STRUCTURAL ENGINEERING REPORT

97 King William Street, Kent Town

Job reference: 2018-7161

Rev	Date	By	Reviewed by
A	6/06/2018	JT	BR

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1.0 INTRODUCTION

GINOS has been commissioned to design a four storey office building/carpark with a basement located at 97 King William Street, Kent Town.

This design basis report outlines the guidelines and certain parameters for the design of the building. The building is approximately 40 meters in length accommodating three levels of office space and two levels of car parking. The highest point of the building is approximately 16 meters located at the roof. The substructure consists of a basement formed by concrete piles and a concrete slab on ground. The superstructure is made up of steel beams, precast panels, composite suspended slabs and a steel framed roof.

2.0 DESIGN STANDARDS AND REFERENCES

All engineering services design provided by GINOS will generally comply with the latest requirements of the following:

2.1 Statutory

Occupational Health and Safety and Welfare Regulations, 1995

Building Code of Australia (BCA), 2009

Private Certification

2.2 Codes and Standards

Relevant Australian Standards including and others:

AS/NZS 1170.0–2002 - Structural design actions - Part 0: General principles

AS/NZS 1170.1-2002 - Structural design actions - Part 1: Permanent, imposed and other actions

AS/NZS 1170.2-2002 - Structural design actions –Part 2: Wind actions

AS 1170.4-2007 - Structural design actions –Part 4: Earthquake actions in Australia

AS 2159-1995 - Piling - design and installation

AS 2327.1-2003 – Composite Structures – Simply Supported Beams

AS 2870-1996 - Residential Slabs and Footings-Construction

AS 3600-2001 - Concrete structures

AS 3700-2001 - Masonry structures

AS 4100-1998 - Steel structures

AS/NZS 4600-1996 - Cold formed steel structures

2.3 Site Investigations

Geotechnical site investigations were undertaken by Structural Stability Consulting and consist of the following:

Surface Soil Borelog – S13050 – 27/3/13

2.4 Software

Excel	Spreadsheet
Spacegass	General structural analysis
Word	Word processing
RAPT	Reinforced concrete analysis
Fielders KingFlor	Composite beam & slab analysis

3.0 DESIGN CRITERIA

3.1 Loads

3.1.1 Dead loads

Unit weights

Dead loads should be calculated on the basis of the following material densities:

- Reinforced concrete 2,500 kg/m^3
- Structural steel 7,850 kg/m^3

Dead loads from other building elements shall be taken as:

- Metal deck sheeting: 0.055 kPa
- Insulation: 0.05 kPa
- Purlins/battens: 0.1 kPa
- Services: 0.2 kPa
- Ceiling (normal 13mm plasterboard): 0.1 kPa
- Glazing: 0.3 kPa

3.1.2 Superimposed Dead and Live Loads

The following design loads are taken from AS 1170 Part I - 2002 Structural design actions - permanent, imposed and other actions. Description:

SDL		LL	
Superimposed dead load (kPa)		Distributed live load (kPa)	Point load (kN)
Partitions	0.50		
Office areas		3.0	2.7
Balconies		4.0	4.5
Car Park		2.5	13
Non-trafficable roof		0.25	1.4

3.1.3 Wind loads

Wind loads applied to structural elements shall be assessed in accordance with:

AS 1170 Part 2 - Structural design actions - Wind loads.

The following parameters shall be adopted:

- Building importance: Level 2
- Region: A1
- Annual probability of exceedance: 1:500
- Regional wind speeds: Vu = 45 m/sec
  Vs = 37 m/sec

- Terrain Category: 3
- Structure height: Z = 16.16m
- Variation of wind speed with height: $M_{z,cat}$ = 0.90
- Topographical multiplier: 1.0
- Shielding multiplier: 1.0
- Directional multiplier: 1.0

Pressure Coefficients as follows:

- Internal pressure coefficients - in accordance with the requirements of Section 5.3 and Tables 5.1(A) and 5.1(B) as appropriate;
- External pressure coefficients - in accordance with the requirements of Section 5.4 and Tables 5.2(A, B and C) and 5.3(A, B and C) as appropriate;
- Apply combination factors (Kc) as appropriate. Apply local pressure coefficients or area reduction factors as required.

3.1.4 Seismic loads

Seismic loads to the structure shall be assessed in accordance with AS 1170 Part 4 - 2007 Structural design actions - Earthquake actions in Australia.

The following parameters shall be adopted:

Strength limit state

- Building importance: 2
- Annual probability of exceedance: 1:500
- Probability factor: 1.0
- Hazard factor: Z = 0.08
- Site sub-soil classification De
- Earthquake design category EDCII
- Lateral load resisting elements Limited ductile shear walls
- Structural ductility factor: $\mu = 2$
- Structural performance factor: $S_p = 0.77$
- Structure height: Z = 16.16m
- Structural base: Ground (0m)
- Natural period of structure: T = 0.50 secs
- Spectral shape factor: Ch(T) = 3.68

3.2 Serviceability

3.2.1 Deflection limits

The following deflection limits in are proposed for the building structure:

Element	Deflection limit under Dead + Live = Total Load (U.N.O.)
Floor Beams & Slabs	L/250
Roof with ceiling	L/300 (dead only)
Roof without ceiling	L/150
Floor Cantilevers	L/125
Lintels	L/240 (12mm max.)
Wind beams	L/200 (serviceability wind)

Floor beams were checked for vibration in accordance with AS/NZS 1170.0:2002 – Table C1.

3.2.2 Fire Protection

The fire ratings for different areas in the building are as per the following table:

Area	Fire Rating
All levels	2 hrs
Roof	0 hr

Cover to reinforcement for general areas in concrete elements to achieve the 120 mins for structural insulation and adequacy shall be a minimum of the values in the following table.

Elements	Min thickness (mm)	a_s, Cover to centre of steel (mm)
Slabs	120	20
Walls	120	25

For areas with a higher fire rating refer to Section 5 of AS3600 for increased covers. Structural steel shall have a protective coating suitable of achieving the above fire ratings for each area.

APPENDIX A – STRUCTURAL CALCULATIONS

Reference: 2018-7161
Project: 97 King William Street, Kent Town **Checked by:** BR
Designer: JT Date: 6/06/2018 **Index:** TOC

CALCULATIONS	REF./COMMENT

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2) P - PILES/CAPPING BEAM

3) RW - RETAINING WALLS

4) F - FOOTINGS

5) PR - PRECAST

6) LP - LIFT PIT

7) SL - SLABS

8) CF - COMPOSITE FLOOR BEAMS

9) B - FLOOR BEAMS

10) MS - MISCELLANEOUS STEEL FRAMING

11) R - ROOF FRAMING

12) BR - ROOF BRACING

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	Date: 6/06/2018	**Index:**	L - 1

CALCULATIONS			REF./COMMENT
1) DESIGN LOADS/DESIGN CRITERIA			
Refer to Structural Design Report for further details.			
a) General			
	Design life:	50 years	
	Inportance level:	2	AS 1170.0 Tab F1
	ULS Prob. of exceedance =	1/500	
	SLS Prob. of exceedance =	1/25	
b) Loading			AS/NZS 1170.1
i) Dead loads (G)			
ROOF	Sheeting =	0.055 kPa	
	Purlins =	0.10 kPa	
	Commercial steel roof =	0.45 kPa (includes ceilings & typical services)	
WALLS	180 Precast =	4.50 kPa	
	130 Precast =	3.25 kPa (roof)	
	Partitions =	0.50 kPa	
	Glazing =	0.30 kPa	
	140 blockwork =	3.50 kPa	
FLOORS	170 slab =	4.25 kPa	
	120 slab =	3.00 kPa	
	Steel beams =	0.20 kPa	
	Services =	0.20 kPa	
	Screed =	1.00 kPa	
ii) Live loads (Q)			
ROOF	Roof =	0.25 kPa	
	or	1.8/A + 0.12 (whichever is greater)	
	or	1.4 kN	
FLOORS	Floor =	3.0 kPa	Office
	or	2.7 kN	
	Balcony =	4.0 kPa	
	or	1.8 kN	
	Carpark (light traffic) =	2.5 kPa	Vehicles < 2500kg
	or	13 kN	

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CALCULATIONS	REF./COMMENT

iii) *Wind Loads (W)* — AS/NZS 1170.2

Wind region =	A1		
Structure's height, Z =	16.16	m	
Terrain category =	3		
Prob. of exceedance =	1/500	years	(ULS)
	1/25	years	(SLS)
Regional wind speed, $V_{R,u}$ =	45	m/s	
$V_{R,s}$ =	37	m/s	
Terrain/height multiplier, $M_{z,cat}$ =	0.90		Interpolated
Sheilding multiplier, M_s =	1.00		
Topographic multiplier, M_t =	1.00		
Directional multiplier, M_d =	1.00	(Any direction)	
	0.95	(N-W)	Long direction
	0.80	(N-E)	Short direction

Basic design wind speed: $V_{sit,\beta} = V_r M_d (M_{z,cat} M_s M_t)$

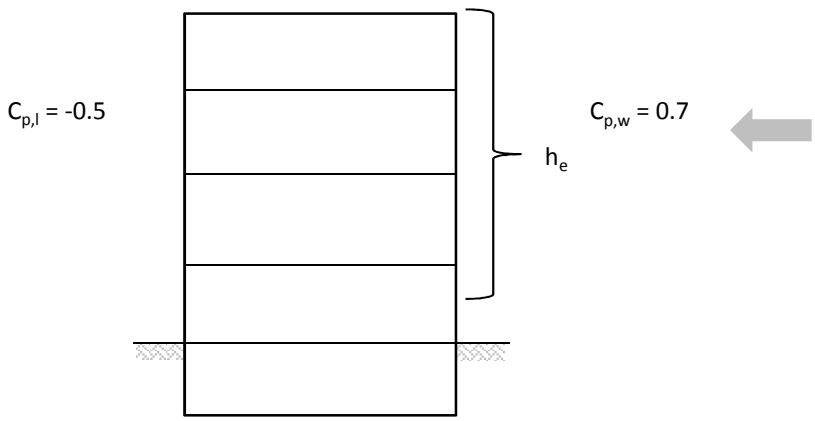
	(Any)	(N-W)	(N-E)	
V_{sit,β_ult} =	40.6	38.5	32.5	m/s
V_{sit,β_serv} =	33.4	31.7	26.7	m/s

Basic design wind pressure: $q^* = 0.0006 \times V_{sit2}$

	(Any)	(N-W)	(N-E)	
q^*_{ult} =	0.99	0.89	0.63	kPa
q^*_{serv} =	0.67	0.60	0.43	kPa
Ratio =	0.68			

BUILDING DIMENSIONS

Height =	16.16	m	
Length =	40.4	m	(not incl. ramp)
Width =	12	m	(clear span)
Floor height =	3.5	m	(average)
Effective height, h_e =	14.24	m	



Taking into account wind coefficients

C_{fig} =	1.2		
p* =	1.07	kPa (N-W)	
	0.76	kPa (N-E)	
A =	170.9	m^2 (N-W)	(Surface area in elevation view)
	575.3	m^2 (N-E)	
Base wind force, F_W =	183	kN (N-W)	
	436	kN (N-E)	

CALCULATIONS	REF./COMMENT

WIND ANALYSIS

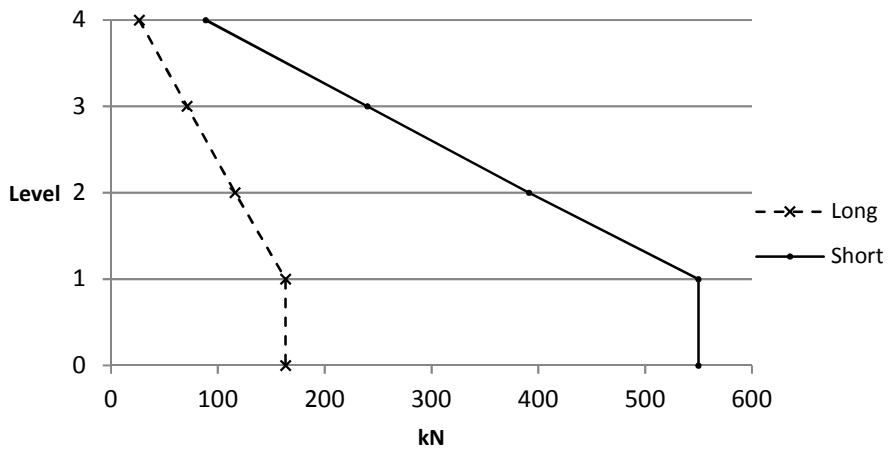
Short direction

	Height(m)	LW(m)	F_w (kN)	SUM (kN)
Roof (4)	14.95	2.05	89	89
L3 (3)	10.85	3.50	151	240
L2 (2)	7.35	3.50	151	391
L1 (1)	3.85	3.675	159	550
G (0)	0	0	0	550

Long direction

	Height(m)	LW(m)	F_w (kN)	SUM (kN)
Roof (4)	14.95	2.05	26	26
L3 (3)	10.85	3.50	45	71
L2 (2)	7.35	3.50	45	116
L1 (1)	3.85	3.675	47	163
G (0)	0	0	0	163

Figure 1: Global wind loads per floor



CALCULATIONS	REF./COMMENT

iv) Earthquake Loads (EQ) — AS 1170.4

Probability factor, k_p = 1.0
Hazard factor, Z = 0.1 (Adelaide)
Site sub-soil class = De — Assumed
Structures height, h_n = 16.16 m > 15

Earthquake design catagory = **EDCII**

k_pZ = 0.10

Period: $T_1 = 1.25k_t h_n^{0.75}$

k_t = 0.05 — Cl. 6.2.3
h_n = 16.16 m
T_1 = 0.50 secs

Spectral shape factor, Ch(T1) = 3.68 — Table 6.4
C(T1) = $k_pZCh(T1)$ = 0.368

"Limited ductile shear walls"
Structural performance factor, S_p = 0.77 — Table 6.5(A)
Structural ductility factor, μ = 2.00 — Table 6.5(A)

Gravity Load (Total weight of structure): Wi = ΣGi + ΣψcQi ψ_c = 0.3

	Areas (m^2)						
	Slab	Precast	Roof				
Roof	0	0	485	=	218	kN	(Neglect roof Q for EQ)
L3	485	283	0	=	3769	kN	
L2	485	283	0	=	3769	kN	
L1	485	283	0	=	3769	kN	
G	0	141	0	=	636	kN	

Note: Assume earthquake loads applied to half the height of G go to ground.

TOTAL, W_t = 12162 kN

Base shear: $F_{EQ} = [k_p ZCh(T1)S_p/\mu]W_t$

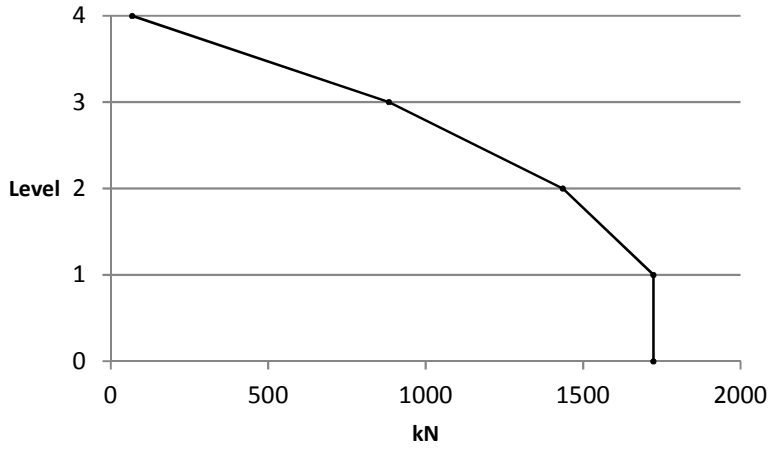
F_{EQ} = 1723 kN
Percentage of total weight = 14.2 %

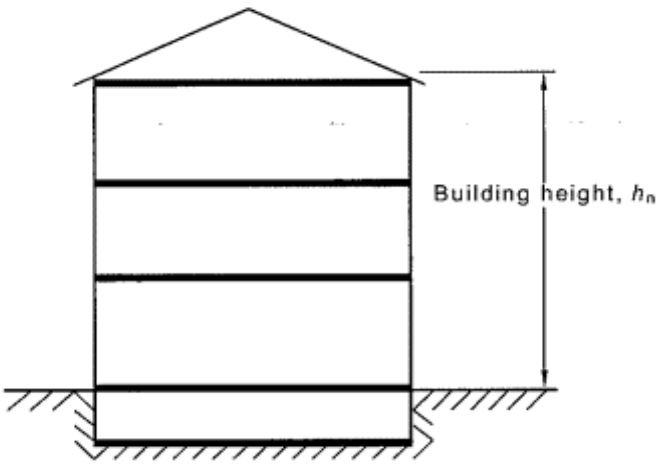
CALCULATIONS | **REF./COMMENT**

SEISMIC ANALYSIS *- Equivalent Static Analysis (all directions):*

	Height (m)	Wi (kN)	k	Wihi^k	kF,i	Fi (kN)	Vi (kN)
Roof (4)	15.45	218	1.0	3405	0.0391	67	67
L3 (3)	10.85	3769	1.0	41264	0.4734	816	883
L2 (2)	7.35	3769	1.0	27912	0.3202	552	1435
L1 (1)	3.85	3769	1.0	14585	0.1673	288	1723
G (0)	0.00	0				0	1723
			Total =	87167			

Figure 2: Global earthquake shear per floor (any direction)





SURFACE SOIL BORELOG

Sheet BH01

DATE: 27/3/13 **JOB NO:** S13050 **SAMPLE METHOD:** Hyd Push Tube/Portable Push Tube
CLIENT: **LOGGED BY:** In-depth Drilling

SITE ADDRESS: 97 King William St, Kent Town

HORIZON DEPTH (m)			SOIL DESCRIPTION	SOIL COLOUR	UNIFIED SYMBOL (USCS)	MOISTURE CONTENT	CONSISTENCY /DENSITY	PLASTICITY/ REACTIVITY	BEARING CAPACITY	ESTIMATED I_{pt}
BH.1	BH.2	BH.3								
0-0.05	0-0.15	0-0.1		-	Fill	-	-	-	-	-
0.05-0.25	0.15-0.4	0.1-0.35	Sandstone gravel	Yellow brown	Fill	D	Fb/L	NP	H	-
0.25-0.50	0.4-0.6	0.35-0.5	Clayey silt	Brown	ML	<=PL	F	M	M	0.018
0.50-1.3	0.6-1.2	0.5-1.2	Silty clay	Red brown	CH	<PL	VSt	VH	L	0.032
1.3-2.5	1.2-3.5	1.2-3.2	Silty calcareous clay	Orange brown	CH	<=PL	St	H	M	0.025
2.5-3.4		3.2-4.0	Calcareous silty clay	Light orange brown	CH	<PL	F/St	H	M	0.022
3.4-4.8	3.5-4.5	4.0-4.2	Silty sandy clay	Orange brown	CH	>PL	F	H	M	0.020
4.8-6.0	4.5-6.0	4.2-6.0	Silty sandy clay	Pale orange brown	CH	>PL	F	H	M/H	0.025

COMMENTS:

CLASSIFICATION: ''

Ys = mm
Yst = mm

Abbreviations & Symbols

MC	Moisture Content
PL	Plastic Limit
>	Greater Than
<	Less Than
~	Approximately equal to
DCP	Dynamic Cone Penetrometer

Consistency*

VS	Very Soft
S	Soft
F	Firm
St	Stiff
VSt	Very Stiff
H	Hard

Density*

Fb	Friable
VL	Very Loose
L	Loose
M	Medium Dense
D	Dense
VD	Very Dense

Moisture

D	Dry
Da	Damp
M	Moist
W	Wet
Sat	Saturated

Plasticity/Reactivity

L	Low
M	Medium
H	High
NP	Non Plastic

USCS Abbreviations

GW, GP, GM, GC	Gravelly Soils
SW, SP, SM, SC	Sandy Soils
ML, MH	Silty Soils
CL, CH	Clay Soils

* Field estimate only

Structural Stability Consulting Pty Ltd ACN 136 766 679 ATF Structural Stability Consulting Trust ABN 68997844272
2 Greer Place, Magill SA 5072
Telephone (08) 8333 1157 **E**mail sscadmin@adam.com.au

CALCULATIONS				REF./COMMENT

2) CONCRETE PILE & CAPPING BEAM DESIGN

Concrete piles are to be max. 300mm diameter due to space restrictions. Due to the loadings imposed on the piles, they will need to be propped during construction and designed as simply supported. Piles are designed in accordance with AS 4678:2002 and AS 2159:2009.

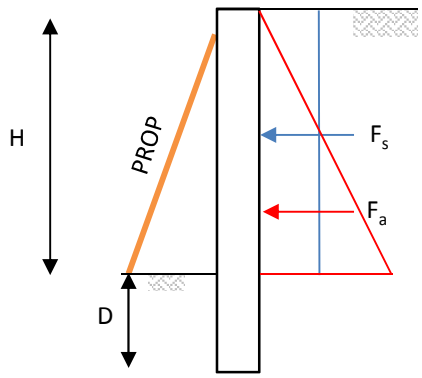
a) Main Piles - P1

INPUT

Pile properties	Diameter, Db =	300	mm	
	Span, h =	3.05	m	(to bot. of capping beam)
	Spacing, S =	0.8	m	
	Density of conc., ρ_c =	25	kN/m^3	Assumed
				"
Soil properties	Ultimate shaft friction, f_s =	50	kPa (firm, unsaturated caly)	
	Ultimate base resistance, f_b =	600	kPa	
	strength reduction factor, ϕ_{gb} =	0.67		cl. 4.3.2

LOADING

Retaining height, L =	3.05	m	
Clay, γ =	18	kN/m^3	
Angle of friction, ϕ =	30	degrees	
Factor of safety, FS =	1.5		ULS
At rest pressure coefficient, K_o =	0.50	(as pile is simply supported)	
Surcharge, q =	10.0	kPa	
Lateral earth pressure, p_o =	27.45	kPa (at base of retaining wall)	
Surcharge loading, K_o x q =	5.00	kPa	
Soil loading, F_a =	50.2	kN	
Surcharge loading, F_s =	18.3	kN	
L_a =	1.02	m from base	
L_s =	1.53	m from base	
M* =	43.3	kNm per pile	@ approx. mid span
V* =	42.6	kN	@ base



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CALCULATIONS	REF./COMMENT

OUTPUT

Pile section capacity

See Response2000 output on the next page for design of reinforcement.

ϕM cap =	47.2	kNm	> M*	***0.92 OK***

Design of pile depth

M* =	0.00	kNm @ base	
V* =	42.64	kN	
Ult. horizontal bearing capacity, f_b =	400	kPa	(refer sec.PR c))
C =	120	kN/m	
D = (3.6V*+√(12.96V*^{2}+16.2CM*))/(2C) =	1.28	m	
Depth =	1.30	m	
Total depth req. =	4.35	m	(from bot. of capping beam)
Total depth adopted, Df =	5.80	m	(from bot. of capping beam)
Depth adopted =	2.75	m	(below basement)

Geotechnical strength - Compression (AS 2159 cl. 4.4.1)

$R_{d,ug} = f_{m,s} A_s + f_b A_b$ Df/Db = 14.5 > 4

Type	p*	Load area				
	kPa	m	G =	98	kN/m	
			Q =	18	kN/m	
Precast	4.50	15.5	G+Q =	93	kN/m	Allowbale
Slab	4.25	6.0	1.2G+1.5Q =	144	kN/m	Ult.
Roof	0.45	6.0				
Live	3.00	6.0	E_d =	115	kN (per pile)	

Surface area, A_s =	4.03	m^2		
Base area, A_b =	0.07	m^2		
$R_{d,ug}$ =	243.9	kN		
Geotech strength in compression, $R_{d,g} = \phi_g R_{d,ug}$ =	163.4	kN	> E_d	***0.71 OK***

Geotechnical strength - Tension (AS 2159 cl. 4.4.2)

Design for tension (pullout) was not critical as the structure does not go into uplift under any load case.

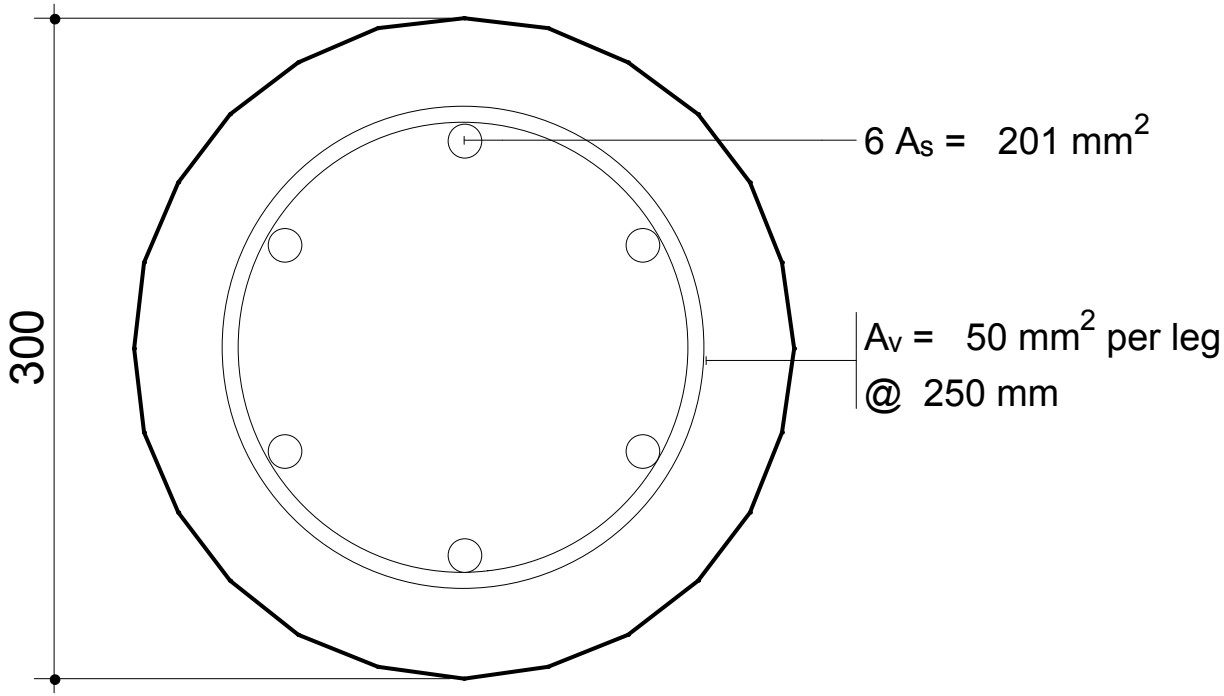
Reinforcement requirements - cl. 5.3.3

6 x N16 vert, A_s =	1206	mm^2		
Min. reo, 0.005Ag =	353	mm^2	< A_s	***OK***
Max. reo, 0.04Ag =	2827	mm^2	> A_s	***OK***

Adopt: 300 diameter piles with 6-N16 vert @ 800mm centres

Geometric Properties

	Gross Conc.	Trans (n=7.79)
Area (mm^2) x 10^3	69.9	78.1
Inertia (mm^4) x 10^6	388.6	424.8
y_t (mm)	150	150
y_b (mm)	150	150
S_t (mm^3) x 10^3	2590.9	2832.0
S_b (mm^3) x 10^3	2590.9	2832.0



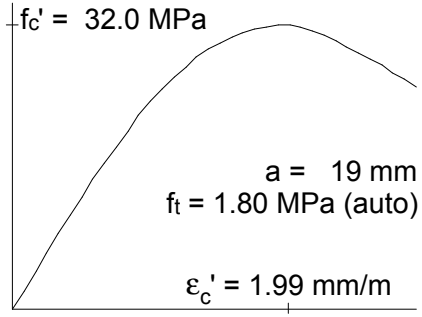
Crack Spacing

2 x dist + 0.1 d_b /ρ

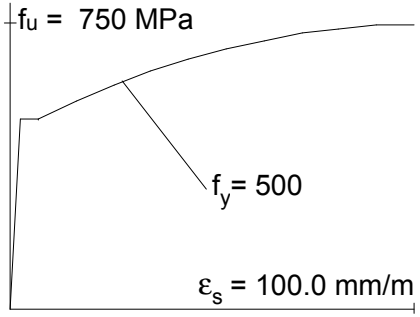
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in millimetres
Clear cover to transverse reinforcement = 40 mm

P1

2018/3/16

Response-2000 v 1.0.5

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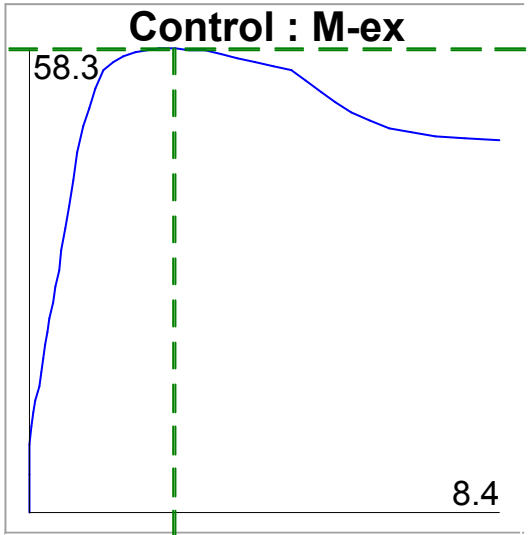
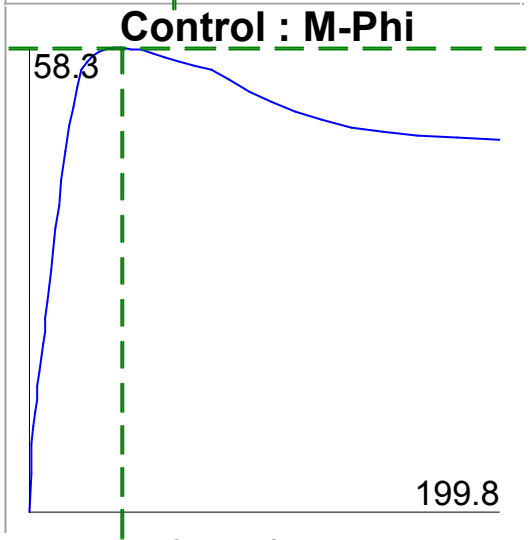
2018/3/16 - 9:41 am





