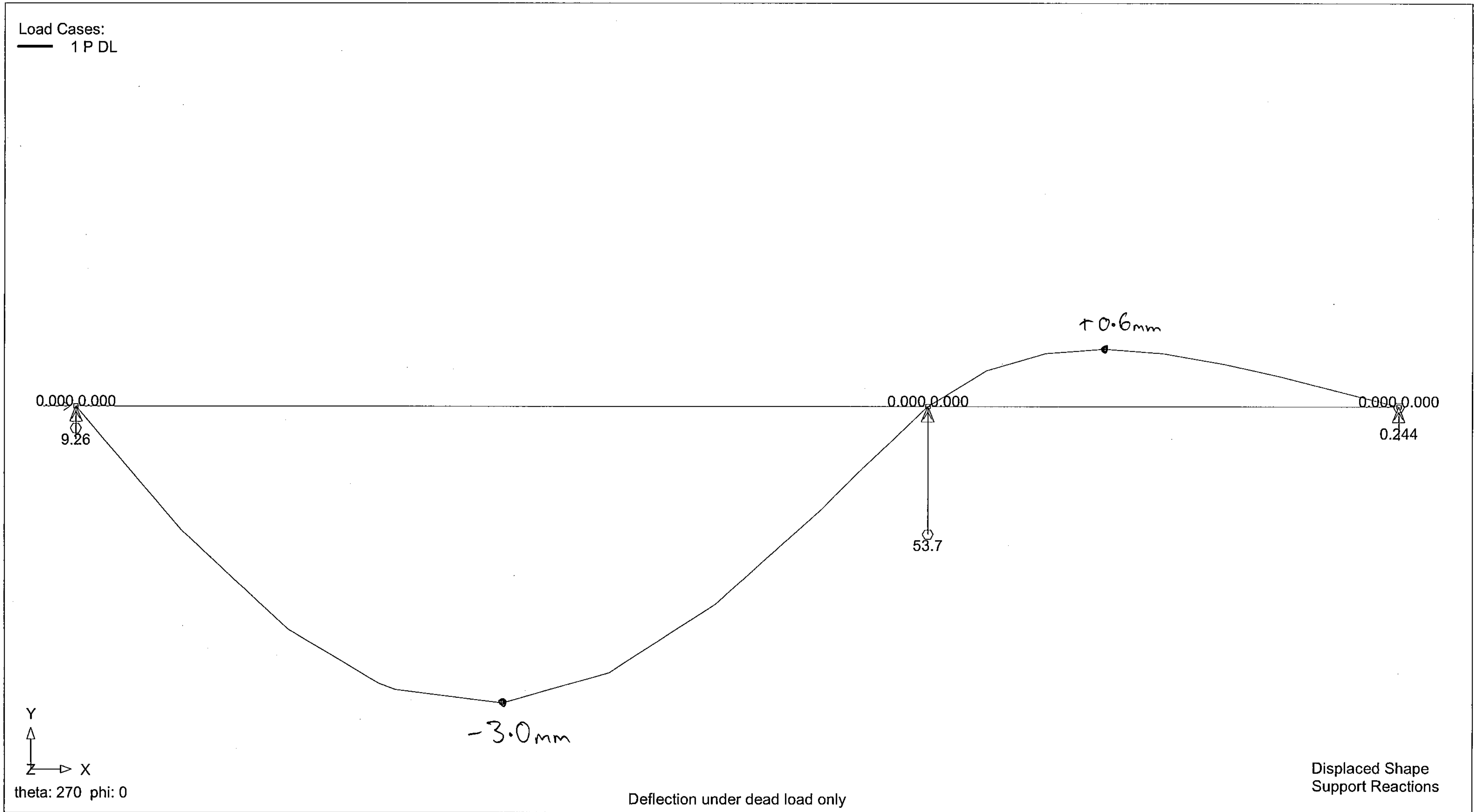
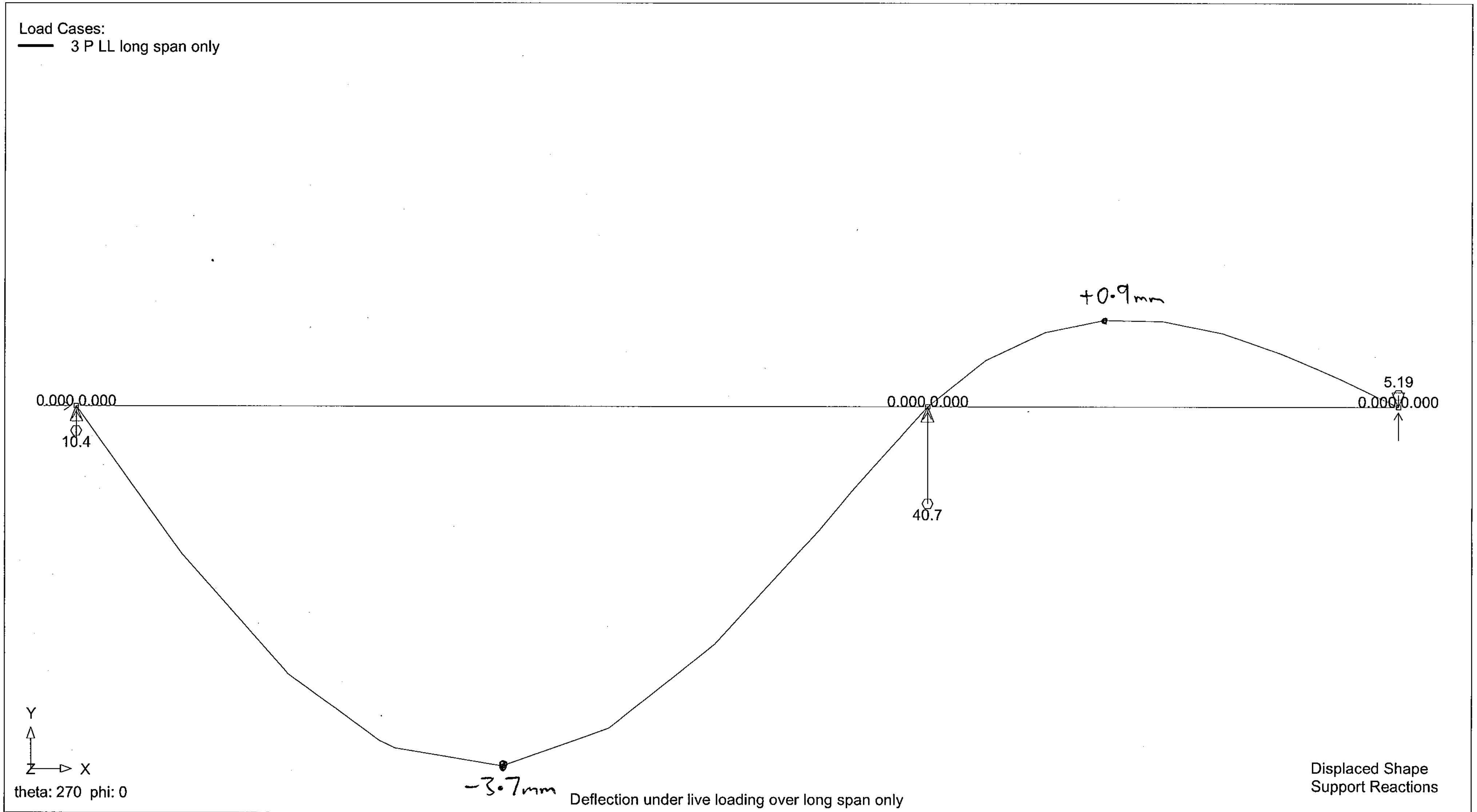
S23



Deflection under dead load only



Deflection under live loading over long span only

S24

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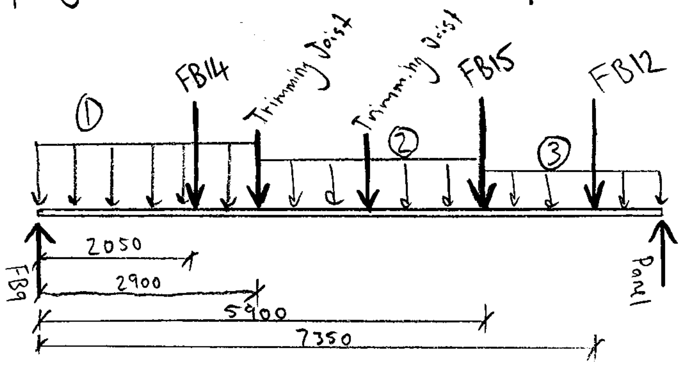
FB7 Steel bearer supporting party wall between offices & Apartment 1

Span = 8250 mm



UDL ①

7m double stud party wall (1.0 kPa)
3m T/F load bearing wall (0.4 kPa)
Office Roof Load = 6.2m
Apartment FLW = 2.8m
Office FLW = 2.95m

$W_{DL} = (7\times1.0) + (3\times0.4) + (6.2\times0.4) + (5.75\times1.0) + 1.0$
$= 17.43$ kN/m

$W_{LL} = (6.2\times0.25) + (2.8\times1.5) + (2.95\times3)$
$= 14.6$ kN/m

UDL ②

↳ Same as above but only 2.6m office roof load and 1.4m office floor load

$W_{DL} = (7\times1.0) + (3\times0.4) + (2.6\times0.4) + (4.2\times1.0) + 1.0$
$= 14.44$ kN/m

$W_{LL} = (2.6\times0.25) + (2.8\times1.5) + (1.4\times3)$
$= 9.05$ kN/m

UDL ③

Same as UDL① but no apartment floor & office floor only 1.1m

$W_{DL} = (7\times1) + (3\times0.4) + (6.2\times0.4) + (1.1\times1.0) + 1.0$
$= 12.78$ kN/m

$W_{LL} = (6.2\times0.25) + (1.1\times3)$
$= 4.85$ kN/m

Point Loads

$P_{FB14} \rightarrow P_{DL} = 8.55$ kN
$P_{LL} = 12.4$ kN

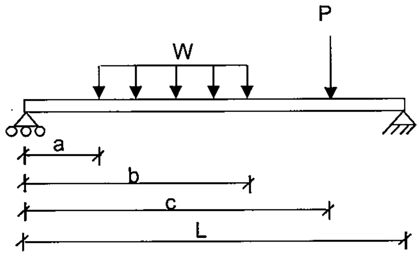
$P_{FB15} \rightarrow P_{DL} = 7.3$ kN
$P_{LL} = 3.8$ kN

$P_{FB12} \rightarrow P_{DL} = 12.5$ kN
$P_{LL} = 10.2$ kN

Select 530 UB 92 & enter loading into beam design spreadsheet
↳ refer next page

Floor Bearer FB7 Design

Length = 8.25 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	554.00	X10^6 mm4
EI	1.11E+05	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	17.43	14.44	12.78	0	0
a_i	0	2.9	5.9	0	0
b_i	2.9	5.9	8.25	0	0

	1	**2**	**3**	**4**	**5**
P_i	8.55	7.3	12.5	0	0
c_i	2.05	5.9	7.35	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	14.6	9.05	4.85	0	0
a_i	0	2.9	5.9	0	0
b_i	2.9	5.9	8.25	0	0

	1	**2**	**3**	**4**	**5**
P_i	12.4	3.8	10.2	0	0
c_i	2.05	5.9	7.35	0	0

M_{DL} = 149.8 kNm M_{LL} = 106.1 kNm

M* = 1.2 DL + 1.5 LL = 338.9 kNm

OR 1.35 DL = 202.2 kNm

$R1_{DL}$ = 76.0 kN $R2_{DL}$ = 76.2 kN

$R1_{LL}$ = 60.7 kN $R2_{LL}$ = 46.6 kN

MAX DL DEFLECTION = 9.75 mm

MAX LL DEFLECTION = 6.86 mm

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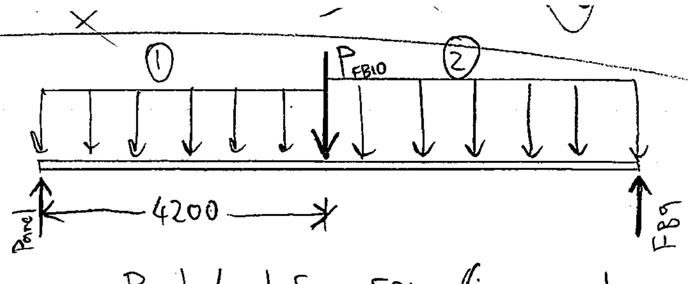
FB8 Span = 8150 mm



UDL ① Same as UDL ① above in FB7 design but no apartment Floor load

$W_{DL} = 13.6$ kN/m
$W_{LL} = 10.4$ kN/m

Point Load from FB10 (ignore continuous action → assume FB10 is single span between FB8 and first column)

UDL ② Same as UDL ① above in FB7 design

$W_{DL} = 16.4$ kN/m
$W_{LL} = 14.6$ kN/m

$$P_{DL} = \frac{2.775}{3.1}(0.65 \times 6.14) + \frac{2.45}{3.1}(9.76) + \frac{1.875}{3.1}(2.45 \times 1.59)$$
$$= 13.6 \text{ kN}$$

$$P_{LL} = \frac{2.775}{3.1}(0.65 \times 1.23) + \frac{2.45}{3.1}(6) + \frac{1.875}{3.1}(2.45 \times 2)$$
$$= 8.42 \text{ kN}$$

Select 530 UB 82 → $I_x = 477 \times 10^6 \text{ mm}^4$

Enter into Beam design spreadsheet

RCI CONSULTING ENGINEERS

Sheet : S28

Job No : **17799**

Floor Bearer FB8 Design

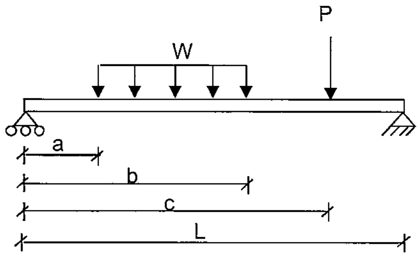
Length = 8.15 m

Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35



Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	477.00	X10^6 mm4
EI	9.54E+04	kN.m2

Dead Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	13.6	16.4	0	0	0
a_i	0	4.2	0	0	0
b_i	4.2	8.15	0	0	0

	1	2	3	4	5
P_i	13.6	0	0	0	0
c_i	4.2	0	0	0	0

Live Load

Distrubuted Load (W)

	1	2	3	4	5
W_i	10.4	14.6	0	0	0
a_i	0	4.2	0	0	0
b_i	4.2	8.15	0	0	0

	1	2	3	4	5
P_i	8.42	0	0	0	0
c_i	4.2	0	0	0	0

M_{DL} = 150.7 kNm M_{LL} = 119.9 kNm

M^* = 1.2 DL + 1.5 LL = 360.7 kNm

OR 1.35 DL = 203.4 kNm

$R1_{DL}$ = 64.7 kN $R2_{DL}$ = 70.8 kN

$R1_{LL}$ = 50.5 kN $R2_{LL}$ = 59.3 kN

MAX DL DEFLECTION = 10.58 mm

MAX LL DEFLECTION = 8.44 mm

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FB9: Cantilever Bearer Supporting steel bearers FB7 & FB8 at free end.

Backspan = 4550 mm
Cantilever = 1450 mm

$P_{DL} = 76 + 70.8 = 147$ kN $P^* = 356$ kN

$P_{LL} = 60.7 + 59.3 = 120$ kN $M^*_{-ve} = 517$ kNm

For strength Select 530 UB

↳ $\phi M_b = 584$ kNm ($l_e = 2.0$m)

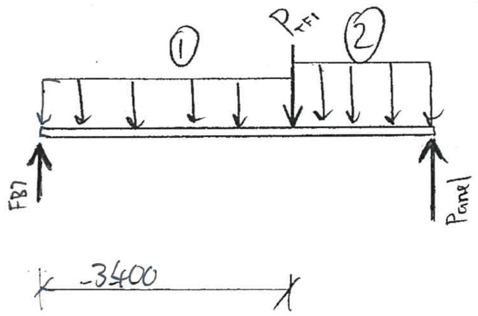
$I_x = 554 \times 10^6$ mm⁴

lateral restraint of compression flange provided by cantilever column

$$\Delta_{DL} = \frac{147000 \times 1450^2 \times 6000}{3 \times 200\times10^3 \times 554 \times 10^6}$$

$= 5.58$ mm

FB12 Span = 5400 mm



UDL ① Same as UDL ② from FB6 design but add extra 1.0 kN/m DL for S.W

$W_{DL} = 2.6$ kN/m
$W_{LL} = 3.0$ kN/m

UDL ② Same as UDL ① from FB6 design but add extra 1.0 kN/m DL for S.W

$W_{DL} = 4.67$ kN/m
$W_{LL} = 3.8$ kN/m

P_{TF1}

↳ $P_{DL} = 1.76$ kN
$P_{LL} = 1.5$ kN

Select 310UB32 → $I_x = 63.2 \times 10^6$ mm⁴ & $\phi M_b = 109$ kNm ($l_e = 2.0$m)

↳ Enter into beam design spreadsheet (refer next page)

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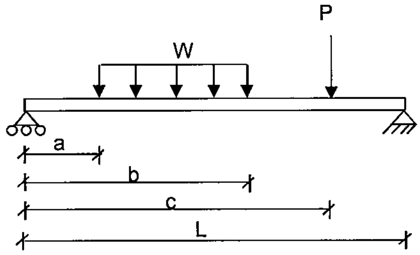
Sheet : S30

Job No : **17799**

Floor Bearer FB12 Design

Length = 5.4 m



Ultimate Limit State

DL	1.2
LL	1.5

OR

DL	1.35

Sevriceability Limit State

DL	1
LL	0.7

E	2.00E+008	kPa
Ix	63.20	X10^6 mm4
EI	1.26E+04	kN.m2

Dead Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	2.6	4.67	0	0	0
a_i	0	3.4	0	0	0
b_i	3.4	5.4	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.76	0	0	0	0
c_i	3.4	0	0	0	0