ε_{x0} = 2.59 mm/m

ϕ = 39.53 rad/km

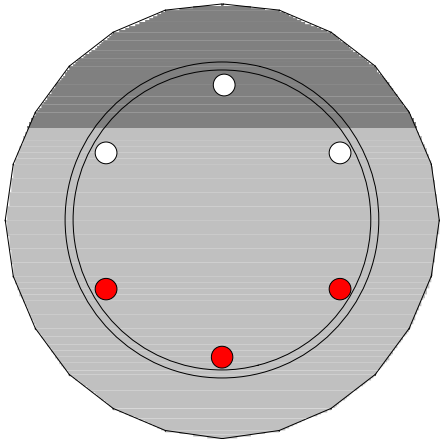
γ_{xy}(avg) = 0.00 mm/m

Axial Load = 0.0 kN

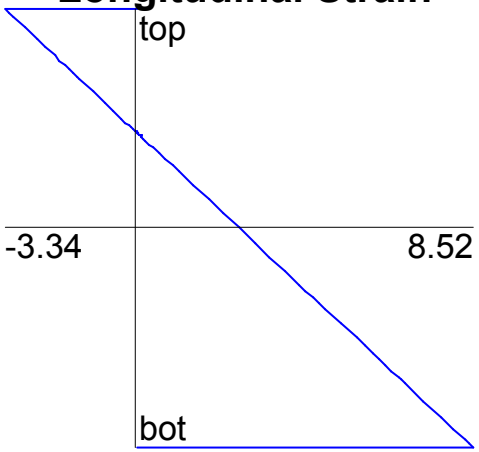
Moment:= 58.3 kNm

Shear = 0.0 kN

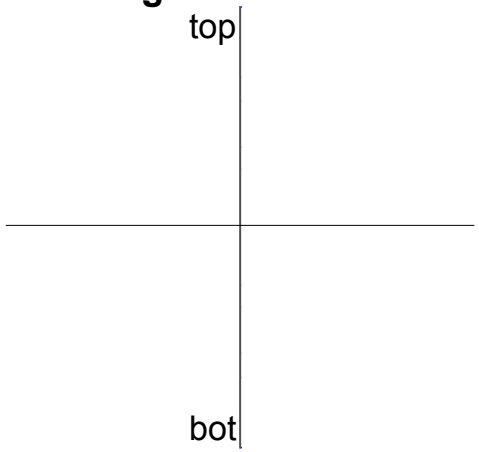
Cross Section



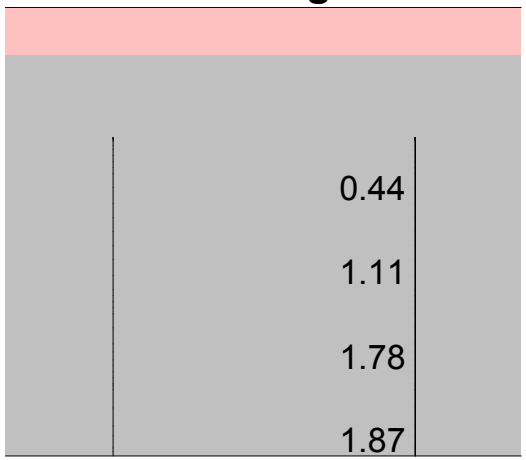
Longitudinal Strain



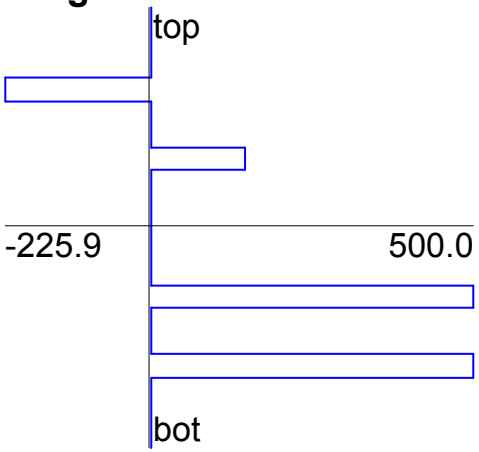
Shrinkage & Thermal Strain



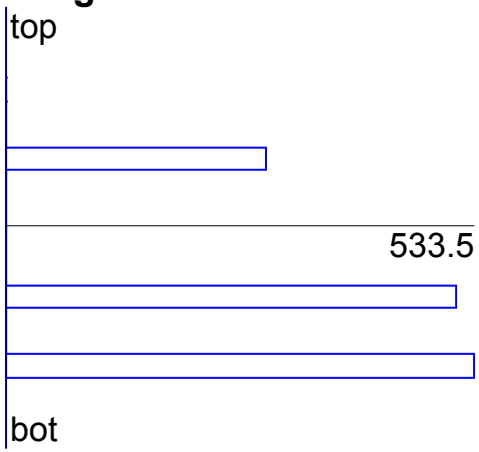
Crack Diagram



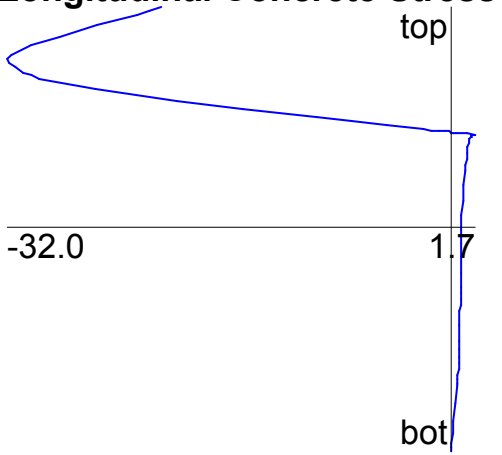
Long. Reinforcement Stress



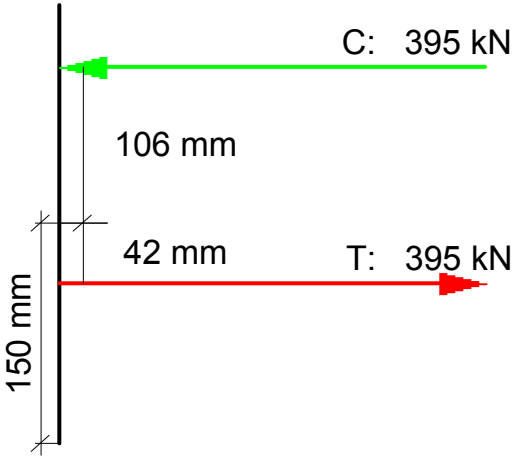
Long. Reinf Stress at Crack



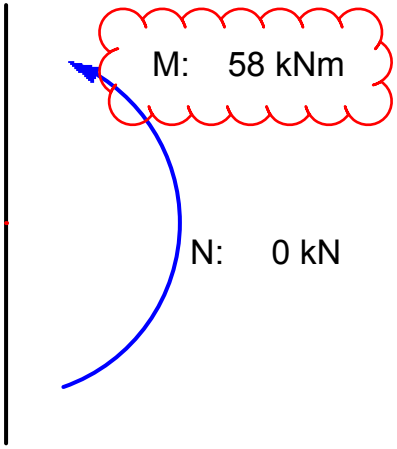
Longitudinal Concrete Stress



Internal Forces



N+M

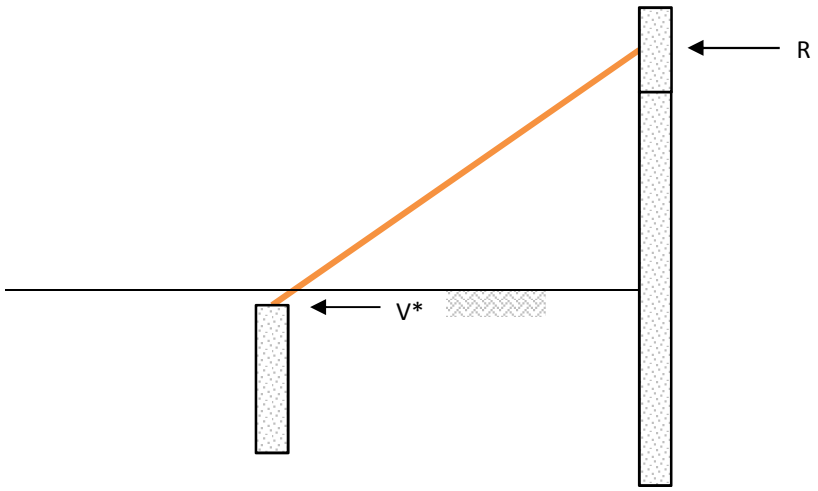


CALCULATIONS	REF./COMMENT

b) Propping Piles - PP

INPUT Diameter, Db = 300 mm

LOADING R = 25.89 kN



OUTPUT V* = 25.89 kN

Ult. horizontal bearing capacity, f_b = 400 kPa — Assumed

C = 120 kN/m

D = (3.6V*+√(12.96V*^{2}+16.2CM*))/(2C) = 0.78 m

Depth = 0.80 m

Adopt: 300mm dia. x 1000 mm deep mass concrete

CALCULATIONS	REF./COMMENT

c) Capping Beam - CB1

Design capping to span inbetween props until ground floor slab is poured.

INPUT

Span =	3.0	m
Vertical loadwidth =	1.53	m

LOADING

Surcharge, q =	10	kPa	
Clay, γ =	18	kN/m^3	
Angle of friction, ϕ =	30	degrees	
Factor of safety, FS =	1.5		ULS
At rest pressure coefficient, K_o =	0.50	(as cap is simply supported)	
Lateral earth pressure, p_o =	13.7	kPa	
Surcharge pressure, K_o x q =	5.0	kPa	
Lateral earth loading, p* =	15.7	kN/m	
Surcharge loading, q* =	11.4	kN/m	
w* = p* + q* =	27.1	kN/m	
$M^* = w^*L^2/8$ =	30.5	kNm	

OUTPUT

Try: 300 W x 600 D
Reinforcement: 3-N12 T & B

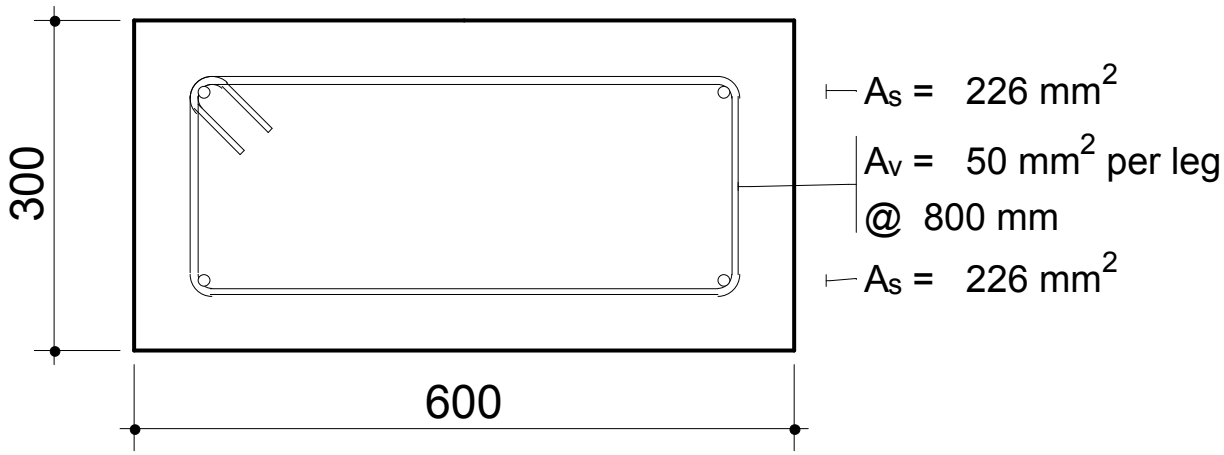
See Response2000 output on the next page for design of reinforcement. As the section is bending laterally about its weak axis the concrete section was analysed as a 600 W x 300 D section with 2-N12 top and bottom, W8-800 ligs.

ϕM cap =	32.8	kNm	> M*	***0.93 OK***

Adopt: 300 W x 600 D, 3-N12 Top and bottom. W8-800 ligs

Geometric Properties

	Gross Conc.	Trans (n=8.51)
Area (mm^2) x 10^3	180.0	183.4
Inertia (mm^4) x 10^6	1350.0	1375.1
y_t (mm)	150	150
y_b (mm)	150	150
S_t (mm^3) x 10^3	9000.0	9167.5
S_b (mm^3) x 10^3	9000.0	9167.5



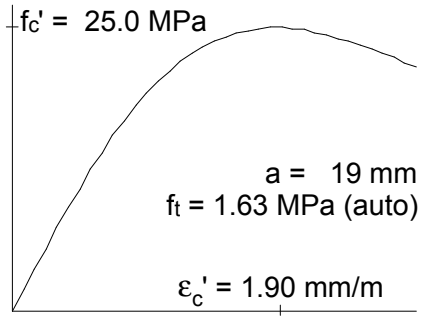
Crack Spacing

2 x dist + 0.1 d_b /ρ

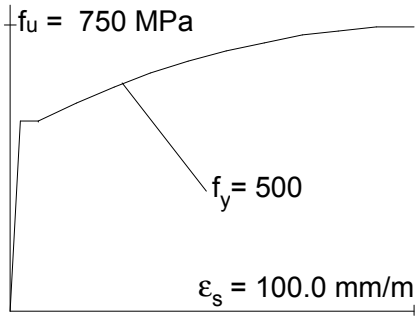
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in millimetres
Clear cover to transverse reinforcement = 50 mm

CB1

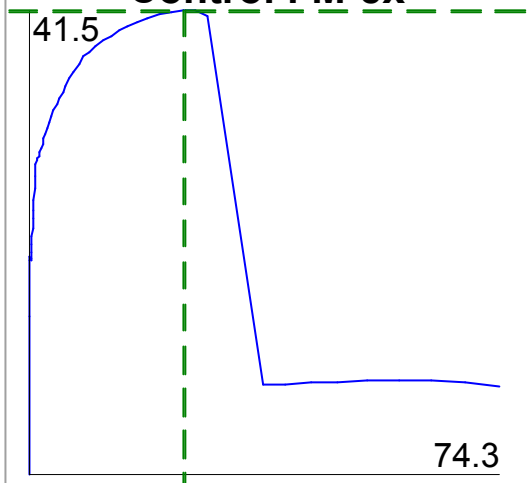
JT **2018/6/6**

Response-2000 v 1.0.5

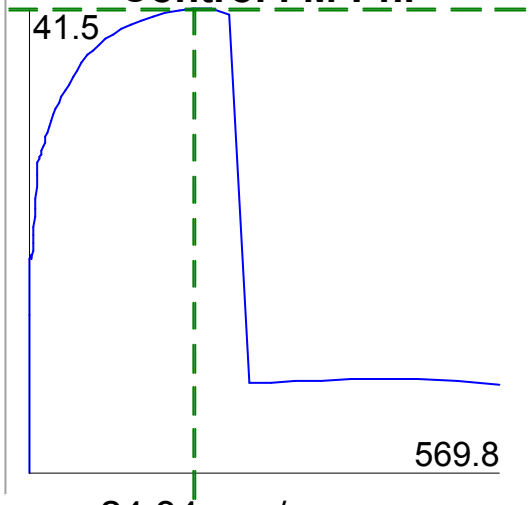
CB1

JT 2018/6/6 - 11:28 am

Control : M-ex



Control : M-Phi



ε_{x0} = 24.34 mm/m

ϕ = 199.70 rad/km

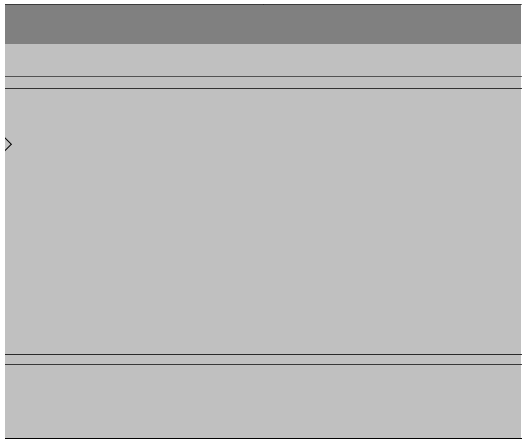
γ_{xy}(avg) = 0.00 mm/m

Axial Load = -0.0 kN

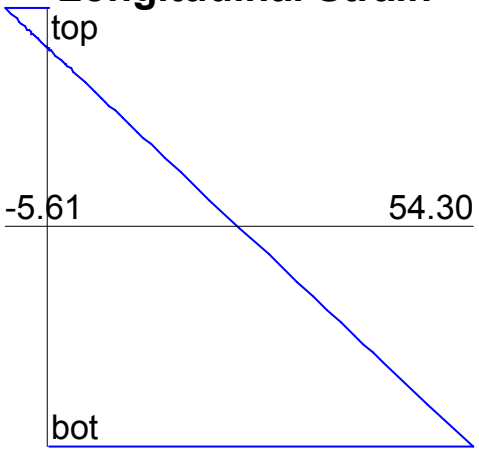
Moment:= 41.5 kNm

Shear = 0.0 kN

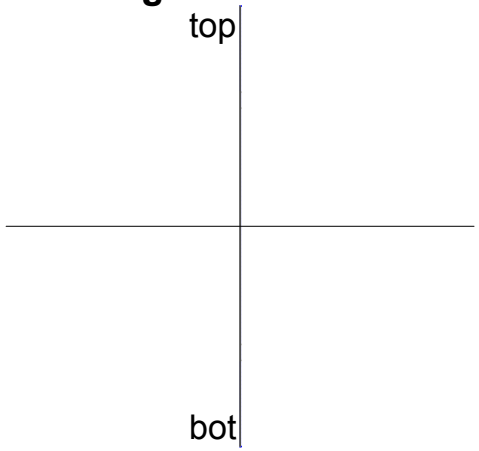
Cross Section



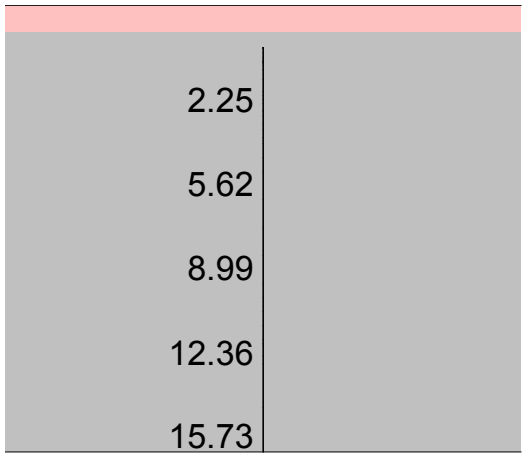
Longitudinal Strain



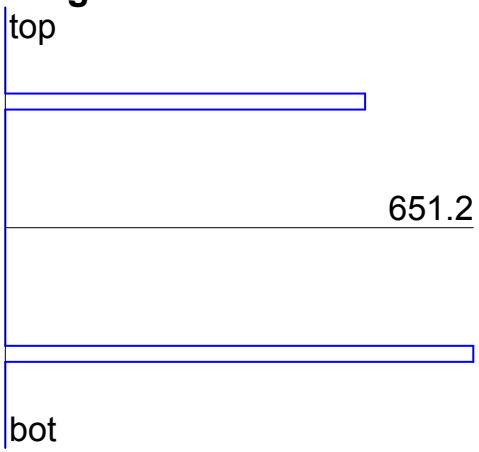
Shrinkage & Thermal Strain



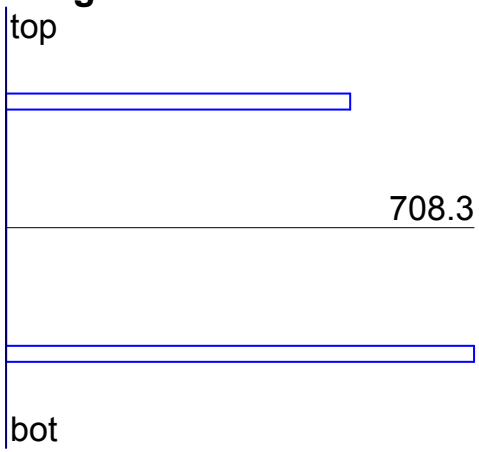
Crack Diagram



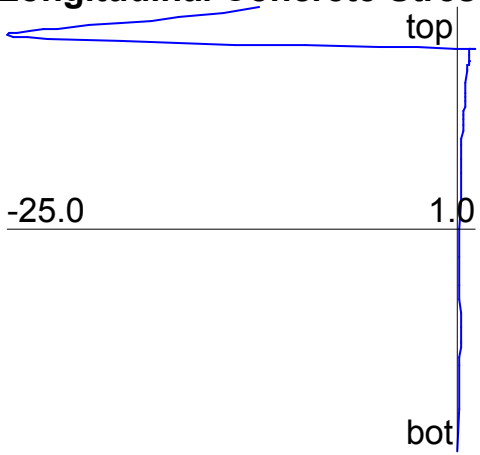
Long. Reinforcement Stress



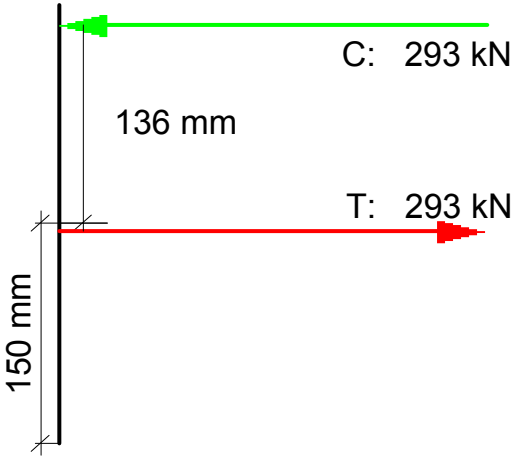
Long. Reinf Stress at Crack



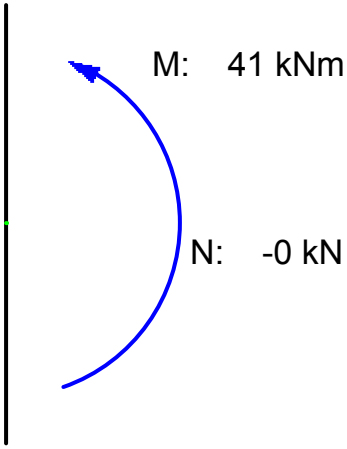
Longitudinal Concrete Stress



Internal Forces



N+M



Reference: 2018-7161
Project: 97 King William Street, Kent Town **Checked by:** BR
Designer: JT Date: 6/06/2018 **Index:** RW - 1

CALCULATIONS				REF./COMMENT
3) RETAINING WALL DESIGN				
Design the retaining walls required in the areas where piles are not present.				
a) In situ conc. retaining wall - RW1				
INPUT	Retaining height, L =	3.20	m	
	Design loadwidth =	1.0	m	
	Surcharge, q =	10	kPa	
	Clay, γ =	18	kN/m^3	
	Angle of friction, ϕ =	30	degrees	
	At rest pressure coefficient, K_o =	0.50	(as simply supported)	
	Factor of safety, FS =	1.5		ULS
LOADING	Lateral earth pressure, p_o =	28.8	kPa at base of retaining wall	
	Surcharge loading, K_o x q =	5.0	kPa	
	Soil loading, F_a =	69.1	kN	
	Surcharge loading, F_s =	24.0	kN	
	L_a =	1.07	m from base	
	L_s =	1.60	m from base	
	M* =	62.0	kNm per pile	@ approx. mid span
	V* =	58.1	kN	@ base
OUTPUT	Thickness, t =	180	mm	
	f'_c =	32	MPa	
	γ =	0.822		
	α_2 =	0.85		
	Cover =	35	mm (to tensile steel)	
	d_o =	145	mm	
	C_c =	670	kN	
	Reinforcement yield strength, f_{sy} =	500	MPa	
	A_{st} =	1340	mm^2	N16-150 vert
	Assuming tensile steel yields, $T_s = A_{st}f_{sy}$ =	670	kN	
	As $C_c = T_s$, solve for d_n		0.00	
	d_n =	29.97	mm	
Taking moments about reinforcement	M_u =	87.1	kNm	
	$\phi M_u = 0.8M_u$ =	69.7	kNm > M*	***0.89 OK***

Adopt: 180 thk. N16-150 vert, N12-400 horiz (inner face). SL72 (outer face).

Reference: 2018-7161
Project: 97 King William Street, Kent Town **Checked by:** BR
Designer: JT Date: 6/06/2018 **Index:** RW - 2

CALCULATIONS				REF./COMMENT
b) 100 thk precast adjacent to ramp				
INPUT	Retaining height, L =	2.35	m (Ave. ret height for worst case panel)	
	Design loadwidth =	1.0	m	
	Surcharge, q =	10	kPa	
	Clay, γ =	18	kN/m^3	
	Angle of friction, ϕ =	30	degrees	
	Active pressure coefficient, K_a =	0.50		
	Factor of safety, FS =	1.5		
LOADING	Lateral earth pressure, p_a =	21.15	kPa at base of retaining wall	
	Surcharge loading, K_a x q =	5.00	kPa	
	Soil loading, F_a =	37.3	kN	
	Surcharge loading, F_s =	17.6	kN	
	L_a =	0.78	m from base	
	L_s =	1.18	m from base	
	M* =	26.4	kNm per pile	@ approx. mid span
	V* =	33.7	kN	@ base
OUTPUT	Thickness, t =	100	mm	
	f'_c =	50	MPa	
	γ =	0.696		
	α_2 =	0.85		
	Cover =	50	mm (to tensile steel)	
	d_o =	50	mm	
	C_c =	1005	kN	
	Reinforcement yield strength, f_{sy} =	500	MPa	
	A_{st} =	2010	mm^2	N16-100 vert
	Assuming tensile steel yields, $T_s = A_{st}f_{sy}$ =	1005.00	kN	
	As $C_c = T_s$, solve for d_n		0.00	
	d_n =	34.0	mm	
Taking moments about reinforcement	M_u =	33.2	kNm	
	$\phi M_u = 0.8M_u$ =	26.5	kNm > M*	***0.99 OK***

Adopt: 100 thk precast wall with N16-100 vert. N12-400 horiz. Central.

CALCULATIONS	REF./COMMENT

4) FOOTING DESIGN

a) Typical Strip Footing Design - S1

Design strip footings for supporting precast panels that don't form the lift shaft for bearing only. Reinforcement requirements as per AS 3600.

INPUT

Dimensions

Width, b =	700	mm
Depth, d =	400	mm
Self weight =	700	kg/m

Concrete properties

f'_c =	25	MPa
f'_{cf} =	3.0	MPa
f_{sy} =	500	MPa
E_s =	200000	MPa

Top steel

No. =	4	
Bars diameter =	12	mm
Cover =	35	mm

Bottom steel

No. =	4	
Bars diameter =	12	mm
Cover =	50	mm

Soil properties Allowable bearing capacity, q_{all} = 200 kPa

LOADING

Bearing load

Type	p* kPa	Loadwidth m
Precast	4.50	7.6
Slab	4.25	3.0
Live	3.00	3.0

*No roof loading as the roof spans to outside walls

Self weight =	6.87	kN/m	
G =	92	kN/m	
Q =	36	kN/m	
1.2G+1.5Q, w =	164	kN/m	ULS
Alllowable, w_{all} = G+Q =	128	kN/m	Allowable load

OUTPUT

Check bearing per metre

Base width, A =	0.70	m		
Allowable capacity, q* =	140	kN/m	> w_{all}	***0.91 OK***

Check minimum reinforcement cl. 16.3.1

$$\rho_{min} = 0.19\left(\frac{D}{d}\right)^2 \frac{f'_{ct.f}}{f_{sy}}$$

ρ_{min} =	0.0015			
A_{st} =	905	mm^2		
ρ =	0.0032		> ρ_{min}	***0.46 OK***

Adopt: 700 x 400, 4-N12 top and bottom, W8-1000 ligs.

CALCULATIONS	REF./COMMENT

5) PR - DESIGN OF PRECAST PANELS — AS 3600 Sec. 11

a) Short direction internal panels

Lateral force distribution in the short direction. Refer to precast element layout sketch.

i) Stiffness

$k = 3EI/L^3$

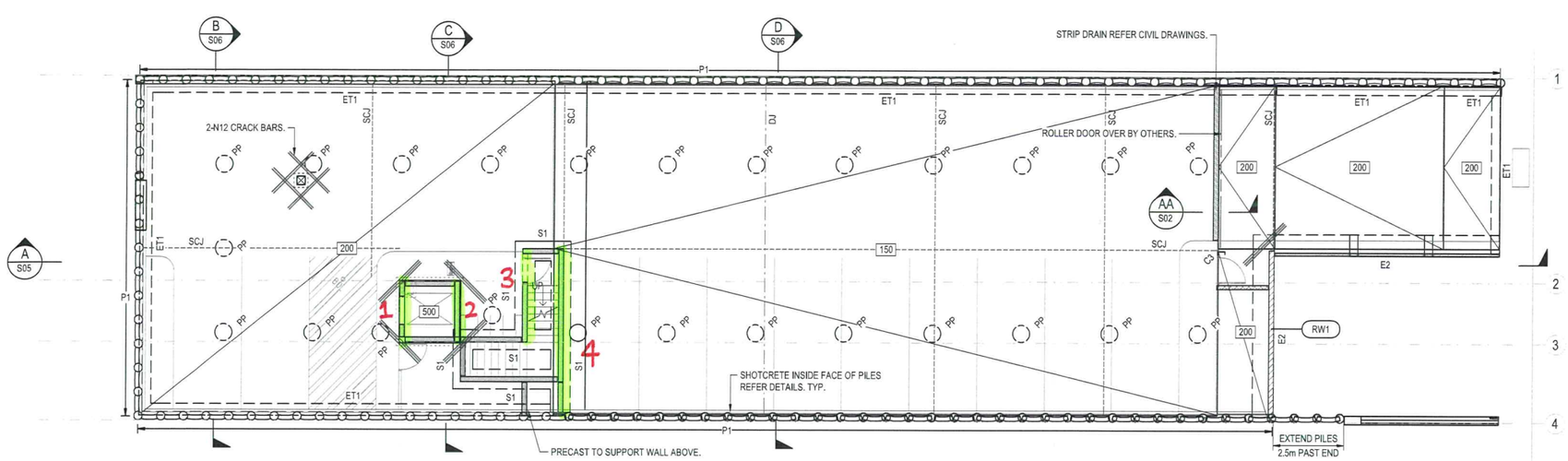
E = 34800 MPa

BASEMENT - GROUND						
Element	Thk, b	Length, d	I	Height, L	k	Ratio
	mm	mm	mm^4	mm		
1	*180*	*2360*	*1.97E+11*	*3800*	*375125.8*	*0.06*
2	*180*	*2360*	*1.97E+11*	*3800*	*375125.8*	*0.06*
3	180	3200	4.92E+11	3800	935170.7	0.14
4	180	5600	2.63E+12	3800	5011931	0.75
GROUND - LEVEL 1						
Element	Thk, b	Length, d	I	Height, L	k	Ratio
	mm	mm	mm^4	mm		
1	*180*	*2360*	*1.97E+11*	*3850*	*360699.5*	*0.18*
2	*180*	*2360*	*1.97E+11*	*3850*	*360699.5*	*0.18*
3	180	3210	4.96E+11	3850	907663.1	0.46
4	180	2320	1.87E+11	3850	342668	0.17
LEVEL 1 - LEVEL 2						
Element	Thk, b	Length, d	I	Height, L	k	Ratio
	mm	mm	mm^4	mm		
1	*180*	*2360*	*1.97E+11*	*3500*	*480091.1*	*0.50*
2	*180*	*2360*	*1.97E+11*	*3500*	*480091.1*	*0.50*
LEVEL 2 - LEVEL 3						
Element	Thk, b	Length, d	I	Height, L	k	Ratio
	mm	mm	mm^4	mm		
1	*180*	*2360*	*1.97E+11*	*3500*	*480091.1*	*0.50*
2	*180*	*2360*	*1.97E+11*	*3500*	*480091.1*	*0.50*
LEVEL 3 - ROOF						
Element	Thk, b	Length, d	I	Height, L	k	Ratio
	mm	mm	mm^4	mm		
1	*180*	*2360*	*1.97E+11*	*4600*	*211472.7*	*0.50*
2	*180*	*2360*	*1.97E+11*	*4600*	*211472.7*	*0.50*

**Italics = lift shaft panels*

When considering earthquake in the short direction, 100% of the load was considered. As per AS 1170.4, 30% of the load needs to be considered in the other direction. This load is assumed to be resisted by the longitudinal panels on the either side of the building.

PRECAST ELEMENT LAYOUT (SHORT DIRECTION)



CONSTRUCTION SEQUENCE FOR PILES

1. CONSTRUCT CONTIGUOUS PILES AN CAPPING BEAM.
2. CONSTRUCT PROPPING PIERS.
3. DIG TRENCH BETWEEN PROPPING PIERS AND CAPPING BEAM.
4. PROP CAPPING BEAM AT EACH PROPPING PIER LOCATION
5. EXCAVATE BASEMENT
6. CONTRUCT GROUND FLOOR SLAB TO RESTRAIN PILES.
7. REMOVE PROPS AFTER 28 DAYS POST GROUND FLOOR CONSTRUCTION.
8. CONSTRUCT BASEMENT.