Live Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	3	3.8	0	0	0
a_i	0	3.4	0	0	0
b_i	3.4	5.4	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.5	0	0	0	0
c_i	3.4	0	0	0	0

M_{DL} = 13.7 kNm M_{LL} = 13.3 kNm

M* = 1.2 DL + 1.5 LL = 36.5 kNm

OR 1.35 DL = 18.5 kNm

$R1_{DL}$ = 8.4 kN $R2_{DL}$ = 11.5 kN

$R1_{LL}$ = 9.0 kN $R2_{LL}$ = 10.3 kN

MAX DL DEFLECTION = 3.23 mm

MAX LL DEFLECTION = 3.19 mm

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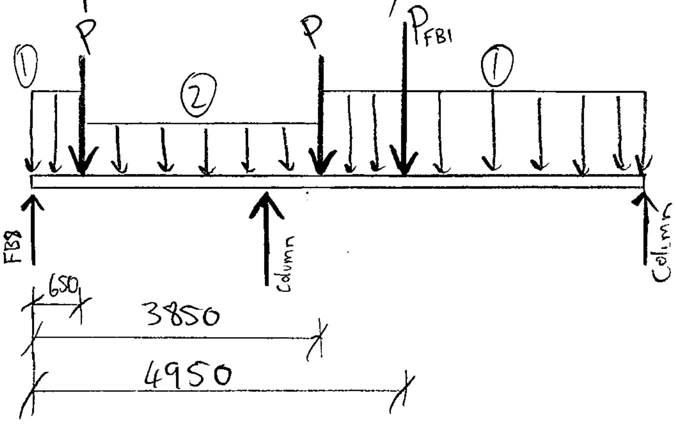
FB10 Continuous over two spans 3100mm / 4950mm



UDL ① 5.4m external wall
Upper RLW = 4.9m

$W_{DL} = (5.4 \times 0.7) + (4.9 \times 0.4) + 0.4$
$= 6.14$ kN/m

$W_{LL} = 4.9 \times 0.25$
$= 1.23$ kN/m

UDL ② Balcony FLW = 1.0m
3.9m window/door

$W_{DL} = (1 \times 0.8) + (3.9 \times 0.1) + 0.4$
$= 1.59$ kN/m

$W_{LL} = 1 \times 2$
$= 2.0$ kN/m

Point Load P_{FB1}

$P_{DL} = 40.2$ kN

$P_{LL} = 24.2$ kN

Point Load P (from BB1 & Column C4 above)

3.2 m² of Canopy roof
8.0 m² of main roof
1.6 m² of balcony floor
8.0 m² of balcony side wall

$P_{DL} = (3.2 \times 0.4) + (8 \times 0.4) + (1.6 \times 0.8) + (8 \times 0.5)$
$= 9.76$ kN

$P_{LL} = (11.2 \times 0.25) + (1.6 \times 2)$
$= 6.0$ kN

Enter into computer software Microstran
↳ Select 310UB40 $\phi M_b = 127$ kNm ($l_e = 3.0$m)

Refer following 3 pages

$\Delta_{DL} = 5.1$ mm

$\Delta_{LL} = 2.5$ mm

Microstran V9
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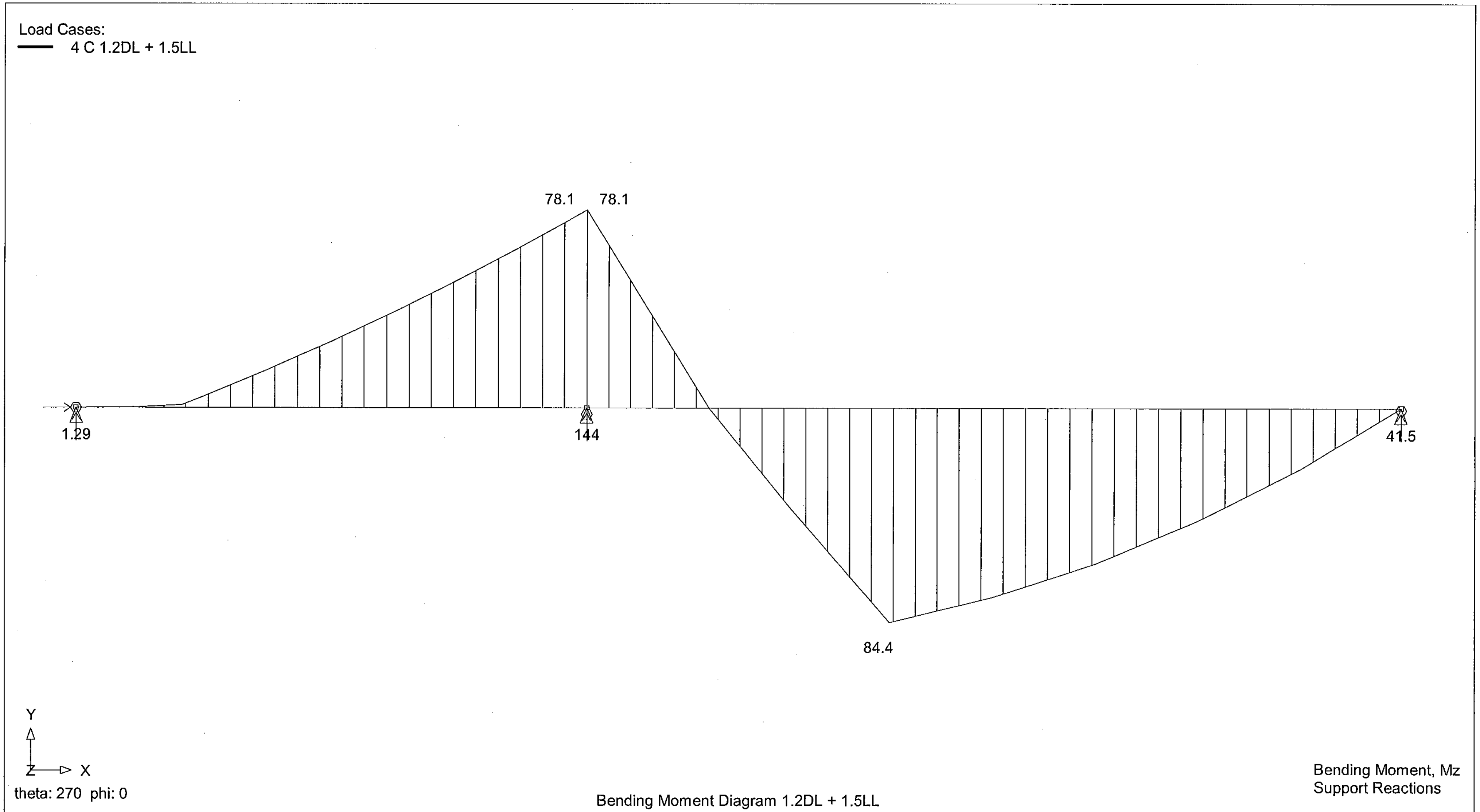

Bending Moment Diagram 1.2DL + 1.5LL

Bending Moment, Mz
Support Reactions

S31

Microstran V9.00.111024 {1767411}

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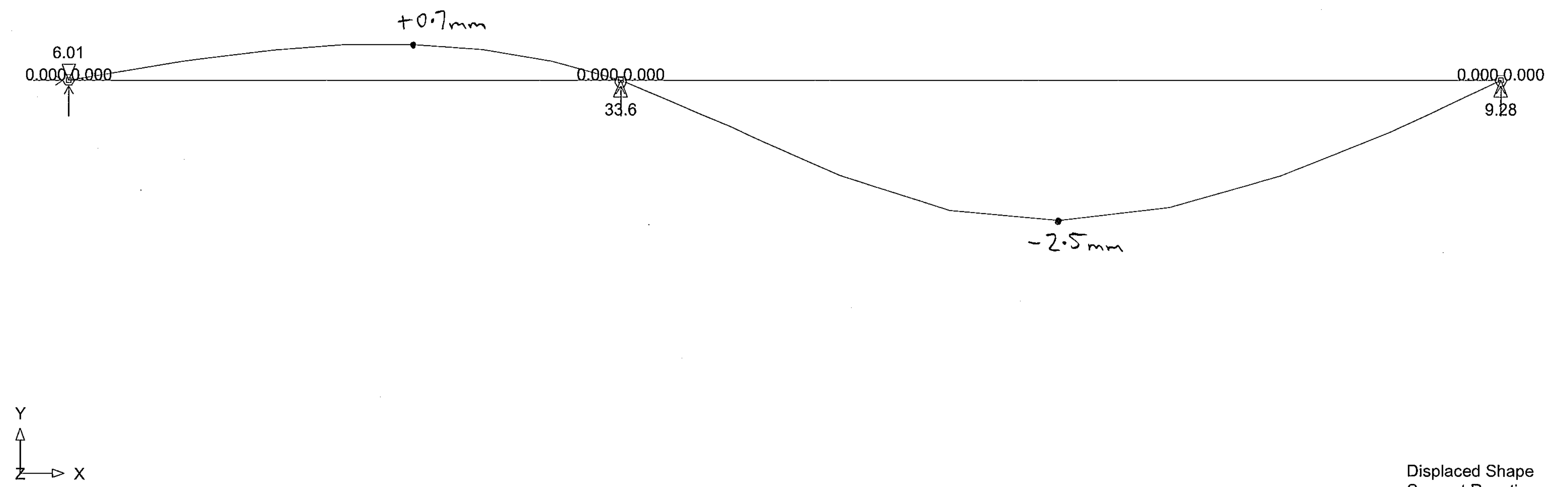
Load Cases:
3 P LL over long span only



theta: 270 phi: 0

FB10 - Deflection live load over long span only

Displaced Shape
Support Reactions

S32

Microstran V9.00.111024 {1767411}

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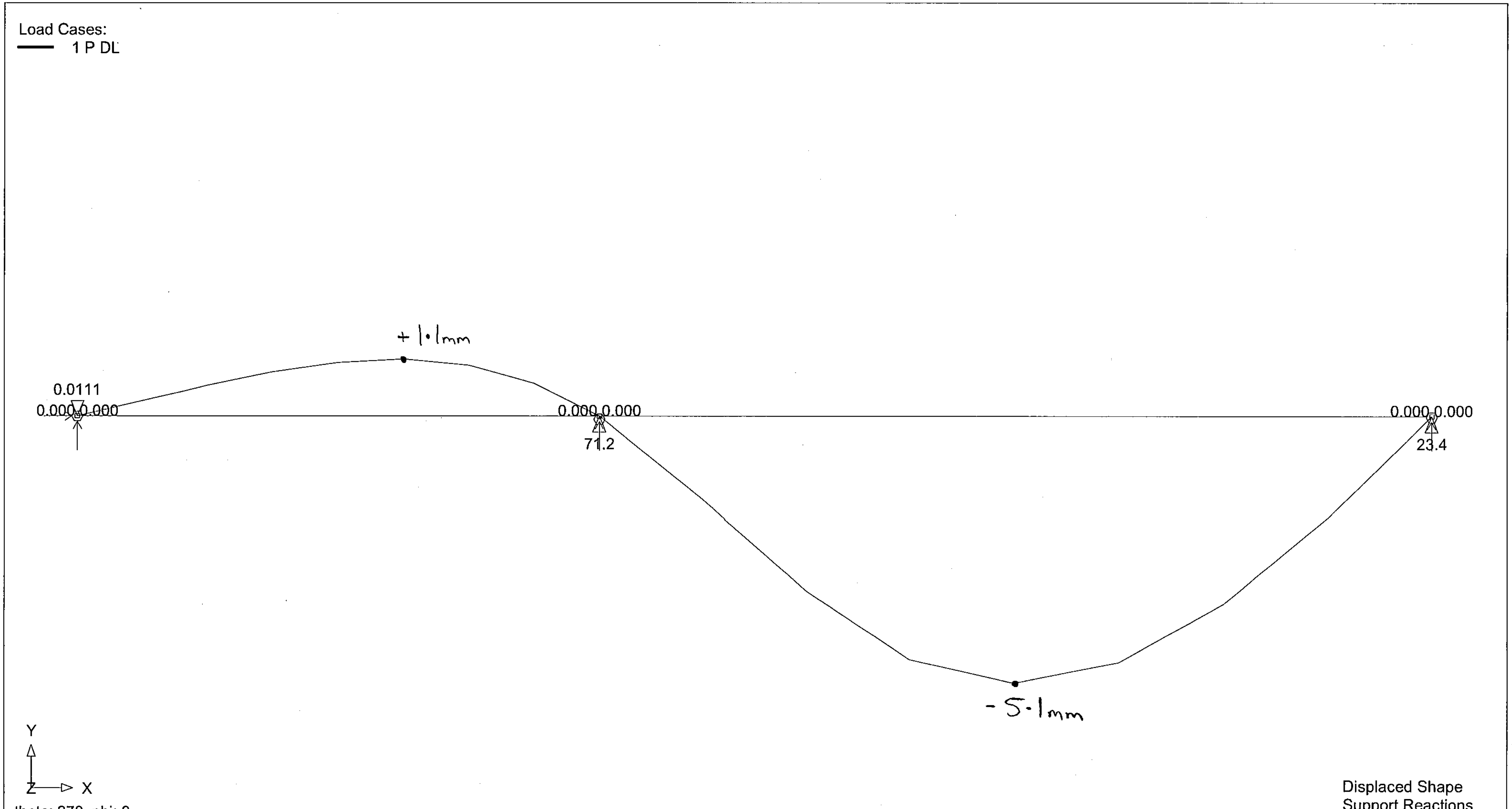

FB10 - Deflection under dead load only

S33

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$\boxed{FB13}$ Span = 2150 mm

Beam supporting western end of FB12

$P_{DL} = 60.7$ kN

$P_{LL} = 42.4$ kN

$P^* = 136$ kN

Select 300 PFC → $I_x = 72.4 \times 10^6 \text{ mm}^4$ → $\phi M_b = 131$ kNm ($L = 1.5$m)

$$\Delta_{DL} = \frac{60700 \times 2150^3}{48 \times 2 \times 10^5 \times 72.4 \times 10^6}$$

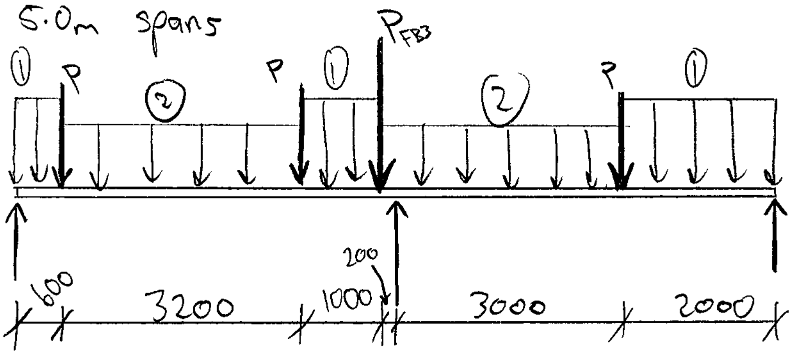
$= 0.9$ mm

$$M^* = \frac{136 \times 2.15}{4}$$

$= 73.1$ kNm

$\boxed{FB11}$ Supporting northern external wall (Similar to FB10)

Continuous over two 5.0m spans



UDL ① (Same as for FB10 design)

$W_{DL} = 6.14$ kN/m

$W_{LL} = 1.23$ kN/m

UDL ② (Same as for FB10 design)

$W_{DL} = 1.59$ kN/m

$W_{LL} = 2.0$ kN/m

Point Load P

$P_{DL} = 9.76$ kN

$P_{LL} = 6.0$ kN

Point Load P_{FB3}

$P_{DL} = 55.5 + 9.76$
$= 65.3$ kN

$P_{LL} = 44.7 + 6$
$= 50.7$ kN

Select Same as FB10

→ 310 UB 40

Enter into Microstran (refer following 3 pages)

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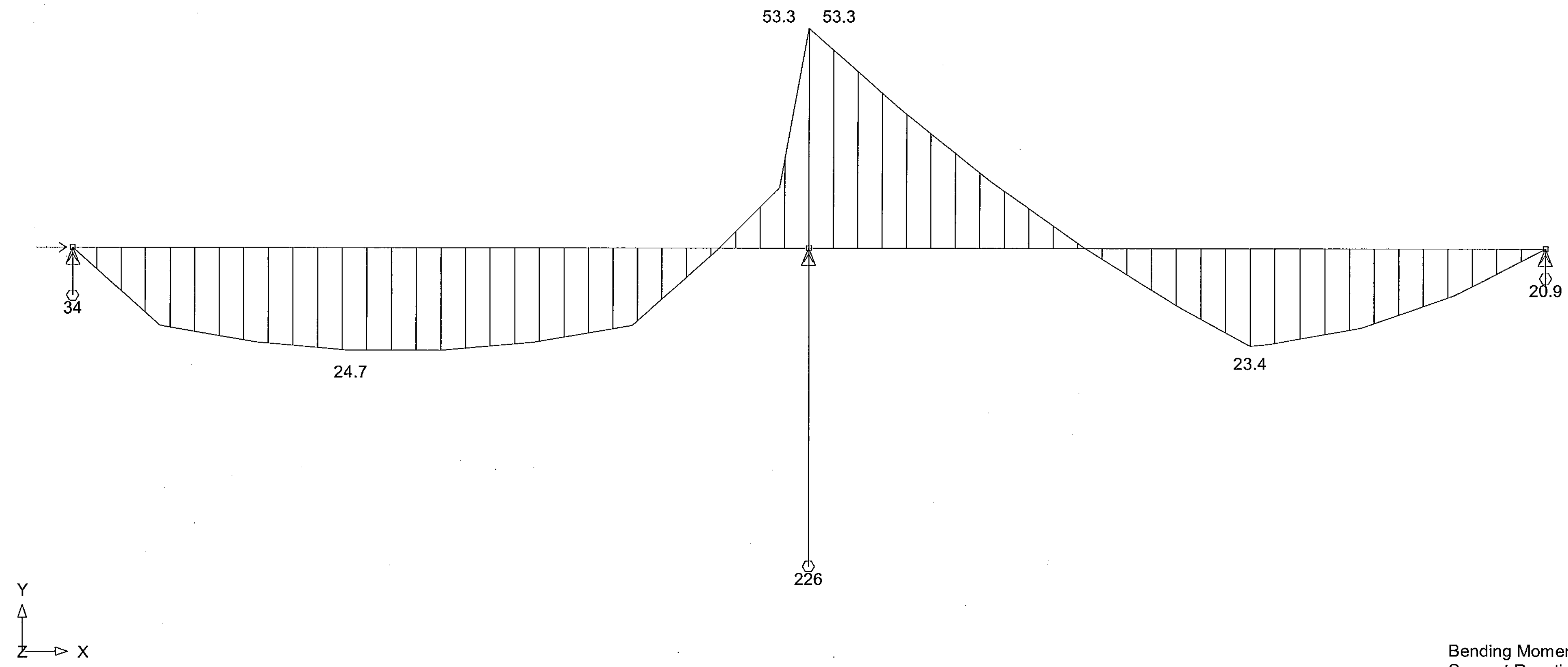
Load Cases:
3 C 1.2DL + 1.5LL



theta: 270 phi: 0

Bending Moment Diagram 1.2DL + 1.5LL

Bending Moment, Mz
Support Reactions

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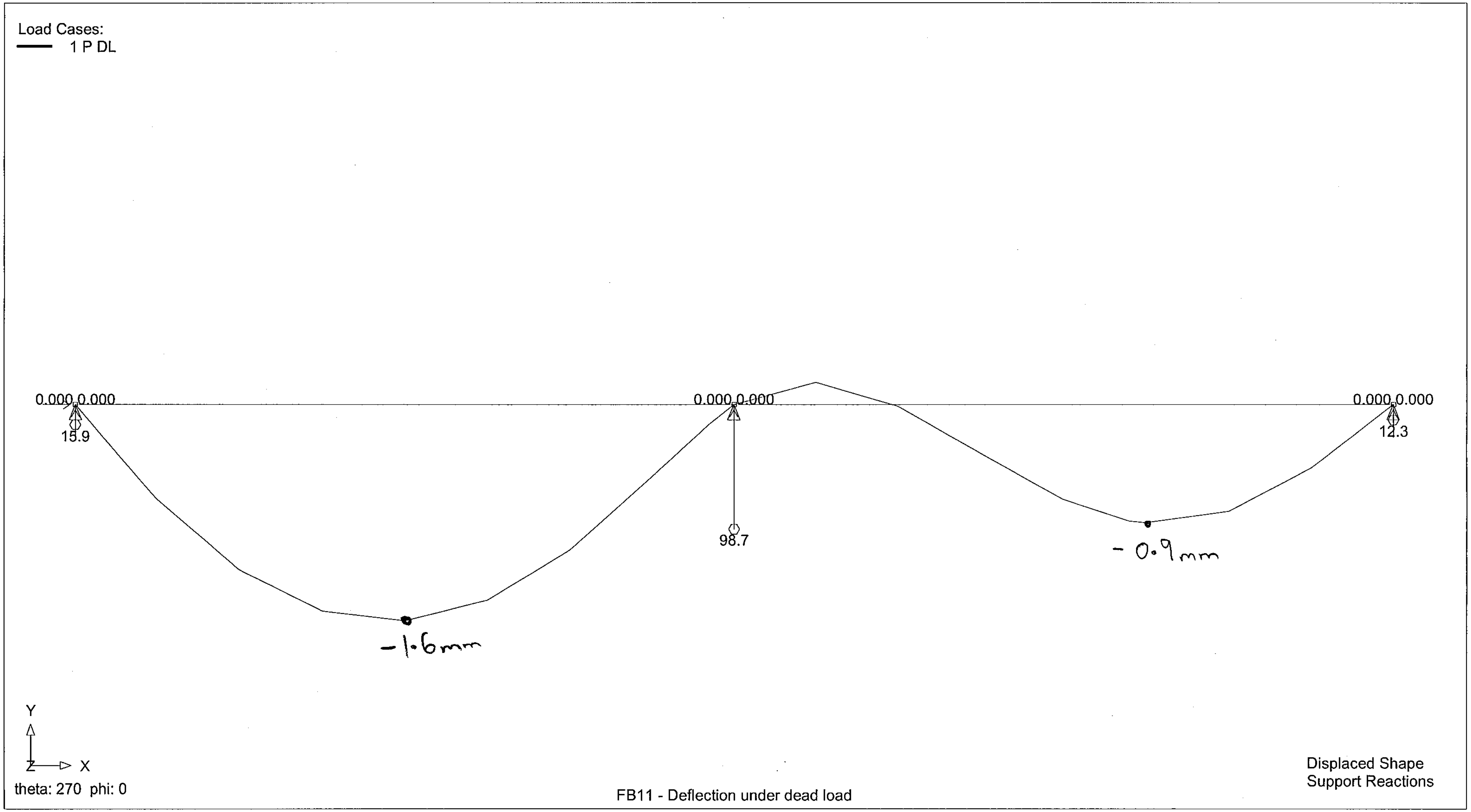
S36



FB11 - Deflection under dead load

Microstran V9

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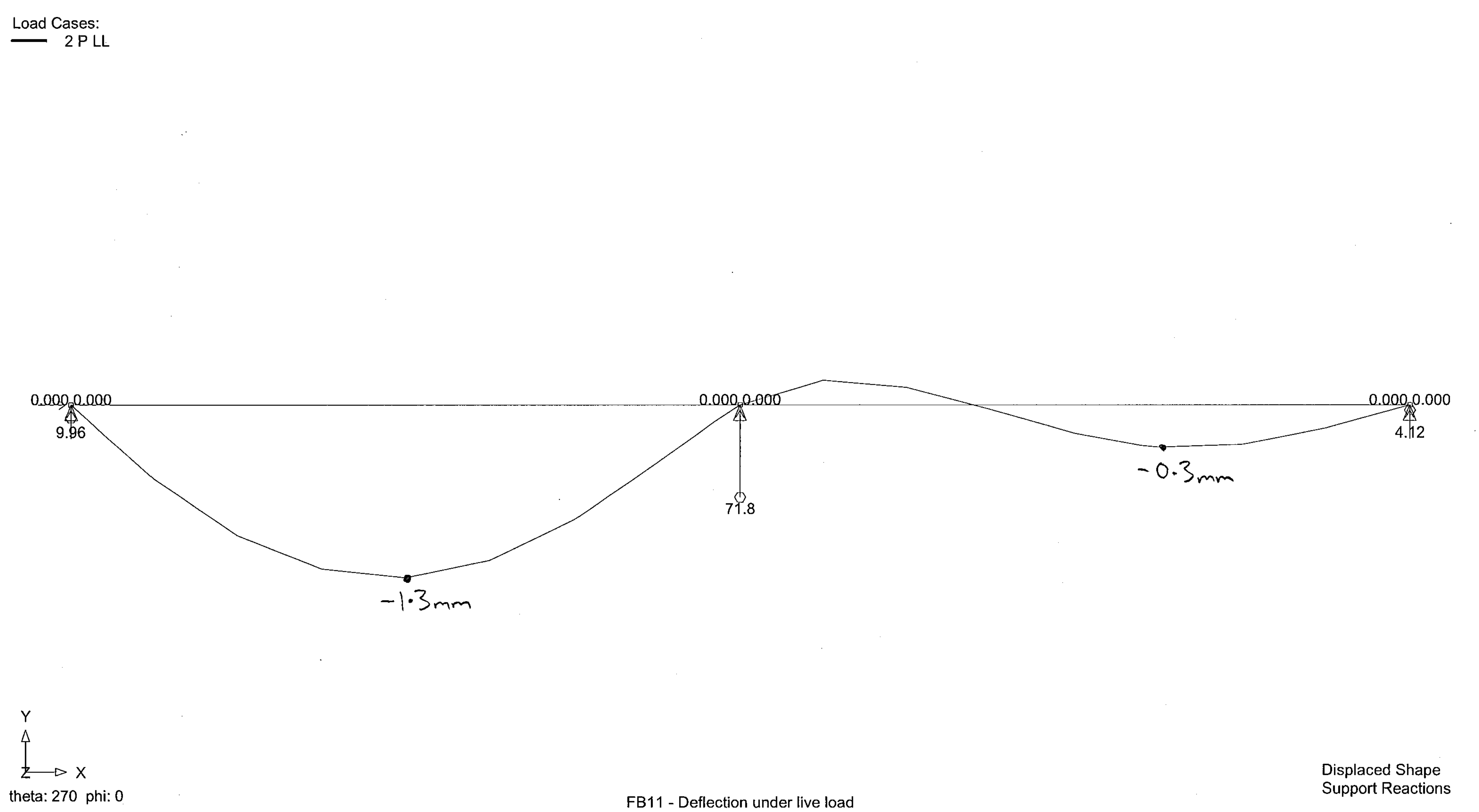

FB11 - Deflection under live load

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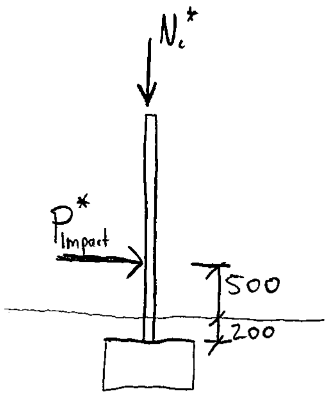
Columns

C1 Columns supporting building over

Worst case for total Axial load is central support under FB11

↳ $1.2DL + 1.5LL \rightarrow N_c^* = 226$ kN



Due to column being located within carpark must design for impact load from a misguided vehicle (P^*_{impact}).

Limited design criteria for this situation is available.

↳ The Australian/New Zealand Standard for Structural actions (AS 1170.1) does nominate design loads for 'barriers' within carparks for 'Light Traffic Areas'

The static design load 30 kN is applied 500mm above ground level. Although this is intended for the design of barriers this office can see no reason that the design load could not be applied to a column in the same carpark situation. The big difference being the implications of if this design load was exceeded.

The commentary for AS 1170.1 provides details behind the calculations used to determine the 30 kN Static load. The load has been calculated assuming a 1500 kg vehicle with a crumple zone of 100mm travelling at 2 m/s

While this office considers the derivation/calculation of this load a vast oversimplification of what is a very complex problem (converting impact load of a vehicle into an equivalent static load) it is is appreciated that at some point some large assumptions must be made in order to easily calculate the the design static load, suitable for an almost infinite different cases (car type, speed etc)

Due to the significant consequence of a column 'failing' compared to that of a barrier as well as the simplified derivation method this office proposes to increase the vehicle velocity by 50% up to 3.0 m/s

Design Load = 67.5 kN

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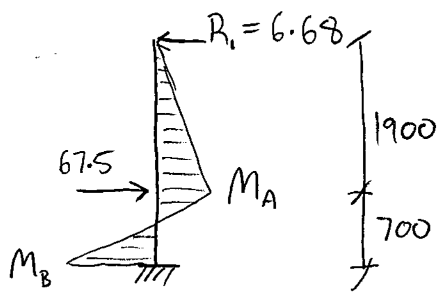
Top of column is restrained laterally by floor diaphragm
Base of column is fixed.
↳ Propped Cantilever

Consider eccentricity (e) of N_c^* to be half column width in both directions

$$M_x = M_y = 0.075 \times 226 = 16.95 \text{ kN}$$

Bending moment from P^*_{impact}



$$M_A = 6.68 \times 1.9 = 12.69 \text{ kNm}$$

$$M_B = \frac{67.5 \times 1.9 \times 0.7 \times (1.9 + 2.6)}{2 \times 2.6^2} = 29.88 \text{ kNm}$$

Critical Ultimate Design Loads

Case ①

$N_c^* = 226$ kN

$M_x^* = 16.95$ kNm

$M_y^* = 16.95 + 1.5(29.88) = 61.8$ kNm

150 × 8.0 SHS 450 grade column → $\phi N_c = 1572$ kN
$k_e = 0.85$ → $l_e = 0.85 \times 2600 = 2210$ mm
$\phi M_s = 91.5$ kNm

$$\phi M_i = \phi M_o = 91.5\left(1 - \frac{226}{1572}\right) = 78.3 \text{ kNm}$$

Check biaxial bending (case ①)

$$\left(\frac{M_x^*}{\phi M_{cx}}\right)^{1.4} + \left(\frac{M_y^*}{\phi M_{iy}}\right)^{1.4} = \left(\frac{16.95}{78.3}\right)^{1.4} + \left(\frac{61.8}{78.3}\right)^{1.4} = 0.835 < 1.0 \Rightarrow \text{ok}$$

Adopt 150 × 8.0 SHS

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Precast Panel Design

Select 150 thick panels, N32 concrete

↳ Max Allowable effective height, $H_{max} = 50 \times 150 = 7500\,mm$

Min steel for both horizontal & vertical

↳ $A_{s,min} = 0.0015 \times 150 \times 1000$
$= 225\ mm^2$

Adopt SL82 mesh central ($A_{st} = 227\ mm^2$ per m in both directions)

Bending capacity of panel (assume worst case $d = \frac{150}{2} - 8 - 4 = 63\,mm$)

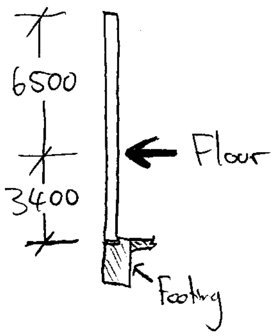
$$\phi M_u = 0.8 \times 500 \times 227 \times 63 \left(1 - \frac{0.6 \times 500 \times 227}{32 \times 63 \times 1000}\right)$$

$= 5.53$ kNm per metre

Under normal conditions full height panels are supported by both floor & roof/ceiling framing by inspection / experience panels are ok!!

Check Full Height Panels in fire event where upper roof/ceiling members damaged → no lateral support at top

Panels on Northern boundary are worst case



Check panel under serviceability wind loads

$WL = (0.837 \times (0.7 + 0.3) \times 0.9) \times 0.676$
$= 0.509$ kN/m per m

Bending in panel

↳ $M^* = \dfrac{0.509 \times 6.5^2}{2}$

$= 10.8$ kNm per m

150 panel with SL82 central → not suitable

Try extra N16 bars @ 600 crs → $\frac{10}{6} \times 200 = 333\ mm^2$ per m

$d = \frac{150}{2} - 8 - 8$
$= 59\,mm$

$$\phi M_u = 0.8 \times 500 \times 560 \times 59 \left(1 - \frac{0.6 \times 500 \times 560}{32 \times 59 \times 1000}\right)$$

$= 12.0$ kNm per m > 10.8 kNm ⇒ ok!!

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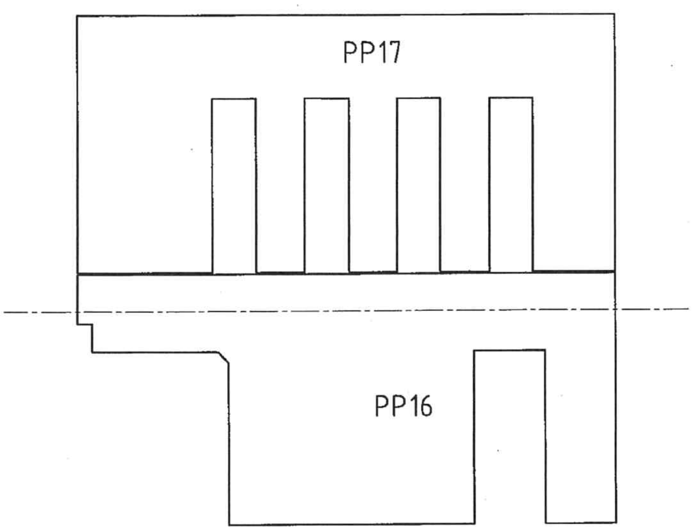
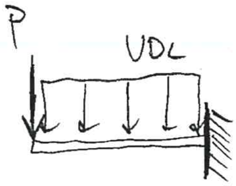
Panel 16

Panel 16 includes 1050mm deep section that cantilevers out from main panel supporting both panel above for loading and point load at the end.