CONCRETE LEGEND

- WET AREA OR BALCONY SET DOWN TO BUILDING. 50mm. REFER BALCONY DETAILS FOR FALLS.
- 200 — INDICATES 200mm CONCRETE SLAB ON 100mm COMPACTED QUARRY RUBBLE. SL92 TOP AND SL72 BOTTOM. REFER TO CIVIL DRAWINGS FOR TOP OF SLAB LEVELS AND SLOPES.
- 170 — INDICATES 170mm CONCRETE SLAB ON KF57 1.0 BMT. SL82 TOP FACE. REINFORCEMENT AS SHOWN OR N16-200 U.N.O. 1-N16 BOTTOM EACH PAN. 35mm COVER.
- 130 — INDICATES 130mm MIN CONCRETE SLAB ON KF57 1.0 BMT. SL82 TOP FACE. REINFORCEMENT AS SHOWN OR N16-200 U.N.O. 1-N16 BOTTOM EACH PAN. 35mm COVER. BALCONY SLAB TAPERS 1:100 FALL.
- 120 — INDICATES 120mm CONCRETE SLAB ON KF57 1.0 BMT. SL82 TOP FACE. REINFORCEMENT AS SHOWN OR N16-200 U.N.O. 1-N16 BOTTOM EACH PAN. 35mm COVER.
- 130 — INDICATES 130mm MIN OFF FORM CONCRETE SLAB. SL82 TOP & BOTTOM. ADDITIONAL BARS AS NOTED ON PLAN. BALCONY SLAB TAPERS 1:100 FALL.
- PROVIDE 3-N12-T CRACK CONTROL RODS, 2000mm LONG UNDER MESH AT ALL RE-ENTRANT CORNERS. TYPICAL.
- ST — INDICATES STEP IN CAPPING BEAM. REFER TO DETAILS.
- INTICATES DIRECTION OF SHEETING.
- INDICATES PENETRATION THROUGH SLAB.
- INDICATES DIRECTION TOP BARS OF MESH MUST RUN. ADDITIONAL TOP REINFORCEMENT TO RUN IN LINE WITH MESH.
- INDICATES STEP IN SLAB
- RAMP — INDICATES RAMP. REFER TO ARCHITECTS DRAWINGS.
- SCJ — INDICATES SAW CUT JOINT. REFER TYPICAL DETAIL.
- DJ — INDICATES DOWELLED JOINT. REFER TYPICAL DETAIL.
- PP — INDICATES PROPPING PIERS. ø300mm 2.3m DEEP MASS CONCRETE. MAX 3.0m C/C SET IN 3.5m.
- INDICATES 180THK PRECAST WALLS.
- INDICATES 140 BLOCKWORK WALLS. N12-600 CENTRAL E.W.
- INDICATES ARCHITECTURAL NON-LOADBEARING STUDWORK.
- INDICATES SUMP LOCATIONS. REFER TO CIVIL DRAWINGS.
- RW1 — INDICATES 180THK CAST IN SITU WALL. N12-200 (V) BARS, N12-400 (H) BARS INTERNAL FACE, SL72 EXTERNAL FACE. f'c = 32 MPa. PROP UNTIL GROUND FLOOR SLAB IS POURED AND HAS ACHIEVED FULL DESIGN STRENGTH.
- B#(U) — INDICATES BEAM UNDER.

FOOTING SCHEDULE

MEMBER	SIZE	REMARKS
	390Wx600D	
CB1	300Wx600D	CAPPING BEAM , REFER DETAILS
E2	750Wx300D	4-N12 T&B, W8-300 CTS.
ET1	300Wx300D	2-N12 T&B, W8-1000 CTS
S1	700Wx400D	4-N12 T&B, W8-1000 CTS. SET BELOW SLAB
S2	300Wx600D	3-N12 T&B, W8-1000 CTS.

CONTIGULATED PILE SCHEDULE

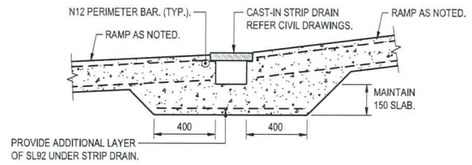
MEMBER	SIZE	REMARKS
P1	Ø300	PILES, 5.8m DEEP BELOW BOTTOM OF CAPPING BEAM, 6-N16 VERTICAL, W8-250 HOOPS CTS. 0.8m CTS

BASEMENT LAYOUT PLAN

SCALE 1 : 100

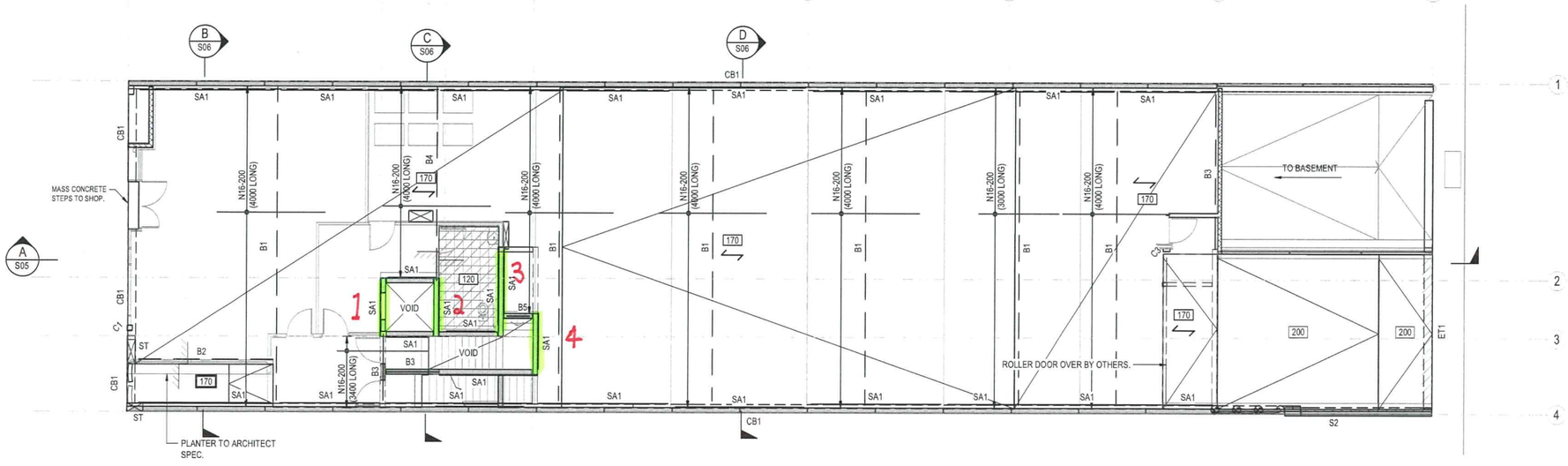
WATERPROOFING NOTES

1. FOR ALL CONCRETE IN BASEMENT AREA USE XYPEX ADDITIVE C-1000 REFER TO MANUFACTURERS SPECIFICATIONS FOR INSTALLATION DETAILS.



SECTION AA S02

SCALE 1 : 20



GROUND LEVEL LAYOUT PLAN

SCALE 1 : 100

FRAMING SCHEDULE

MEMBER	SIZE	REMARKS
B1	610UB101	COMPOSITE FLOOR BEAM - ø19mm STUDS @ 250 CTS, PRE-CAMBER 40mm.
B2	380PFC	FLOOR BEAM
B3	310UB32	FLOOR BEAM
B4	530UB82	COMPOSITE FLOOR BEAM - ø19mm STUDS @ 220 CTS, NO PRECAMBER
B5	200PFC	FLOOR BEAM
B6	310UC137	COMPOSITE FLOOR BEAM - ø19mm STUDS @ 250 CTS, PRE-CAMBER 40mm.
B7	610UB125	COMPOSITE FLOOR BEAM - ø19mm STUDS @ 230 CTS, PRE-CAMBER 40mm.
BR1	Ø20 ROD BRACING	ROOF BRACING FIX TO SEATING ANGLE WITH 1-M20 8.8/S BOLT. PROVIDE HANGER OFF OF PURLINS TO REDUCE SAG.
C1	200x200x5.0 SHS	COLUMN
C2	200x200x5.0 SHS	COLUMN
C3	250PFC	FIXED TO FLOOR SLAB AT BOTH LEVELS
FB1	C30024	FASCIA BEAM FIXED TO OUTRIGGERS
FB2	360UB45	FLOOR BEAM
FB3	250UB26	FLOOR BEAM
H1	89x89x3.5 SHS	HANGER @ 1200 CTS
L1	360UB45	LINTEL
L2	310UB32	LINTEL
OR1	Z30030	OUTRIGGER AT 1200 CTRS
PL1	Z30030	PURLINS AT 1200 CTRS 2 ROWS OF BRIDGING
RB1	300PFC	ROOF BEAM
SA1	90x90x6 EA	SEAT ANGLE
WB1	250PFC	WALL HEAD ON FLAT

NOTES:

1. BEAMS MARKED COMPOSITE IN STEELWORK SCHEDULE SHALL BE COMPOSITE WITH THE SLAB
2. FILDER KF57 TO BE MIN 2 SPANS LONG WHERE APPLICABLE.
3. SHEAR STUDS TO BE ø19mm x 95mm HIGH HEADED STUDS, REFER SCHEDULE FOR CENTRES.

PROPPING NOTES:

1. SLAB SPAN BETWEEN 2.5-5m SHALL REQUIRE 1 ROW OF PROPS AT MIDSPAN.
2. SLAB SPANS BETWEEN 5-6m SHALL REQUIRE 2 ROWS OF PROPS AT THIRD SPANS.

Issue	Date	Amendment
D	1/6/18	CERTIFICATION
C	29/5/18	CERTIFICATION
B	26/4/18	PRELIMINARY WIP
A	29/3/18	PRELIMINARY WIP

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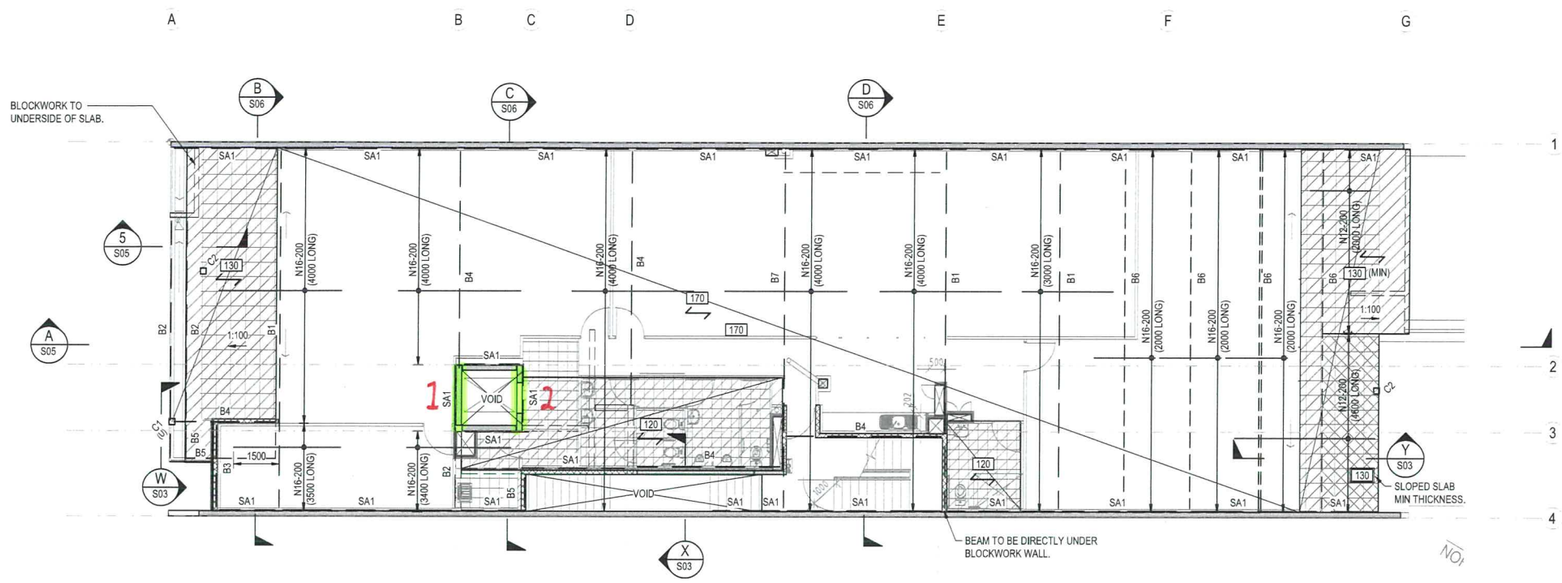
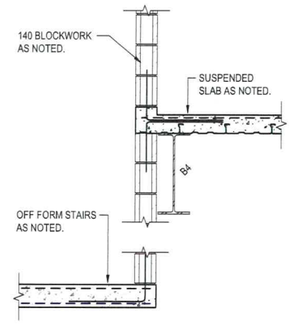
97 KING WILLIAM STREET
KENT TOWN SA
ANTHONY DONATO ARCHITECTS
BASEMENT AND GROUND FLOOR LAYOUT PLANS

Drawn	SGP	Scale	As indicated on A1
Design	JT	Drawing Number	2018-7161 S02
Approved			
Date	MARCH 18		



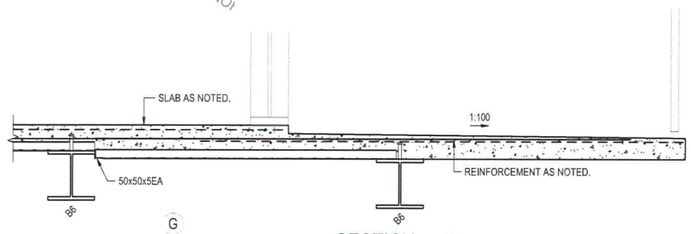
FIRST FLOOR LAYOUT PLAN

SCALE 1 : 100



SECTION

SCALE 1 : 20



SECTION

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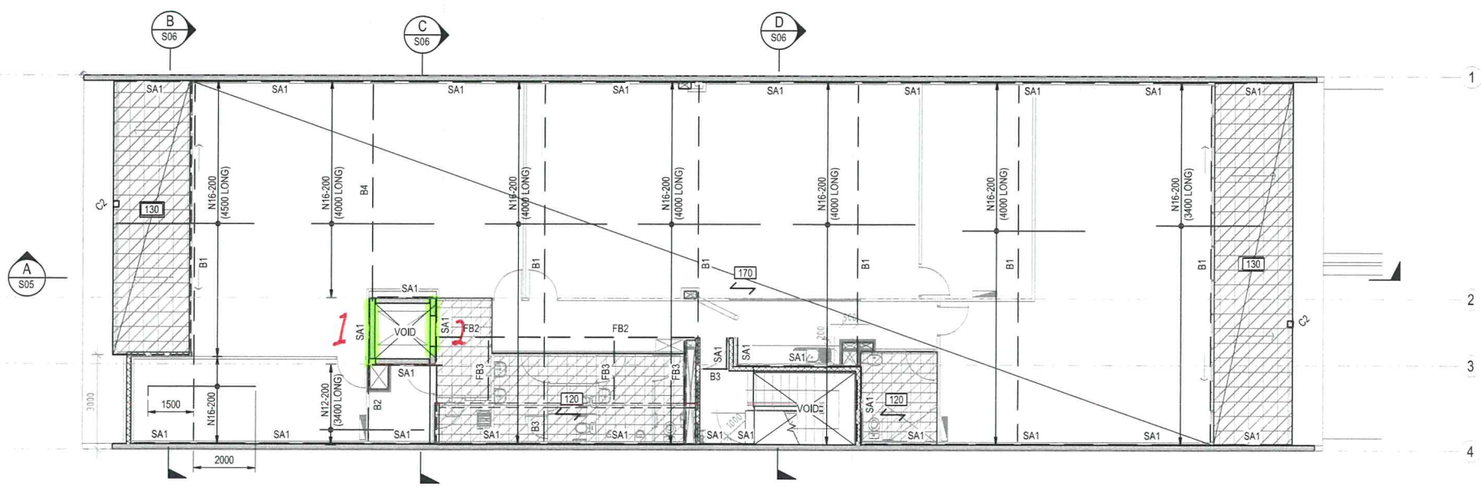
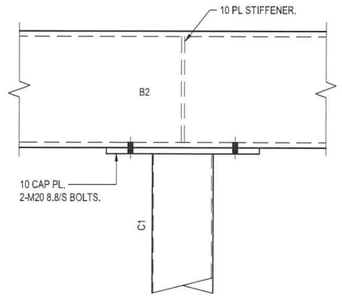


SECOND FLOOR LAYOUT PLAN

SCALE 1 : 100



SECTION

SCALE 1 : 10

CONCRETE LEGEND

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- 200 — INDICATES 200mm CONCRETE SLAB ON 100mm COMPACTED QUARRY RUBBLE. SL92 TOP AND SL72 BOTTOM. REFER TO CIVIL DRAWINGS FOR TOP OF SLAB LEVELS AND SLOPES.
- 170 — INDICATES 170mm CONCRETE SLAB ON KF57 1.0 BMT. SL82 TOP FACE. REINFORCEMENT AS SHOWN OR N16-200 U.N.O. 1-N16 BOTTOM EACH PAN. 35mm COVER.
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- 130 — INDICATES 130mm MIN OFF FORM CONCRETE SLAB. SL82 TOP & BOTTOM. ADDITIONAL BARS AS NOTED ON PLAN. BALCONY SLAB TAPERS 1:100 FALL.
- PROVIDE 3-N12-T CRACK CONTROL RODS, 2000mm LONG UNDER MESH AT ALL RE-ENTRANT CORNERS. TYPICAL.
- S1 — INDICATES STEP IN CAPPING BEAM. REFER TO DETAILS.
- INTICATES DIRECTION OF SHEETING.
- INDICATES PENETRATION THROUGH SLAB.
- INDICATES DIRECTION TOP BARS OF MESH MUST RUN. ADDITIONAL TOP REINFORCEMENT TO RUN IN LINE WITH MESH.
- INDICATES STEP IN SLAB
- RAMP — INDICATES RAMP. REFER TO ARCHITECTS DRAWINGS.
- SCJ — INDICATES SAW CUT JOINT. REFER TYPICAL DETAIL.
- DJ — INDICATES DOWELLED JOINT. REFER TYPICAL DETAIL.
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- B#(U) — INDICATES BEAM UNDER.

FRAMING SCHEDULE

MEMBER	SIZE	REMARKS
B1	610UB101	COMPOSITE FLOOR BEAM - ø19mm STUDS @ 250 CTS, PRE-CAMBER 40mm.
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B5	200PFC	FLOOR BEAM
B6	310UC137	COMPOSITE FLOOR BEAM - ø19mm STUDS @ 250 CTS, PRE-CAMBER 40mm.
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C1	200x200x5.0 SHS	COLUMN
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RB1	300PFC	ROOF BEAM
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WB1	250PFC	WALL HEAD ON FLAT

Issue	Date	Amendment
C	29/5/18	CERTIFICATION
B	26/4/18	PRELIMINARY WIP
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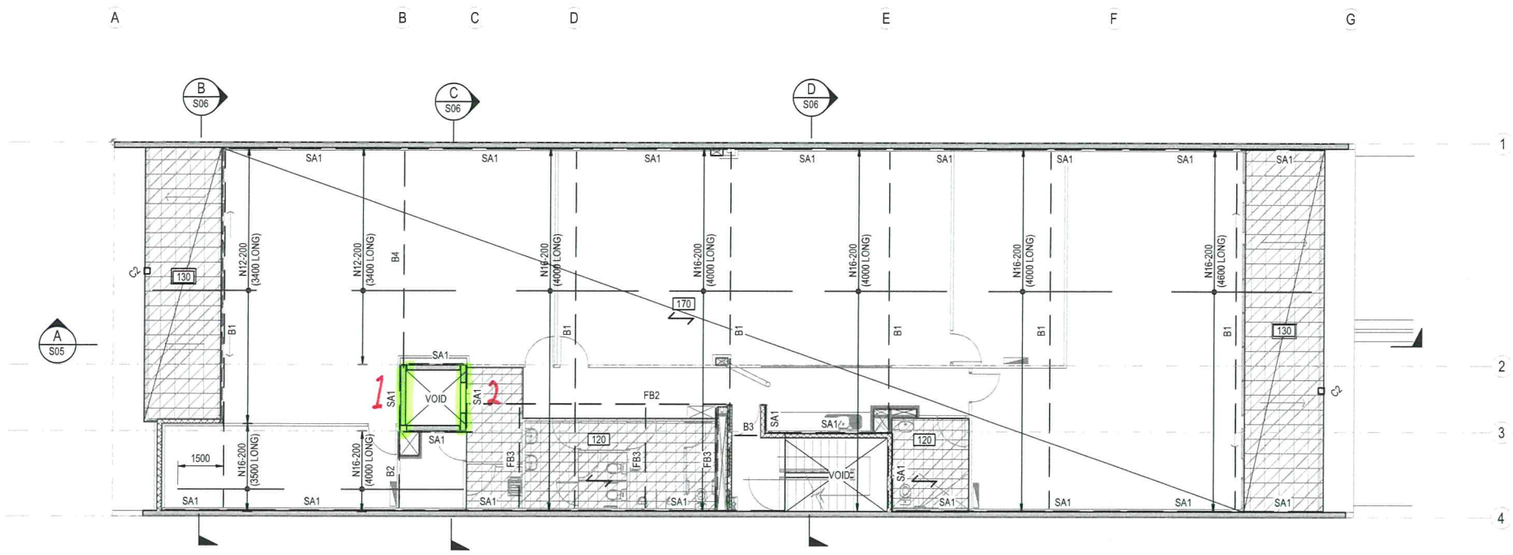
KENT TOWN SA

ANTHONY DONATO ARCHITECTS

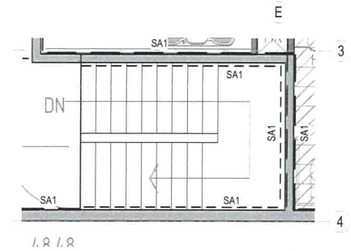
FIRST FLOOR AND SECOND FLOOR LAYOUT PLANS

Drawn	SGP	Scale	As indicated on A1
Design	JT	Drawing Number	2018-7161 S03
Approved			
Date	MARCH 18		



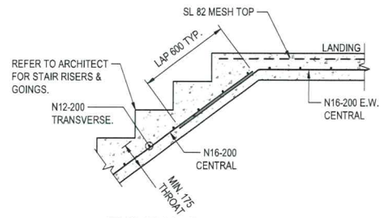
THIRD FLOOR LAYOUT PLAN

SCALE 1 : 100



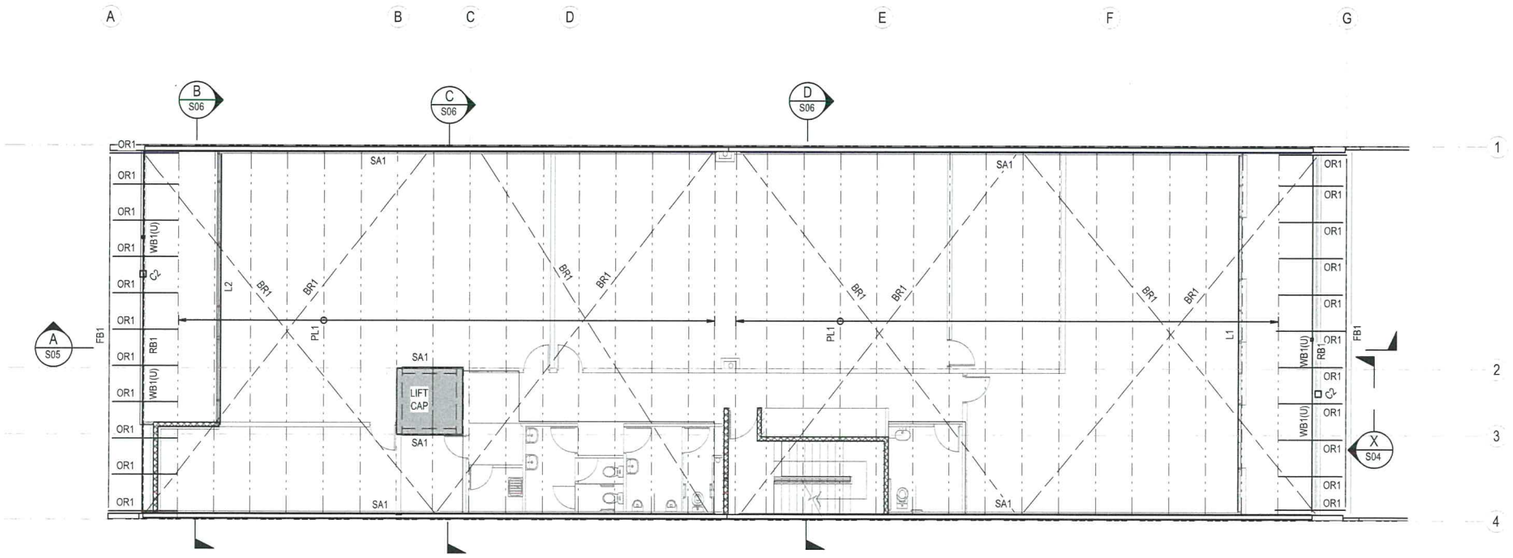
TYPICAL STAIR PLAN

SCALE 1 : 50



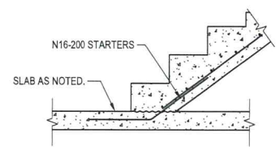
TYPICAL STAIR DETAIL

SCALE 1:20



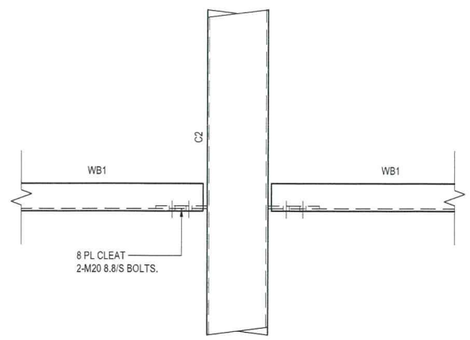
ROOF FRAMING PLAN

SCALE 1 : 100



TYPICAL STAIR TO SLAB DETAIL

SCALE 1:20



SECTION X S04

SCALE 1 : 10

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Issue	Date	Amendment
D	29/5/18	CERTIFICATION
C	15/5/18	PRELIMINARY WIP
B	26/4/18	PRELIMINARY WIP
A	29/3/18	PRELIMINARY WIP

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KENT TOWN SA

ANTHONY DONATO ARCHITECTS

THIRD FLOOR AND ROOF FRAMING LAYOUT PLAN

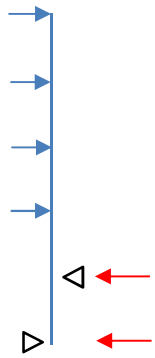
Drawn	SGP	Scale	As indicated on A1
Design	JT	Drawing Number	2018-7161 S04
Approved			
Date	MARCH 18		

CALCULATIONS	REF./COMMENT

ii) *Loading*

It was assumed that the worst shear in the short direction is resisted by the lift shaft elements. Using the stiffness ratios above the governing earthquake forces per level calculated earlier are broken down to give the forces in the lift shaft elements parallel to the short direction. The lift shaft or core is analysed as a pinned-pinned base, where one support is at ground level and the other at basement.

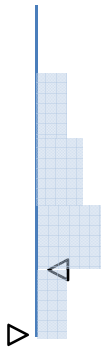
FORCES



Roof	0 kN		15.45 m
Level 3	408 kN		10.85 m
Level 2	276 kN		7.35 m
Level 1	53 kN		3.85 m
Ground		-299 kN	0 m
Basement		-437 kN	-3.80 m

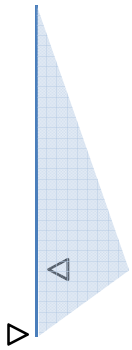
The roof load is not resisted by the lift shaft as it is assumed that the external panels (parallel with the long direction) act as cantilevers above level 3 and resist the roof load about their weak axis.

SHEAR



Roof	0 kN	15.45 m
Level 3	408 kN	10.85 m
Level 2	684 kN	7.35 m
Level 1	737 kN	3.85 m
Ground	437 kN	0 m
Basement	0 kN	-3.80 m

OVERTURNING MOMENT *(M_o)*



Roof		15.45 m
Level 3		10.85 m
Level 2		7.35 m
Level 1		3.85 m
Ground	6656 kNm	0 m
Basement		-3.80 m

CALCULATIONS	REF./COMMENT

The overturning moment calculated above, is resisted by the self weight of the structure supported by the precast, plus the factored live load acting on the structure. Again roof load was neglected. The area contributing to the resisting moment is considered to be slightly larger than the loadwidth of the lift shaft panels as the structural system is interconnected with the lift shaft and during an earthquake the majority of the structure would have to overturn for failure to occur.

VERTICAL LOADS (per level)

Slab =	4.25 kPa x	85.4 m^2 =	362.95 kN
Precast =	4.50 kPa x	64.4 m^2 =	289.8 kN
Live =	3.00 kPa x	42.7 m^2 =	128.1 kN

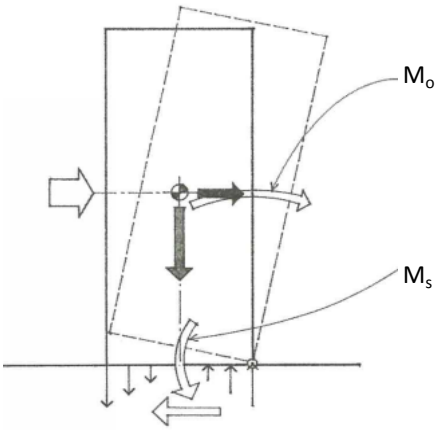
Self weight =	164.1 kN	
G =	817 kN	
Q =	128 kN	
G+0.3Q, w =	855 kN (per level)	
Total, w =	3421 kN	(4 levels incl. ground)

STABILISING MOMENT

$M_s = wL_a$

Lever arm, L_a = 1180 mm (half the width of panel)

ϕM^*_a = 3633 kNm (ϕ = 0.9)



RESULTANT MOMENT *(in-plane)* M* = 3023 kNm

DESIGN ACTIONS IN PANEL

V* = 737 kN

M* = 3023 kNm

Under G+0.3Q and from overturning moment, N_c* = 543 kN/m

The panel undergoes uplift and in-plane bending. Compressive force (N_c*) in the panel is largest from the overturning moment.