↳ cantilever = 2.1m



Design as fixed end cantilever beam



UDL → 4.6m high of 150 panel (self weight of beam included)

$W_{DL} = (4.6 \times 0.15 \times 25)$
$= 17.3$ kN/m

$w^* = 20.8$ kN/m

Point Load from FB7 & BB1 (BB1 → $2m^2$ of balcony + $3m^2$ of balustrade wall)

$P_{DL} = 76.2 + (2 \times 1.0) + (3 \times 0.5)$
$= 79.7$ kN

$P_{LL} = 46.6 + (2 \times 2)$
$= 50.6$ kN

$P^* = 172$ kN

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Negative Design Bending Moment

$$\hookrightarrow M^* = \frac{20.8 \times 2.1^2}{2} + 172 \times 2.1$$
$$= 407 \text{ kNm}$$

Shear Force

$$V^* = (20.8 \times 2.1) + 172$$
$$= 216 \text{ kN}$$

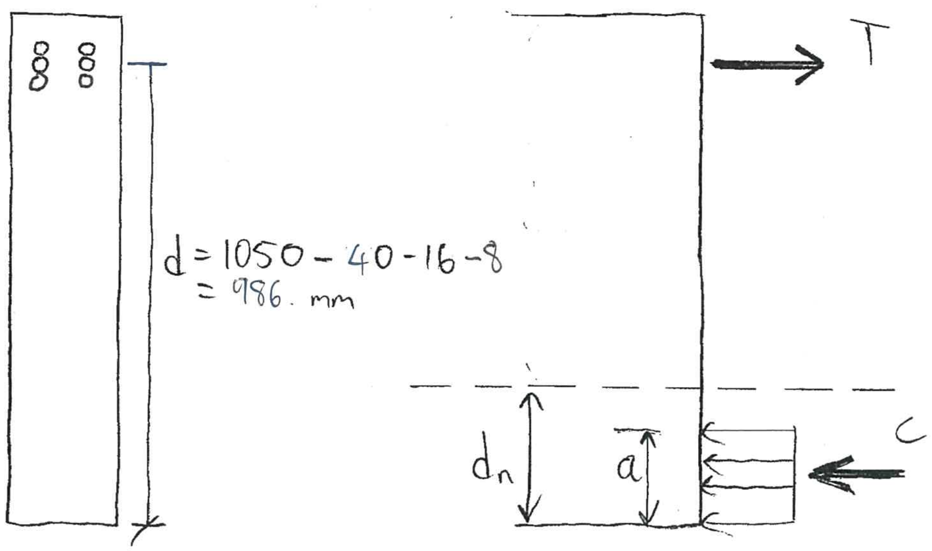
Beam dimensions 150 wide × 1050 deep

Approx tensile reinforcement required   take $d = 1050 - 90 = 960$

$$T_y = \frac{M^*}{\phi 0.9 d} = \frac{407 \times 10^6}{0.8 \times 0.9 \times 960} = 589 \times 10^3 \text{ N}$$

$$A_{Req} = \frac{589 \times 10^3}{500} = 1178 \text{ mm}^2$$

Select 6/N16 bars $A_{st} = 1200 \text{ mm}^2$



Assume tensile steel @ yield

$$T = 1200 \times 500 = 600 \times 10^3 \text{ N}$$

$$C = 0.85 \times 32 \times 150 \times a$$
$$= 4080\,a$$

$$T = C \Rightarrow a = \frac{600 \times 10^3}{4080} = 147.06 \text{ mm}$$

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$$M_u = 600\left(0.986 - \frac{0.14706}{2}\right)$$
$$= 547.5 \text{ kNm}$$

$$\phi M_u = 0.8 \times 547.5$$
$$\approx 438 \text{ kNm} > M^* \Rightarrow \text{ok for flexural strength}$$

check depth of neutral axis

$$d_n = \frac{a}{\gamma} \quad \text{where } \gamma = 0.822 \text{ for } f'_c = 32$$
$$d_n = \frac{147.06}{0.822} = 178.9 \text{ mm}$$

$$d_n = k_u d \Rightarrow k_u = \frac{178.9}{986} = 0.181$$

k_u is less than 0.4 $\Rightarrow$ Section considered ductile!!
↳ ok!

Shear Reinforcement

Beam depth > 750mm min shear reinforcement $A_{sv,min}$ required

Shear strength of beam excluding shear reinforcement

$$V_{uc} = \beta_1 \beta_2 \beta_3 b_v d_o f_{cv}\left(\frac{A_{st}}{b_v d_o}\right)^{1/3}$$
$$= 1.1 \times 150 \times 2 \times 3.17\left(\frac{1200}{150 \times 1002}\right)^{1/3}$$
$$= 104.7 \text{ kN}$$

$\beta_1 = 1.1$
$\beta_2 = \beta_3 = 1.0$
$b_v = 150$ mm
$d_o = 1050 - 40 - 8 = 1002$ mm
$A_{st} = 1200$ mm²
$f'_{cv} = 32^{1/3} = 3.17$ MPa

Shear strength with min shear reinforcement

$$\phi V_{u,min} = 0.7\left[104.7 + \left(0.1 \times \sqrt{32} \times 150 \times 1002\right)\right]$$
$$= 133 \text{ kN} < V^* \Rightarrow \text{Must adopt more than } A_{sv,min}$$

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$$\phi V_{uc} + \phi V_{us} \geq V^*$$

$$\Rightarrow \phi V_{us} \geq V^* - (0.7 \times 104.7)$$

$$\geq 142.7 \text{ kN}$$

Select 6mm ligs $\Rightarrow A_{sv} = 2 \times 28 = 56 \text{ mm}^2$

$$\phi V_{us} = 0.7\left(\frac{A_{sv}}{s} \times f_{sy} \times d_o \times \cot\theta^\circ\right) \quad \Rightarrow \theta^\circ = 45 \Rightarrow \cot\theta^\circ = 1.0$$

$$= 0.7\left(\frac{56}{s} \times 500 \times 1002 \times 1\right)$$

$$= \frac{19.639 \times 10^6}{s}$$

Try 200mm spacing ($s = 200$mm)

$\hookrightarrow \phi V_{us} = 98.2$ kN $\rightarrow$ Not enough

Increase to 100mm spacing ($s = 100$mm)

$\hookrightarrow \phi V_{us} = 196$ kN $> 142.7 \Rightarrow$ OK

Shear capacity $= 196 + 0.7(104.7)$

$= 269$ kN $> V^*$

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Sub-Floor Bracing Below Apartments $\boxed{\text{East-West}}$

Area of elevation on eastern side of building (apartments only)

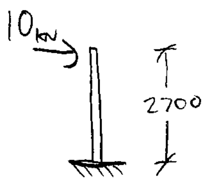
↳ 100 m^2

Windward Pressure (WD3) $C_{p,e} = +0.7$

Leeward Pressure (WD1) $C_{p,e} = -0.5$

WD1 is critical Design Racking Force, $F^* = 0.837 \times 0.5 \times 100$
$= 41.9$ kN

6 × Cantilevered Columns to resist 10 kN each



$M^* = 27$ kNm

150 × 8.0 SHS ok by inspection for strength

Check deflection $P_{serv} = \frac{37^2}{45^2} \times 10 = 6.76$ kN

$$\Delta_{sway} = \frac{6760 \times 2700^3}{3 \times 2 \times 10^5 \times 14.1 \times 10^6}$$
$$= 15.7 \text{ mm} \approx \frac{\text{Height}}{172} < H/150 \Rightarrow \text{Acceptable}$$

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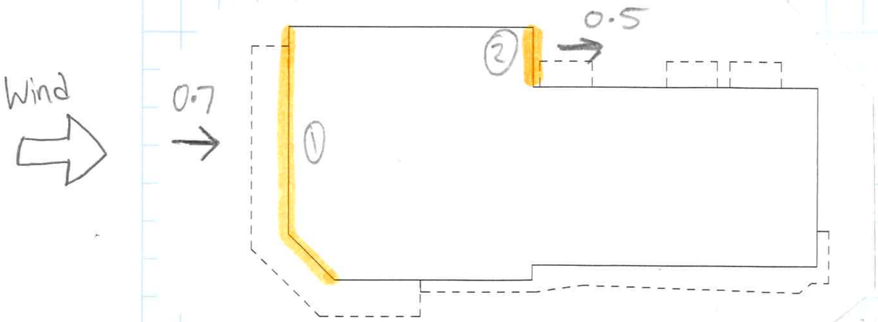
Ground Floor Bracing for Office

East - West

WD1 ⇒ $q_z = 0.837$ kPa



Area ① ⇒ 134 m² (windward only)

Area ② ⇒ 35 m² (leeward only)

Total Racking Force

Take $k_c = 0.9$

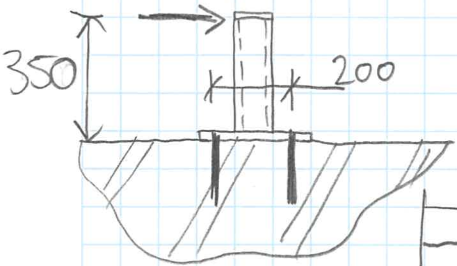
$$F = \left[(0.837 \times 0.7 \times 134) + (0.837 \times 0.5 \times 35)\right] \times 0.9$$

$$= 83.8 \text{ kN}$$

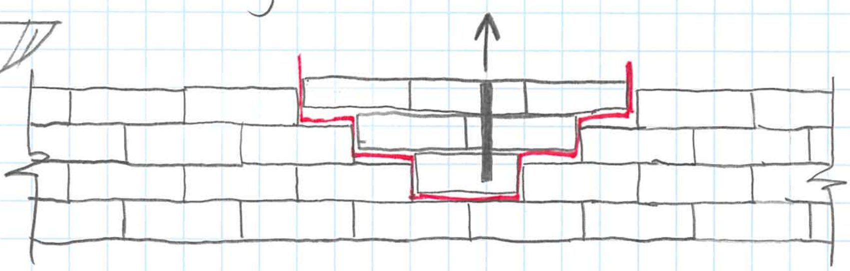
Bracing force will be resisted by a combination of precast panel walls, existing masonry (via cantilever stub columns) and one cantilever column.

cantilever column to resist 10 kN

Masonry wall bracing capacity will be limited by fixing capacity of cantilever stub columns



Adopt M10 threaded rod embeded 250 into masonry wall with Hilti Hit-RE500 adhesive



For fixing to fail a total of 6 bricks must 'shear' free from the wall

↳ Total vertical shear plane height $2 \times (3 \times 86) = 516$ mm

Shear plane width = 110 − 20 = 90 mm

Shear strength of mortar joint 0.35 MPa

↳ Shear Capacity = $\phi \times 0.35 \times$ Area = $0.6 \times 0.35 \times 90 \times 516 = \underline{\underline{9.75 \text{ kN}}}$

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Max moment with 200mm lever arm

↳ $M^*_{max} = 9.75 \times 0.2 \times \frac{1}{1.2}$ ← prying

$= 1.63$ kN

Max horizontal load

↳ $P_{max} = \frac{1.63}{0.3}$

$= 4.66$ kN

3 × Stub columns ⇒ 3× 4.66 = 13.98 kN

Remaining Racking Force = 83.8 − 10 − 13.98

= 59.8 kN

By Inspection long precast wall on northern boundary, panels on southern elevation and panels around lift & stair voids will provide more than enough capacity/resistance to cover the remaining 59.8 kN

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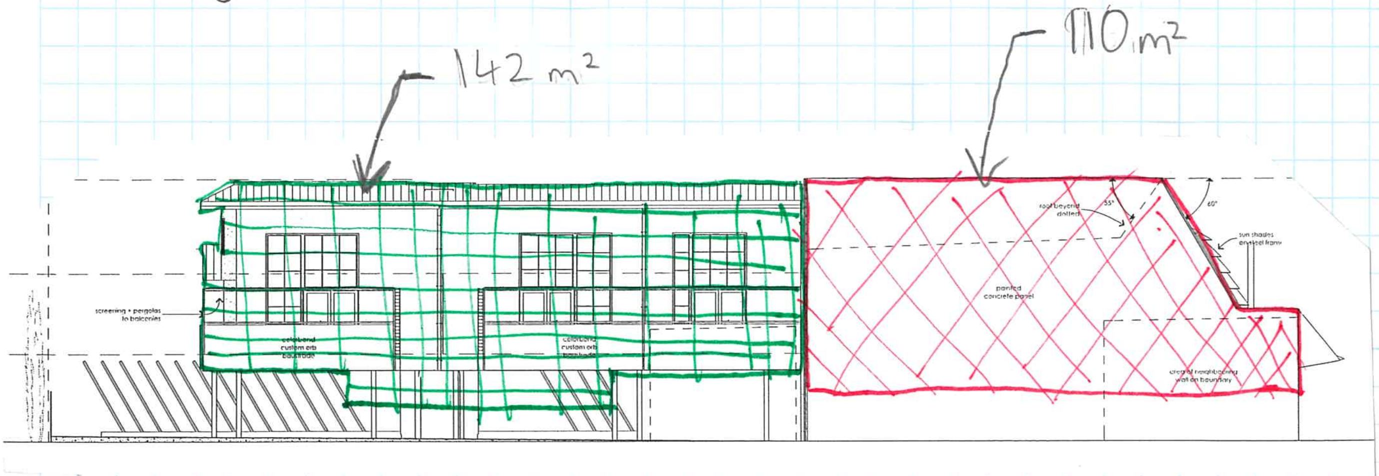
Bracing in North-South Direction – Ground Floor/Carpark



Below Apartments

↳ Area = 142 m²

$$\text{Racking Force} = 0.755 \times (0.7 + 0.5) \times \underset{K_c}{0.8} \times 142$$
$$= 102.9 \text{ kN}$$

6x Cantilevered columns have total capacity 60 kN (10 each)
By inspection precast panels either side of both stairwells
are adequate to resist remaining 42.9 kN

Ground Floor Office → Area = 110 m²

$$\text{Racking Force} = 0.755 \times 1.2 \times 0.8 \times 110$$
$$= 79.7 \text{ kN}$$

By inspection the precast panel walls (rear of office, stairwell, lift shaft) will have enough capacity to resist entire 79.7 kN however the walls are all located within a 4m strip of the building

↳ Provide cantilever column to the rear and sway frame to the front in order to give better spread.

Design Sway Frame and Column for 8 kN each

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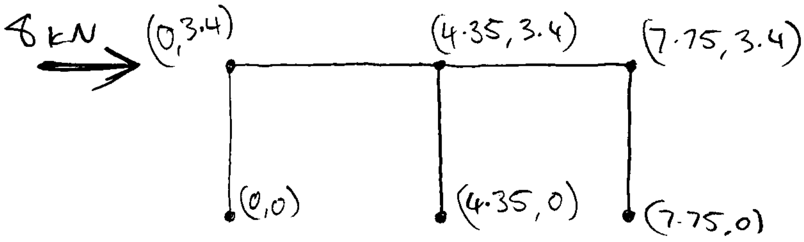


Pinned base connections
125×6.0 SHS columns
250 PFC cross member

Design Sway frame to resist 8 kN (Ultimate Wind Speed)

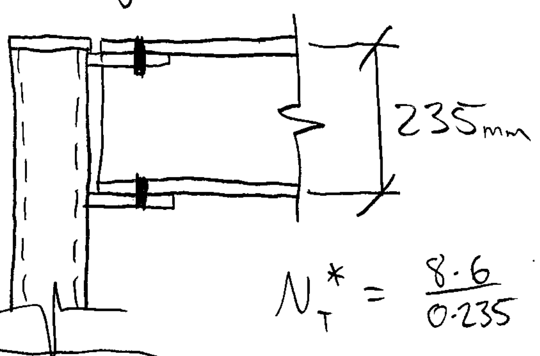
Use computer software Microstran version 9.01
↳ Refer Appendix A

125×6.0 SHS & 250 PFC are ok for strength by inspection!

Deflection/Sway under serviceability wind load
↳ $\Delta_{sway} = 22.2$ mm $\Rightarrow \frac{\text{Height}}{153} \Rightarrow$ Acceptable

deflection will in reality be slightly less than this due to small amount of 'partial fixity' at the base of all 3 columns.

Design Top Connections



$M^* = 8.6$ kNm (from Microstran ↳ refer appendix A for output)

6CFW both sides of cleat (150 total weld length) ok by Inspection

$$N_T^* = \frac{8.6}{0.235} = 36.6 \text{ kN}$$

Adopt 1/M20 bolt top & bottom
↳ Shear capacity $\phi V_{fn} = 92.6$ kN > 36.6 kN

Try 75 wide × 10 thick cleats

$$\phi N_t = \phi\, 0.85 f_u A_n = 0.9 \times 0.85 \times 440 \times 10 \times (75-22) = \underline{178 \text{ kN}}$$

or

$$\phi N_t = \phi f_y A_g = 0.9 \times 300 \times 10 \times 75 = 203 \text{ kN}$$

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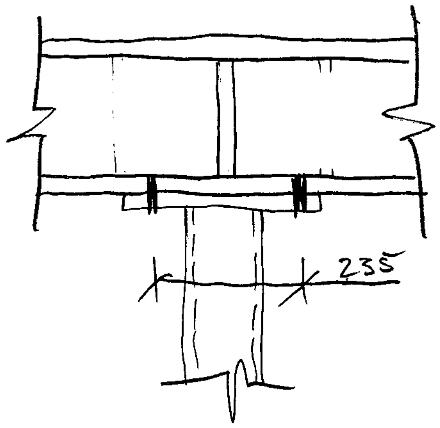
Middle Connection



$M^* = 6.7$ kNm

$N_T^* = \frac{6.7}{0.235} \times 1.2$

$= 34.2$ kN

M20 bolt → $\phi N_{tf} = 163$ kN

Bending in cap plate/cleat

$M_{PL}^* = 34.2 \times 0.055 = 1.88$ kN

Select 20 thick x 90 wide

$\phi M_{PL} = 0.9 \times 300 \frac{90 \times 20^2}{4}$

$= 2.43$ kNm $> M_{PL}^*$ ⇒ ok

Provide 10 thick stiffener inside PFC → bottom flange strengthened/stiffened by both stiffener & web

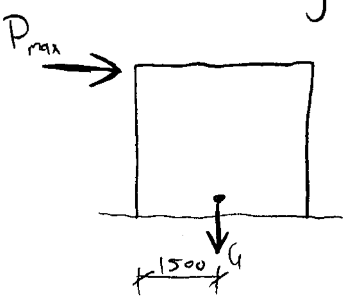
↳ OK by Inspection!

$79.7 - 8 - 8 = 63.7$ kN

↳ remaining racking force

Precast panel around lift void & stairs as well as new precast panel wall along eastern side of offices will provide more than enough capacity → Panel PPS alone has 17.3 kN capacity (see below)

PPS 3.2 high x 3.0 wide



Restoring moment from overturning based on self weight only

Panel weight $= 3.2 \times 0.15 \times 3 \times 24$
$= 34.6$ kN

$M_{MAX} = 34.6 \times 1.5$
$= 51.9$ kNm

Max Horizontal Force

↳ $P_{max} = \frac{51.9}{3} = \underline{17.3 \text{ kN}}$

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Earthquake Design Loading

Importance Level II Select 1 in 500 annual prob. of exceedance

$k_p = 1.0$

$Z = 0.1$ (Hazard factor for Adelaide - Table 3.2 AS1170.4)

Sub-soil class $\boxed{D_e}$

Must Design building for $\boxed{\text{EDC II}}$ (Table 2.1 AS1170.4)

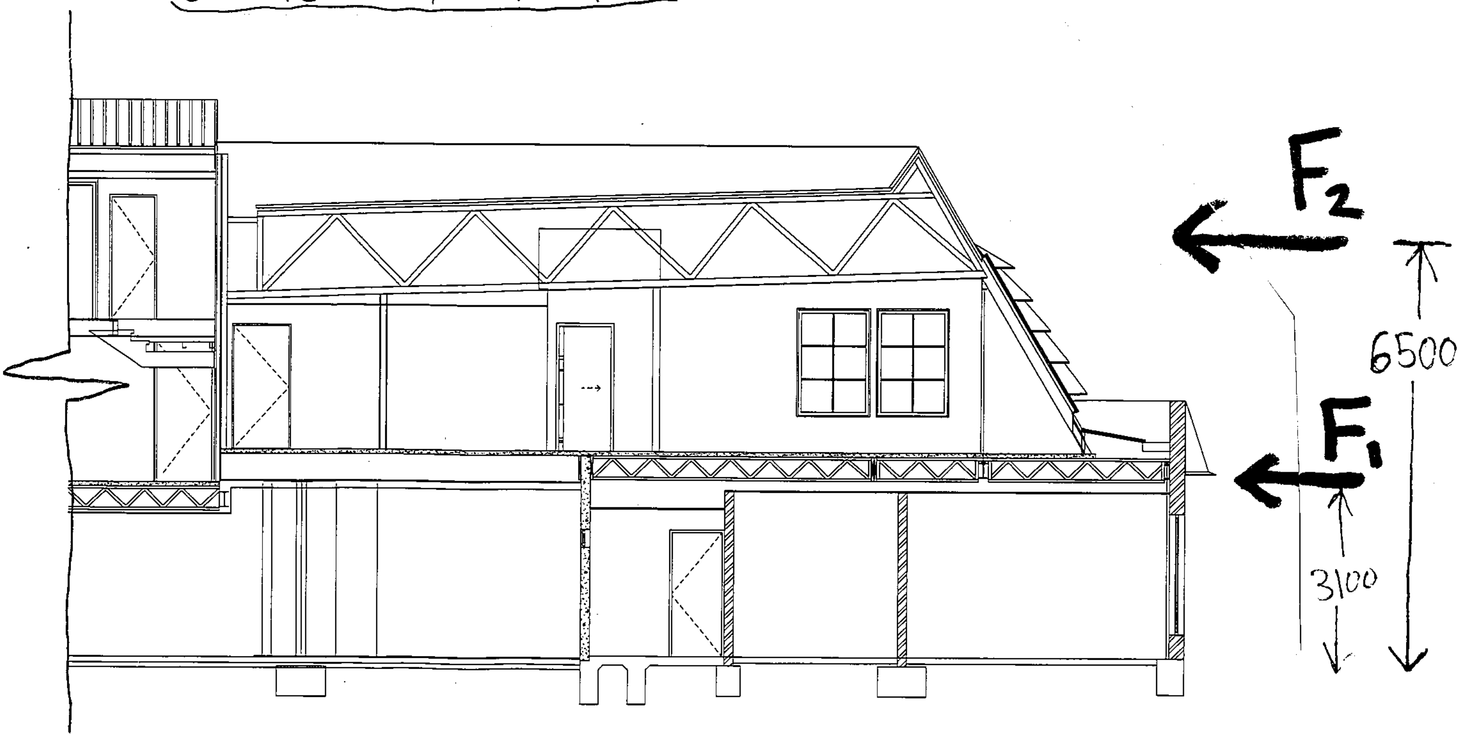
Building height less than 15m tall → determine earthquake forces in accordance with clause 5.4.2.3 AS1170.4 (Simplified Method).

Determine Earthquake design force (F_i) acting at each level.

Where $F_i = k_s \frac{k_p Z S_p}{\mu} W_i$

Check / Design office tenancies & apartments seperately

OFFICE TENANCIES



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Re-design of Earthquake loading for office Area (western part)

Seismic Weights

Roof/ceiling Level $\boxed{W_2}$

245 m² Timber trussed sheet roof
20 m internal non-LBW (timber frame) → consider top 1.5 m
8.5 m external precast Southern side → top 2.4 m
21 m external walls (South & West) → top 2.0 m
16.5 double Stud party wall (east) → top 4.8 m
17.5 external precast Northern Side → top 4.8 m
8.0 m internal precast wall (lift) → top 3.0 m

$$G = (245 \times 0.4) + (20 \times 1.5 \times 0.4) + (8.5 \times 2.4 \times 0.15 \times 25) + (21 \times 2 \times 1) + (16.5 \times 4.8 \times 1) + (17.5 \times 4.8 \times 0.15 \times 25) + (8 \times 3 \times 0.15 \times 25)$$
$$= 712.7 \text{ kN}$$

$$Q = (245 \times 0.25)$$
$$= 61.3 \text{ kN}$$

$$\Rightarrow W_2 = 712.7 + 0.3(61.3) = \underline{731 \text{ kN}}$$

First Floor Level $\boxed{W_1}$

195 m² of Timber trussed, hebel floor
60 m² Canopy roof
20 m upper level internal walls (bottom 1.5 m)
8.5 m precast wall Southern side → 3.2 m
16.5 m party wall eastern side → 2.0 m
17.5 m external precast Northern side → 3.5 m
8.0 m preast lift panels → 3.5 m
25 m internal lower level panels → 2.0 m

Existing ceiling must remain to provide restraint/diaphragm to top of existing masonry walls

↳ by inspection masonry walls will provide enough bracing capacity to 'brace' themselves

* Do not include existing masonry walls in Seismic weight calcs

$$G = (195 \times 1.0) + (60 \times 0.4) + (20 \times 1.5 \times 0.4) + (8.5 \times 3.2 \times 0.15 \times 25) + (16.5 \times 2 \times 1) + (17.5 \times 3.5 \times 0.15 \times 25) + (8 \times 3.5 \times 0.15 \times 25) + (25 \times 2 \times 0.15 \times 25)$$
$$= 888.2 \text{ kN}$$

$$Q = (195 \times 3) + (60 \times 0.25)$$
$$= 600 \text{ kN}$$

$$\Rightarrow W_2 = 888.2 + 0.3(600)$$
$$= 1068 \text{ kN}$$

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Design Forces (F_i) $\Rightarrow$ $F_i = k_s \dfrac{k_p Z S_p}{\mu} W_i$

Roof/ceiling Level $\boxed{F_2}$ $k_s = 4.9$ $k_p = 1.0$ $Z = 0.1$ $\mu = 2$ $S_p = 0.77$

$$F_2 = 4.9 \frac{1 \times 0.1 \times 0.77}{2} \times 731$$

$$= \underline{138 \text{ kN}}$$

First Floor Level $\boxed{F_1}$ As above but $k_s = 2.5$

$$F_1 = 2.5 \frac{1 \times 0.1 \times 0.77}{2} \times 1068$$

$$= \underline{103 \text{ kN}}$$

Upper Level Must have minimum 138 kN capacity in both directions
Lower Level must have minimum 241 kN capacity in both directions

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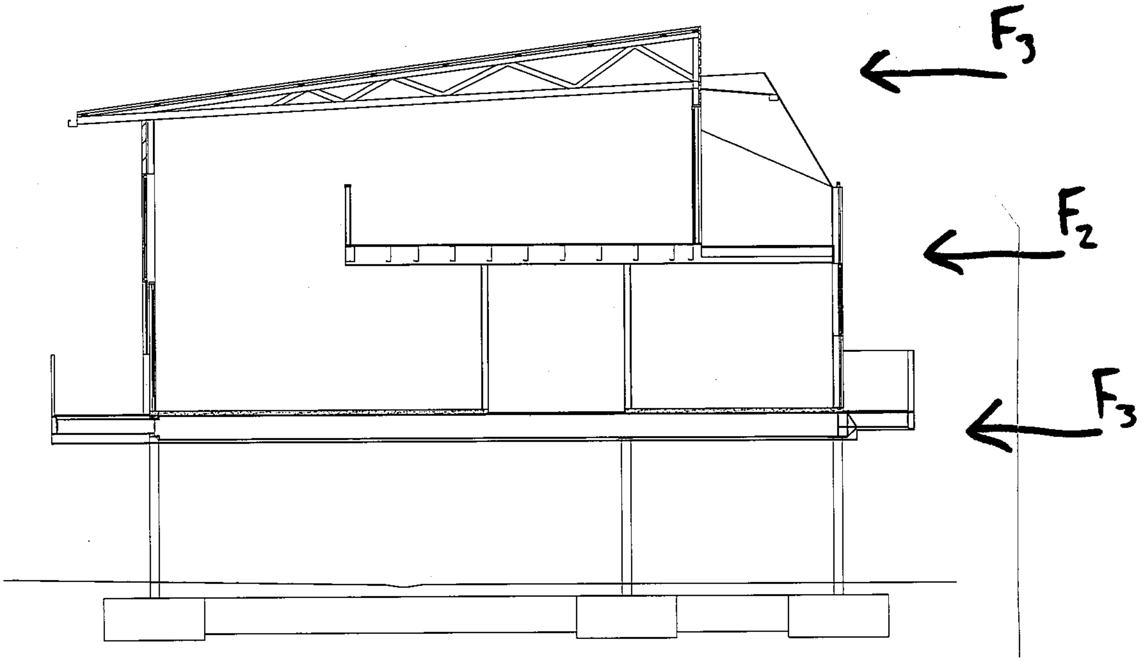
Apartments

↳ Residental Structure but top of roof is > 8.5m

Must Design as $\boxed{\text{EDC II}}$



Seismic Weights

$\boxed{W_3}$

28m² of precast panel
218m² of Timber trussed sheet roof
46.2m of external wall - 1.4
17m of double stud party wall - 1.4
15m of internal timber non-LBW wall - 1.3m

$G = (218 \times 0.4) + (46.2 \times 1.4 \times 1.0) + (17 \times 1.4 \times 1.0) + (15 \times 1.3 \times 0.4) + (28 \times 0.15 \times 25)$

$= 288$ kN

$Q = (218 \times 0.25)$

$= 54.5$ kN

$W_3 = 304$ kN

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$\boxed{W_2}$

36 m^2 of precast panel
95 m^2 Mez floor (timber truss with Structaflor)
40 m^2 balcony floor timber joists tiled floor
17 m double stud party wall - 2.6 m
48 m external wall - 2.6 m
15 m internal non-LBW walls above - 1.3 m
35 m internal walls below - 1.2 m
13 m balustrade to Mez floor - 1.0 m
20 m balustrade to balcony - 1.0 m

$$G = (36 \times 0.15 \times 24) + (95 \times 0.5) + (40 \times 0.8) + (17 \times 2.6 \times 1.0) + (48 \times 2.6 \times 1.0) + (15 \times 1.3 \times 0.4) + (35 \times 1.2 \times 0.4) + (33 \times 1.0 \times 0.8)$$

$$= 429 \text{ kN}$$

$$Q = (95 \times 1.5) + (40 \times 2.0)$$

$$= 223 \text{ kN}$$

$$W_2 = 496 \text{ kN}$$

$\boxed{W_1}$

65 m^2 of precast panel
192 m^2 of first floor (timber truss Hebel floor) - Take 1.1 kPa
65 m^2 of balcony
17 m double stud party wall (1.3)
48 m external wall - 1.3 m
35 m internal non-LBW - 1.2 m
50 m of balcony balustrade

allow for Steelwork

$$G = (65 \times 0.15 \times 25) + (192 \times 1.1) + (65 \times 0.8) + (17 \times 1.3 \times 1.0) + (48 \times 1.3 \times 1.0) + (35 \times 1.2 \times 0.4) + (50 \times 1.0 \times 0.8)$$

$$= 648 \text{ kN}$$

$$Q = (192 \times 1.5) + (65 \times 2)$$

$$= 418 \text{ kN}$$

$$W_1 = 773 \text{ kN}$$

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Design Loads

F_3 $k_s = 5.5$ $\mu = 2.0$ $S_p = 0.77$

$$F_3 = 5.5 \frac{0.1 \times 0.77}{2} \times 304$$
$$= 64.4 \text{ kN}$$

F_2 $k_s = 3.6$ $\mu = 2.0$ $S_p = 0.77$

$$F_2 = 3.6 \times \frac{0.1 \times 0.77}{2} \times 496$$
$$= 68.7 \text{ kN}$$

F_1 $k_s = 1.8$ $\mu = 2.0$ $S_p = 0.77$

$$F_1 = 1.8 \frac{0.1 \times 0.77}{2} \times 773$$
$$= 53.6 \text{ kN}$$

Upper Level Requires: min 64.4 kN in both directions
Middle level " " 133.1 kN in both directions
Lower Level " " 186.7 kN in both directions

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Earthquake bracing for office

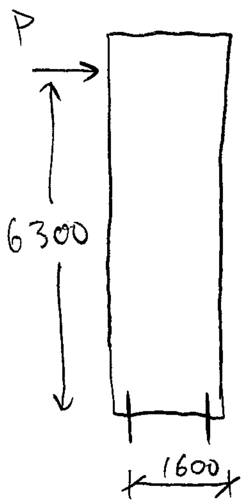
North South

Upper level must have 138 kN capacity

Adopt 2.7 + 1.4 lengths of hardboard brace (or similar) rated at minimum 6.0 kN/m (for 2700 walls)

↳ Capacity $= \frac{2.7}{2.9} \times 6 \times 4.1 = 22.9$ kN

Precast Panels either side of lift shaft (6500 high × 1900 wide)



Panel S.W $= 6.5 \times 1.9 \times 0.15 \times 24$
$= 44.5$ kN

Consider capacity of dowel fixing 20 kN (conservative)

Total resisting moment

$$M_R = 0.9\left(44.5 \times \frac{1.9}{2}\right) + \left(20 \times 1.6\right)$$
$$= 70.0 \text{ kNm}$$

$P_{max} = \frac{70}{6.3} = 11.1$ kN per panel

$22.9 + 2(11.1) = 45.1$ kN

Require further 92.9 kN

Hardboard brace entire length of back wall & Kitchen wall (13.6 + 2.1)

↳ 15.7 m ⇒ $\frac{2.7}{2.9} \times 6 \times 15.7 = 87.7$ kN

Still 5.2 kN remaining → by inspection panel PP12 will take it (also panel PP19 is wider at base so will have more capacity than calculated above)

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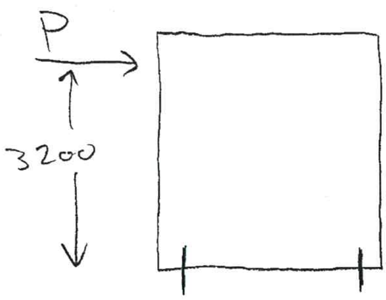
Lower Level must have min 241 kN Capacity

Stairwell Panels 11.1 kN each & Cantilever Column 10 kN

Precast wall seperating office from carpark

↳ 4 x panels 2950 wide



Panel ⇒ 3.3 x 2.95

$S.W = 3.3 \times 2.95 \times 0.15 \times 24$
$= 35.0$ kN

dowel fixing at base 200mm in from panel edges

N20 dowel bar fixed with Hilti Hit-RE 500 adhesive

↳ Take pull out capacity of 25 kN

Maximum moment at base of panel

↳ $M_b = 0.9\left(35 \times \frac{2.95}{2}\right) + (25 \times 2.75)$
$= 115.2$ kNm

Bracing capacity of each panel

↳ $P = \frac{115.2}{3.2} = 36.0$ kN

3 x panels ⇒ 144 kN capacity

Sway frame   column

Achieved Capacity = $(2 \times 11.1) + 144 + 8 + 10$
$= 184.2$ kN

Still require a further 56.8 kN by observation/comparison to above calcs remaining precast panels (PP6, PP22, PP20) will be adequate

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East – West

Upper level – 138 kN required

Adopt 4.2 m long hardboard or similar brace

↳ $4.2 \times 6 \times \frac{2.7}{2.9} = 23.5$ kN

By inspection/comparison to previous calcs precast panels on both northern & southern walls as well as front & rear walls of lift shaft will be more than adequate to resist the remaining 114.5 kN

Lower Level – 241 kN required

Combination of precast panels & cantilever column will provide required capacity

↳ ok by inspection/comparison to previous design calcs.

Earthquake Bracing Below Apartments

Required Capacity → 186.7 kN (both directions)

6 × Cantilevered 150×8.0 SHS Columns

Timber framed apartments to be braced by timber estimator
↳ ensure min capacity of all 3 apartments meet min requirements for earthquake
(64.4 kN upper
133.1 kN lower)

Each column to resist 25 kN

$M^* = 25 \times 2.65 = 66.3$ kNm

→ $\phi M_s = 91.5$ kNm $\phi N_c = 1572$ kN

Columns resist earthquake in both directions → consider 100% & 30%

$M_x = 66.3$ kNm
$M_y = 19.9$ kNm

Worst case column under FB11 (axial load)

↳ $P_{DL} = 98.7$ kN
↳ $P_{LL} = 71.8$ kN

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Check column for strength under combined load case $\boxed{G + E_u + \psi_c Q}$ $\psi_c = 0.4$

$$N_c^* = 98.7 + (0.4 \times 71.8)$$
$$= 127 \text{ kN}$$

$$M_x^* = 66.3 + (0.075 \times 127)$$
$$= 75.8 \text{ kNm}$$

$$M_y^* = 19.9 + (0.075 \times 127)$$
$$= 29.4 \text{ kNm}$$

$$\phi M_i = \phi M_o = 91.5\left(1 - \frac{127}{1572}\right) = 84.1 \text{ kNm}$$

Biaxial Bending Check

$$\left(\frac{75.8}{84.1}\right)^{1.4} + \left(\frac{29.4}{84.1}\right)^{1.4} = 1.09 \Rightarrow \text{Not Acceptable} > 1.0$$

reduce to 23.0 kN per column

Revised Moments

$$M_x^* = 70.5 \text{ kNm}$$

$$M_y^* = 27.8 \text{ kNm}$$

$$\left(\frac{70.5}{84.1}\right)^{1.4} + \left(\frac{27.8}{84.1}\right)^{1.4} = 0.993 \Rightarrow \text{less than } 1.0 \Rightarrow \underline{\text{ok!!}}$$

6 × columns each resist 23 kN ⇒ 138 kN

Require a further 48.7 kN capacity

$\boxed{\text{East-West}}$

2 × Short panels below stairs to resist 24.5 kN each

↳ Combination of side walls & concrete stairs to transfer force down to top of panels.