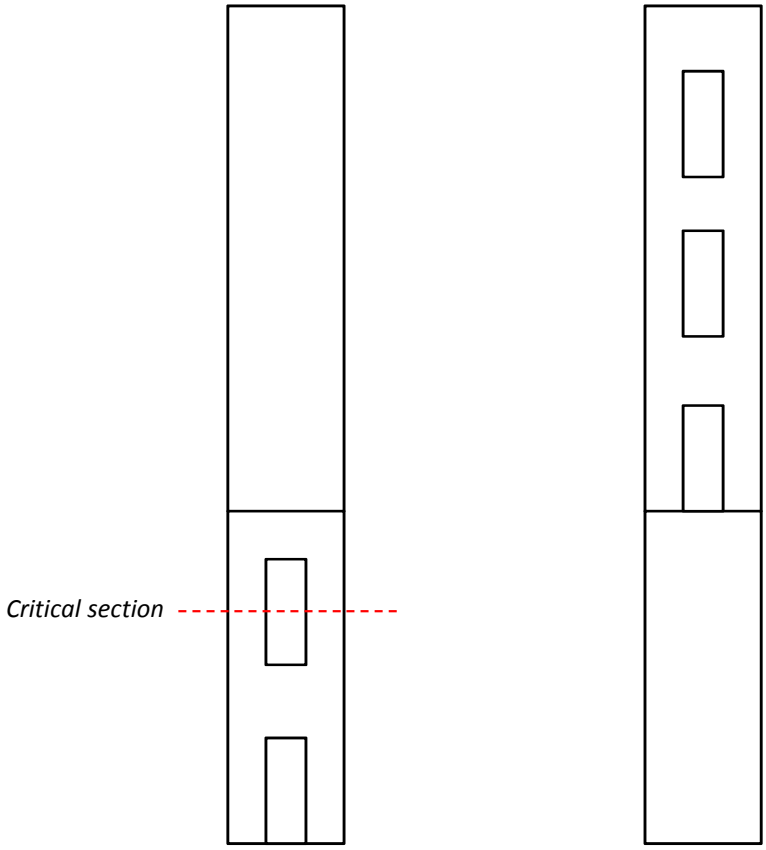
CALCULATIONS	REF./COMMENT

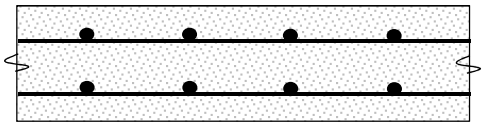
iii) *Output*

GEOMETRY/PROPERTIES

Span, H_w =	3850	mm	(worst case floor-to-floor)
Length, L_W =	2360	mm	
=	1180	mm	(small section with void)
Thickness, t_w =	180	mm	
f'_c =	50	MPa	
Refer to "Structural Toolkit" output E =	34800	MPa	

SCHEMATIC





Adopt: 180 thk, f'c = 50 MPa, N16-150 Horiz, N12-350, EF

Geometry:	**(Short direction lift shaft panel) Hwe = 3850mm, 180mm thick, f'c=50MPa**
Reinf't:	**N12-350 cts - each face vertical, N16-150 cts - each face horizontal**
Capacity:	**N* = 1051kN/m < øNu = 1406kN/m, Hwe/tw = 21.4** **OK (0.75)**
FRL:	**FRL = 120 minutes (Refer Cl 5.7.4 for additional limitations for chases and recesses)**

Geometry **σ.mhi ≥ 0 MPa (all compression under in-plane) - Design as wall - Cl 11.1(a) (wall or column - Cl 11.2.1(a))**

Concrete strength (f'c) =	50	MPa				
			Load eccentricity =	30.0	mm	
Wall thickness (tw) =	180	mm	Min. Ecc = 0.05*tw =	9.0	mm	Cl 11.5.2
Height of wall (Hw) =	3850	mm	ea = Hwe²/(2500*tw) =	32.9	mm	Cl 11.5.1
Effective height factor (k) =	1.00	(Refer below)	Design ecc. (e) =	30.0	mm	
Effective height (Hwe = k*Hw) =	3850	mm	30 * tw =	5400	mm	
Length of wall (Lw) =	1180	mm	50 * tw =	9000	mm	
Cover to outer bars =	35	mm	Hwe/tw =	21.4		
Axis distance (as) =	41	mm Cl 5.2.2				
			Formwork =	S	(S)tandard,(R)igid	
Ductile shear wall =	Y	(Y)es,(N)o	Exposure =	B2	Tab 4.10.3.2	

Loading Wall under in-plane uniform compression

Dead load (Ndl) =	438	kN/m	Load type =	F	(F)loor,(S)tore,(R)oof,(O)ther
Live load (Nll) =	350	kN/m	Long term LL factor (Ψl.f) =	0.60	Table 4.1 (Concentrated)
Mid height SWt =	8.7	kN/m			
Axial comp. N* = 1.2*Ndl+1.5*Nll =	1050.6	kN/m	Include S.Wt =	N	(Y)es,(N)o
Axial comp. fire N*f = Ndl+Ψl.f*Nll =	648.0	kN/m	**In-Plane:**		
Out-of-plane moment (Mo*) =	0	kNm/m	Max. mid height stress (σ.mhi.max) =	5.84	MPa
In-Plane moment (Mi*) =	0	kNm	Min. mid height stress (σ.mhi.min) =	5.84	MPa
Shear Force =	0	kN	0.03*f'c =	1.50	MPa

Reinforcement If designed as column - restraint provisions of Cl 10.7.4 not required if N* ≤ 0.5*øNu - Cl 11.7.4

Reinforcement may be provided in single central layer - Cl 11.7.3

Vertical reinf't =	**N12-350 cts - each face**		Class =	N	
No. Reinf't layers =	2				
Bar size =	12	mm	Steel ratio (pw) =	0.0036	
Bar cts =	350	mm	pw.min =	0.0025	Cl C5.2
Reinf't yield strength (fsy) =	500	MPa	Ast.min =	450	mm²/m
Area steel (Astv) =	323	mm²/m each face			
Clear gap =	338	mm			
Max reinf't cts = min(350 & 2.5*tw) =	350	mm	Cl 11.7.3		
Horizontal reinf't =	**N16-150 cts - each face**		Unrestrained =	N	(Y)es, (N)o
Bar size =	16	mm	Steel ratio (pw) =	0.0149	
Bar cts =	150	mm	pw.min =	0.0025	Cl C5.2
Reinf't yield strength (fsy) =	500	MPa	Ast.min =	450	mm²/m
Area steel (Asth) =	1340	mm²/m each face	Horz. shrinkage control - Strong B1-C2		
Clear gap =	134	mm			
Max reinf't cts = min(350 & 2.5*tw) =	350	mm	Cl 11.7.3		

Design axial strength - Cl 11.5

Hwe/tw =	21.4		
Strength reduction factor (ø) =	0.6	Cl 11.5.1	
øNuo = 0.03*tw*f'c =	0.0	kN/m	N/A - Not within slab design limits, øNuo=0kN
Nus = (tw-1.2*e-2*ea)*0.6*f'c =	2343.7	kN/m	Eq 11.5.1
øNus =	1406.2	kN/m	Hwe/tw ≤ 30, Within simplified limits
øNu = øNus =	1406.2	kN/m	
Max. height when øNu = 0kN, Hwe =	5692	mm	
Strength reduction factor - tension (øt) =	0.8		Table 2.2.2
Tensile capacity (øNuot = øt*fsy*Ast) =	129.3	kN/m	(When fully anchored)

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Fire resistance - Cl 5.7

Fire exposed on 1 side only =	Y	(Y)es,(N)o	Cl 5.7.2
Lateral support on 1 side only =	N	(Y)es,(N)o	Cl 5.7.2
Top lateral support requires FRL =	Y	(Y)es,(N)o	Cl 5.7.3

				Interpolated Values
Load level (μ.fi = Nf*/øNu) =	0.461			
Insulation =	240	mins.	Cl 5.7.1	240 mins.
Adequacy =	120	mins.	Cl 5.7.2	169 mins.
Max. eff. height for FRL = 40*tw (FRL of support) =	7200	mm	Cl 5.7.3	
FRL =	120	mins.		169 mins.

Effective Heights - Cl 11.4 No openings

Height of wall (Hw) =	3850	mm	Openings =	N (Y)es,(N)o
Length of wall (L1=Lw) =	1180	mm		
Area of Wall =	4.54	m²		
Length of return wall for lateral restraint = 0.2*Hw =	770	mm		

One way buckling - Cl 11.4(a)

			Hwe = k*Hw	
Restrained against rotation top and bottom k =	0.750	Hwe =	2888	mm
Not restrained against rotation top or bottom k =	1.000	Hwe =	3850	mm

Two way buckling - Cl 11.4(b)

Lateral support three sides k = [1/(1+(Hw/(3*L1))²)] ≥ 0.3 =	0.458	Hwe =	1764	mm

Two way buckling - Cl 11.4(b)

Lateral support four sides, Hw > L1, k = L1/(2*Hw) =	0.153	Hwe =	590	mm

Horizontal Crack Control - Cl 11.7.2

Where restrained from shrinkage, the total horizontal reinforcement shown below is to be used.

Degree of crack control	p	Ast	
Minor (A1 & A2)	0.0025	450	mm²
Moderate and hidden (A1 & A2)	0.0035	630	mm²
Strong for appearance (A1 & A2)	0.006	1080	mm²
Exposure (B1, B2, C1, C2)	0.006	1080	mm²

Licensee: MLEI Consulting Engineers

Geometry: **(Short direction lift shaft panel) Hwe = 3850mm, 180mm thick, f'c=50MPa**
Reinf't: **N12-350 cts - each face vertical, N16-150 cts - each face horizontal**
Capacity: **V* = 737kN < øVu = 1029kN** **OK (0.72)**

Geometry

Concrete strength (f'c) =	50	MPa		
Height of wall (Hw) =	3850	mm	Hw/Lw =	3.26
Length of wall (Lw) =	1180	mm		
Wall thickness (tw) =	180	mm		
Reinf't yield strength (fsy) =	500	MPa		

Loading

In-plane shear (V*) =	737	kN

Strength in shear - Cl 11.6.2

Strength reduction factor (ø) =	0.7		Table 2.2.2
Vu.max = 0.2*f'c*(0.8*Lw*tw) =	1699.2	kN	
øVu.max =	1189.4	kN	

Excluding wall reinforcement - Cl 11.6.3

Vuc.min = 0.17*√f'c*(0.8*Lw*tw) =	204.3	kN	(Critical)
øVuc.min =	143.0	kN	
Vuc.a = (0.66*√f'c-0.21*Hw/Lw*√f'c)*0.8*Lw*tw =	-30.2	kN	For Hw/Lw ≤ 1
Vuc.b = (0.05*√f'c+0.1*√f'c/(Hw/Lw-1))*0.8*Lw*tw =	113.2	kN	For Hw/Lw > 1 (Applicable)
Vuc = Max(Vuc.min,Min(Vuc.a,Vuc.b)) =	204.3	kN	
øVuc =	143.0	kN	

With wall reinforcement - Cl 11.6.4

Vertical area steel (Asth) =	323	mm²/m each face	
Vert. steel ratio (pwv) =	0.0036		
Horz. Area steel (Asth) =	1340	mm²/m each face	
Horz. steel ratio (pwh) =	0.0149		
pw = pwh =	0.0149		Hw/Lw > 1, pw=pwh - Cl 11.6.4(b)
Reinf't contribution Vus = pw*fsy*(0.8*Lw*tw) =	1265.3	kN	
Reinf't contribution øVus =	885.7	kN	
Vu = min(Vuc + Vus & Vu.max) =	1469.6	kN	
Shear capacity (øVu = min(øVuc + øVus & øVu.max) =	1028.7	kN	

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CALCULATIONS	REF./COMMENT

iv) *Check capacity of dowels* - *as per AS 4100*

The shear capacity need to be transfered between the panels through the use dowel bars.

INPUT

Try: N32 Dowels

Diameter = 32 mm

f_{sy} = 500 MPa

A_{st} = 804 mm^2

LOADING

Total shear, V = 737 kN

OUTPUT

Shear capacity, $\varphi V_f = 0.62 f_{sy} k_r A_{st}$

ϕ = 0.8

k_r = 1.0

ϕV_f = 199.5 kN

Required # of dowels, n = 3.7

Adopt = 4.0

Development Length of a bar in tension

f_{sy} =	500	MPa
f'_c =	50	MPa
k_1 =	1	all other bars
k_2 =	1.7	bars in slabs and walls if the clear distance is greater than 150mm
clear cover =	90	mm

Bar size	d_b	A_b	$L_{sy.t}$
N12	12	110	300
N16	16	200	400
N20	20	310	500
N24	24	450	600
N28	28	620	700
N32	32	800	800
N36	36	1020	900

Adopt: 4 x N32 Dowels

CALCULATIONS	REF./COMMENT

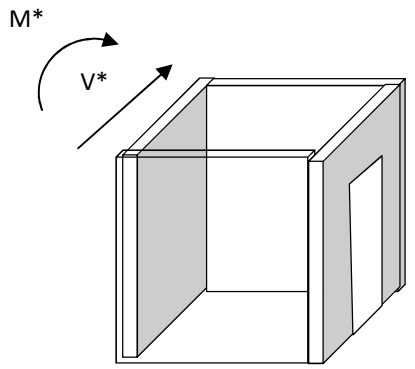
By inspection the moment in the panel is too large to resist, espically when the voids of the lift doorways are considered. For this reason, we will consider the lift shaft to act as a box section with tension and compression forces in the panels parallel with the long direction.

As the lift shaft is made up of two panels in the short direction, the loads previously calculated will be doubled, including the area of the weights resisting. We already know the panels are adquate in shear, therefore we will consider bending only.

DESIGN ACTIONS IN LIFT SHAFT

M* =	6046	kNm	@ Ground level
Width of shaft, b =	2180	mm (c/c of panels)	
N_t* =	2773	kN	
N_c* =	2773	kN	

Due to the panels now acting as rectangular columns refer to column spreadsheet for capacities.



Schematic of lift shaft design actions

Adopt: 180 thk precast panel. F'c = 50. 12-N20 each face. W6-180 ligs for each row.

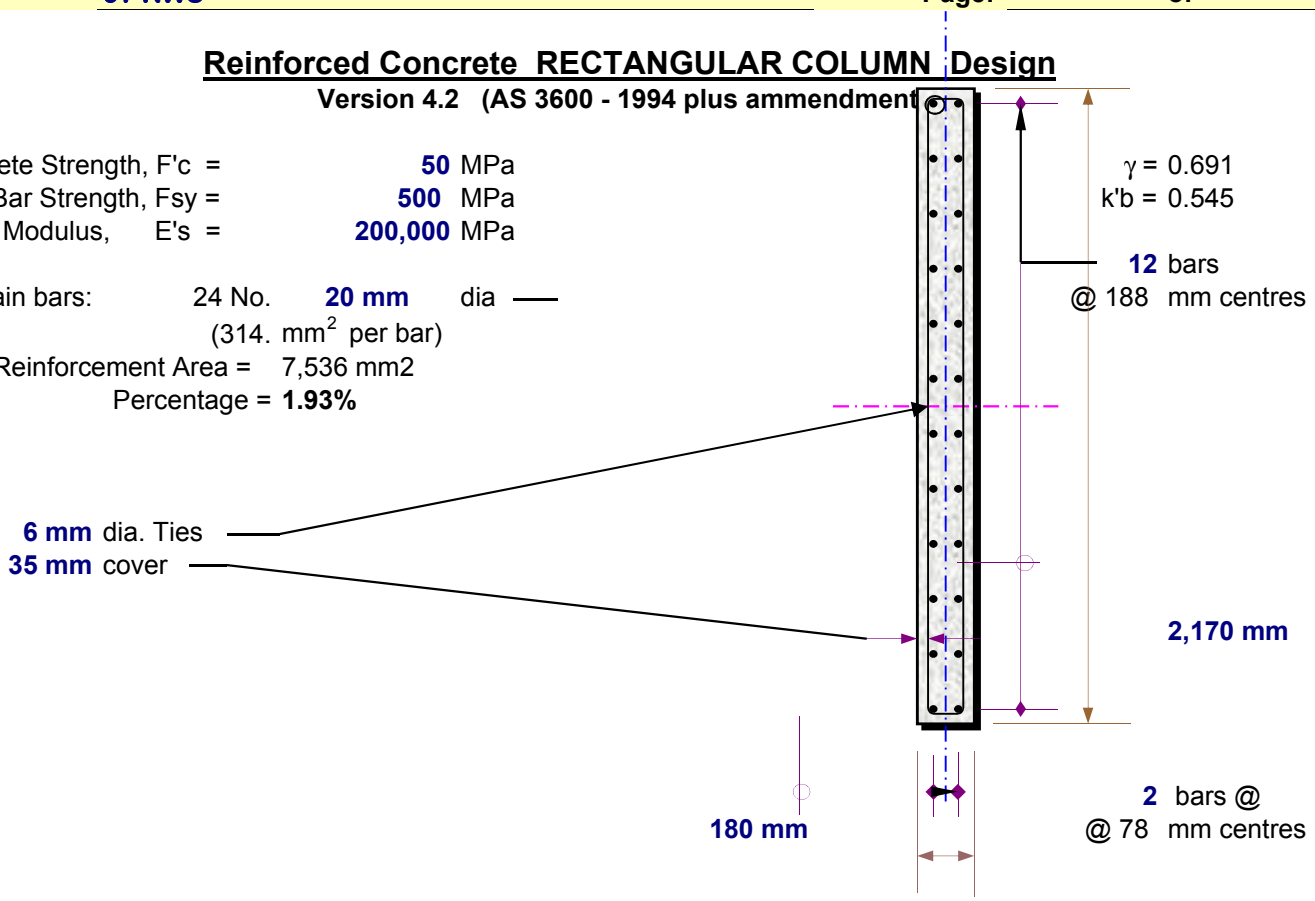
Designer:	**JT**	**Date:**	**6/06/2018**	**Doc Code:**	
Title:	**97 KWS**			**Page:**	**of**

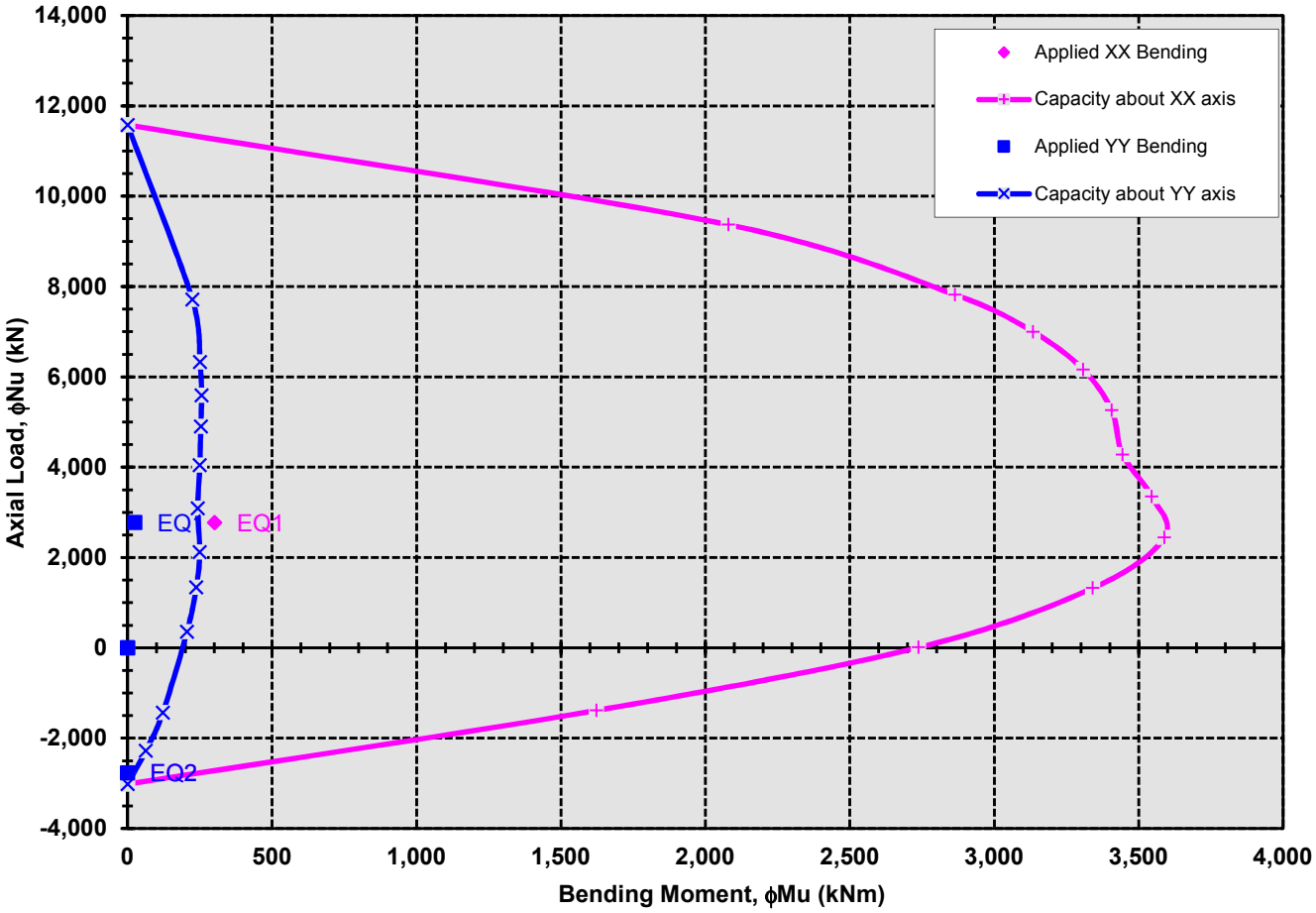
Reinforced Concrete RECTANGULAR COLUMN Design

Version 4.2 (AS 3600 - 1994 plus ammendment

Concrete Strength, F'c = **50** MPa
Main Bar Strength, Fsy = **500** MPa
Reinf. Modulus, E's = **200,000** MPa

Main bars: 24 No. **20 mm** dia
(314. mm^2 per bar)
Total Reinforcement Area = 7,536 mm2
Percentage = **1.93%**





KINHILL

Designer:	**JT**	**Date:**	**6/06/2018**	**Doc Code:**	
Title:	**97 KWS**			**Page:**	**of**

Reinforced Concrete RECTANGULAR COLUMN Design

Version 4.2 (AS 3600 - 1994 plus ammendments)

	About XX axis	About YY axis
Effective Length, Le (mm) =	**3,850**	**0**
Slenderness Ratio, Le/r =	5.91	0
Laterial column restraint	**Braced**	**Braced**
Frame Sway Factor, δs (default=1.0)	**1.0**	**1.0**

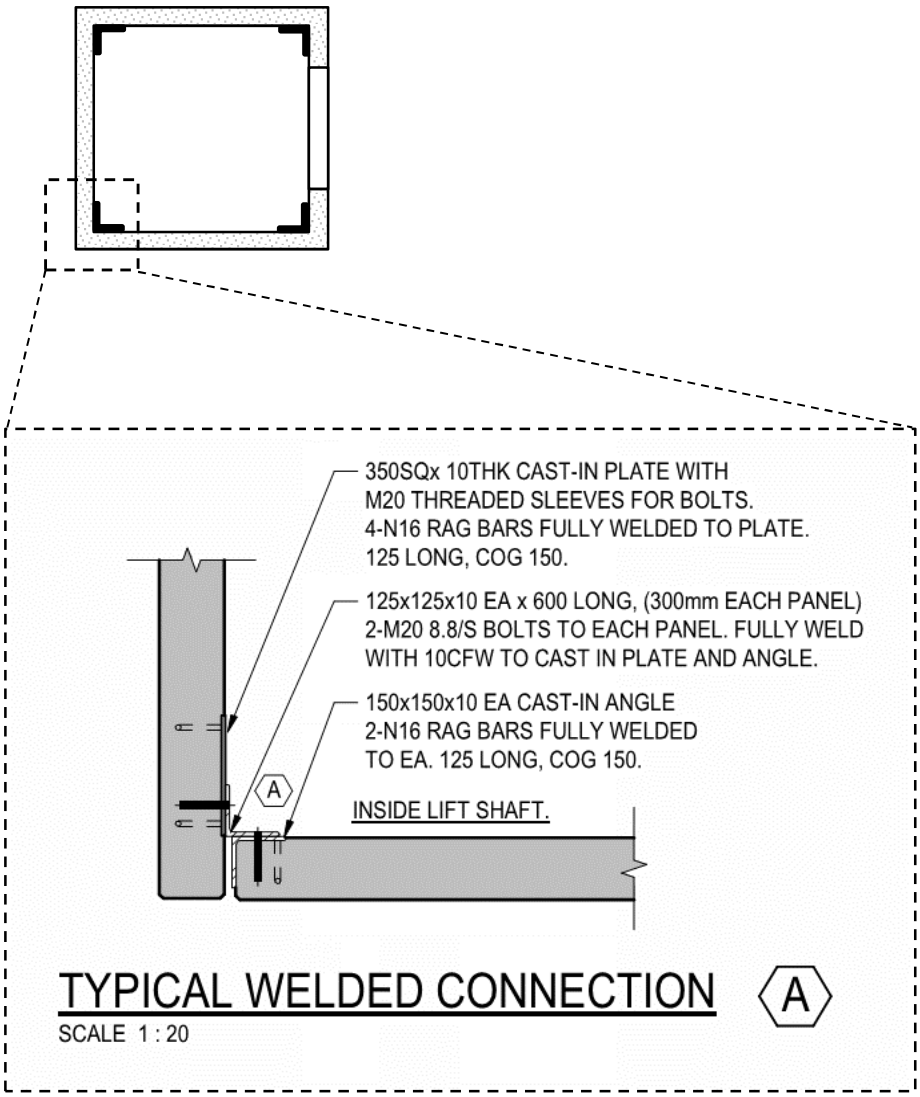
Load Case Reference	**EQ1**	**EQ2**			
Applied Axial Load, N* =	**2.77E+03**	**-2773**	**0**	**0**	kN
Load Ratio, G*/(G*+Q*) =	**0.86**	**0.86**	**0**	**0**	
Squash Load, Nuo =	19295	19295	19295	19295	kN
Comp. Capacity, ϕNuo =	11577	11577	11577	11577	kN
Tens. Capacity, ϕNt =	-3014	-3014	-3014	-3014	kN
About YY axis					
Applied Ultimate Moment, M* =	**0**	**0**	**0**	**0**	kNm
End Moment Ratio (-1=DC< x <1=SC)	**1**	**0**	**0**	**0**	
Load Eccentricity, ex/b =	0.00	0.00	0.00	0.00	
About YY Column is	Short	Short	Short	Short	
Beta.d =	#N/A	#N/A	#N/A	#N/A	
Buckling Load, Nc =	#N/A	#N/A	#N/A	#N/A	kN
km =	#N/A	#N/A	#N/A	#N/A	
Moment Magnification =	100.0%	100.0%	100.0%	100.0%	
Min Moment =	24.957	0	0	0	kNm
Design Moment, M* =	**25**	**0**	**0**	**0**	kNm
Moment Capacity, fMu.yy =	**245**	**21**	**189**	**189**	kNm
About XX axis					
Applied Ultimate Moment, M* =	**0**	**0**	**0**	**0**	kNm
End Moment Ratio (-1=DC< x <1=SC)	**1**	**0**	**0**	**0**	
Load Eccentricity, ex/b =	0.00	0.00	0.00	0.00	
About XX Column is	Short	Short	Short	Short	
Beta.d =	#N/A	#N/A	#N/A	#N/A	
Buckling Load, Nc =	#N/A	#N/A	#N/A	#N/A	kN
km =	#N/A	#N/A	#N/A	#N/A	
Moment Magnification =	100.0%	100.0%	100.0%	100.0%	
Min Moment =	300.8705	0	0	0	kNm
Design Moment, M* =	**301**	**0**	**0**	**0**	kNm
Moment Capacity, fMu.xx =	**3573**	**240**	**2730**	**2730**	kNm
Interaction coefficent, Alpha.n =	1.11	1.00	1.00	1.00	
Load/Capacity Ratio =	**0.14**	**0.00**	**0.00**	**0.00**	**ALL OK**

Reference: 2018-7161
Project: 97 King William Street, Kent Town **Checked by:** BR
Designer: JT Date: 6/06/2018 **Index:** PR - 7

CALCULATIONS	REF./COMMENT

To treat the lift shaft as a box, the tension and compression forces must be transferred between the connected panels. For this welded connections are to be used.

TRY: 10mm CFW (high strength, SP grade)
Design capacity, ϕVw = 1.63 kN/mm
Required length of weld, L_w = 1702 mm
= 851 mm (per corner)



Adopt: 125 x 10 EA. 10mm CFW all around to cast in plates.

CALCULATIONS	REF./COMMENT

b) Long direction external panels

Lateral force distribution in the short direction. Refer to precast element layout sketch.

i) Stiffness

As the external walls are the same in length and are divided into the same number panels we can assume the load is evenly distributed to each panel in the long direction.

Total number panels in long direction = 32 (External walls only)

The governing earthquake forces per level calculated earlier are broken down to give the forces in each of the external panels in the long direction. The external panels are analysed as a fixed base at ground level. See loading below for force breakdown.

CALCULATIONS	REF./COMMENT

ii) *Loading*

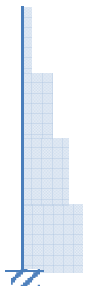
FORCES

Roof	2 kN	15.45 m
Level 3	25 kN	10.85 m
Level 2	17 kN	7.35 m
Level 1	9 kN	3.85 m
Ground		0 m

In the long direction the roof load is resisted by the panels.

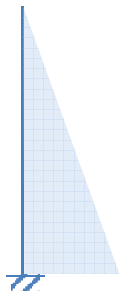
SHEAR

Roof	2 kN	15.45 m
Level 3	28 kN	10.85 m
Level 2	45 kN	7.35 m
Level 1	54 kN	3.85 m
Ground	54 kN	0 m



OVERTURNING MOMENT *(M_o)*

Roof		15.45 m
Level 3		10.85 m
Level 2		7.35 m
Level 1		3.85 m
Ground	471 kNm	0 m



CALCULATIONS	REF./COMMENT

The overturning moment calculated above, is resisted by the self weight of the structure supported by the precast, plus the factored live load acting on the structure.

VERTICAL LOADS (per level)

Roof =	0.45 kPa x		15 m^2 =		6.75	kN
Slab =	4.25 kPa x		45 m^2 =		191.3	kN
Precast =	4.50 kPa x	-	m^2 =	-		kN
Live =	3.00 kPa x		45 m^2 =		135	kN

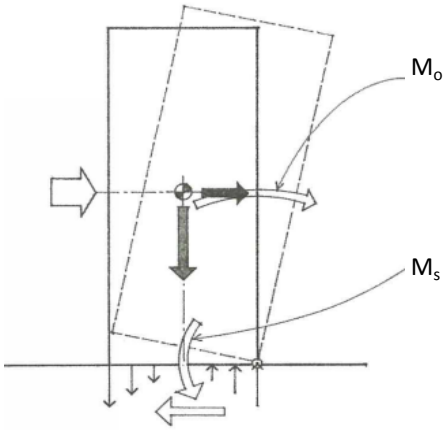
Self weight =	173.8	kN
G =	372	kN
Q =	135	kN
G+0.3Q, w =	412	kN (per level)
Total, w =	1649	kN

STABILISING MOMENT

$M_s = wL_a$

Lever arm, L_a = 1250 mm (half the width of the panel)

ϕM^*_a = 1855 kNm (ϕ = 0.9)



RESULTANT MOMENT *(in-plane)* M* = -1385 kNm**

DESIGN ACTIONS IN PANEL

V* =	54	kN
M* =	-1385	kNm**
Under 1.2G+1.5Q and from overturning moment, N_c* =	649	kN

**As the overturning moment is less than the stabilising moment, the panel does not undergo uplift or in-plane bending. Therefore, only shear and compression need to be checked.

CALCULATIONS	REF./COMMENT

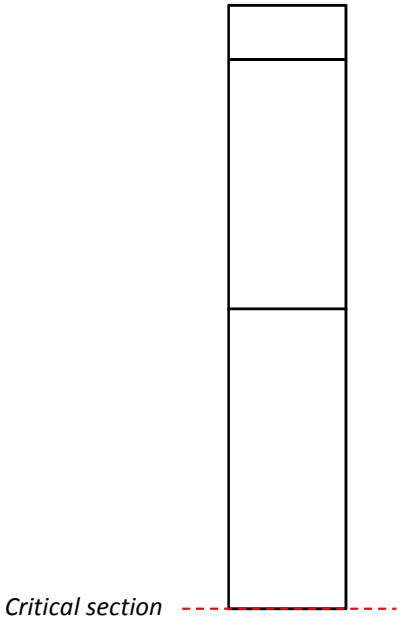
iii) *Output*

GEOMETRY/PROPERTIES

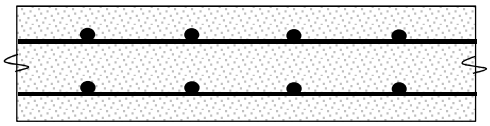
Span, H_w =	3850	mm	(worst case floor-to-floor)
Length, L_W =	2500	mm	
Thickness, t_w =	180	mm	
f'_c =	50	MPa	
E =	34800	MPa	

Refer to "Structural Toolkit" output

SCHEMATIC



East & West side



Adopt: 180 thk, f'c = 50 MPa, SL92 EF

Geometry: **(Long direction external wall panels) Hwe = 3850mm, 180mm thick, f'c=50MPa**
Reinf't: **SL92 (main wires in main direction) - each face**
Capacity: **N* = 649kN/m < øNu = 1406kN/m, Hwe/tw = 21.4** **OK (0.46)**
FRL: **FRL = 120 minutes (Refer Cl 5.7.4 for additional limitations for chases and recesses)**

Geometry **σ.mhi ≥ 0 MPa (all compression under in-plane) - Design as wall - Cl 11.1(a) (wall or column - Cl 11.2.1(a))**

Concrete strength (f'c) =	50	MPa				
			Load eccentricity =	30.0	mm	
Wall thickness (tw) =	180	mm	Min. Ecc = 0.05*tw =	9.0	mm	Cl 11.5.2
Height of wall (Hw) =	3850	mm	ea = Hwe²/(2500*tw) =	32.9	mm	Cl 11.5.1
Effective height factor (k) =	1.00	(Refer below)	Design ecc. (e) =	30.0	mm	
Effective height (Hwe = k*Hw) =	3850	mm	30 * tw =	5400	mm	
Length of wall (Lw) =	2500	mm	50 * tw =	9000	mm	
Cover to outer bars =	35	mm	Hwe/tw =	21.4		
Axis distance (as) =	39	mm Cl 5.2.2				
			Formwork =	S	(S)tandard,(R)igid	
Ductile shear wall =	Y	(Y)es,(N)o	Exposure =	B2	Tab 4.10.3.2	

Loading Wall under in-plane uniform compression

Dead load (Ndl) =	372	kN/m	Load type =	F	(F)loor,(S)tore,(R)oof,(O)ther	
Live load (Nll) =	135	kN/m	Long term LL factor (Ψl.f) =	0.60	Table 4.1 (Concentrated)	
Mid height SWt =	8.7	kN/m				
Axial comp. N* = 1.2*Ndl+1.5*Nll =	648.9	kN/m	Include S.Wt =	N	(Y)es,(N)o	
Axial comp. fire N*f = Ndl+Ψl.f*Nll =	453.0	kN/m	**In-Plane:**			
Out-of-plane moment (Mo*) =	0	kNm/m	Max. mid height stress (σ.mhi.max) =	3.61	MPa	
In-Plane moment (Mi*) =	0	kNm	Min. mid height stress (σ.mhi.min) =	3.61	MPa	
Shear Force =	0	kN	0.03*f'c =	1.50	MPa	

Reinforcement If designed as column - restraint provisions of Cl 10.7.4 not required if N* ≤ 0.5*øNu - Cl 11.7.4

Reinforcement may be provided in single central layer - Cl 11.7.3

Vertical reinf't =	**SL92 (main wires in main direction) - each fa**		Class =	L	
No. Reinf't layers =	2				
Bar size =	8.6	mm	Steel ratio (pw) =	0.0032	
Bar cts =	200	mm	pw.min =	0.0025	Cl C5.2
Reinf't yield strength (fsy) =	500	MPa	Ast.min =	450	mm²/m
Area steel (Astv) =	290	mm²/m each face			
Clear gap =	191	mm			
Max reinf't cts = min(350 & 2.5*tw) =	350	mm	Cl 11.7.3		

Horizontal reinf't =	**SL92 (main wires in main direction) - each fa**		Unrestrained =	N	(Y)es, (N)o
Bar size =	8.6	mm	Steel ratio (pw) =	0.0032	
Bar cts =	200	mm	pw.min =	0.0025	Cl C5.2
Reinf't yield strength (fsy) =	500	MPa	Ast.min =	450	mm²/m
Area steel (Asth) =	290	mm²/m each face	Horz. shrinkage control - Minor A1 & A2		
Clear gap =	191	mm			
Max reinf't cts = min(350 & 2.5*tw) =	350	mm	Cl 11.7.3		

Design axial strength - Cl 11.5

Hwe/tw =	21.4		
Strength reduction factor (ø) =	0.6	Cl 11.5.1	
øNuo = 0.03*tw*f'c =	0.0	kN/m	N/A - Not within slab design limits, øNuo=0kN
Nus = (tw-1.2*e-2*ea)*0.6*f'c =	2343.7	kN/m	Eq 11.5.1
øNus =	1406.2	kN/m	Hwe/tw ≤ 30, Within simplified limits
øNu = øNus =	1406.2	kN/m	
Max. height when øNu = 0kN, Hwe =	5692	mm	
Strength reduction factor - tension (øt) =	0.64		Table 2.2.2
Tensile capacity (øNuot = øt*fsy*Ast) =	92.9	kN/m	(When fully anchored)

Licensee: MLEI Consulting Engineers

Fire resistance - Cl 5.7 Warning - μ.fi < 0.35, 0.35 used

Fire exposed on 1 side only =	N	(Y)es,(N)o	Cl 5.7.2	
Lateral support on 1 side only =	N	(Y)es,(N)o	Cl 5.7.2	
Top lateral support requires FRL =	Y	(Y)es,(N)o	Cl 5.7.3	
Load level (μ.fi = Nf*/øNu) =	0.322			Interpolated Values
Insulation =	240	mins.	Cl 5.7.1	240 mins.
Adequacy =	120	mins.	Cl 5.7.2	150 mins.
Max. eff. height for FRL = 40*tw (FRL of support) =	7200	mm	Cl 5.7.3	
FRL =	120	mins.		150 mins.

Effective Heights - Cl 11.4 No openings

Height of wall (Hw) =	3850	mm	Openings =	N (Y)es,(N)o
Length of wall (L1=Lw) =	2500	mm		
Area of Wall =	9.63	m²		
Length of return wall for lateral restraint = 0.2*Hw =	770	mm		

One way buckling - Cl 11.4(a) **Hwe = k*Hw**

Restrained against rotation top and bottom k =	0.750	Hwe =	2888	mm
Not restrained against rotation top or bottom k =	1.000	Hwe =	3850	mm

Two way buckling - Cl 11.4(b)

Lateral support three sides k = $[1/(1+(Hw/(3*L1))^2)] \geq 0.3$ =	0.791	Hwe =	3047	mm

Two way buckling - Cl 11.4(b)

Lateral support four sides, Hw > L1, k = L1/(2*Hw) =	0.325	Hwe =	1250	mm

Horizontal Crack Control - Cl 11.7.2

Where restrained from shrinkage, the total horizontal reinforcement shown below is to be used.

Degree of crack control	p	Ast	
Minor (A1 & A2)	0.0025	450	mm²
Moderate and hidden (A1 & A2)	0.0035	630	mm²
Strong for appearance (A1 & A2)	0.006	1080	mm²
Exposure (B1, B2, C1, C2)	0.006	1080	mm²

Licensee: MLEI Consulting Engineers

Geometry: **(Long direction external wall panels) Hwe = 3850mm, 180mm thick, f'c=50MPa**
Reinf't: **SL92 (main wires in main direction) - each face**
Capacity: **V* = 54kN < øVu = 826kN** **OK (0.07)**

Geometry

Concrete strength (f'c) =	50	MPa		
Height of wall (Hw) =	3850	mm	Hw/Lw =	1.54
Length of wall (Lw) =	2500	mm		
Wall thickness (tw) =	180	mm		
Reinf't yield strength (fsy) =	500	MPa		

Loading

In-plane shear (V*) =	54	kN

Strength in shear - Cl 11.6.2

Strength reduction factor (ø) =	0.7		Table 2.2.2
Vu.max = 0.2*f'c*(0.8*Lw*tw) =	3600.0	kN	
øVu.max =	2520.0	kN	

Excluding wall reinforcement - Cl 11.6.3

Vuc.min = 0.17*√f'c*(0.8*Lw*tw) =	432.7	kN	
øVuc.min =	302.9	kN	
Vuc.a = (0.66*√f'c-0.21*Hw/Lw*√f'c)*0.8*Lw*tw =	856.8	kN	For Hw/Lw ≤ 1
Vuc.b = (0.05*√f'c+0.1*√f'c/(Hw/Lw-1))*0.8*Lw*tw =	598.7	kN	For Hw/Lw > 1 (Applicable) (Critical)
Vuc = Max(Vuc.min,Min(Vuc.a,Vuc.b)) =	598.7	kN	
øVuc =	419.1	kN	

With wall reinforcement - Cl 11.6.4

Vertical area steel (Asth) =	290	mm²/m each face	
Vert. steel ratio (pwv) =	0.0032		
Horz. Area steel (Asth) =	290	mm²/m each face	
Horz. steel ratio (pwh) =	0.0032		
pw = pwh =	0.0032		Hw/Lw > 1, pw=pwh - Cl 11.6.4(b)
Reinf't contribution Vus = pw*fsy*(0.8*Lw*tw) =	580.9	kN	
Reinf't contribution øVus =	406.6	kN	
Vu = min(Vuc + Vus & Vu.max) =	1179.6	kN	
Shear capacity (øVu = min(øVuc + øVus & øVu.max) =	825.7	kN	

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CALCULATIONS	REF./COMMENT

iv) *Check capacity of dowels* - *as per AS 4100*

The shear capacity need to be transfered between the panels through the use dowel bars.

INPUT

Try: N20 Dowels

Diameter = 20 mm

f_{sy} = 500 MPa

A_{st} = 314 mm^2

LOADING

Total shear, V = 54 kN

OUTPUT

Shear capacity, $\varphi V_f = 0.62 f_{sy} k_r A_{st}$

ϕ = 0.8

k_r = 1.0

ϕV_f = 77.9 kN

Required # of dowels, n = 0.7

Adopt = 2.0

Development Length of a bar in tension

f_{sy} =	500	MPa
f'_c =	50	MPa
k_1 =	1	all other bars
k_2 =	1.7	bars in slabs and walls if the clear distance is greater than 150mm
clear cover =	90	mm

Bar size	d_b	A_b	$L_{sy.t}$
N12	12	110	300
N16	16	200	400
N20	20	310	500
N24	24	450	600
N28	28	620	700
N32	32	800	800
N36	36	1020	900

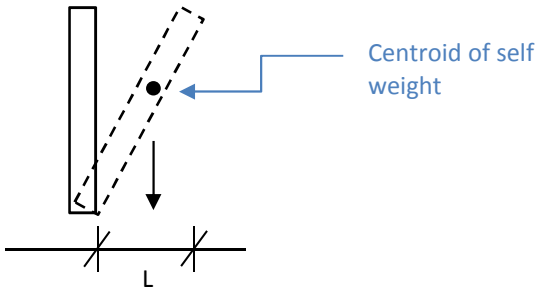
Adopt: 2 x N20 Dowels per panel

CALCULATIONS	REF./COMMENT

v) Check external panels for fire - as per NCC spec C1.11

The external panels of the building have to be checked for a further requirement of the NCC. Refer to spec C1.11.

LOADING



Design loadwidth =	1.0	m	
Total height =	7350	mm	
Total height/thickness =	41	< 50	***OK***
L =	735	mm	
Self weight =	33.1	kN per m	
M* =	12.2	kNm	

OUTPUT

Thickness, t =	180	mm	
f'_c =	50	MPa	
γ =	0.696		
α_2 =	0.85		
Cover =	35	mm (to tensile steel)	
d_o =	145	mm	
C_c =	145	kN	
Reinforcement yield strength, f_{sy} =	500	MPa	
A_{st} =	290	mm^2	SL92
Assuming tensile steel yields, $T_s = A_{st}f_{sy}$ =	145	kN	
As $C_c = T_s$, solve for d_n			0.00
d_n =	4.90	mm	

Taking moments about reinforcement	M_u =	20.7	kNm	
	$\phi M_u = 0.8M_u$ =	16.5	kNm > M*	***0.74 OK***

CALCULATIONS	REF./COMMENT

6) LIFT PIT

Design lift pit for bearing loads and shear loads against the soil.

i) Base of pit - vertical bearing/strength

INPUT

Width, b =	2400	mm
Length, L =	2400	mm
Base area, A =	5.8	m^2

SOIL PROPERTIES

Minimal soil information was avaliable apart from serveral borelogs (attached). Borelogs indicate sandy clay at the lift pit depth.

Looking at geotechnical reports of similar soil profiles an allowable vertical bearing capacity of 200 kPa was assumed. For the ulimate soil bearing capacity a value of 600 kPa was adopted (factor of safety of 3 for allowable).

For short term loads such as wind and earthquake, a 25% increase in the ulitmate bearing capacity may be used.

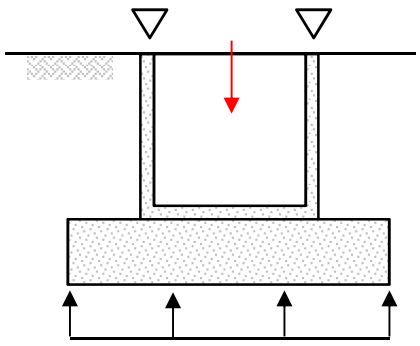
The lateral bearing capacity, 2/3 of the vertical was adopted.

LOADING

Dead, G:	4.50 kPa x	152 m^2 =	684	kN	Precast
	4.25 kPa x	238 m^2 =	1012	kN	Slab
Live, Q:	3.00 kPa x	238 m^2 =	714	kN	Office
		ULS 1, 1.35G =	2289	kN	
		ULS 2, 1.2G+1.5Q =	3106	kN	

OUTPUT

Bearing:	q_{all} =	200	kPa		
	q_{ult} =	600	kPa		
	q* = ULS 2/A =	539	kPa	< q_{ult}	***0.90 OK***
Strength:	q_{ult} =	600	kPa		
	w =	600	kN/m		1m design width
	Span, L =	1.81	m (short direction)		
	M* =	246	kNm		
	ϕM_u =	271	kNm	> M*	***0.91 OK***



Adopt: 2.4 x 2.4 x 0.5 thk base. N20-200 EW EF.

Strength Reo

D	500	mm
d	450	mm
b	1000	mm
A_{st}	1570	mm^2
ϕ	0.8	
f_{sy}	500	MPa
f'_c	25	MPa

ϕM_u	271.0	kNm
k_u	0.097	OK

Ductility Checks

b_w	1000	mm
f'_{cf}	3.0	MPa
Z	41.7E+6	mm^3
M_{cr}	125.0	kNm
M_u/M_{cr}	2.71	OK
Check for yielding		
E_s	205000	MPa
ε_c	0.003	
ε_{st}	0.0281	
ε_{sy}	0.0024	OK

CALCULATIONS	REF./COMMENT

ii) Side of pit - lateral bearing/strength

INPUT

Width, b = 2000 mm
depth, d = 1500 mm
Side area, A = 3.0 m^2 (plus increase side area from pad)

LOADING

The base shear at the lift pit is double the base element shear as there are two parallel elements (two outside lift panels in the short direction) carrying the shear to the base of the lift shaft.

Earthquake, V* = 874 kN (refer precast calcs)

OUTPUT

Bearing:

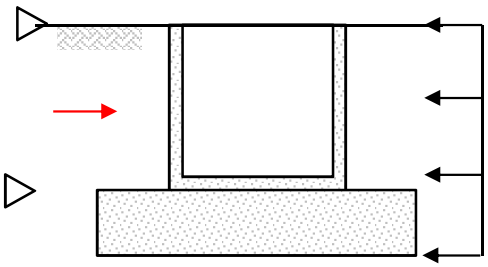
q_{all} = 200 kPa — As above
q_{ult} = 600 kPa — "
q_{ultL} = 400 kPa — 2/3 ult

q* = V*/A = 291 kPa < q_{ultL} — ***0.73 OK***

Strength:

q_{ultL} = 400 kPa
w = 400 kN/m — 1m design width
Span, L = 1.0 m (depth of pit)
M* = 50 kNm

ϕM_u = 61.6 kNm > M* — ***0.81 OK***



Strength Reo

D	200	mm
d	165	mm
b	1000	mm
A_{st}	1005	mm^2
ϕ	0.8	
f_{sy}	500	MPa
f'_c	25	MPa

ϕM_u	61.6	kNm
k_u	0.169	OK

Ductility Checks

b_w	1000	mm
f'_{cf}	3.0	MPa
Z	6.7E+6	mm^3
M_{cr}	20.0	kNm
M_u/M_{cr}	3.85	OK
Check for yielding		
E_s	205000	MPa
ε_c	0.003	
ε_{st}	0.0148	
ε_{sy}	0.0024	OK

Adopt: 200 thk wall. N16-200 EW EF

CALCULATIONS	REF./COMMENT

7) SUSPENDED SLAB DESIGN

Suspended slab design was done in *Fielders - KingFlor Designer Suite 5.4*. Further analysis was done in *RAPT* for cantilevers and setdowns.

a) Typical 6m cont. suspended slab

Span, L = 6.0 m (max.)
Loadwidth, b = 1.20 m — Design

METAL DECKING *All slabs use "Fielders" KF57 metal decking*

BMT = 1.0 mm
A_{st} = 1593 mm^2/m

A design width of 1200mm was adopted (4 pans of KF57)

b = 1200 mm

CONCRETE

f'_c = 32 MPa
E_c = 30100 MPa

REINFORCEMENT

Negative

Bar = N16
Spacing = 200 mm
Length = as per plan
Cover = 25 mm

Mesh

Mesh = SL82
Crack control: Minor
Cover = 35 mm

Positive/fire

Exposure: 120 mins
Bar = N16
Spacing = 1 each pan
Cover = 35 mm (from bottom face)

LOADING

Dead load, G

Services = 0.2 kPa
Screed (wet areas only) = 1.0 kPa
Partitions = 0.50 kPa

Plus the self weight of the slab

Live load, Q

Office = 3.0 kPa
Balcony = 4.0 kPa
Carpark = 2.5 kPa

LOAD COMBINATIONS

ULS 1 = 1.2G + 1.5Q
SLS 1 = G + 0.7Q (short term)
SLS 2 = G + 0.4Q (long term)

Refer to Fielders - KingFlor output on the next page. Slab is to be propped at thirds points.

Adopt: 170 thk, KF57 1.0 BMT, SL82 top, N16-200 additional top (length as shown on plan).
1-N16 each pan for fire, 35mm cover.

COMPANY NAME:		Date:	22/05/2018
CLIENT NAME:		Time:	09:14
JOB REFERENCE:		Job No:	
DECK REFERENCE:		Calcs by:	
FILENAME:	Typical 2 span 170 thk.pmd	Checked by:	

SUMMARY OUTPUT

Construction Stage:	**PASS**	**Max Unity Factor =**	**0.80**
Composite Stage:	**PASS**	**Max Unity Factor =**	**0.78**
Serviceability:	**PASS**	**Max Unity Factor =**	**0.86**
Fire State:	**PASS**	**Max Unity Factor =**	**0.80**

***** SECTION ADEQUATE *****

FLOOR PLAN DATA : (propped composite construction with Fielders KF57 decking)

Support Centres	6.000 m	Span Type	Double
Beam or wall width	230 mm	Row of props	Two at 1/3 span
Shortest Sheet Length	12.000 m	Prop width	100 mm

PROFILE DATA : (Fielders KF57 decking. Grade G 550)

Overall Depth	58 mm	Design sheet thickness	1.00 mm
Trough width	300 mm	Nominal sheet thickness	1.04 mm
Pitch of deck ribs	300 mm	Deck weight	0.13 kPa
Average Inertia, Ixx	0.502 * 10^6 mm4/m	Area of profile, Ap	1593 mm²/m
Yield strength	550 MPa		

CONCRETE DATA : Normal Weight Concrete (NWC):

Overall slab depth	170 mm	Concrete wet density	2450 kg/m³
Concrete characteristic strength	32 MPa	Concrete dry density	2400 kg/m³
Modular ratio	7.0	Concrete Volume	0.170 m³/m²
Transverse crack control	Minor	Concrete Gamma	0.822

REINFORCEMENT DATA:

Reinforcement Name	SL82	Yield Strength	500 MPa
Top Cover	35 mm		
Bar reinforcement diameter	16 mm	Bar yield strength	500 MPa
Bar Top Cover	27 mm	Bar Spacing	200 mm
Fire Bar diameter	16 mm	Fire Bar yield strength	500 MPa
Fire Bar Bottom Cover	35 mm	Fire Bar Spacing	1 per 300 mm

SECTION PROPERTIES :

*NOTE - 1: All values of inertia are expressed in steel units.
*NOTE - 2: Effective inertia is used for deflection calculations in continuous spans.

COMPOSITE:

Inertia, Ixx - Uncracked	67.43 * 10^6 mm4/m	Cracked Inertia (+ve)	28.09 * 10^6 mm4/m
Effective Inertia	41.91 * 10^6 mm4/m	Cracked Inertia (-ve)	14.04 * 10^6 mm4/m

LOADS ACTING ON SLAB :

*** NOTE: Slab subjected to uniformly distributed loads (UDL) only

Occupancy Live Loads (Pattern Off)	3.00 kPa	Partition loads	0.50 kPa
Ceilings and services	0.35 kPa	Construction load	1.00 kPa
Self weight of concrete slab (wet)	4.14 kPa	Self weight of decking	0.13 kPa
Self weight of concrete slab (dry)	4.06 kPa	Screeds	None

FIRE DATA :

Design method	VUT, April 2002	Fire resistance period	120 mins

LOAD FACTORS :

Dead (self weight)	1.20	Live Strength Limit		1.50
Superimposed dead	1.20	Live Combination	ψ_c	0.40
Fire Limit	1.00		ψ_l	0.40
			ψ_s	0.70

UNITY FACTORS :

Transverse Reinforcement Area (Minor)	0.86
Minimum Positive Tensile Reinforcement Area	0.17
Minimum Negative Tensile Reinforcement Area	0.46
Long Reinforcement Spacing	0.67
Long Reinforcement Stress	0.79

CALCULATIONS	REF./COMMENT
b) <u>Typical 6m cont. suspended slab with cantilevers at each end (levels 2 and 3)</u> Additional cases were modelled in RAPT. The second and third floor slab systems were modelled with the 2500mm cantilever and the 2600mm cantilevers at the Southern and Northern ends respectively taken into account. The results can be seen in the RAPT output on the next page. The cantilevers were off form (no sheeting) with SL82 mesh top and bottom with additional bars top as shown. Deflections were limited to L/250 with cantilevers limited to L/125 for long term loading conditions. As the slab runs over beams, all connections were treated as "knife-edge", i.e. pinned.	

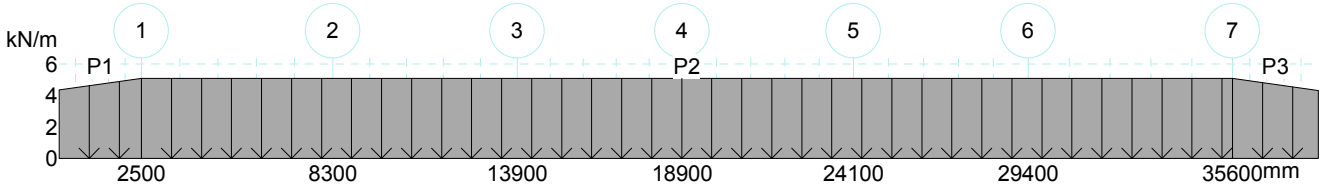
Project Name: 97 King William Street, Kent Town
Project Number: 2018-7161
Frame Description: b) Slab system (levels 2 and 3)
Designer: JT
S:\01 - PROJECTS (FROM 14-7-2010)\2018-7161 - 97 King William St, Kent Town\02_Technical\2_Calculations\Structural\Slabs\RAPT\L2,L3 Slab system.rpf

RAPT - Version: 6.5.6.0
Reinforced And Post-Tensioned Concrete Analysis & Design Package
Copyright(C) 1988 - 2015 PCDC Pty. Ltd. All Rights Reserved

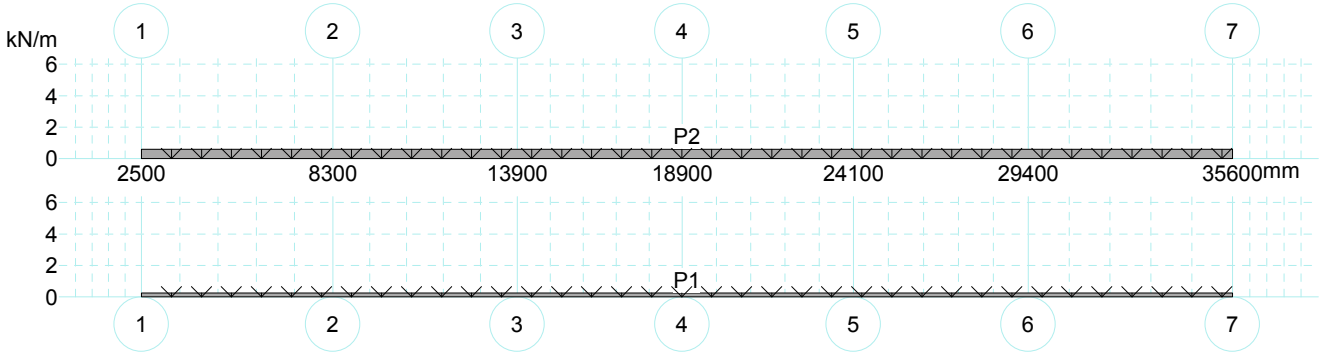
Licensee
MLEI Consultants
Level 5
19 Gilles Street
Adelaide SA 5000
11104065151117WPN1

Input

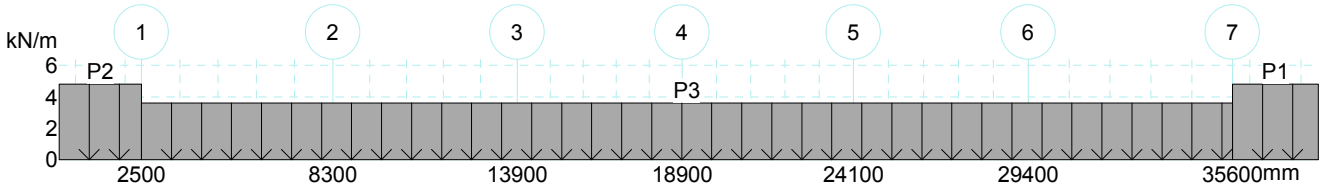
Load Case 1 : 1. Self Weight

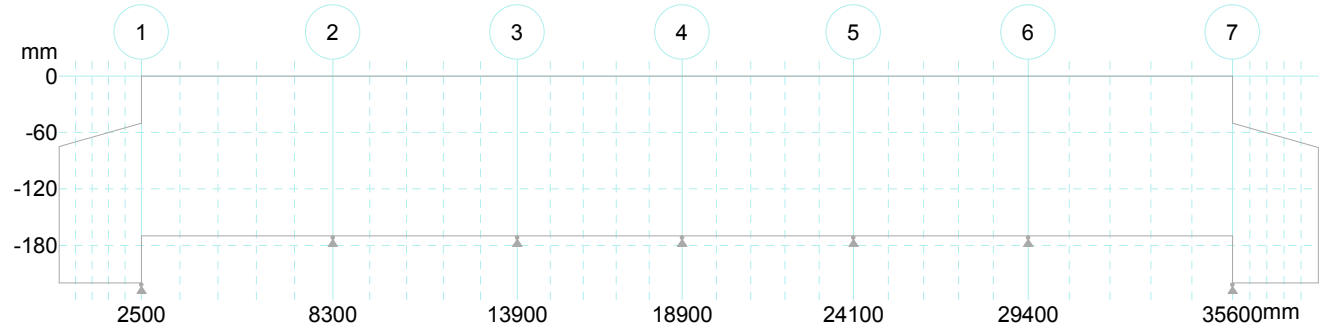


Load Case 2 : 2. Extra Dead Load

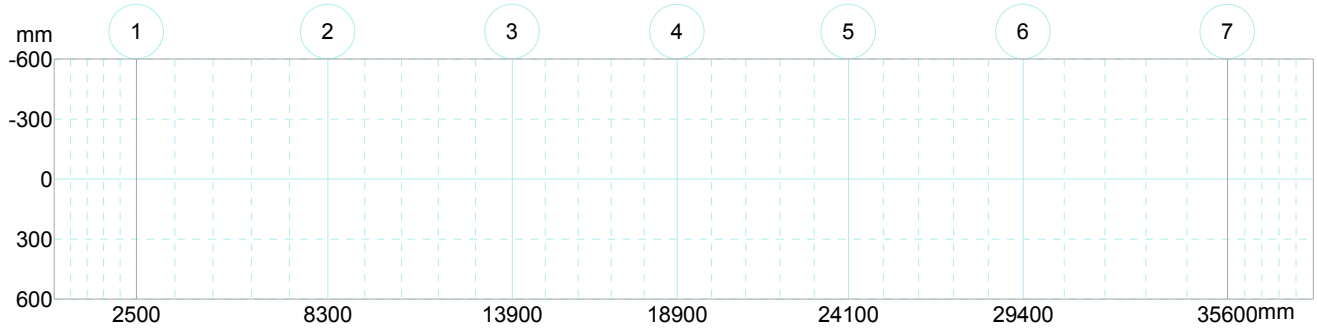


Load Case 3 : 3. Live Load





Plan view



Warnings

Input

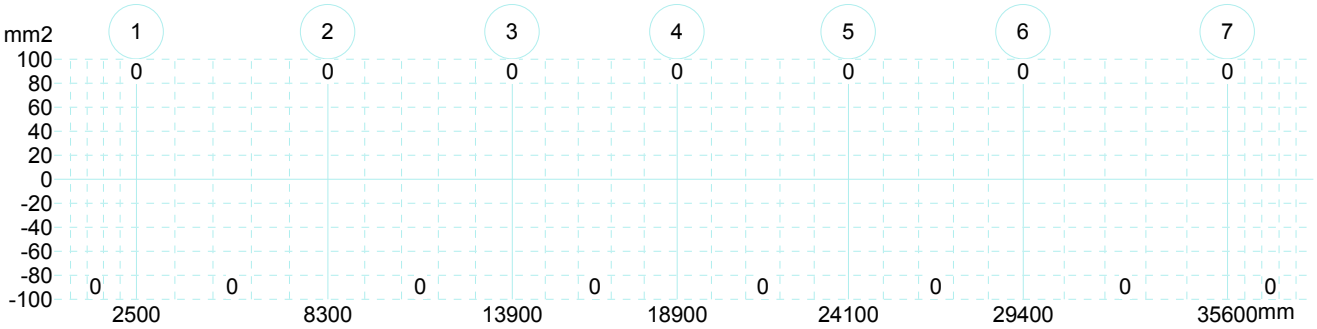
No errors or warnings were found.

Output

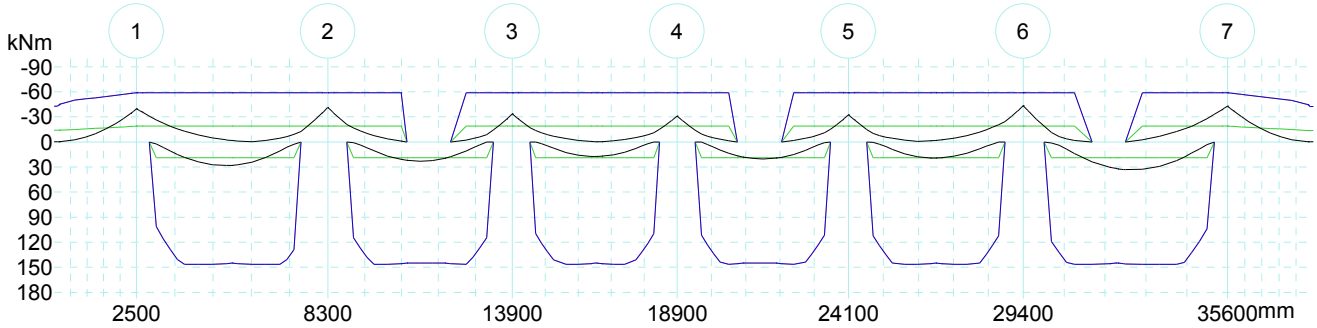
Warning:Total Deflection span/deflection ratio in at least one span is less than defined limit.