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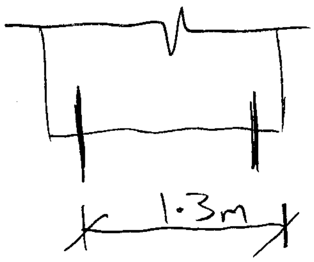
Moment at panel base

↳ $M_b^* = 24.5 \times 2.1 = 51.5$ kN



restoring moment (ignoring S.W → conservative)

Tensile Uplift required $= \frac{51.5}{1.3} = 39.6$ kN

By inspection N20 dowel bar adequate to resist required uplift!!

North-South

Remaining 48.7 kN resisted by panels → ok by inspection!!

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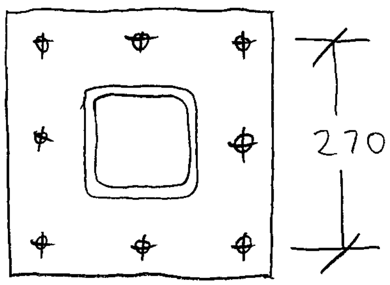
Base Connection of Cantilever Columns $\boxed{C1}$

Try 350 square × 20 thick base plate with 8 bolts



Critical case → central column supporting FB11

↳ Design case ① 1.2DL + 1.5LL + 1.5 Impact

$N_1^* = 226$ kN

$V_1^* = 91.2$ kN

$M_{x_1}^* = 16.95$ kNm

$M_{y_1}^* = 61.8$ kNm

↳ Design Case ② DL + E_u + $\psi_c Q$

$N_2^* = 127$ kN

→ $V_2^* = 23$ kN

↑ $V_2^* = 23$ kN

$M_{x_2}^* = 70.5$ kNm

$M_{y_2}^* = 27.8$ kNm

Case ② critical

M_x → $N_T^* = \frac{70.5}{0.27} \times 1.2 = 313$ kN → 3 bolts 104 kN per bolt

M_y → $N_T^* = \frac{27.8}{0.27} \times 1.2 = 124$ kN → 2 bolts 112 kN per Bolt

Select M24 4.6 grade hold down bolts

↳ $\phi N_{tf} = 113$ kN ⇒ ok!!

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Total Tensile pull out load over the 5 bolts (ignore axial load on column)
↳ 313 + 124 = 437 kN

Check concrete pull out → Select 400mm embed

$A_{ps} = 400^2 \times \pi = 502,655\ mm^2$ (Conservative as only allowed for one bolt)

$f'_c = 20MPa$

$\phi N_{cc} = 0.8 \times 0.33 \times \sqrt{20} \times 502655 \times 10^{-3}$

$= 593\ kN > 437 \Rightarrow$ ok!!

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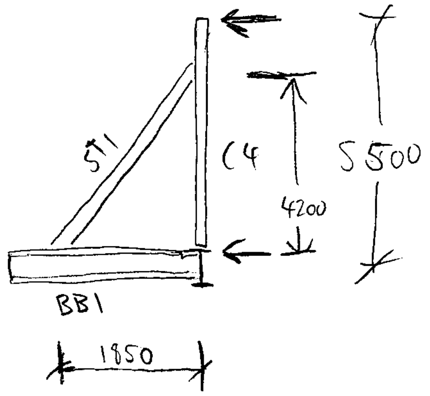
Column C4 Supporting Lintel L1



Column height = 5.5 m

Supporting Balcony via 89x3.5 SHS strut

Point load @ free end from 1.8 m² of both balcony floor & canopy above
Also 1.6 m² of balustrade

$P_{DL} = (1.8 \times 0.8) + (1.8 \times 0.4) + (1.6 \times 0.5)$
$= 2.96$ kN

$P_{LL} = (1.8 \times 2) + (1.8 \times 0.25)$
$= 4.05$ kN

UDL on BB1 from wall above $\rightarrow$ $w_{DL} = 4.5 \times 0.5 = 2.25$ kN/m

Vertical load supported by ST1

↳ $P_{DL} = 2.96 + (1.075 \times 2.25)$
$= 5.38$ kN

$P_{LL} = 4.05$ kN

Tension in strut — Strut length $= \sqrt{4200^2 + 1850^2} = 4589$ mm

$T_{DL} = 5.38 \dfrac{4.589}{4.2}$
$= 5.88$ kN

$N_T^* = 13.7$ kN

$T_{LL} = 4.05 \dfrac{4.589}{4.2}$
$= 4.43$ kN

↳ 89x3.5 SHS ok for strut by inspection!!

Horizontal component acting on column C4

$P_{DL} = 5.88 \dfrac{1.85}{4.589} = 2.37$ kN

$P_{LL} = 4.43 \dfrac{1.85}{4.589} = 1.79$ kN

$P^*_{\rightarrow} = 5.53$ kN

$M^* = \dfrac{5.53 \times 1.3 \times 4.2}{5.5} = 5.49$ kNm

Limit Deflection under DL to maximum 8mm

$I_x \geq \dfrac{2370 \times 4200 \times 1300 \times 6800 \sqrt{3 \times 4200 \times 6800}}{27 \times 200 \times 10^3 \times 8 \times 5500}$

$\geq 3.43 \times 10^6$ mm⁴

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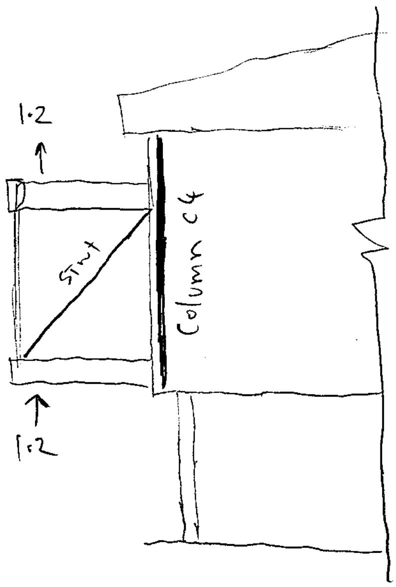
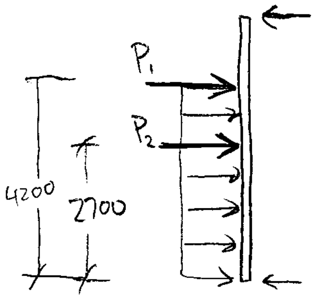
Select 2/89 x 6.0 SHS welded together!!

↳ $I_{x_{combined}} = 4.12 \times 10^6 \, mm^4$



Check deflection from northerly wind



P_1 from Strut & Window Trimmer
↳ 1.0 m² of wall

↳ take $C_{p,n} = 1.2$ over both balcony & canopy → conservative

Wind up $= (1.8 + 1.8) \times 0.755 \times 1.2$
$= 3.26$ kN

minus half dead load (conservative

$W_{\uparrow} = 3.26 - \frac{1}{2}(5.38)$
$= 0.57$ kN

$P_1 = 1.85 \frac{0.57}{4.2} + 1.0 \times (0.7 + 0.3) \times 0.9 \times 0.755)$

$= 0.93$ kN

P_2 from Wind beam WB1

↳ worst case wall Area 3.0 m²

$P_2 = 3.0 \times 1.0 \times 0.9 \times 0.755$
$= 2.04$ kN

UDL → 1.8m load width

$W_{WL} = 1.8 \times 1.0 \times 0.9 \times 0.755$
$= 1.22$ kN/m

$P_{1\,serv} = \frac{37^2}{45^2} \times 1.85 \frac{3.26}{4.2} = 0.971$ kN

$P_{2\,serv} = \frac{37^2}{45^2} \times 2.04 = 1.38$ kN

$W_{serv} = \frac{37^2}{45^2} \times 1.22 = 0.825$ kN/m

Enter into design spreadsheet (refer next page)

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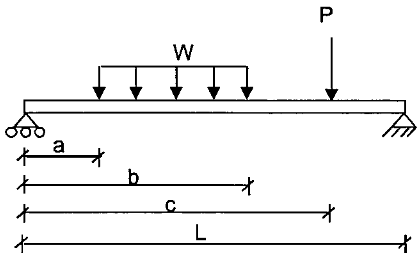
Sheet : S65

Job No : **17799**

Mullion / Column C4 Design

Height = 5.5 m



E	2.00E+008	kPa
Ix	4.12	X10^6 mm4
EI	8.24E+02	kN.m2

Ultimate Wind Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	1.22	0	0	0	0
a_i	0	0	0	0	0
b_i	4.2	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	2.04	0.93	0	0	0
c_i	2.7	4.2	0	0	0

Serviceability Wind Load

Distrubuted Load (W)

	1	**2**	**3**	**4**	**5**
W_i	0.825	0	0	0	0
a_i	0	0	0	0	0
b_i	4.2	0	0	0	0

	1	**2**	**3**	**4**	**5**
P_i	1.38	0.971	0	0	0
c_i	2.7	4.2	0	0	0

M^* = 7.5 kNm

$R1_{ULT}$ = 4.4 kN $R2_{ULT}$ = 3.7 kN

$R1_{serv}$ = 3.1 kN $R2_{serv}$ = 2.7 kN

DEFLECTION (serviceability) = 18.84 mm < 20mm → Acceptable

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Footing Design

Soil Classification : H1-D

$y_s = 58\,mm$

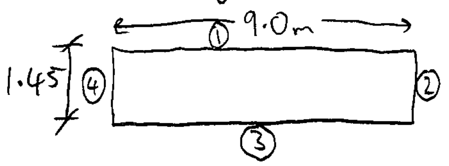
Building Construction

- 2 Storey
- Sheet roof
- Timber Framed hebel floor
- Wall construction → Part Precast Panel, part lightweight

Use Computer Software program CORD

3 Rectangles (refer next page)

Rectangle 1



1 beam in long direction
5 beams in short direction

Live Load 3.0 kPa ⇒ UDL for footing design 1.5 kPa (0.5 Q)

Side 1

8.6m high 150 thick precast panel wall + 600mm RLW

$W_1 = (8.6 \times 0.15 \times 25) + (0.6 \times 0.4) + \frac{1}{2}(0.6 \times 0.25)$

$= 32.6\ kN/m$

Side 2

3.3m high 150 thick precast panel wall + 6.2m FLW

$W_2 = (3.3 \times 0.15 \times 25) + (6.2 \times 1.0) + \frac{1}{2}(6.2 \times 3)$

$= 27.9\ kN/m$

Side 3 → Nil

Side 4 Lightweight wall only → $W_4 = 1.0\ kN/m$

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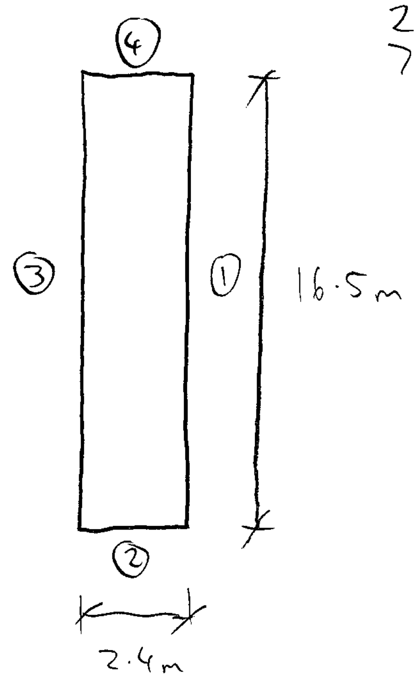
Rectangle 2



2 × Long Beams
7 × Short beams

Side 1 (Same as side 2 on rectangle 1)

$W_{1} = 27.9$ kN/m

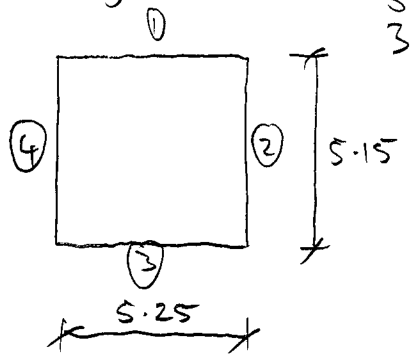
Side 2 – Nil

Side 3 – Nil

Side 4 (Same as side 1 on rectangle 1)

$W_{4} = 32.6$ kN/m

Rectangle 3



5 × Long beams
3 × Short beams

Side 1

3.4m high panel wall

$W_{1} = (3.4 \times 0.15 \times 25)$
$= 12.8$ kN/m

Side 2

7.2m high panels + 2m FLW + 2m RLW

$W_{2} = (7.2 \times 0.15 \times 25) + [2 \times (1.0 + 0.4)] + \frac{1}{2}[2 \times (3 + 0.25)]$

$= 33.1$ kN/m

Side 3 7.0m high panel + 2.5 FLW + 2 RLW

$W_{3} = (7 \times 0.15 \times 25) + (2.5 \times 1.0) + \frac{1}{2}(2.5 \times 3) + (2 \times 0.4) + \frac{1}{2}(2 \times 0.25) = 33.6$ kN/m

Side 4

7.2m high panel + 5m RLW + 1.8m FLW

$W_{4} = (7.2 \times 0.15 \times 25) + (5 \times 0.4) + (1.8 \times 1.0) + \frac{1}{2}(5 \times 0.25) + \frac{1}{2}(1.8 \times 3)$

$= 34.1$ kN/m

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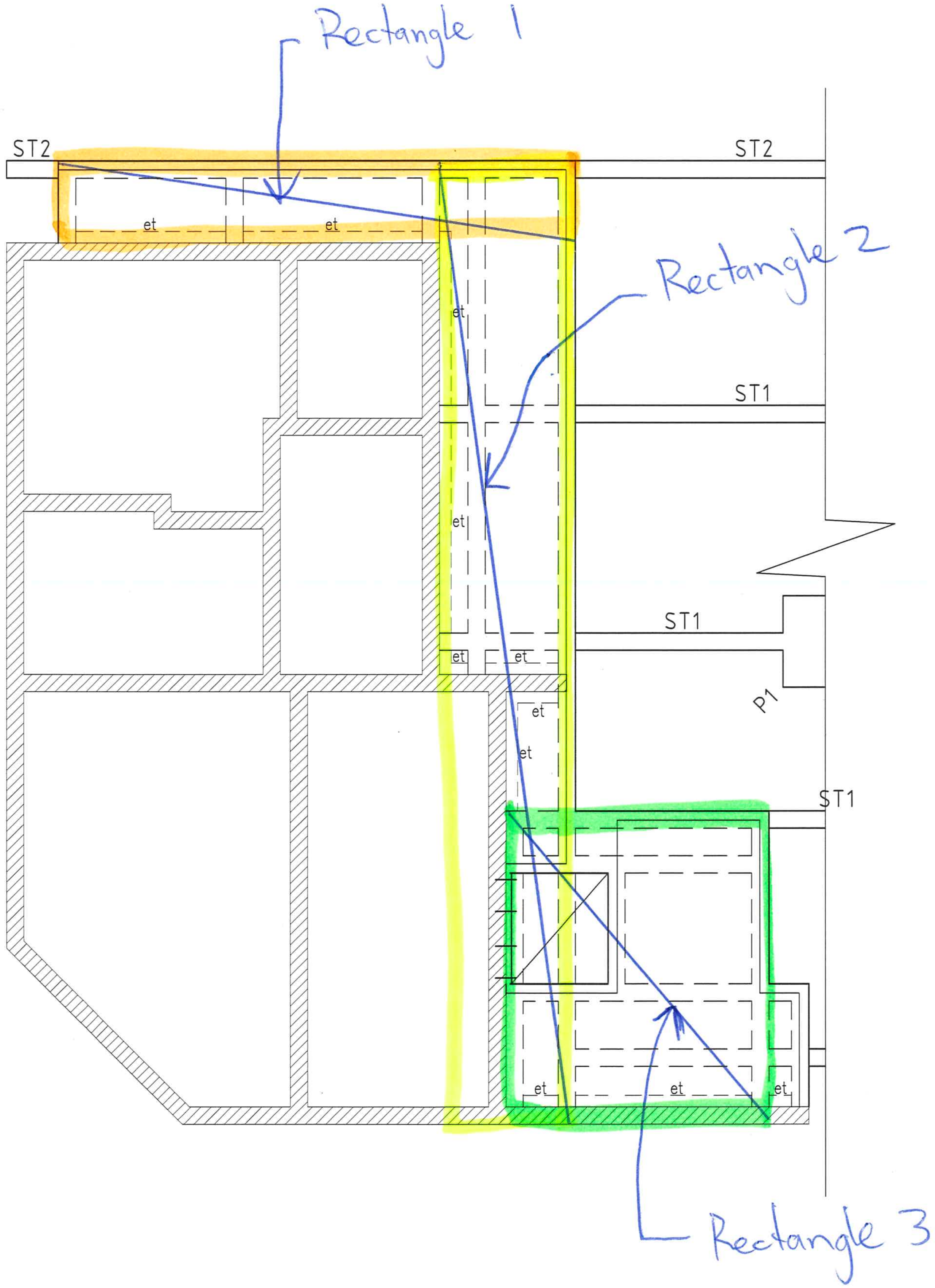
Centre Line Load (Direction 2) due to lift shaft/stair well panels

$W_{centre} = (7.2 \times 0.15 \times 25)$

$= 27$ kN/m

Adopt 300 x 800 beams with 3/N16 bars top & bottom. 125 thick slab with SL82 mesh top (20 cover).