Flexural Design

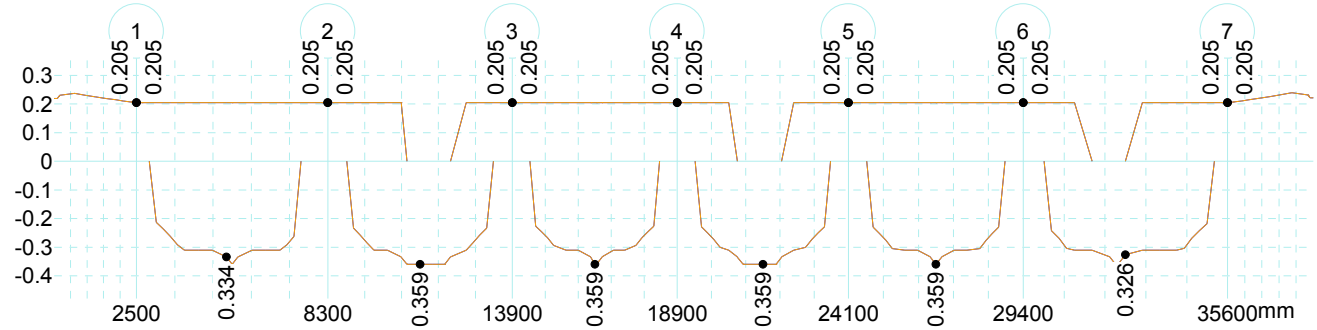
Ultimate



Reinforcement Top Total | Bottom Total | Top Ultimate | Bottom Ultimate | Min Top | Min Bot



Capacity Minimum | Ultimate Design | Initial | Final

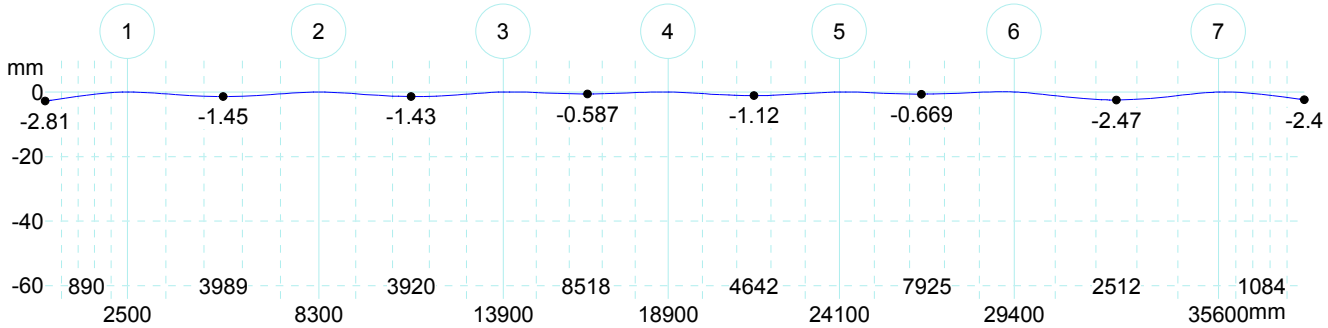


Neutral Axis Depth Initial | Final

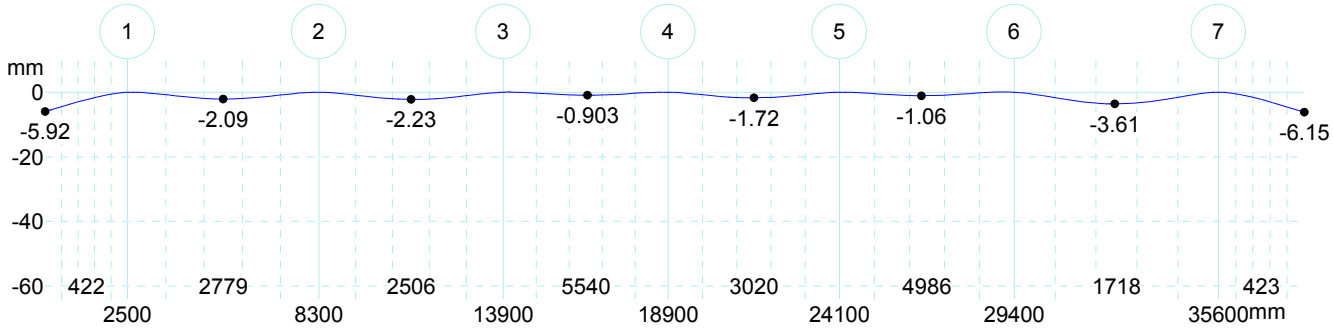
Deflections

All Spans Loaded

Transfer

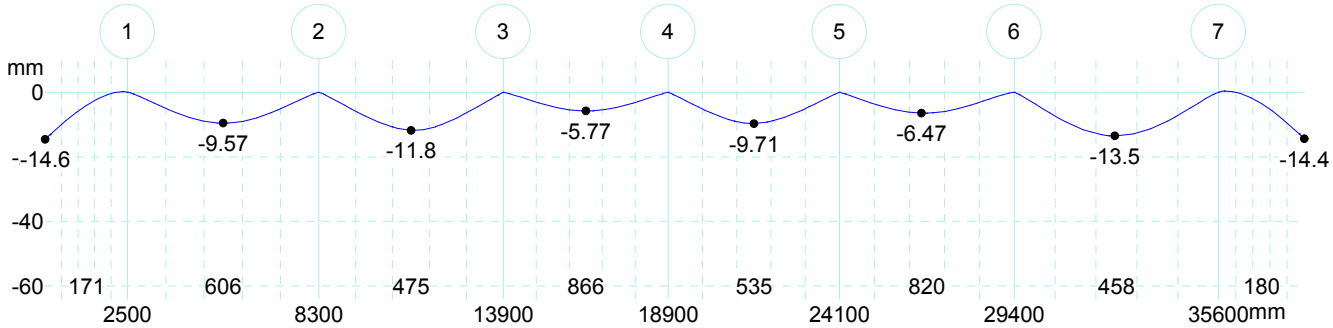


Short Term



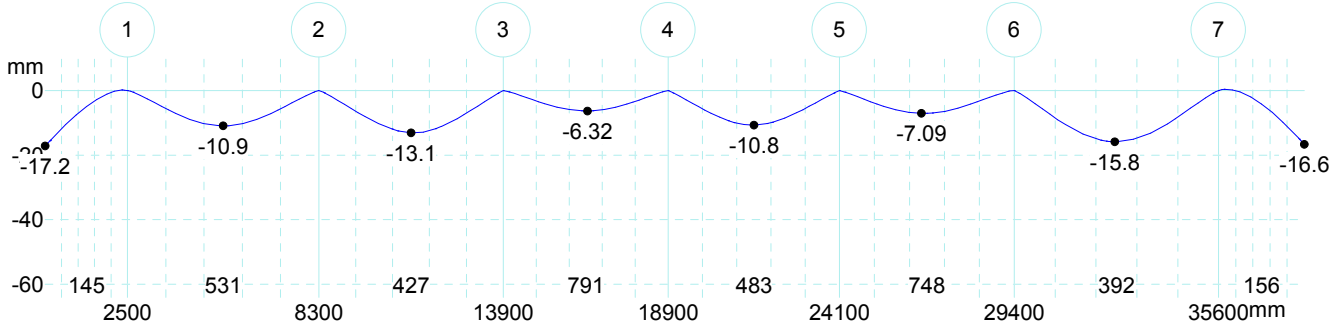
Even Spans Loaded | Odd Spans Loaded | All Spans Loaded

Incremental



Even Spans Loaded | Odd Spans Loaded | All Spans Loaded

Total Long Term



Even Spans Loaded | Odd Spans Loaded | All Spans Loaded

CALCULATIONS	REF./COMMENT

c) <u>50mm wet area setdown adjacent to stairs</u>

50mm setdowns were required to all wet areas. As the soffit of the slab was not setdown, the thickness of the slab in the wet area reduced by 50mm making the total overal depth of the slab 120mm in these areas.

Initial analysis showed the slab system failing over the 6m span due to the reduced thickness, therefore intermediate beams were considered reducing the span to approximately 2.3m. The results can be seen in the Fielder - KingFlor output on the next page.

This was mainly a concern on levels two and three as the slab in this area on level 1 is supported by walls below. Ground floor does not have a setdown to this extent.

COMPANY NAME:		Date:	22/05/2018
CLIENT NAME:		Time:	10:14
JOB REFERENCE:		Job No:	
DECK REFERENCE:		Calcs by:	
FILENAME:	Short span setdown.pmd	Checked by:	

SUMMARY OUTPUT

Construction Stage:	**PASS**	**Max Unity Factor =**	**0.27**
Composite Stage:	**PASS**	**Max Unity Factor =**	**0.30**
Serviceability:	**PASS**	**Max Unity Factor =**	**0.83**
Fire State:	**PASS**	**Max Unity Factor =**	**0.71**

***** SECTION ADEQUATE *****

FLOOR PLAN DATA : (propped composite construction with Fielders KF57 decking)

Support Centres	2.300 m	Span Type	Double
Beam or wall width	100 mm	Row of props	One at 1/2 span
Shortest Sheet Length	4.600 m	Prop width	100 mm

PROFILE DATA : (Fielders KF57 decking. Grade G 550)

Overall Depth	58 mm	Design sheet thickness	1.00 mm
Trough width	300 mm	Nominal sheet thickness	1.04 mm
Pitch of deck ribs	300 mm	Deck weight	0.13 kPa
Average Inertia, Ixx	0.502 * 10^6 mm4/m	Area of profile, Ap	1593 mm²/m
Yield strength	550 MPa		

CONCRETE DATA : Normal Weight Concrete (NWC):

Overall slab depth	120 mm	Concrete wet density	2450 kg/m³
Concrete characteristic strength	32 MPa	Concrete dry density	2400 kg/m³
Modular ratio	7.0	Concrete Volume	0.120 m³/m²
Transverse crack control	N.A.	Concrete Gamma	0.822
Screed depth	50 mm		

REINFORCEMENT DATA:

Reinforcement Name	SL82	Yield Strength	500 MPa
Top Cover	35 mm		
Bar reinforcement diameter	16 mm	Bar yield strength	500 MPa
Bar Top Cover	27 mm	Bar Spacing	200 mm
Fire Bar diameter	16 mm	Fire Bar yield strength	500 MPa
Fire Bar Bottom Cover	35 mm	Fire Bar Spacing	1 per 300 mm

SECTION PROPERTIES :

*NOTE - 1: All values of inertia are expressed in steel units.
*NOTE - 2: Effective inertia is used for deflection calculations in continuous spans.

COMPOSITE:

Inertia, Ixx - Uncracked	24.25 * 10^6 mm4/m	Cracked Inertia (+ve)	11.35 * 10^6 mm4/m
Effective Inertia	23.40 * 10^6 mm4/m	Cracked Inertia (-ve)	4.95 * 10^6 mm4/m

LOADS ACTING ON SLAB :

*** NOTE: Slab subjected to uniformly distributed loads (UDL) only

Occupancy Live Loads (Pattern On)	3.00 kPa	Partition loads	0.50 kPa
Ceilings and services	0.35 kPa	Construction load	1.00 kPa
Self weight of concrete slab (wet)	2.89 kPa	Self weight of decking	0.13 kPa
Self weight of concrete slab (dry)	2.83 kPa	Self weight of screeds	0.98 kPa

FIRE DATA :

Design method	VUT, April 2002	Fire resistance period	120 mins

LOAD FACTORS :

Dead (self weight)	1.20	Live Strength Limit		1.50
Superimposed dead	1.20	Live Combination	ψ_c	0.40
Fire Limit	1.00		ψ_l	0.40
			ψ_s	0.70

UNITY FACTORS :

Transverse Reinforcement Area (N.A.)	N.A.
Minimum Positive Tensile Reinforcement Area	0.11
Minimum Negative Tensile Reinforcement Area	0.33
Long Reinforcement Spacing	0.83
Long Reinforcement Stress	0.19

CALCULATIONS	REF./COMMENT
d) <u>50mm wet area set down on the Northern side of the blockwork stairwell</u> Adjacent to the blockwork stairwell on levels one, two and three is a wet area setdown. As the stairwell causes the slab to not be continuous in this area, analysis was undertaken to ensure deflections were adequate. The results can be seen in the RAPT output on the next page. 30% Stiffness was considered at the fixing of the slab to the blockwork.	

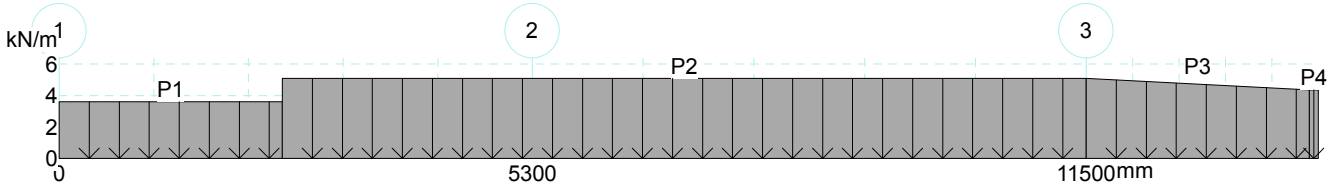
Project Name: 97 King William Street, Kent Town
Project Number: 2018-7161
Frame Description: d) Setdown to the Northern side of blockwork stairwell
Designer: JT
S:\01 - PROJECTS (FROM 14-7-2010)\2018-7161 - 97 King William St, Kent Town\02_Technical\2_Calculations\Structural\Slabs\RAPT\Levels 2 _3 adjacent to block stairwell w setdown.rpf

RAPT - Version: 6.5.6.0
Reinforced And Post-Tensioned Concrete Analysis & Design Package
Copyright(C) 1988 - 2015 PCDC Pty. Ltd. All Rights Reserved

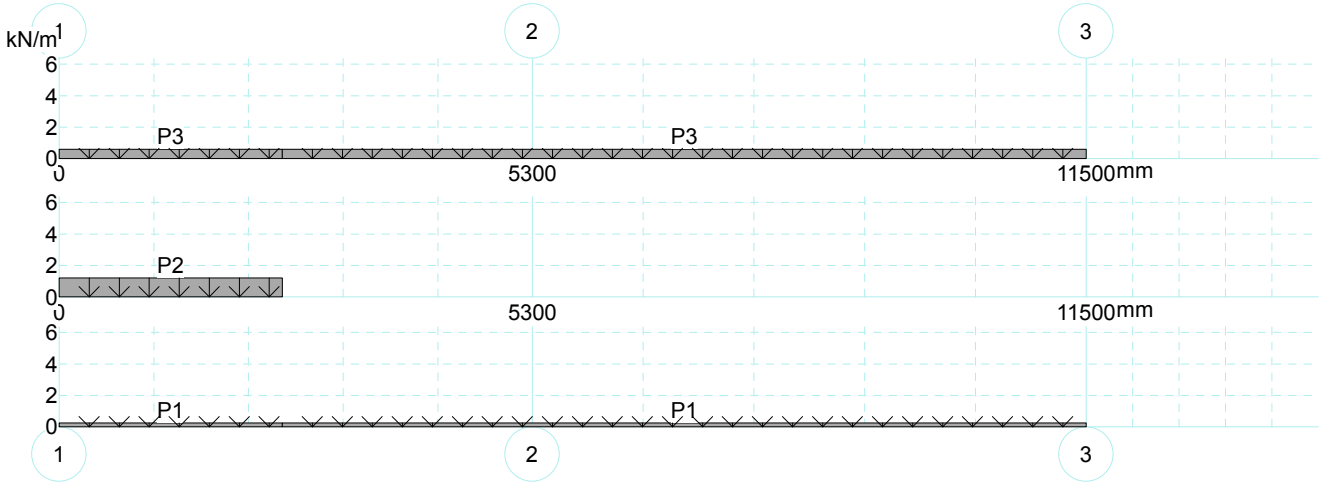
Licensee
MLEI Consultants
Level 5
19 Gilles Street
Adelaide SA 5000
11104065151117WPN1

Input

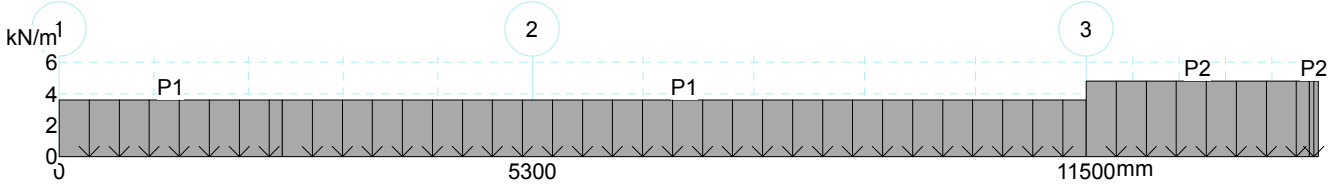
Load Case 1 : 1. Self Weight



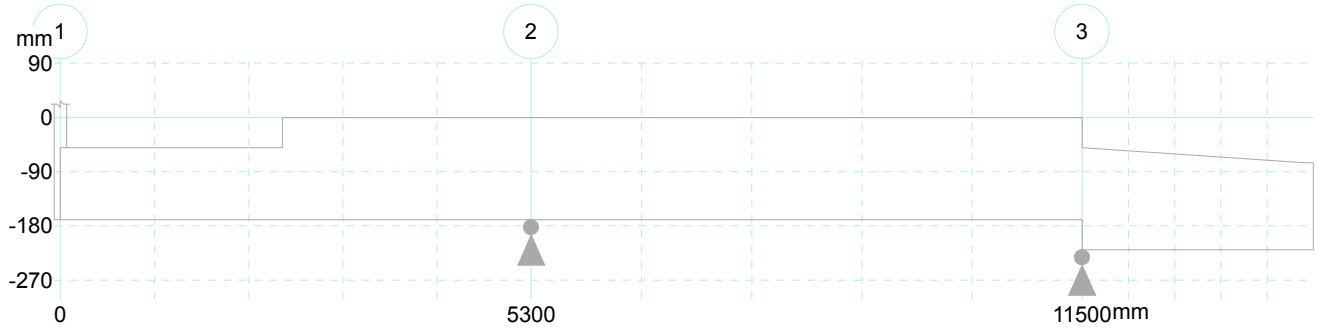
Load Case 2 : 2. Extra Dead Load



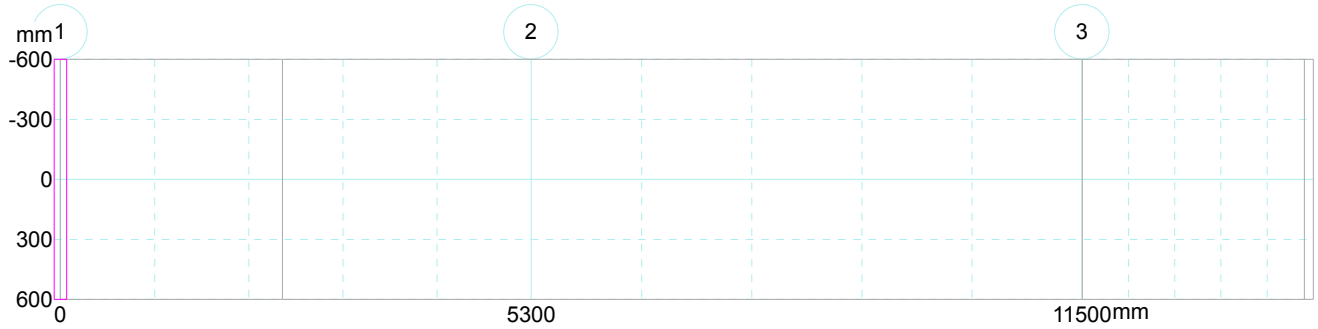
Load Case 3 : 3. Live Load



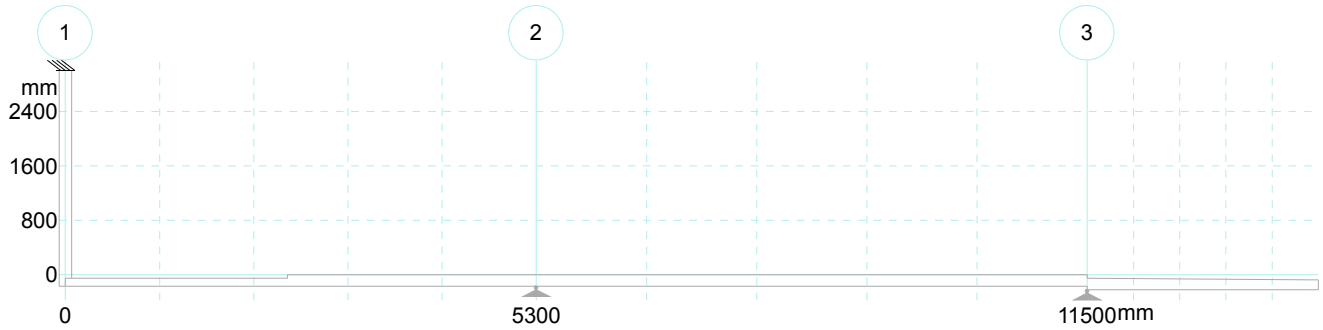
Elevation view



Plan view



Full Elevation View



Warnings

Input

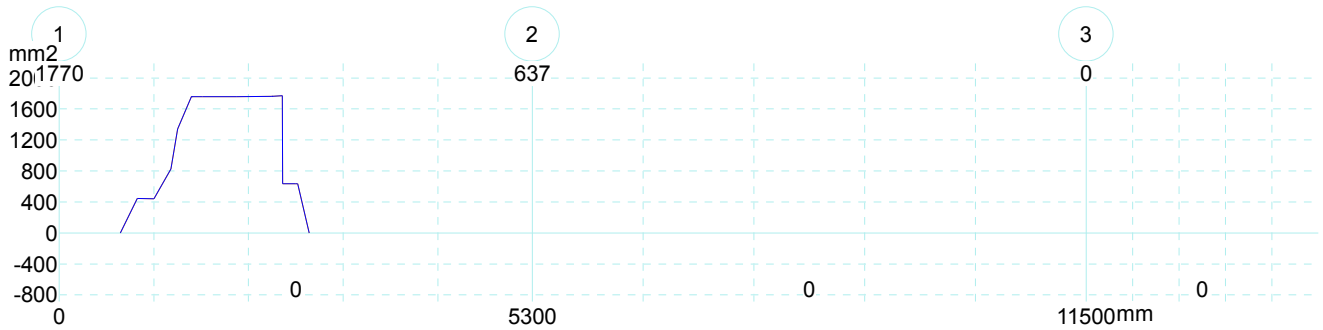
No errors or warnings were found.

Output

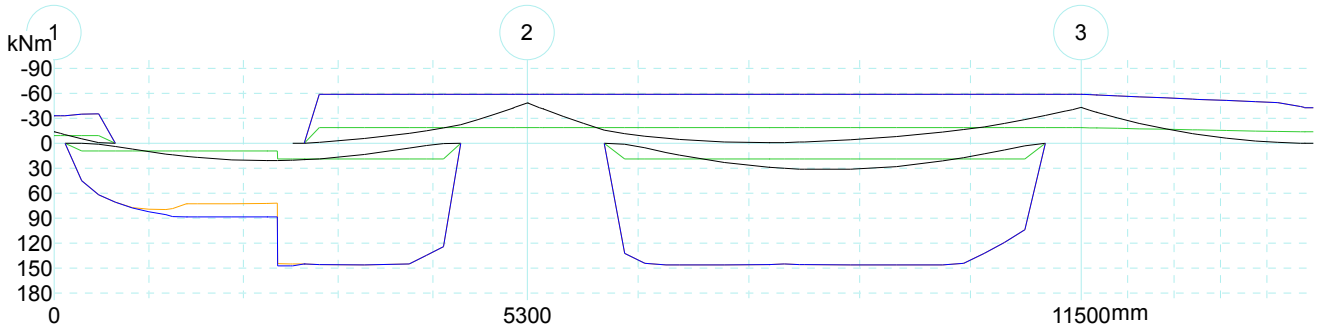
Warning:Incremental Deflection span/deflection ratio in at least one span is less than defined limit.
Warning:Total Deflection span/deflection ratio in at least one span is less than defined limit.

Flexural Design

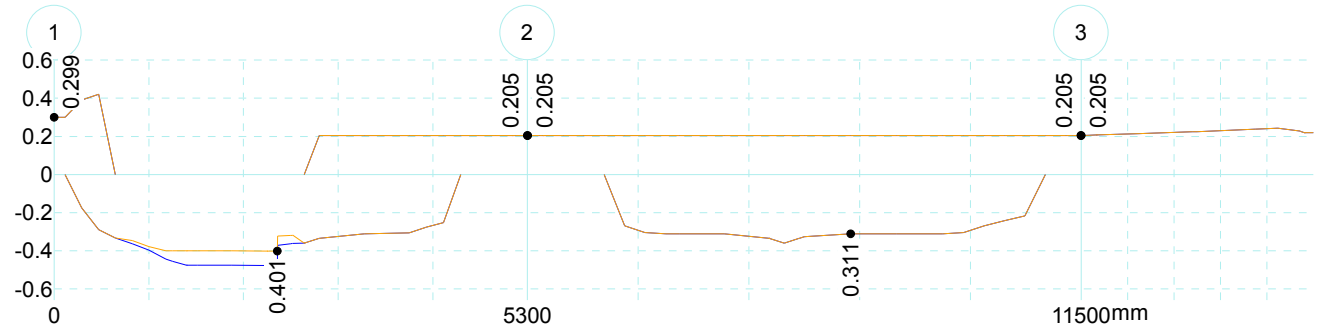
Ultimate



Reinforcement Top Total | Bottom Total | Top Ultimate | Bottom Ultimate | Min Top | Min Bot



Capacity Minimum Ultimate Design Initial Final



Neutral Axis Depth Initial Final

Span 1

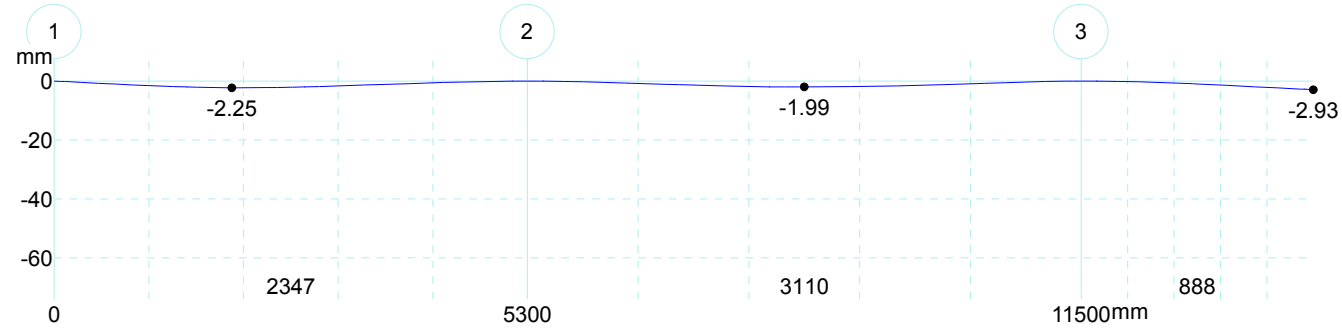
Design Comments:-

- Compression reinforcement added at span hinge location for ductility in accordance with AS3600 cl.8.1.5. Asc = 1770.43mm2 @ 2499mm

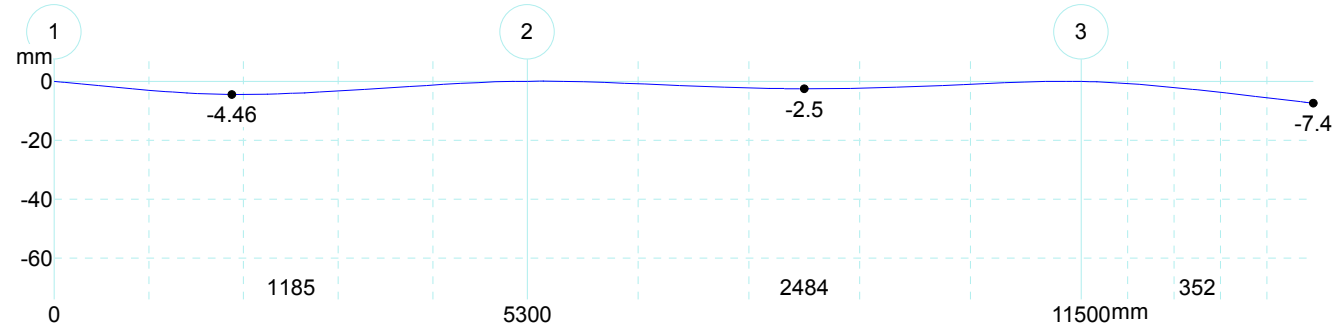
Deflections

All Spans Loaded

Transfer

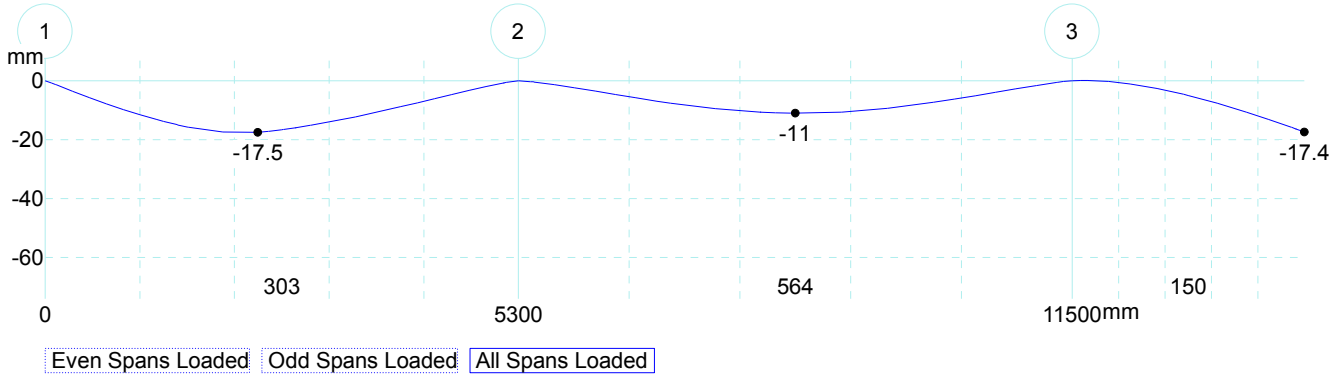


Short Term

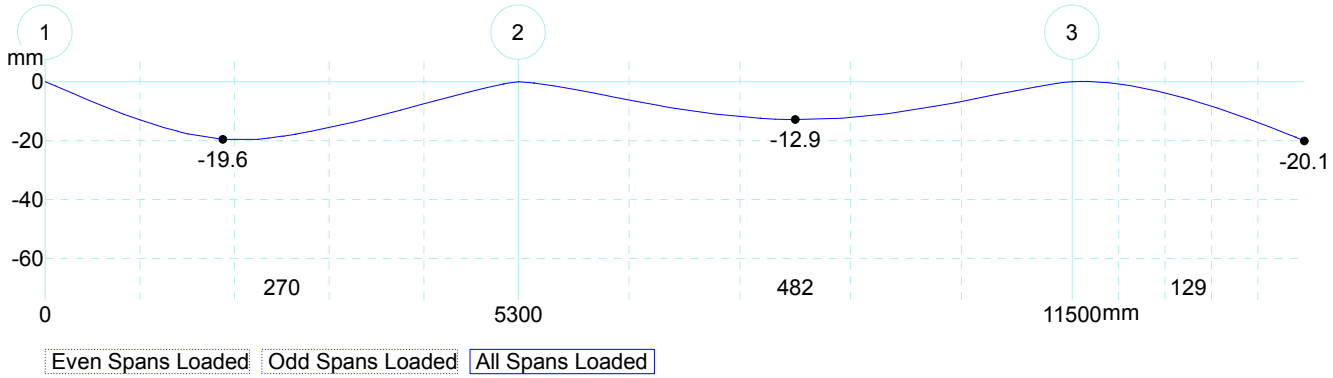


Even Spans Loaded Odd Spans Loaded All Spans Loaded

Incremental



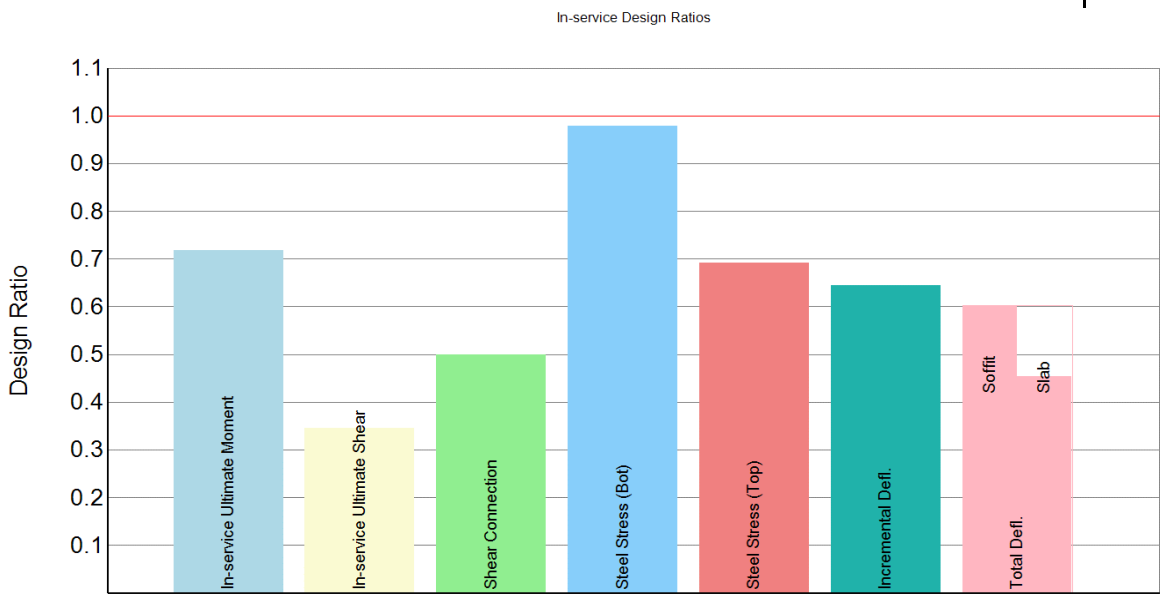
Total Long Term



CALCULATIONS				REF./COMMENT
8) COMPOSITE BEAM DESIGN				
Using Fielders "KingBeam" software to design composite beam. This beam is the typical floor beam spanning 12m with a maximum loadwidth of 6m. Design of composite beams is in accordance with AS 2327.1.				
a) Typical composite floor beam - B1				
INPUT	Member:	610 UB 101		
	Span, L =	12000	mm	
	Loadwidth, b_1 =	6000	mm = b_2 (internal)	
	b_2 =	2600	mm (overhang)	Worst case
	Flange width, b_{sf1} =	228	mm = b_x	
LOADING				
Stage 1&2 Loads	Live load, Q =	1.0	kPa	Stacked mats on sheet
	or =	2.0	kN at any point	
Stage 3 (calculated by KingFlor)	Live load, Q =	1.0	kPa	
	or =	2.0	kPa over 2.6 m^2 area	
Stage 5&6	Superimposed dead load, G =	0.5	kPa	Partitions
	=	0.2	kPa	Services
In Service	Live load, Q =	3.0	kPa	Office
	or =	2.7	kN at any point	"
i) Design of Shear Studs				
19mm shear stud, 32MPa concrete	f_{vs} =	93	MPa	Table 8.1
	F_{cp} =	1982	kN	KingFlor software
$n = F_{cp}/f_{vs}$	n =	24	each side of midspan	
$k_n = 1.18-(0.18/\sqrt{n})$	k_n =	1.1		
	f_{ds} =	90	MPa	
	nf_{ds} =	2169	kN > F_{cp}	***0.91 OK***
$f_{ds} = \emptyset k_n f_{vs}$	s =	250	mm	
Spacing must be less than 4 times the slab depth and 600mm				
	D_c =	170	mm	
	4D =	680	mm	
	spacing =	250	mm	

CALCULATIONS	REF./COMMENT

ii) Strength and serviceability checks



In-service Design Moment, M* =	1143.5 kNm		
Moment Capacity, ϕM_{bc} =	1592 kNm		
In-service Design Shear, V* =	381.2 kN		
Shear Capacity, ϕV_u =	1102 kN		
Stress in Bottom Flange =	264.0 MPa	(tension)	
Stress in Top Flange =	186.6 MPa	(compression)	
$0.9f_y$ =	270 MPa		
Dead Load Deflection, δ_{DL} =	49.9 mm		
Total Beam Deflection, δ =	68.9 mm		
Total Beam Deflection with pre-camber, δ_{pc} =	28.9 mm		
Final Beam Deflection, = span/	415	> 250	***OK***

Note: Beam is pre-cambered 40mm.