S70



Rectangle 1
ST2
ST2
et
et
Rectangle 2
et
ST1
et
ST1
et
et
P1
et
et
ST1
et
et
et
Rectangle 3

Licensed User: RCI Consulting Engineers
Job Number: 17799 Date: 17/02/2017 (16:43)

Code Oriented Raft Design (Version 8.0)
****

FOOTING DESIGN TO AS2870 - 2011
-- Raft Footing --

****
RECTANGLE 1 of 3 (9m x 1.5m)

THE FOLLOWING VALUES WILL BE USED:

Deflection Ratio ....= 1 / 400
E conc. long term (max) = 15.48 GPa
Hs ....= 4 m
Footing design modified for tree effects - No

Ys ....= 58 mm

Ym (centre) --> 0.7Ys ..= 41 mm

Ym (edge) --> 0.5Ys ....= 29 mm

S72


```
                           Side 1
                  -------------------------
                  |                       |
                  |          |            |
                  | (dir 1)  |            |
            Side 4|  <-------+--------    |  Side 2
                  |          |            |
                  |          v (dir 2)    |
                  |                       |
                  -------------------------
                           Side 3
***********************************************************************
LOAD CALCULATION (Note: Footing self-weight is generated automatically)
-----------------------------------------------------------------------
  Design edge load (kN/m)
    Side  1 ...................................................= 32.60 kN/m
    Side  2 ...................................................= 27.90 kN/m
    Side  3 ...................................................= 0.00 kN/m
    Side  4 ...................................................= 1.00 kN/m
  Footing self weight:-
    Direction 1 (0.675 x 0.3 x 24).............................= 4.86 kN/m
    Direction 2 (0.675 x 0.3 x 24).............................= 4.86 kN/m
- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
  PE (Side 1) .................................................= 37.46 kN/m
  PE (Side 2) .................................................= 32.76 kN/m
  PE (Side 3) .................................................= 0.00 kN/m
  PE (Side 4) .................................................= 5.86 kN/m
-----------------------------------------------------------------------
Distributed internal load W (kPa)
  Design UDL ..................................................= 1.5 kPa
  Slab self weight  (0.125 x 24) ..............................= 3.00 kPa
  Footing self weight:-
      Direction 1       ((0 x 0.3 x (0.8 - 0.125) x 24) / 1.5)..= 0.00 kPa
      Direction 2       ((3 x 0.3 x (0.8 - 0.125) x 24) / 9)....= 1.62 kPa
  Sub Total (Omega) ...........................................= 6.12 kPa
Longitudinal edge loads

  Direction 1       ((37.46 + 0.00) / 1.5).....................= 24.97 kPa

  Direction 2       ((32.76 + 5.86) / 9).......................= 4.29 kPa

- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -

  W (Direction 1) .............................................= 31.09 kPa

  W (Direction 2) .............................................= 10.41 kPa

-----------------------------------------------------------------------

Total distributed load Q (kPa)

  Omega .......................................................= 6.12 kPa

  Line loads

    Direction 1       ((37.46 + 0.00 + 0.00) x 9) / (9 x 1.5) ..= 24.97 kPa

    Direction 2       ((32.76 + 0.00 + 5.86) x 1.5) / (1.5 x 9) = 4.29 kPa

- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -

  Q ...........................................................= 35.38 kPa

-----------------------------------------------------------------------
```

S73

****

RECTANGLE 2 of 3 (16.5m x 2.4m)

THE FOLLOWING VALUES WILL BE USED:

Deflection Ratio ....= 1 / 400
E conc. long term (max) = 15.48 GPa
Hs ....= 4 m
Footing design modified for tree effects - No

Ys ....= 58 mm

Ym (centre) --> 0.7Ys ..= 41 mm

Ym (edge) --> 0.5Ys ....= 29 mm

S74


```
                          Side 1
                 -------------------------
                |                         |
                |             |           |
                |  (dir 1)    |           |
        Side 4  |     <-------+--------   |  Side 2
                |             |           |
                |             v (dir 2)   |
                |                         |
                 -------------------------
                          Side 3
```

****

LOAD CALCULATION (Note: Footing self-weight is generated automatically)

Design edge load (kN/m)
- Side 1 ....= 27.90 kN/m
- Side 2 ....= 0.00 kN/m
- Side 3 ....= 0.00 kN/m
- Side 4 ....= 32.60 kN/m

Footing self weight:-
- Direction 1 (0.675 x 0.3 x 24)..........= 4.86 kN/m
- Direction 2 (0.675 x 0.3 x 24)..........= 4.86 kN/m

- - - - - - - - - - - - - - - - - - - -

- PE (Side 1) ....= 32.76 kN/m
- PE (Side 2) ....= 4.86 kN/m
- PE (Side 3) ....= 4.86 kN/m
- PE (Side 4) ....= 37.46 kN/m

Distributed internal load W (kPa)
- Design UDL ....= 1.5 kPa
- Slab self weight (0.125 x 24) ....= 3.00 kPa
- Footing self weight:-
  - Direction 1 ((0 x 0.3 x (0.8 - 0.125) x 24) / 2.4)......= 0.00 kPa
  - Direction 2 ((5 x 0.3 x (0.8 - 0.125) x 24) / 16.5).....= 1.47 kPa
- Sub Total (Omega) ....= 5.97 kPa

Longitudinal edge loads

- Direction 1 ((32.76 + 4.86) / 2.4)..........= 15.68 kPa
- Direction 2 ((4.86 + 37.46) / 16.5)..........= 2.56 kPa

- - - - - - - - - - - - - - - - - - - -

- W (Direction 1) ....= 21.65 kPa
- W (Direction 2) ....= 8.54 kPa

Total distributed load Q (kPa)

Omega ....= 5.97 kPa

Line loads

- Direction 1 ((32.76 + 0.00 + 4.86) x 16.5) / (16.5 x 2.4) = 15.68 kPa
- Direction 2 ((4.86 + 0.00 + 37.46) x 2.4) / (2.4 x 16.5) .= 2.56 kPa

- - - - - - - - - - - - - - - - - - - -

Q ....= 24.21 kPa

S75

****

RECTANGLE 3 of 3 (5.25m x 5.15m)

THE FOLLOWING VALUES WILL BE USED:

Deflection Ratio ....= 1 / 400
E conc. long term (max) = 15.48 GPa
Hs ....= 4 m
Footing design modified for tree effects - No

Ys ....= 58 mm

Ym (centre) --> 0.7Ys ..= 41 mm

Ym (edge) --> 0.5Ys ....= 29 mm

S76


```
                                   Side 1
                          -------------------------
                         |                         |
                         |           |             |
                         | (dir 1)   |             |
             Side 4      |   <-------+--------     |   Side 2
                         |           |             |
                         |           v (dir 2)     |
                         |                         |
                          -------------------------
                                   Side 3
***********************************************************************
LOAD CALCULATION (Note: Footing self-weight is generated automatically)
-----------------------------------------------------------------------
  Design edge load (kN/m)
    Side  1 ...........................................................= 12.80 kN,
    Side  2 ...........................................................= 33.10 kN,
    Side  3 ...........................................................= 33.60 kN,
    Side  4 ...........................................................= 34.10 kN,
  Design centre load (kN/m)

    Dir  1 ............................................................= 0.00 kN/r

    Dir  2 ............................................................= 27.00 kN,

  Footing self weight:-
    Direction 1 (0.675 x 0.3 x 24).....................................= 4.86 kN/r
    Direction 2 (0.675 x 0.3 x 24).....................................= 4.86 kN/r
- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
  PE (Side 1) .........................................................= 17.66 kN,
  PE (Side 2) .........................................................= 37.96 kN,
  PE (Side 3) .........................................................= 38.46 kN,
  PE (Side 4) .........................................................= 38.96 kN,
-----------------------------------------------------------------------
Distributed internal load W (kPa)
  Design UDL ..........................................................= 1.5 kPa
  Slab self weight  (0.125 x 24) ......................................= 3.00 kPa
  Footing self weight:-
      Direction 1      ((3 x 0.3 x (0.8 - 0.125) x 24) / 5.15).........= 2.83 kPa
      Direction 2      ((1 x 0.3 x (0.8 - 0.125) x 24) / 5.25).........= 0.93 kPa
  Sub Total (Omega) ...................................................= 8.26 kPa
Longitudinal edge loads

  Direction 1      ((17.66 + 38.46) / 5.15)............................= 10.90 kPa

  Direction 2      ((37.96 + 38.96) / 5.25)............................= 14.65 kPa

Longitudinal centre loads (kN/m)

  Direction 1      (0.00 / 5.15) ......................................= 0.00 kPa

  Direction 2      (27.00 / 5.25) .....................................= 5.14 kPa

- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -

  W (Direction 1) .....................................................= 19.15 kPa

  W (Direction 2) .....................................................= 28.05 kPa

-----------------------------------------------------------------------

Total distributed load Q (kPa)

  Omega ...............................................................= 8.26 kPa
```

Line loads

Direction 1 ((17.66 + 0.00 + 38.46) x 5.25) / (5.25 x 5.15) .= 10.90 kPa

Direction 2 ((37.96 + 27.00 + 38.96) x 5.15) / (5.15 x 5.25) = 19.79 kPa

- -

Q ....= 38.95 kPa

Rectangle 1 of 3	DIRECTION 1	DIRECTION 2
L(m)	9	1.5
B(m)	1.5	9
P Edge (kN/m)	37.46	32.76
P Centre (kN/m)	0.00	0.00
W(kPa)	56.07	10.41
k(kPa/m)	3538.44	3538.44
Delta(mm)	22.5	3.8
No. of Beams	1	5
	CENTRE HEAVE	
Delta > Ymc ?	NO	NO
Edge Dist. (m)	1.628	1.628
M work (kNm/m)	55.35	0.94
I req(x10^6 mm4/m)	1131.60	0.80
	EDGE HEAVE	
Delta > Yme ?	NO	NO
Edge Dist. (m)	1.760	0.300
M work (kNm/m)	41.12	3.00
I req(x10^6 mm4/m)	909.46	0.80

S79

Rectangle 2 of 3	DIRECTION 1	DIRECTION 2
L(m)	16.5	2.4
B(m)	2.4	16.5
P Edge (kN/m)	32.76	4.86
P Centre (kN/m)	0.00	0.00
W(kPa)	33.27	8.54
k(kPa/m)	2421.26	2421.26
Delta(mm)	30.0	6.0
No. of Beams	2	7
	CENTRE HEAVE	
Delta > Ymc ?	NO	NO
Edge Dist. (m)	1.628	1.628
M work (kNm/m)	47.49	5.66
I req(x10^6 mm4/m)	695.84	0.45
	EDGE HEAVE	
Delta > Yme ?	YES	NO
Edge Dist. (m)	1.760	0.480
M work (kNm/m)	4.52	1.34
I req(x10^6 mm4/m)	7.52	0.45

S80

Rectangle 3 of 3	DIRECTION 1	DIRECTION 2
L(m)	5.25	5.15
B(m)	5.15	5.25
P Edge (kN/m)	17.66	37.96
P Centre (kN/m)	0.00	27.00
W(kPa)	15.12	22.91
k(kPa/m)	3894.82	3894.82
Delta(mm)	13.1	12.9
No. of Beams	5	3
	CENTRE HEAVE	
Delta > Ymc ?	NO	NO
Edge Dist. (m)	1.628	1.628
M work (kNm/m)	66.50	67.68
I req(x10^6 mm4/m)	959.97	891.74
	EDGE HEAVE	
Delta > Yme ?	NO	NO
Edge Dist. (m)	1.050	1.030
M work (kNm/m)	36.51	31.37
I req(x10^6 mm4/m)	423.38	426.62

TRIAL FOOTING PROPERTIES :-

Edge Beams:
- Beam Width = 300 mm
- Beam Depth = 800 mm
- Reinforcement
  - top = 3 x N16 bars, 40 mm cover
  - bottom = 3 x N16 bars, 65 mm cover

Internal Beams:
- Beam Width = 300 mm
- Beam Depth = 800 mm
- Reinforcement
  - top = 3 x N16 bars, 40 mm cover
  - bottom = 3 x N16 bars, 65 mm cover

Slab:
- Thickness = 125 mm
- Reinforcement
  - layer 1 = 227 mm^2/m in both directions, 20 mm cover

Material Properties:
- Fsy = 500 MPa
- F'c = 20 MPa

A COMPARISON OF THE REQUIRED DESIGN PROPERTIES AND THOSE OBTAINED FOR THE ABOVE FOOTING SYSTEM IS TABULATED BELOW

- Note that where relevant, the properties are expressed in units per metre width of total footing cross section
- The I required values have been factored up to take account of the variation in the long term creep factor for concrete, refer to AS3600, clause 8.5.3.3

S82

RECTANGLE 1 of 3 (9m x 1.5m)

BEAM DEFLECTED SHAPE	CENTRE HEAVE		EDGE HEAVE	
	REQUIRED	ACTUAL	REQUIRED	ACTUAL
DIRECTION 1				
Moment of Inertia (x 10^9 mm^4/m)	1.132 (Ireq)	12.043 (Ieff)	0.909 (Ireq)	12.043 (Ieff)
Flexural Strength (kNm/m)	55.4 (M*)	149.4 (øMu)	41.1 (M*)	116.5 (øMu)
Ductility Check (kNm/m)	76.9 (1.2Mcr)	186.7 (Mu)	83.6 (1.2Mcr)	145.6 (Mu)
Flange Width (m)	External	Internal		
	0.75	0		
DIRECTION 2				
Moment of Inertia (x 10^9 mm^4/m)	0.001 (Ireq)	7.159 (Ieff)	0.001 (Ireq)	7.159 (Ieff)
Flexural Strength (kNm/m)	0.9 (M*)	94.2 (øMu)	3.0 (M*)	77.0 (øMu)
Ductility Check (kNm/m)	42.8 (1.2Mcr)	117.7 (Mu)	52.3 (1.2Mcr)	96.2 (Mu)
Flange Width (m)	External	Internal		
	0.45	0.6		

583

RECTANGLE 2 of 3 (16.5m x 2.4m)

BEAM DEFLECTED SHAPE	CENTRE HEAVE		EDGE HEAVE	
	REQUIRED	ACTUAL	REQUIRED	ACTUAL
DIRECTION 1				
Moment of Inertia (x 10^9 mm^4/m)	0.696 (Ireq)	18.060 (Ieff)	0.008 (Ireq)	18.060 (Ieff)
Flexural Strength (kNm/m)	47.5 (M*)	211.1 (øMu)	4.5 (M*)	148.2 (øMu)
Ductility Check (kNm/m)	132.3 (1.2Mcr)	263.9 (Mu)	114.7 (1.2Mcr)	185.3 (Mu)
Flange Width (m)	External	Internal		
	1.2	0.8		
DIRECTION 2				
Moment of Inertia (x 10^9 mm^4/m)	0.000 (Ireq)	7.484 (Ieff)	0.000 (Ireq)	7.484 (Ieff)
Flexural Strength (kNm/m)	5.7 (M*)	94.0 (øMu)	1.3 (M*)	74.0 (øMu)
Ductility Check (kNm/m)	47.3 (1.2Mcr)	117.5 (Mu)	52.5 (1.2Mcr)	92.5 (Mu)
Flange Width (m)	External	Internal		
	0.54	0.78		

584

RECTANGLE 3 of 3 (5.25m x 5.15m)

BEAM DEFLECTED SHAPE	CENTRE HEAVE		EDGE HEAVE	
	REQUIRED	ACTUAL	REQUIRED	ACTUAL
DIRECTION 1				
Moment of Inertia (x 10^9 mm^4/m)	0.960 (Ireq)	19.563 (Ieff)	0.423 (Ireq)	19.563 (Ieff)
Flexural Strength (kNm/m)	66.5 (M*)	235.2 (øMu)	36.5 (M*)	171.5 (øMu)
Ductility Check (kNm/m)	137.8 (1.2Mcr)	294.0 (Mu)	128.3 (1.2Mcr)	214.3 (Mu)
Flange Width (m)	External	Internal		
	0.64375	1.2875		
DIRECTION 2				
Moment of Inertia (x 10^9 mm^4/m)	0.892 (Ireq)	10.599 (Ieff)	0.427 (Ireq)	11.399 (Ieff)
Flexural Strength (kNm/m)	67.7 (M*)	136.8 (øMu)	31.4 (M*)	100.8 (øMu)
Ductility Check (kNm/m)	78.9 (1.2Mcr)	171.0 (Mu)	75.3 (1.2Mcr)	126.0 (Mu)
Flange Width (m)	External	Internal		
	0.815	1.33		

FOR FOOTINGS USE :-

EXTERNALLY:- 300 mm (Wide) x 800 mm (Deep)
- With 6 /N16 Bars - 3 Top And 3 Bottom

INTERNALLY:- 300 mm (Wide) x 800 mm (Deep)
- With 6 /N16 Bars - 3 Top And 3 Bottom

WARNING:
In the following case(s), the analysis was NOT a reactive soils analysis because soil mound height was less than the allowable beam deflection:
Rectangle 2, Edge Heave, Direction 1

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CLIENT:					
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DATE:		BY:	TH	REV:	

Stub Column (ST1)

worst case → Supporting FB22 & FB23 → 21m² of Office floor

(Allow 1.0 kPa)

$P_{DL} = 21 \times 1.0$
$= 21.0$ kN

$P_{LL} = 21 \times 3$
$= 63$ kN

$P^* = 1.2(21) + 1.5(63)$
$= 119.7$ kN

Bearing onto existing brickwork
↳ 300 × 100 base plate

$$\text{Bearing} = \frac{119.7 \times 10^3}{300 \times 100}$$
$$= 3.99 \text{ MPa}$$

Existing brickwork assume 15 MPa unconfined compressive strength

$f'_{uc} = 15$ MPa

$f'_m = 1.4\sqrt{15}$
$= 5.4$ MPa

Allowable bearing pressure $\phi f'_m = 0.75 \times 5.4 = 4.05$ MPa > 3.99
↳ OK!!