Check frequency

Apply 1.0 kN load at midspan and see if defleciton is less than 2mm as per AS/NZS 1170.1. No dead load including self weight is considered. Beam is considered as fixed-fixed as no rotation occurs under such a small loading. No composite action is considered (conservative).

$$\delta = \frac{PL^3}{192EI}$$

P =	1.0 kN		
E =	200000 MPa		
I =	761 x 10^6 mm^4		
δ =	0.06 mm	< 2mm	***OK***

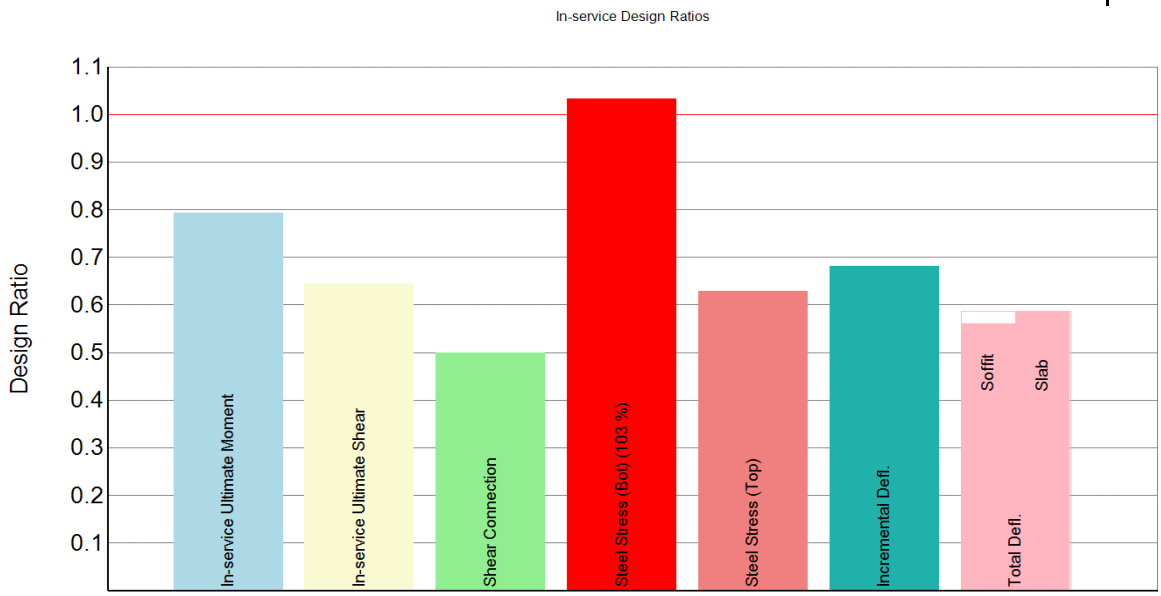
CALCULATIONS				REF./COMMENT
iii) Check longitudinal reinforcement				Sec. 9
Try: N16 @ 200 centres	Effective length, L_{ef} =	12000 mm		
	Stud Height, h_c =	90 mm		
	Stud Diameter =	19 mm		
	Shear Reo Diameter =	16 mm		
	Spacing =	200 mm		
	A_{sv} =	1005 mm^2/m		
	Yield strength of reinforcement, f_{yr} =	500 MPa		
φV_L is the lesser of φV_{L1} and φV_{L2}, where;				
$\emptyset V_{L1} = 0.32 f_c' u$				eq. 9.6(1)
$\emptyset V_{L2} = \emptyset u \left(0.36\sqrt{f_c'}\right) + 0.9 A_{sv} f_{yr}$				eq. 9.6(2)
Type 1	V_L* =	271 kN/m		eq. 9.5(1)
	u = D_c =	170 mm		cl. 9.4.2.3 b)
	ϕV_{L1} =	1741 kN/m		
	ϕV_{L2} =	799 kN/m		
	Therefore, ϕV_L =	799 kN/m	> V_L*	***0.34 OK***
Type 2	V_L* =	542 kN/m		eq. 9.5(2)
	u = $b_x + 2h_c$ =	408 mm		
	ϕV_{L1} =	4178 kN/m		
	ϕV_{L2} =	1193 kN/m		
	Therefore, ϕV_L =	1193 kN/m	> V_L*	***0.45 OK***
Type 3	Not applicable as metal decking runs perpedicular to beam in all cases			
Check development length of reinforcement				
$k_3 = 0.7 < 1.0 - \frac{0.15(c_d - d_b)}{d_b} < 1.0$	k_1 =	1.0		
	k_2 =	1.16		
	k_3 =	0.70		
$L_{sy.tb} = \frac{0.5 k_1 k_3 f_{sy} d_b}{k_2 \sqrt{f_c'}} > 29 k_1 d_b$	$L_{sy.tb}$ =	427 mm		
	$29k_1d_b$ =	464 mm		
Min. development length required = 470mm				

Adopt: 610 UB 101 Composite Beam with 19mm Φ Headed Stud Shear Connectors at 250mm CTS

CALCULATIONS				REF./COMMENT
b) Composite floor beam with additional point loads between grids D and E				
INPUT	Member:	610 UB 125		
	Span =	12000	mm	
	Loadwidth, b_1 =	6000	mm = b_2 (internal)	
	Flange width, b_{sf1} =	229	mm = b_x	
LOADING				
Stage 1&2 Loads	Live load, Q =	1.0	kPa	Stacked mats on sheet
	or =	2.0	kN at any point	
Stage 3 (calculated by KingFlor)	Live load, Q =	1.0	kPa	
	or =	2.0	kPa over 2.6 m^2 area	
Stage 5&6	Superimposed dead load, G =	0.5	kPa	Partitions
	=	0.2	kPa	Services
	=	3.5	kPa	Blockwork
In Service Refer to loading breakdown on the next page.				
i) Design of Shear Studs				
19mm shear stud, 32MPa concrete	f_{vs} =	93	MPa	Table 8.1
	F_{cp} =	2278	kN	KingFlor software
$n = F_{cp}/f_{vs}$	n =	26	each side of midspan	
$k_n = 1.18-(0.18/\sqrt{n})$	k_n =	1.1		
	f_{ds} =	90	MPa	
	nf_{ds} =	2353	kN > F_{cp}	***0.97 OK***
$f_{ds} = \emptyset k_n f_{vs}$	s =	230	mm	
Spacing must be less than 4 times the slab depth and 600mm				
	D_c =	170	mm	
	4D =	680	mm	
	spacing =	230	mm	

CALCULATIONS	REF./COMMENT

ii) Strength and serviceability checks



3% overstress is deemed ok due to the factored live loads

In-service Design Moment, M* =	1455.1	kNm	
Moment Capacity, ϕM_{bc} =	1834	kNm	
In-service Design Shear, V* =	759.3	kN	
Shear Capacity, ϕV_u =	1178	kN	
Stress in Bottom Flange =	260.4	MPa	(tension)
Stress in Top Flange =	158.2	MPa	(compression)
$0.9f_y$ =	270	MPa	
Dead Load Deflection, δ_{DL} =	47.1	mm	
Total Beam Deflection, δ =	66.9	mm	
Total Beam Deflection with pre-camber, δ_{pc} =	26.9	mm	
Final Beam Deflection, = span/	446		> 250

OK

Note: Beam is pre-cambered 40mm.

Check frequency

Apply 1.0 kN load at midspan and see if defleciton is less than 2mm as per AS/NZS 1170.1. No dead load including self weight is considered. Beam is considered as fixed-fixed as no rotation occurs under such a small loading. No composite action is considered (conservative).

$$\delta = \frac{PL^3}{192EI}$$

P =	1.0	kN	
E =	200000	MPa	
I =	986 x 10^6	mm^4	
δ =	0.046	mm	< 2mm

OK

CALCULATIONS	REF./COMMENT

iii) Check longitudinal reinforcement

Try: N16 @ 200 centres

Effective length, L_{ef} =	12000	mm	
Stud Height, h_c =	90	mm	
Stud Diameter =	19	mm	
Shear Reo Diameter =	16	mm	
Spacing =	200	mm	
A_{sv} =	1005	mm^2/m	
Yield strength of reinforcement, f_{yr} =	500	MPa	

φV_L is the lesser of φV_{L1} and φV_{L2}, where;

$\emptyset V_{L1} = 0.32 f_c' u$ — eq. 9.6(1)

$\emptyset V_{L2} = \emptyset u \left(0.36\sqrt{f_c'}\right) + 0.9 A_{sv} f_{yr}$ — eq. 9.6(2)

Type 1

V_L* =	271	kN/m		eq. 9.5(1)
u = D_c =	170	mm		cl. 9.4.2.3 b)
ϕV_{L1} =	1741	kN/m		
ϕV_{L2} =	799	kN/m		
Therefore, ϕV_L =	799	kN/m	> V_L*	***0.34 OK***

Type 2

V_L* =	541	kN/m		eq. 9.5(2)
u = $b_x + 2h_c$ =	409	mm		
ϕV_{L1} =	4188	kN/m		
ϕV_{L2} =	1195	kN/m		
Therefore, ϕV_L =	1195	kN/m	> V_L*	***0.45 OK***

Type 3 Not applicable as metal decking runs perpedicular to beam in all cases

Check development length of reinforcement

$$k_3 = 0.7 < 1.0 - \frac{0.15(c_d - d_b)}{d_b} < 1.0$$

$$L_{sy.tb} = \frac{0.5 k_1 k_3 f_{sy} d_b}{k_2 \sqrt{f_c'}} > 29 k_1 d_b$$

k_1 =	1	
k_2 =	1.16	
k_3 =	0.70	
$L_{sy.tb}$ =	427	mm
$29k_1d_b$ =	464	mm

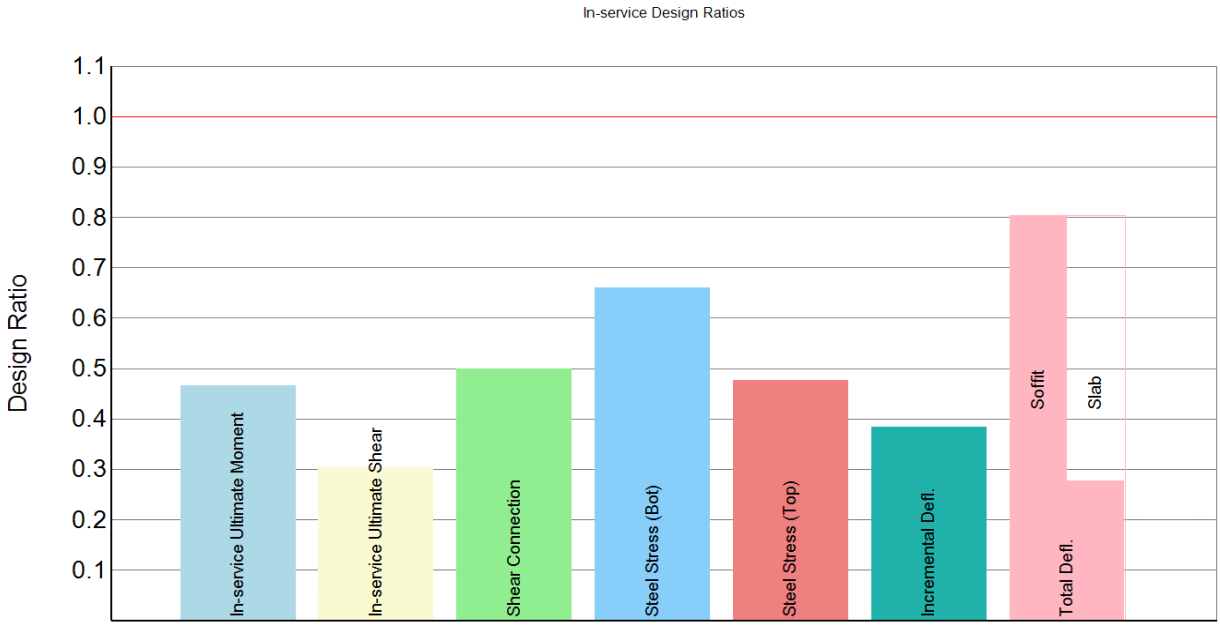
Min. development length required = 470mm

Adopt: 610 UB 125 Composite Beam with 19mm Φ Headed Stud Shear Connectors at 230mm CTS

CALCULATIONS				REF./COMMENT
c) Shorter span composite beam				
INPUT	Member:	530 UB 82		
	Span =	8200	mm	
	Loadwidth, b_1 =	6000	mm = b_2 (internal)	
	Flange width, b_{sf1} =	209	mm = b_x	
LOADING				
Stage 1&2 Loads	Live load, Q =	1.0	kPa	Stacked mats on sheet
	or =	2.0	kN at any point	
Stage 3 (calculated by KingFlor)	Live load, Q =	1.0	kPa	
	or =	2.0	kPa over 2.6 m^2 area	
Stage 5&6	Superimposed dead load, G =	0.5	kPa	Partitions
	=	0.2	kPa	Services
In Service	Live load, Q =	3.0	kPa	Office
	or =	2.7	kN at any point	"
i) Design of Shear Studs				
19mm shear stud, 32MPa concrete	f_{vs} =	93	MPa	Table 8.1
	F_{cp} =	1599	kN	KingFlor software
$n = F_{cp}/f_{vs}$	n =	18	each side of midspan	
$k_n = 1.18-(0.18/\sqrt{n})$	k_n =	1.1		
	f_{ds} =	90	MPa	
	nf_{ds} =	1619	kN > F_{cp}	***0.99 OK***
$f_{ds} = \emptyset k_n f_{vs}$	s =	220	mm	
Spacing must be less than 4 times the slab depth and 600mm				
	D =	170	mm	
	4D =	680	mm	
	spacing =	220	mm	

CALCULATIONS	REF./COMMENT

ii) Strength and serviceability checks



In-service Design Moment, M* =	544.7	kNm	
Moment Capacity, ϕM_{bc} =	1167	kNm	
In-service Design Shear, V* =	265.7	kN	
Shear Capacity, ϕV_u =	877	kN	
Stress in Bottom Flange =	178.5	MPa	(tension)
Stress in Top Flange =	128.6	MPa	(compression)
$0.9f_y$ =	270	MPa	
Dead Load Deflection, δ_{DL} =	18.2	mm	
Total Beam Deflection, δ =	26.3	mm	
Total Beam Deflection with pre-camber, δ_{pc} =	26.3	mm	
Final Beam Deflection, = span/	312		> 250

OK

Note: Beam is pre-cambered 0mm.

Check frequency

Apply 1.0 kN load at midspan and see if defleciton is less than 2mm as per AS/NZS 1170.1. No dead load including self weight is considered. Beam is considered as fixed-fixed as no rotation occurs under such a small loading. No composite action is considered (conservative).

$$\delta = \frac{PL^3}{192EI}$$

P =	1.0	kN	
E =	200000	MPa	
I =	477 x 10^6	mm^4	
δ =	0.030	mm	< 2mm

OK

CALCULATIONS	REF./COMMENT

iii) Check longitudinal reinforcement

Try: N16 @ 200 centres

Effective length, L_{ef} =	8200	mm
Stud Height, h_c =	90	mm
Stud Diameter =	19	mm
Shear Reo Diameter =	16	mm
Spacing =	200	mm
A_{sv} =	1005	mm^2/m
Yield strength of reinforcement, f_{yr} =	500	MPa

φV_L is the lesser of φV_{L1} and φV_{L2}, where;

$\emptyset V_{L1} = 0.32 f_c' u$ — eq. 9.6(1)

$\emptyset V_{L2} = \emptyset u \left(0.36\sqrt{f_c'}\right) + 0.9 A_{sv} f_{yr}$ — eq. 9.6(2)

Type 1

V_L* =	178	kN/m		eq. 9.5(1)
u = D_c =	170	mm		cl. 9.4.2.3 b)
ϕV_{L1} =	1741	kN/m		
ϕV_{L2} =	799	kN/m		
Therefore, ϕV_L =	799	kN/m	> V_L*	***0.22 OK***

Type 2

V_L* =	356	kN/m		eq. 9.5(2)
u = $b_x + 2h_c$ =	389	mm		
ϕV_{L1} =	3983	kN/m		
ϕV_{L2} =	1154	kN/m		
Therefore, ϕV_L =	1154	kN/m	> V_L*	***0.31 OK***

Type 3 Not applicable as metal decking runs perpedicular to beam in all cases

Check development length of reinforcement

$$k_3 = 0.7 < 1.0 - \frac{0.15(c_d - d_b)}{d_b} < 1.0$$

$$L_{sy.tb} = \frac{0.5 k_1 k_3 f_{sy} d_b}{k_2 \sqrt{f_c'}} > 29 k_1 d_b$$

k_1 =	1.0	
k_2 =	1.16	
k_3 =	0.70	
$L_{sy.tb}$ =	427	mm
$29k_1d_b$ =	464	mm

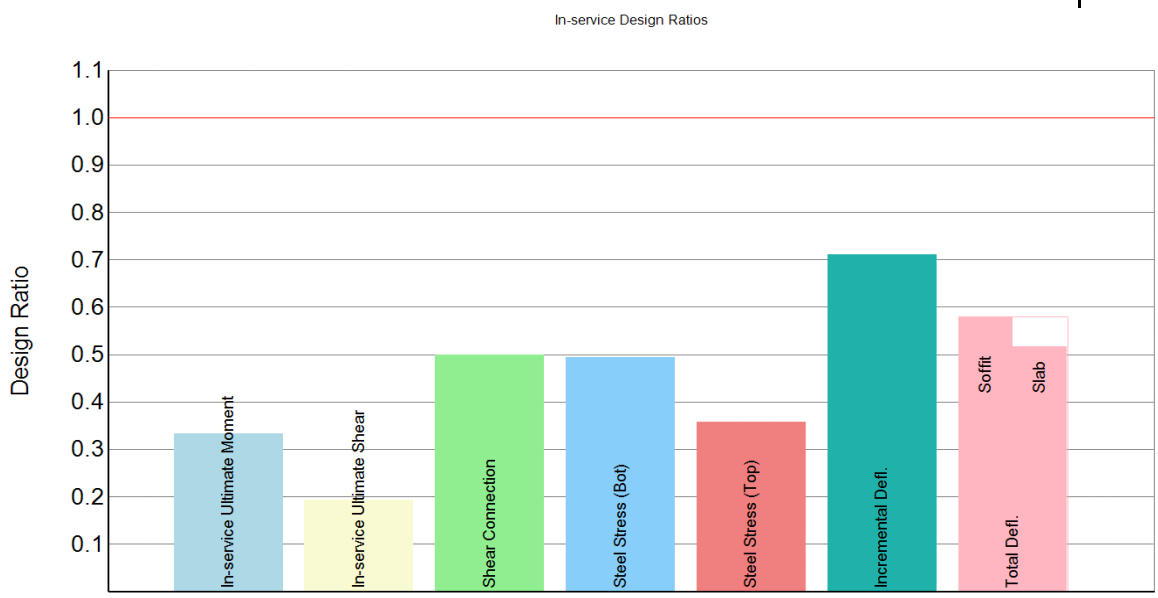
Min. development length required = 470mm

Adopt: 530 UB 82 Composite Beam with 19mm Φ Headed Stud Shear Connectors at 220mm CTS

CALCULATIONS				REF./COMMENT
d) Special case composite beam for reduced head room - B6				
INPUT	Member:	310 UC 137		
	Span =	12000	mm	
	Loadwidth, b_1 =	2150	mm = b_2 (internal)	
	b_2 =	1850	mm (overhang)	Worst case
	Flange width, b_{sf1} =	309	mm = b_x	
LOADING				
Stage 1&2 Loads	Live load, Q =	1.0	kPa	Stacked mats on sheet
	or =	2.0	kN at any point	
Stage 3 (calculated by KingFlor)	Live load, Q =	1.0	kPa	
	or =	2.0	kPa over 2.6 m^2 area	
Stage 5&6	Superimposed dead load, G =	0.5	kPa	Partitions
	=	0.2	kPa	Services
In Service	Live load, Q =	3.0	kPa	Office
	or =	2.7	kN at any point	"
i) Design of Shear Studs				
19mm shear stud, 32MPa concrete	f_{vs} =	93	MPa	Table 8.1
	F_{cp} =	2451	kN	KingFlor software
$n = F_{cp}/f_{vs}$	n =	29	each side of midspan	
$k_n = 1.18-(0.18/\sqrt{n})$	k_n =	1.1		
	f_{ds} =	91	MPa	
	nf_{ds} =	2628	kN > F_{cp}	***0.93 OK***
$f_{ds} = \emptyset k_n f_{vs}$	s =	200	mm	
Spacing must be less than 4 times the slab depth and 600mm				
	D =	170	mm	
	4D =	680	mm	
	spacing =	200	mm	

CALCULATIONS	REF./COMMENT

ii) Strength and serviceability checks



In-service Design Moment, M* =	422.8	kNm	
Moment Capacity, ϕM_{bc} =	1272	kNm	
In-service Design Shear, V* =	138.9	kN	
Shear Capacity, ϕV_u =	716	kN	
Stress in Bottom Flange =	124.3	MPa	(tension)
Stress in Top Flange =	90.2	MPa	(compression)
$0.9f_y$ =	270	MPa	
Dead Load Deflection, δ_{DL} =	45.2	mm	
Total Beam Deflection, δ =	57.8	mm	
Total Beam Deflection with pre-camber, δ_{pc} =	27.8	mm	
Final Beam Deflection, = span/	432		> 250

OK

Note: Beam is pre-cambered 40mm.

Check frequency

Apply 1.0 kN load at midspan and see if defleciton is less than 2mm as per AS/NZS 1170.1. No dead load including self weight is considered. Beam is considered as fixed-fixed as no rotation occurs under such a small loading. No composite action is considered (conservative).

$$\delta = \frac{PL^3}{192EI}$$

P =	1.0	kN	
E =	200000	MPa	
I =	329 x 10^6	mm^4	
δ =	0.137	mm	< 2mm

OK

CALCULATIONS	REF./COMMENT

iii) Check longitudinal reinforcement

Try: N16 @ 200 centres

Effective length, L_{ef} =	12000	mm
Stud Height, h_c =	90	mm
Stud Diameter =	19	mm
Shear Reo Diameter =	16	mm
Spacing =	200	mm
A_{sv} =	1005	mm^2
Yield strength of reinforcement, f_{yr} =	500	MPa

φV_L is the lesser of φV_{L1} and φV_{L2}, where;

$$\emptyset V_{L1} = 0.32 f_c' u$$ eq. 9.6(1)

$$\emptyset V_{L2} = \emptyset u \left(0.36\sqrt{f_c'}\right) + 0.9 A_{sv} f_{yr}$$ eq. 9.6(2)

Type 1

V_L* =	263	kN/m		eq. 9.5(1)
u = D_c =	170	mm		cl. 9.4.2.3 b)
ϕV_{L1} =	1741	kN/m		
ϕV_{L2} =	799	kN/m		
Therefore, ϕV_L =	799	kN/m	> V_L*	***0.33 OK***

Type 2

V_L* =	526	kN/m		eq. 9.5(2)
u = $b_x + 2h_c$ =	489	mm		
ϕV_{L1} =	5007	kN/m		
ϕV_{L2} =	1358	kN/m		
Therefore, ϕV_L =	1358	kN/m	> V_L*	***0.39 OK***

Type 3 Not applicable as metal decking runs perpedicular to beam in all cases

Check development length of reinforcement

$$k_3 = 0.7 < 1.0 - \frac{0.15(c_d - d_b)}{d_b} < 1.0$$

$$L_{sy.tb} = \frac{0.5 k_1 k_3 f_{sy} d_b}{k_2 \sqrt{f_c'}} > 29 k_1 d_b$$

k_1 =	1	
k_2 =	1.16	
k_3 =	0.70	
$L_{sy.tb}$ =	427	mm
$29k_1d_b$ =	464	mm

Min. development length required = 470mm

Adopt: 310 UC 137 Composite Beam with 19mm Φ Headed Stud Shear Connectors at 200mm CTS

CALCULATIONS	REF./COMMENT
9) <u>BEAM DESIGN</u> Critical non-composite floor beams where designed using "Structural Toolkit". The output can be found on the following pages. Refer to drawings for member marks.	