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CLIENT:	Bert Faring Constructions				
JOB NO:	17799	SHEET:	S86	CHECK:	
DATE:	7-4-17	BY:	TH	REV:	

Re-design Mez Floor Framing

MFB1 Span = 5.3 m

Use design IT software

FLW = 1.7 m   50 kg/m² D.L

Adopt 3/300×45 LVL 13 (hyspan) bolt laminated

Rigidity Ratio = 1.2
Capacity Ratio = 3.3

MFB6 Cantilever Bearer supporting the end of MFB1

↳ Floor Area = 5.0 m²

$P_{DL} = (5 \times 0.5) = 2.5$ kN $P_{LL} = (5 \times 1.5) = 7.5$ kN $P^* = 14.3$ kN

cantilever 550 mm
Backspan 900 mm

$M^* = 0.55 \times 14.3 = 7.87$ kNm

Limit to max 2 mm deflection @ free end under DL

$$I_x \geqslant \overset{j_2}{2} \times \frac{2500 \times 550^2 \times 1450}{3 \times 13200 \times 2}$$

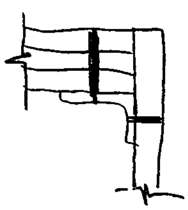
$$\geqslant 27.7 \times 10^6 \text{ mm}^4$$

Adopt 300 × 63 LVL 13 (hyspan)

↳ $I_x = \frac{63 \times 300^3}{12} = 142 \times 10^6$ mm⁴

Strength ok by inspection!!

Check capacity of 2/M16 bolted connection of MFB1 to MFB6



LVL 13 → JD3 $b_{eff} = 2 \times 63 = 126$ mm ⇒ $Q_{skp} = 16.32$ kN

$\phi N_{dj} = \phi k_1 k_{16} k_{17} n Q_{skp}$ $k_1 = 0.69$ $k_{16} = k_{17} = 1.0$

$= 0.85 \times 0.69 \times 2 \times 16.32$

$= 19.1$ kN $> P^* = 14.3$ ⇒ Ok!!

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Column C6

$N_c^* = 20$ kN

offset cleat → take 150mm eccentricity

$M^* = 0.15 \times 20$
$= 3$ kNm

By inspection/experience adopt 89x3.5 SHS

Change FB18 to steel due to steel column over

↳ by inspection Adopt 200UB18

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CLIENT:	Bert Farina Construction Pty. Ltd.				
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SHEET:	S88	DATE:		REV:	

Existing Masonry Walls (check under concentrated loads from SC1)

$F_o = \phi f'_m A_b$ AS 3700 Clause 7.3.2 Basic compressive capacity

$\phi = 0.75$

$k_h = 1.0$ 76 high brick 10mm mortar joint

$f'_m = 5.4$ MPa ($f'_{mb} = 1.4\sqrt{15} = 5.4$ MPa & $f'_m = k_h f'_{mb}$)

$A_b = 100 \times 1000$ (1.0m length of wall)
$= 100{,}000$ mm^2

$F_o = 0.75 \times 5.4 \times 100 \times 10^3$
$= 405$ kN (per linear metre of wall)

$F_d^* \leq k F_o$

$S_{rs} = \dfrac{\alpha_v H}{k_t t}$

$\alpha_v = 1.0$ Supported laterally by ceiling diaphragm
$H = 2750$
$k_t = 1.0$
$t = 110$ mm

$S_{rs} = 25$

$\rightarrow k = 0.31$ (AS 3700 Table 7.1)

$k F_o = 0.31 \times 405$
$= 125.6$ kN

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Worst case load on wall is from SC1 supporting both FB22 & FB23
↳ 21m² of office floor

$P_{DL} = 21 \times 1.0 = 21$ kN
$P_{LL} = 21 \times 3 = 63$ kN
→ $P^* = 1.2(21) + 1.5(63) = \underline{119.7}$ kN

$P^* = F_d^* \Rightarrow F_d^* < 125.6$ kN $\Rightarrow$ existing wall has sufficient compressive strength!

Check as concentrated load (AS 3700 clause 7.3.5)

Calculate basic compression capacity but only consider 300 length of wall (SC1 has 300 long base plate)

$F_o = 0.75 \times 5.4 \times 100 \times 300$
$= \underline{121.5 \text{ kN}}$

$F_d^* \leq k_b F_o$ Take $k_b = 1.0 \Rightarrow$ conservative

$k_b F_o = 1.0 \times 121.5 = 121.5 > F_d^* = 119.7 \Rightarrow$ ok!!

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Panels on Northern Boundary Performance in Fire

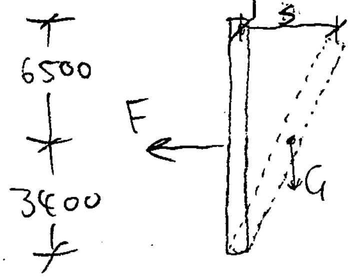
Worst case in a fire assume timber roof framing is burned out leaving panels to cantilever up past the first floor framing level

Panels are fixed/tied back to floor framing M12 cast in ferrules (F4) at 450 crs.

BCA Specification C1.11 part 3 (General requirements)

a) All ferrules to have 300 long cross bar

b) Cast in inserts (ferrules) must be able to withstand ultimate load of two times the force required to resist overturning moment at the panel base arising from outwards displacement at the top of H/10



$$S = \frac{9900}{10} = 990$$

$$G = 0.15 \times 1 \times 9.9 \times 25$$
$$= 37.1 \text{ kN} \quad \text{(S.W of 1m strip of panel)}$$

$$M_o = G\frac{S}{2} = 37.1 \times \frac{0.99}{2} = 18.36 \text{ kN per m}$$

M12 Ferrules must have min. capacity of F^*

$$F^* = \frac{18.36}{3.4} \times 2 = 10.8 \text{ kN per m}$$

Ferrules @ 450 crs $\Rightarrow$ 4.86 kN per Ferrule

From Ramset/Reid design guides M12 elephant foot ferrules x 55 long have Working Load Limit of 9.7 kN each in 32 MPa concrete

↳ Ferrule fixing ok → meets C1.11 requirements!!

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Check Existing Footings Below masonry walls & new SCI point loads

Current loading → 2.75 m wall
3.0 m Floor load
4.0 m roof & ceiling load → Approx

$W_{DL} = (2.75 \times 2.2) + (3 \times 0.4) + (4 \times 0.4)$
$= 8.85$ kN/m

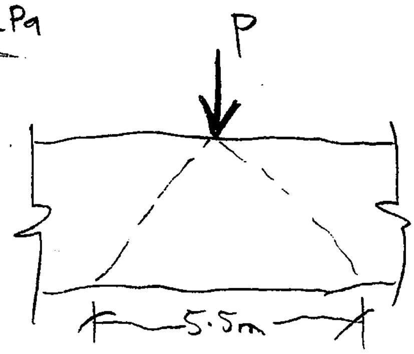
$W_{LL} = (3 \times 3) + (4 \times 0.25)$
$= 10$ kN/m

$W = 8.85 + \frac{1}{2}(10)$
$= 13.85$ kN/m

↳ Assume 250 wide footing beam

↳ $Q_{soil} = \frac{13.85}{0.25} = 55.4$ kPa

New Loading → 2.75 m wall
3.0 m Floor
4.0 m ceiling only
Point load dispersed @ 45°
$P_{DL} = 21$ kN
$P_{LL} = 63$ kN



$W_{DL} = (2.75 \times 2.2) + (3.0 \times 0.4) + (4 \times 0.15) + \frac{21}{5.5}$
$= 11.7$ kN/m

$W_{LL} = (3 \times 3) + \frac{63}{5.5}$
$= 20.5$ kN/m

$W = 11.7 + \frac{1}{2}(20.5)$
$= 22.0$ kN/m

$Q_{soil} = \frac{22}{0.25} = 88$ kPa

less than 100 kPa → consider Acceptable!!

Due to change in loading it is likely some movement re-settlement will occur refer to Structural Adequacy Letter.

Appendix A

Lower Level Sway Frame in Front Elevation Analysis

Microstran Output

INPUT/ANALYSIS REPORT

```
Job:    Sway Frame Front Elevation (lower level)
Title:  Job no. 17799
        147 Marion Road, RICHMOND
Type:   Plane frame
Date:   18 Feb 2016
Time:   10:23 AM

Nodes .............................     6
Members ...........................     5
Spring supports ...................     0
Sections ..........................     2
Materials .........................     1
Primary load cases ................     1
Combination load cases ............     1

Analysis:  Non-linear elastic
Update node coordinates ...........     Y
Small displacement theory .........     Y
Include axial force effects .......     Y
Include flexural shortening .......     N
Convergence criterion:  Residual
Convergence tolerance .........  5.000E-04
```

LOAD CASES

Case	Type	Analysis Type	Flag	Title
1	P	N	CNV	Wind Load
2	C	N	CNV	Serviceability Wind Load

Analysis Types:
S - Skipped (not analysed)
L - Linear
N - Non-linear

Analysis Flag:
CNV - Converged
XSD - Excessive displacements
DNC - Did not converge in iteration limit
UNS - Unstable or local instability

NODE COORDINATES

Node	X m	Y m	Z m	Restraint
1	0.000	0.000	0.000	111000
2	0.000	3.400	0.000	000000
3	4.350	0.000	0.000	111000
4	4.350	3.400	0.000	000000
5	7.750	0.000	0.000	111000
6	7.750	3.400	0.000	000000

MEMBER DEFINITION

Member	A	B	C	Prop	Matl	Rel-A	Rel-B	Length m
1	1	2	-X	1	1	000000	000000	3.400
3	3	4	-X	1	1	000000	000000	3.275
4	2	4	Y	2	1	000000	000000	4.300
6	5	6	-X	1	1	000000	000000	3.400
7	4	6	Y	2	1	000000	000000	3.350

MEMBER OFFSETS

Member	Axes	XA m	YA m	ZA m	XB m	YB m	ZB m
3	GL	0.0000	0.0000	0.0000	0.0000	-0.1250	0.0000
4	GL	0.0500	0.0000	0.0000	0.0000	0.0000	0.0000
7	GL	0.0000	0.0000	0.0000	-0.0500	0.0000	0.0000

LIBRARY SECTIONS

Section	Library	Name	Axis	Comment
1	asw	125X125X5.0SHS	Y	default
2	asw	250PFC	Y	default

SECTION PROPERTIES

Section	Ax m2	Ay m2	Az m2	J m4	Iy m4	Iz m4	fact
1	2.310E-03	0.000E+00	0.000E+00	8.870E-06	5.440E-06	5.440E-06	
2	4.520E-03	0.000E+00	0.000E+00	2.380E-07	3.640E-06	4.510E-05	

MATERIAL PROPERTIES

Material	E kN/m2	u	Density t/m3	Alpha /deg C	
1	2.000E+08	0.2500	7.850E+00	1.170E-05	STEEL

TABLE OF QUANTITIES

MATERIAL 1 STEEL

Section	Name	Length m	Mass tonne	Comment
1	125X125X5.0SHS	10.075	0.183	default
2	250PFC	7.650	0.271	default
		17.725	0.454	

APPLIED LOADING

CASE 1: Wind Load

Node Loads

Node	X Force kN	Y Force kN	Z Force kN	X Moment kNm	Y Moment kNm	Z Moment kNm
2	8.000	0.000	0.000	0.000	0.000	0.000

```
Sum of Applied Loads (Global Axes):
  FX:      8.000    FY:        0.000    FZ:       0.000
 Moments about the global origin:
  MX:      0.000    MY:        0.000    MZ:     -27.200
```

CASE 2: Serviceability Wind Load

Load Combinations

Case	Factor	
1	0.676	Wind Load

```
Sum of Applied Loads (Global Axes):
  FX:      5.408    FY:        0.000    FZ:       0.000
 Moments about the global origin:
  MX:      0.000    MY:        0.000    MZ:     -18.387
```

SIGN CONVENTION

```
Positive Forces (Member Axes):
  Axial  - Tension              Shear  - End A sagging
  Torque - Right-hand twist     Moment - Sagging
Deflections:
  Global deflections are absolute.
  Local deflections are relative to chord joining displaced end nodes.
```

MEMBER FORCES AND DEFLECTIONS

MEMBER 1: Nodes 1 - 2 Section 1: 125X125X5.0SHS Y

CASE 1: Wind Load

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	3.11	-2.47	0.00	0.00	0.00	0.00
2	1.70	3.11	-2.47	0.00	0.00	0.00	4.21
3	3.40	3.11	-2.47	0.00	0.00	0.00	8.41

Point	Offset m	X-glob m	Y-glob m	Z-glob m	y-local m	z-local m
1	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.70	0.0220	-0.0001	0.0000	-0.0056	0.0000
3	3.40	0.0328	-0.0001	0.0000	0.0000	0.0000

CASE 2: Serviceability Wind Load

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	2.09	-1.67	0.00	0.00	0.00	0.00
2	1.70	2.09	-1.67	0.00	0.00	0.00	2.84
3	3.40	2.09	-1.67	0.00	0.00	0.00	5.68

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local

	m	m	m	m	m	m
1	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.70	0.0149	-0.0001	0.0000	-0.0038	0.0000
3	3.40	0.0222	-0.0001	0.0000	0.0000	0.0000

MEMBER 3: Nodes 3 - 4 Section 1: 125X125X5.0SHS Y

CASE 1: Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	1.01	-3.01	0.00	0.00	0.00	0.00
2	1.64	1.01	-3.01	0.00	0.00	0.00	4.93
3	3.28	1.01	-3.01	0.00	0.00	0.00	9.85

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.64	0.0225	-0.0001	0.0000	-0.0061	0.0000
3	3.28	0.0328	-0.0002	0.0000	0.0000	0.0000

CASE 2: Serviceability Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.68	-2.03	0.00	0.00	0.00	0.00
2	1.64	0.68	-2.03	0.00	0.00	0.00	3.33
3	3.28	0.68	-2.03	0.00	0.00	0.00	6.66

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.64	0.0152	-0.0001	0.0000	-0.0041	0.0000
3	3.28	0.0222	-0.0001	0.0000	0.0000	0.0000

MEMBER 4: Nodes 2 - 4 Section 2: 250PFC Y

CASE 1: Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	-5.50	3.08	0.00	0.00	0.00	8.26
2	2.15	-5.50	3.08	0.00	0.00	0.00	1.64
3	4.30	-5.50	3.08	0.00	0.00	0.00	-4.99

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0328	-0.0002	0.0000	0.0000	0.0000
2	2.15	0.0328	-0.0006	0.0000	-0.0004	0.0000
3	4.30	0.0328	-0.0002	0.0000	0.0000	0.0000

CASE 2: Serviceability Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	-3.72	2.08	0.00	0.00	0.00	5.58
2	2.15	-3.72	2.08	0.00	0.00	0.00	1.10
3	4.30	-3.72	2.08	0.00	0.00	0.00	-3.37

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0222	-0.0001	0.0000	0.0000	0.0000
2	2.15	0.0222	-0.0004	0.0000	-0.0003	0.0000
3	4.30	0.0222	-0.0001	0.0000	0.0000	0.0000

MEMBER 6: Nodes 5 - 6 Section 1: 125X125X5.0SHS Y

CASE 1: Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	-4.03	-2.52	0.00	0.00	0.00	0.00
2	1.70	-4.03	-2.52	0.00	0.00	0.00	4.28
3	3.40	-4.03	-2.52	0.00	0.00	0.00	8.55

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.70	0.0221	-0.0001	0.0000	-0.0057	0.0000
3	3.40	0.0328	-0.0002	0.0000	0.0000	0.0000

CASE 2: Serviceability Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	-2.73	-1.70	0.00	0.00	0.00	0.00
2	1.70	-2.73	-1.70	0.00	0.00	0.00	2.89

3	3.40	-2.73	-1.70	0.00	0.00	0.00	5.79

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.70	0.0149	-0.0001	0.0000	-0.0038	0.0000
3	3.40	0.0222	-0.0001	0.0000	0.0000	0.0000

MEMBER 7: Nodes 4 - 6 Section 2: 250PFC Y

CASE 1: Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	-2.48	4.06	0.00	0.00	0.00	5.24
2	1.67	-2.48	4.06	0.00	0.00	0.00	-1.55
3	3.35	-2.48	4.06	0.00	0.00	0.00	-8.35

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0328	-0.0002	0.0000	0.0000	0.0000
2	1.67	0.0328	0.0001	0.0000	0.0002	0.0000
3	3.35	0.0328	-0.0002	0.0000	0.0000	0.0000

CASE 2: Serviceability Wind Load

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	-1.68	2.74	0.00	0.00	0.00	3.54
2	1.67	-1.68	2.74	0.00	0.00	0.00	-1.05
3	3.35	-1.68	2.74	0.00	0.00	0.00	-5.65

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.00	0.0222	-0.0001	0.0000	0.0000	0.0000
2	1.67	0.0222	0.0001	0.0000	0.0002	0.0000
3	3.35	0.0222	-0.0001	0.0000	0.0000	0.0000

SUPPORT REACTIONS

Node	Case	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
		kN	kN	kN	kNm	kNm	kNm
1	1	-2.50	-3.08	0.00	0.00	0.00	0.00
	2	-1.69	-2.08	0.00	0.00	0.00	0.00
3	1	-3.02	-0.98	0.00	0.00	0.00	0.00
	2	-2.04	-0.66	0.00	0.00	0.00	0.00
5	1	-2.48	4.05	0.00	0.00	0.00	0.00
	2	-1.68	2.74	0.00	0.00	0.00	0.00

(Reactions act on structure in positive global axis directions.)

SUM OF REACTIONS

Case	Force-X	Force-Y	Force-Z
	kN	kN	kN
1	-8.00	-0.01	0.00
2	-5.41	0.00	0.00

RESIDUALS

Case	DOFN	Residual
1	11	-3.820E-03