Member:	**(B2 - G, adjacent to entry ramp) 380x100PFC (G300)**	
Bending:	**M(max)* = 90.5kNm < øMb(2600,αm=1.30) = 221.2kNm**	**OK (0.41)**
Shear:	**V.max*=69.6kN < øVvm = 656.6kN (Web area full depth)**	**OK (0.11)**
Deflection:	**δdl = L/1284 (4mm), Ψs.δll = L/3163 (2mm), δtot. = L/913 (6mm), 1kN midspan δ =0.1mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 33.6kN, Rll = 19.5kN, R* = 69.6kN**	

Geometry

Span (L) =	5200	mm	Effective length (Le) =	2600	mm
Centres (cts) =	2500	mm	αm =	1.30	

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 2600 mm
αm = 1.30

Loadings

Floor area =	13.0	m²	Live load type =	N	(N)ormal, (S)torage, (M)anual
Apply reduction =	N	(Y)es,(N)o			AS/NZS 1170.1 - Table 4.1
Floor reduction (Ψa) =	1.00	AS/NZS 1170.1 - Cl 3.4.2			

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	2500	mm +		kN/m =	10.63	kN/m
Partitions (wdl) =	0.50	kPa *	2500	mm +		kN/m =	1.25	kN/m
Services (wdl) =	0.20	kPa *	2500	mm +		kN/m =	0.50	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.55	kN/m
						Σwdl =	12.93	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	2500	mm +		kN/m =	7.50	kN/m
Other live load (wll) =		kPa *	2500	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	7.50 kN/m

Point loads

Dead load (pdl) =		kN	Position =		mm from LHS
Live load (pll) =		kN			

Short term LL (Ψsu) =	0.70		(Ψsp) =	1.00
w* = 1.2*wdl + 1.5*wll =	26.76	kN/m	Rdl =	33.6 kN
p* = 1.2*pdl + 1.5*pll =	0.00	kN	Rll =	19.5 kN
Max M* at =	2600	mm	R* =	69.6 kN
M* =	90.5	kNm (Maximum)		

Capacity

Description =	380x100PFC (G300)		Warping constant (Iw) =	152	x10⁹ mm⁶
Flange yield (fyf) =	280	MPa	Torsional constant (J) =	491	x10³ mm⁴
Web yield (fyw) =	320	MPa	Effective section mod. (Zex) =	946	x10³ mm³
Area (Ag) =	7030	mm²	Effective section mod. (Zey) =	115	x10³ mm³
Stiffness (Ix) =	152	x10⁶ mm⁴	Elastic modulus (E) =	200000	MPa - Cl 1.4
Stiffness (Iy) =	6.48	x10⁶ mm⁴	Shear modulus (G) =	80000	MPa - Cl 1.4
ø =	0.9	Table 3.4			
Msx = min(fyf,fyw)*Zex =	264.9	kNm - Cl 5.2.1	øMsx =	238.4	kNm
Moa =	397.9	kNm - Cl 5.6.1.1(3)	øMsy =	29.0	kNm
αs = 0.714		αm = 1.30	øMbx = αs*αm*øMsx =	221.2	kNm

Deflections

Ireq'd DL (L/360) =	42.6	x10⁶ mm⁴	< Critical	δDL =	4.0 mm	Span / 1284
Ireq'd Ψs.LL (L/360) =	17.3	x10⁶ mm⁴		Ψs.δLL =	1.6 mm	Span / 3163
Ireq'd DL+Ψs.LL (L/250) =	41.6	x10⁶ mm⁴	< Critical	δTotal =	5.7 mm	Span / 913

Max. precamber (0.3%*span) =	16	mm	Min. precamber =	15 mm
Precamber 80% of δDL =	3	mm	Adopted precamber =	0 mm
			1kN midspan δ =	0.1 mm

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Member:	**(B3 - L1, supporting blockwork to the south) 310UB32.0 (G300)**	
Bending:	**M(max)* = 95.8kNm < øMb(1500,αm=1.30) = 134.5kNm**	**OK (0.71)**
Shear:	**V.max*=127.7kN < øVvm = 283.2kN (Web area full depth)**	**OK (0.45)**
Deflection:	**δdl = L/665 (5mm), Ψs.δll = L/3805 (1mm), δtot. = L/566 (5mm), 1kN midspan δ =0.0mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 81.1kN, Rll = 20.3kN, R* = 127.7kN**	

Geometry

Span (L) = 3000 mm
Centres (cts) = 4500 mm
Effective length (Le) = 1500 mm
αm = 1.30

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 1500 mm
αm = 1.30

Loadings

Floor area = 13.5 m²
Apply reduction = N (Y)es,(N)o
Floor reduction (Ψa) = 1.00 AS/NZS 1170.1 - Cl 3.4.2

Live load type = N (N)ormal, (S)torage, (M)anual
AS/NZS 1170.0 - Table 4.1

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	4500	mm +		kN/m =	19.13	kN/m
Blockwork above (wdl) =	3.50	kPa *	9900	mm +		kN/m =	34.65	kN/m
Other dead load (wdl) =		kPa *		mm +		kN/m =	0.00	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.32	kN/m
						Σwdl =	54.10	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	4500	mm +		kN/m =	13.50	kN/m
Other live load (wll) =		kPa *	4500	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	13.50 kN/m

Point loads

Dead load (pdl) = kN
Live load (pll) = kN
Position = mm from LHS

Short term LL (Ψsu) = 0.70
(Ψsp) = 1.00

w* = 1.2*wdl + 1.5*wll = 85.16 kN/m
p* = 1.2*pdl + 1.5*pll = 0.00 kN
Max M* at = 1500 mm
M* = 95.8 kNm (Maximum)

Rdl = 81.1 kN
Rll = 20.3 kN
R* = 127.7 kN

Capacity

Description = 310UB32.0 (G300)
Flange yield (fyf) = 320 MPa
Web yield (fyw) = 320 MPa
Area (Ag) = 4080 mm²
Stiffness (Ix) = 63.2 x10⁶ mm⁴
Stiffness (Iy) = 4.42 x10⁶ mm⁴
ø = 0.9 Table 3.4
Msx = min(fyf,fyw)*Zex = 149.4 kNm - Cl 5.2.1
Moa = 585.5 kNm - Cl 5.6.1.1(3)
αs = 0.897 αm = 1.30

Warping constant (Iw) = 92.9 x10⁹ mm⁶
Torsional constant (J) = 86.5 x10³ mm⁴
Effective section mod. (Zex) = 467 x10³ mm³
Effective section mod. (Zey) = 86.9 x10³ mm³
Elastic modulus (E) = 200000 MPa - Cl 1.4
Shear modulus (G) = 80000 MPa - Cl 1.4

øMsx = 134.5 kNm
øMsy = 25.0 kNm
øMbx = αs*αm*øMsx = 134.5 kNm (=øMsx)

Deflections

Ireq'd DL (L/360) =	34.2 x10⁶ mm⁴	< Critical	δDL =	4.5 mm	Span / 665
Ireq'd Ψs.LL (L/360) =	6.0 x10⁶ mm⁴		Ψs.δLL =	0.8 mm	Span / 3805
Ireq'd DL+Ψs.LL (L/250) =	27.9 x10⁶ mm⁴	< Critical	δTotal =	5.3 mm	Span / 566

Max. precamber (0.3%*span) = 9 mm
Precamber 80% of δDL = 4 mm
Min. precamber = 15 mm
Adopted precamber = 0 mm

1kN midspan δ = 0.0 mm

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Member:	**(B8 - G, above basment carpark ramp) 360UB56.7 (G300), 3-M20 8.8/s bolts**	**OK (0.47)**
Bending:	**M(max)* = 195.9kNm < øMb(3000,αm=1.30) = 259.6kNm**	**OK (0.75)**
Shear:	**V.max*=130.6kN < øVvm = 496.3kN (Web area full depth)**	**OK (0.26)**
Deflection:	**δdl = L/377 (16mm), Ψs.δll = L/3443 (2mm), δtot. = L/340 (18mm), 1kN midspan δ =0.1mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 91.0kN, Rll = 14.3kN, R* = 130.6kN**	

Geometry

Span (L) =	6000	mm	Effective length (Le) =	3000	mm
Centres (cts) =	1900	mm	αm =	1.30	

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 3000 mm
αm = 1.30

Loadings

Floor area =	11.4	m²	Live load type =	N	(N)ormal, (S)torage, (M)anual
Apply reduction =	N	(Y)es,(N)o			AS/NZS 1170.1 - Table 4.1
Floor reduction (Ψa) =	1.00	AS/NZS 1170.1 - Cl 3.4.2			

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	1900	mm +		kN/m =	8.08	kN/m
Blockwork above (wdl) =	3.50	kPa *	2560	mm +		kN/m =	8.96	kN/m
Plant (wdl) =	6.70	kPa *	1900	mm +		kN/m =	12.73	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.57	kN/m
						Σwdl =	30.33	kN/m

Uniform live loads

Floor live load (wll) =	2.50	kPa * Ψa *	1900	mm +		kN/m =	4.75	kN/m
Other live load (wll) =		kPa *	1900	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	4.75 kN/m

Point loads

Dead load (pdl) =		kN	Position =	3000	mm from LHS
Live load (pll) =		kN			

Short term LL (Ψsu) =	0.70		(Ψsp) =	1.00
w* = 1.2*wdl + 1.5*wll =	43.53	kN/m	Rdl =	91.0 kN
p* = 1.2*pdl + 1.5*pll =	0.00	kN	Rll =	14.3 kN
Max M* at =	3000	mm	R* =	130.6 kN
M* =	195.9	kNm (Maximum)		

Capacity

Description =	360UB56.7 (G300)		Warping constant (Iw) =	330	x10⁹ mm⁶
Flange yield (fyf) =	300	MPa	Torsional constant (J) =	338	x10³ mm⁴
Web yield (fyw) =	320	MPa	Effective section mod. (Zex) =	1010	x10³ mm³
Area (Ag) =	7240	mm²	Effective section mod. (Zey) =	193	x10³ mm³
Stiffness (Ix) =	161	x10⁶ mm⁴	Elastic modulus (E) =	200000	MPa - Cl 1.4
Stiffness (Iy) =	11	x10⁶ mm⁴	Shear modulus (G) =	80000	MPa - Cl 1.4
ø =	0.9	Table 3.4			
Msx = min(fyf,fyw)*Zex =	303.0	kNm - Cl 5.2.1	øMsx =	272.7	kNm
Moa =	489.7	kNm - Cl 5.6.1.1(3)	øMsy =	52.1	kNm
αs = 0.732		αm = 1.30	øMbx = αs*αm*øMsx =	259.6	kNm

Deflections

Ireq'd DL (L/360) =	153.6	x10⁶ mm⁴	< Critical	δDL =	15.9	mm	Span / 377
Ireq'd Ψs.LL (L/360) =	16.8	x10⁶ mm⁴		Ψs.δLL =	1.7	mm	Span / 3443
Ireq'd DL+Ψs.LL (L/250) =	118.3	x10⁶ mm⁴	< Critical	δTotal =	17.6	mm	Span / 340

Max. precamber (0.3%*span) =	18	mm	Min. precamber =	15	mm
Precamber 80% of δDL =	13	mm	Adopted precamber =	0	mm
			1kN midspan δ =	0.1	mm

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Member:	**(B9 - L1, supporting B3 and blockwork to the south) 530UB82.0 (G300)**	
Bending:	**M(max)* = 239.6kNm < øMb(1750,αm=1.30) = 558.9kNm**	**OK (0.43)**
Shear:	**V.max*=259.3kN < øVvm = 875.9kN (Web area full depth)**	**OK (0.30)**
Deflection:	**δdl = L/1835 (2mm), Ψs.δll = L/8276 (0mm), δtot. = L/1502 (2mm), 1kN midspan δ =0.0mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(1 End) Rdl.max = 160.0kN, Rll.max = 44.8kN, R.max* = 259.3kN**	
	(1 End) Rdl.min = 134.5kN, Rll.min = 38.5kN, R.min* = 219.1kN	

Geometry

Span (L) = 3500 mm
Centres (cts) = 6000 mm
Effective length (Le) = 1750 mm
αm = 1.30

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 1750 mm
αm = 1.30

Loadings

Floor area = 21.0 m²
Apply reduction = N (Y)es,(N)o
Floor reduction (Ψa) = 1.00 AS/NZS 1170.1 - Cl 3.4.2

Live load type = N (N)ormal, (S)torage, (M)anual
AS/NZS 1170.0 - Table 4.1

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	6000	mm +		kN/m =	25.50	kN/m
Super. dead load (wdl) =	3.50	kPa *	9900	mm +		kN/m =	34.65	kN/m
Other dead load (wdl) =		kPa *		mm +		kN/m =	0.00	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.83	kN/m
						Σwdl =	60.98	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	6000	mm +		kN/m =	18.00	kN/m
Other live load (wll) =		kPa *	6000	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	18.00 kN/m

Point loads B3 reactions

Dead load (pdl) = 81.1 kN
Live load (pll) = 20.3 kN
Position = 1200 mm from LHS
(Point load position within span)

Short term LL (Ψsu) = 0.70
(Ψsp) = 1.00

w* = 1.2*wdl + 1.5*wll = 100.17 kN/m
p* = 1.2*pdl + 1.5*pll = 127.77 kN
Max M* at = 1313 mm
M* = 239.6 kNm (Maximum)

Rdl.max = 160.0 kN
Rll.max = 44.8 kN
R.max* = 259.3 kN

Capacity

Description = 530UB82.0 (G300)
Flange yield (fyf) = 300 MPa
Web yield (fyw) = 320 MPa
Area (Ag) = 10500 mm²
Stiffness (Ix) = 477 x10⁶ mm⁴
Stiffness (Iy) = 20.1 x10⁶ mm⁴
ø = 0.9 Table 3.4
Msx = min(fyf,fyw)*Zex = 621.0 kNm - Cl 5.2.1
Moa = 3413.4 kNm - Cl 5.6.1.1(3)
αs = 0.936 αm = 1.30

Warping constant (Iw) = 1330 x10⁹ mm⁶
Torsional constant (J) = 526 x10³ mm⁴
Effective section mod. (Zex) = 2070 x10³ mm³
Effective section mod. (Zey) = 289 x10³ mm³
Elastic modulus (E) = 200000 MPa - Cl 1.4
Shear modulus (G) = 80000 MPa - Cl 1.4

øMsx = 558.9 kNm
øMsy = 78.0 kNm
øMbx = αs*αm*øMsx = 558.9 kNm (=øMsx)

Deflections (δ ±2.5% with off-centre PL)

Ireq'd DL (L/360) =	93.6 x10⁶ mm⁴	< Critical	δDL =	1.9 mm	Span / 1835
Ireq'd Ψs.LL (L/360) =	20.7 x10⁶ mm⁴		Ψs.δLL =	0.4 mm	Span / 8276
Ireq'd DL+Ψs.LL (L/250) =	79.4 x10⁶ mm⁴	< Critical	δTotal =	2.3 mm	Span / 1502

Max. precamber (0.3%*span) = 11 mm
Precamber 80% of δDL = 2 mm
Min. precamber = 15 mm
Adopted precamber = 0 mm

1kN midspan δ = 0.0 mm

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Member:	**(B9 - L1, supporting blockwork stairwell) 530UB82.0 (G300)**	
Bending:	**M(max)* = 402.5kNm < øMb(2600,αm=1.30) = 558.9kNm**	**OK (0.72)**
Shear:	**V.max*=309.6kN < øVvm = 875.9kN (Web area full depth)**	**OK (0.35)**
Deflection:	**δdl = L/733 (7mm), Ψs.δll = L/3308 (2mm), δtot. = L/600 (9mm), 1kN midspan δ =0.0mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 184.9kN, Rll = 58.5kN, R* = 309.6kN**	

Geometry

Span (L) =	5200	mm	Effective length (Le) =	2600	mm
Centres (cts) =	7500	mm	αm =	1.30	

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 2600 mm
αm = 1.30

Loadings

Floor area = 39.0 m²
Apply reduction = N (Y)es,(N)o
Floor reduction (Ψa) = 1.00 AS/NZS 1170.1 - Cl 3.4.2

Live load type = N (N)ormal, (S)torage, (M)anual
AS/NZS 1170.0 - Table 4.1

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	7500	mm +		kN/m =	31.88	kN/m
Blockwork above (wdl) =	3.50	kPa *	9900	mm +		kN/m =	34.65	kN/m
Partitions (wdl) =	0.50	kPa *	7500	mm +		kN/m =	3.75	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.83	kN/m
						Σwdl =	71.10	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	7500	mm +		kN/m =	22.50	kN/m
Other live load (wll) =		kPa *		mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to	1	members	Σwll =	22.50	kN/m

Point loads

Dead load (pdl) = kN
Live load (pll) = kN
Position = mm from LHS

Short term LL (Ψsu) =	0.70		(Ψsp) =	1.00
w* = 1.2*wdl + 1.5*wll =	119.07	kN/m	Rdl =	184.9 kN
p* = 1.2*pdl + 1.5*pll =	0.00	kN	Rll =	58.5 kN
Max M* at =	2600	mm	R* =	309.6 kN
M* =	402.5	kNm (Maximum)		

Capacity

Description =	530UB82.0 (G300)		Warping constant (Iw) =	1330	x10⁹ mm⁶
Flange yield (fyf) =	300	MPa	Torsional constant (J) =	526	x10³ mm⁴
Web yield (fyw) =	320	MPa	Effective section mod. (Zex) =	2070	x10³ mm³
Area (Ag) =	10500	mm²	Effective section mod. (Zey) =	289	x10³ mm³
Stiffness (Ix) =	477	x10⁶ mm⁴	Elastic modulus (E) =	200000	MPa - Cl 1.4
Stiffness (Iy) =	20.1	x10⁶ mm⁴	Shear modulus (G) =	80000	MPa - Cl 1.4
ø =	0.9	Table 3.4			
Msx = min(fyf,fyw)*Zex =	621.0	kNm - Cl 5.2.1	øMsx =	558.9	kNm
Moa =	1589.4	kNm - Cl 5.6.1.1(3)	øMsy =	78.0	kNm
αs = 0.831		αm = 1.30	øMbx = αs*αm*øMsx =	558.9	kNm (=øMsx)

Deflections

Ireq'd DL (L/360) =	234.3	x10⁶ mm⁴	< Critical	δDL =	7.1	mm	Span / 733
Ireq'd Ψs.LL (L/360) =	51.9	x10⁶ mm⁴		Ψs.δLL =	1.6	mm	Span / 3308
Ireq'd DL+Ψs.LL (L/250) =	198.8	x10⁶ mm⁴	< Critical	δTotal =	8.7	mm	Span / 600

Max. precamber (0.3%*span) =	16	mm	Min. precamber =	15	mm
Precamber 80% of δDL =	6	mm	Adopted precamber =	0	mm
			1kN midspan δ =	0.0	mm

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Member:	**(B9 - L1, supporting precast adjacent to wet area) 530UB82.0 (G300)**	
Bending:	**M(max)* = 137.2kNm < øMb(2500,αm=1.30) = 558.9kNm**	**OK (0.25)**
Shear:	**V.max*=109.7kN < øVvm = 875.9kN (Web area full depth)**	**OK (0.13)**
Deflection:	**δdl = L/2155 (2mm), Ψs.δll = L/11165 (0mm), δtot. = L/1806 (3mm), 1kN midspan δ =0.0mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 68.0kN, Rll = 18.8kN, R* = 109.7kN**	

Geometry

Span (L) =	5000	mm	Effective length (Le) =	2500	mm
Centres (cts) =	2500	mm	αm =	1.30	

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 2500 mm
αm = 1.30

Loadings

Floor area =	12.5	m²	Live load type =	N	(N)ormal, (S)torage, (M)anual
Apply reduction =	N	(Y)es,(N)o			AS/NZS 1170.1 - Table 4.1
Floor reduction (Ψa) =	1.00	AS/NZS 1170.1 - Cl 3.4.2			

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	2500	mm +		kN/m =	10.63	kN/m
Precast above (wdl) =	4.50	kPa *	3500	mm +		kN/m =	15.75	kN/m
Partitions (wdl) =	0.50	kPa *		mm +		kN/m =	0.00	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.83	kN/m
						Σwdl =	27.20	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	2500	mm +		kN/m =	7.50	kN/m
0 (wll) =		kPa *	2500	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	7.50 kN/m

Point loads

Dead load (pdl) =		kN	Position =	2500	mm from LHS
Live load (pll) =		kN			

Short term LL (Ψsu) =	0.70		(Ψsp) =	1.00
w* = 1.2*wdl + 1.5*wll =	43.89	kN/m	Rdl =	68.0 kN
p* = 1.2*pdl + 1.5*pll =	0.00	kN	Rll =	18.8 kN
Max M* at =	2500	mm	R* =	109.7 kN
M* =	137.2	kNm (Maximum)		

Capacity

Description =	530UB82.0 (G300)		Warping constant (Iw) =	1330	x10⁹ mm⁶
Flange yield (fyf) =	300	MPa	Torsional constant (J) =	526	x10³ mm⁴
Web yield (fyw) =	320	MPa	Effective section mod. (Zex) =	2070	x10³ mm³
Area (Ag) =	10500	mm²	Effective section mod. (Zey) =	289	x10³ mm³
Stiffness (Ix) =	477	x10⁶ mm⁴	Elastic modulus (E) =	200000	MPa - Cl 1.4
Stiffness (Iy) =	20.1	x10⁶ mm⁴	Shear modulus (G) =	80000	MPa - Cl 1.4
ø =	0.9	Table 3.4			
Msx = min(fyf,fyw)*Zex =	621.0	kNm - Cl 5.2.1	øMsx =	558.9	kNm
Moa =	1712.8	kNm - Cl 5.6.1.1(3)	øMsy =	78.0	kNm
αs = 0.844		αm = 1.30	øMbx = αs*αm*øMsx =	558.9	kNm (=øMsx)

Deflections

Ireq'd DL (L/360) =	79.7	x10⁶ mm⁴	< Critical	δDL =	2.3	mm	Span / 2155
Ireq'd Ψs.LL (L/360) =	15.4	x10⁶ mm⁴		Ψs.δLL =	0.4	mm	Span / 11165
Ireq'd DL+Ψs.LL (L/250) =	66.0	x10⁶ mm⁴	< Critical	δTotal =	2.8	mm	Span / 1806

Max. precamber (0.3%*span) =	15	mm	Min. precamber =	15	mm
Precamber 80% of δDL =	2	mm	Adopted precamber =	0	mm
			1kN midspan δ =	0.0	mm

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Member:	**(FB2 - L2, L3 wet areas) 360UB44.7 (G300)**	
Bending:	**M(max)* = 107.4kNm < øMb(2500,αm=1.30) = 221.8kNm**	**OK (0.48)**
Shear:	**V.max*=69.1kN < øVvm = 419.7kN (Web area full depth)**	**OK (0.16)**
Deflection:	**δdl = L/1028 (5mm), Ψs.δll = L/1918 (3mm), δtot. = L/669 (7mm), 1kN midspan δ =0.1mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 31.9kN, Rll = 20.6kN, R* = 69.1kN**	

Geometry

Span (L) =	5000	mm	Effective length (Le) =	2500	mm
Centres (cts) =	2000	mm	αm =	1.30	

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 2500 mm
αm = 1.30

Loadings

Floor area =	10.0	m²	Live load type =	N	(N)ormal, (S)torage, (M)anual
Apply reduction =	N	(Y)es,(N)o			AS/NZS 1170.1 - Table 4.1
Floor reduction (Ψa) =	1.00	AS/NZS 1170.1 - Cl 3.4.2			

Uniform dead loads

Floor dead load (wdl) =	4.25	kPa *	2000	mm +		kN/m =	8.50	kN/m
Partitions (wdl) =	0.50	kPa *	2000	mm +		kN/m =	1.00	kN/m
Services (wdl) =	0.20	kPa *		mm +		kN/m =	0.00	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.45	kN/m
						Σwdl =	9.95	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	2000	mm +		kN/m =	6.00	kN/m
Other live load (wll) =		kPa *	2000	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	6.00 kN/m

Point loads

Dead load (pdl) =	14.1	kN	Position =	2500	mm from LHS
Live load (pll) =	11.1	kN	(Point load positioned mid-span)		

Short term LL (Ψsu) =	0.70		(Ψsp) =	1.00
w* = 1.2*wdl + 1.5*wll =	20.94	kN/m	Rdl =	31.9 kN
p* = 1.2*pdl + 1.5*pll =	33.57	kN	Rll =	20.6 kN
Max M* at =	2500	mm	R* =	69.1 kN
M* =	107.4	kNm (Maximum)		

Capacity

Description =	360UB44.7 (G300)		Warping constant (Iw) =	237	x10⁹ mm⁶
Flange yield (fyf) =	320	MPa	Torsional constant (J) =	161	x10³ mm⁴
Web yield (fyw) =	320	MPa	Effective section mod. (Zex) =	770	x10³ mm³
Area (Ag) =	5720	mm²	Effective section mod. (Zey) =	140	x10³ mm³
Stiffness (Ix) =	121	x10⁶ mm⁴	Elastic modulus (E) =	200000	MPa - Cl 1.4
Stiffness (Iy) =	8.1	x10⁶ mm⁴	Shear modulus (G) =	80000	MPa - Cl 1.4
ø =	0.9	Table 3.4			
Msx = min(fyf,fyw)*Zex =	246.4	kNm - Cl 5.2.1	øMsx =	221.8	kNm
Moa =	473.7	kNm - Cl 5.6.1.1(3)	øMsy =	40.3	kNm
αs = 0.773		αm = 1.30	øMbx = αs*αm*øMsx =	221.8	kNm (=øMsx)

Deflections

Ireq'd DL (L/360) =	42.4	x10⁶ mm⁴		δDL =	4.9	mm	Span / 1028
Ireq'd Ψs.LL (L/360) =	22.7	x10⁶ mm⁴		Ψs.δLL =	2.6	mm	Span / 1918
Ireq'd DL+Ψs.LL (L/250) =	45.2	x10⁶ mm⁴	< Critical	δTotal =	7.5	mm	Span / 669

Max. precamber (0.3%*span) =	15	mm	Min. precamber =	15	mm
Precamber 80% of δDL =	4	mm	Adopted precamber =	0	mm
			1kN midspan δ =	0.1	mm

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Member:	**(FB3 - L2, L3 wet areas) 250UB25.7 (G300)**	
Bending:	**M(max)* = 30.2kNm < øMb(2050,αm=1.00) = 67.2kNm**	**OK (0.45)**
Shear:	**V.max*=33.5kN < øVvm = 214.3kN (Web area full depth)**	**OK (0.16)**
Deflection:	**δdl = L/1486 (2mm), Ψs.δll = L/2707 (1mm), δtot. = L/959 (4mm), 1kN midspan δ =0.1mm**	**OK**
Precamber:	**Not required**	
Reactions:	**(Each end) Rdl = 14.1kN, Rll = 11.1kN, R* = 33.5kN**	

Geometry

Span (L) =	3600	mm	Effective length (Le) =	2050	mm
Centres (cts) =	2050	mm	αm =	1.00	

Design at = M mm from LHS, (M)ax, (S)eg

Effective length (Le) = 2050 mm
αm = 1.00

Loadings

Floor area =	7.4	m²	Live load type =	N	(N)ormal, (S)torage, (M)anual
Apply reduction =	N	(Y)es,(N)o			AS/NZS 1170.1 - Table 4.1
Floor reduction (Ψa) =	1.00	AS/NZS 1170.1 - Cl 3.4.2			

Uniform dead loads

Floor dead load (wdl) =	3.00	kPa *	2050	mm +		kN/m =	6.15	kN/m
Partitions (wdl) =	0.50	kPa *	2050	mm +		kN/m =	1.03	kN/m
Services (wdl) =	0.20	kPa *	2050	mm +		kN/m =	0.41	kN/m
Include S.Wt =	Y	(Y)es,(N)o				S.Wt =	0.26	kN/m
						Σwdl =	7.84	kN/m

Uniform live loads

Floor live load (wll) =	3.00	kPa * Ψa *	2050	mm +		kN/m =	6.15	kN/m
Other live load (wll) =		kPa *	2050	mm +		kN/m =	0.00	kN/m
Alternate point live load =	2.70	kN	Distr. to		1	members	Σwll =	6.15 kN/m

Point loads

Dead load (pdl) =		kN	Position =		mm from LHS
Live load (pll) =		kN			

Short term LL (Ψsu) =	0.70		(Ψsp) =	1.00
w* = 1.2*wdl + 1.5*wll =	18.64	kN/m	Rdl =	14.1 kN
p* = 1.2*pdl + 1.5*pll =	0.00	kN	Rll =	11.1 kN
Max M* at =	1800	mm	R* =	33.5 kN
M* =	30.2	kNm (Maximum)		

Capacity

Description =	250UB25.7 (G300)		Warping constant (Iw) =	36.7	x10⁹ mm⁶
Flange yield (fyf) =	320	MPa	Torsional constant (J) =	67.4	x10³ mm⁴
Web yield (fyw) =	320	MPa	Effective section mod. (Zex) =	319	x10³ mm³
Area (Ag) =	3270	mm²	Effective section mod. (Zey) =	61.7	x10³ mm³
Stiffness (Ix) =	35.4	x10⁶ mm⁴	Elastic modulus (E) =	200000	MPa - Cl 1.4
Stiffness (Iy) =	2.55	x10⁶ mm⁴	Shear modulus (G) =	80000	MPa - Cl 1.4
ø =	0.9	Table 3.4			
Msx = min(fyf,fyw)*Zex =	102.1	kNm - Cl 5.2.1	øMsx =	91.9	kNm
Moa =	164.6	kNm - Cl 5.6.1.1(3)	øMsy =	17.8	kNm
αs = 0.732		αm = 1.00	øMbx = αs*αm*øMsx =	67.2	kNm

Deflections

Ireq'd DL (L/360) =	8.6	x10⁶ mm⁴		δDL =	2.4	mm	Span / 1486
Ireq'd Ψs.LL (L/360) =	4.7	x10⁶ mm⁴		Ψs.δLL =	1.3	mm	Span / 2707
Ireq'd DL+Ψs.LL (L/250) =	9.2	x10⁶ mm⁴	< Critical	δTotal =	3.8	mm	Span / 959

Max. precamber (0.3%*span) =	11	mm	Min. precamber =	15	mm
Precamber 80% of δDL =	2	mm	Adopted precamber =	0	mm
			1kN midspan δ =	0.1	mm

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CALCULATIONS	REF./COMMENT

10) MISCELLANEOUS STEEL FRAMING

a) Level 3 Lintel - L1

INPUT

Member: 360 UB 44
Span, L = 12.0 m (simply supported)
Loadwidth = 1.0 m
Self weight = 44 kg/m

LOADING

Dead, G 0.45 kPa x 1.0 m = 0.45 kN/m — Services, etc…
Live, Q -

G = 0.88 kN/m
Q = 0.00 kN/m
ULS 1 1.2G+1.5Q = 1.06 kN/m
SLS 1 G+0.7Q = 0.88 kN/m

	ULS 1	G	Q
M* (kNm)	19.0	-	-
V* (kN)	6.3	5	0

OUTPUT *- as per AS 4100 & ASI Design capacity tables*

Strength
k_r = 1.0
L_e = 12.0 m
E = 200000 MPa
ϕ = 0.9
I_x = 121 x 10^6 mm^4
ϕM_s = 222 kNm
ϕM_b = 33.8 kNm > M* — ***0.56 OK***

Deflection
Under SLS 1, δ = 10 mm < 12mm — ***OK*** AS 1170.0
L/ 1220 > 240 — ***OK*** Tab. C1

Connection
Try: M20 8.8/S Bolts + 6 CFW
= 3
ϕV_{fn} = 92.6 kN
Total shear resistance of bolts, V_b = 277.8 kN > V* — ***0.02 OK***
6mm continuous fillet weld, ϕV_w = 0.835 kN/mm — E41XX/W40X, SP Class
Nominal tensile strength, f_{uw} = 480 MPa
Length of weld, L_w = 980 mm — Welded both sides
Total shear resistance of welds, V_w = 818 kN > V* — ***0.01 OK***

Adopt: 360 UB 44

Reference: 2018-7161
Project: 97 King William Street, Kent Town **Checked by:** BR
Designer: JT Date: 6/06/2018 **Index:** MS - 2

CALCULATIONS	REF./COMMENT

b) Wind beam - WB1

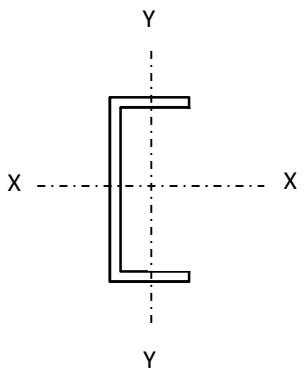
INPUT

Member: 250 PFC

Span, L =	7.6	m (simply supported)
Loadwidth =	1.0	m
Self weight =	35.5	kg/m

LOADING

Dead, G	-	(self weight only)			
Live, Q	-				
Wu	0.69 kPa x	1.0 m =	0.69	kN/m	
Ws	0.47 kPa x	1.0 m =	0.47	kN/m	
$C_{p,w}$ =	0.7				



	G =	0.35	kN/m		
	Q =	0.00	kN/m		
ULS 1	1.2G+1.5Q =	0.42	kN/m		Y-Y
ULS 2	Wu =	0.69	kN/m		X-X
SLS 1	G+0.7Q =	0.35	kN/m		Y-Y
SLS 2	Ws =	0.47	kN/m		X-X

	ULS 1	ULS 2	G	Q
M^*_x (kNm)	-	5.0	-	-
M^*_y (kNm)	3.0	-	-	-
V^*_x (kN)	-	2.6	-	-
V^*_y (kN)	1.6	-	1.3	-

OUTPUT *- as per AS 4100 & ASI Design capacity tables*

Strength

k_r =	1.0				
L_e =	1.2	m	*~1.5m*		
E =	200000	MPa			
ϕ =	0.9				
I_x =	45.1	x 10^6 mm^4			
I_y =	3.64	x 10^6 mm^4			
ϕM_{sx} =	114.0	kNm			
ϕM_{sy} =	24.0	kNm			
ϕM_{bx} =	98.6	kNm	> M^*_x	***0.05***	***OK***
ϕM_{by} =	24.0	kNm	> M^*_y	***0.13***	***OK***

Deflection

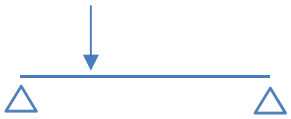
Under SLS 1, δ =	20.8	mm			AS 1170.0
L/	366		> 200	***OK***	Tab. C1
Under SLS 2, δ =	2.3	mm			
L/	15771		> 200	***OK***	

Adopt: 250 PFC

CALCULATIONS						REF./COMMENT
11) ROOF FRAMING						
a) Purlins - PL1						
INPUT			Member:	Z30030 - 2 rows of bridging		
			Span, L =	12 m		
			Spacing, s =	1.2 m		
			Self weight =	10.09 kg/m		
LOADING						
Dead, G	0.45 kPa x	1.20 m =		0.54 kN/m		Sheeting
Live, Q	0.25 kPa x	1.20 m =		0.30 kN/m		Roof
Wu(up)	-0.89 kPa x	1.20 m =		-1.07 kN/m		
Wu(dwn)	0.40 kPa x	1.20 m =		0.47 kN/m		
Ws(up)	-0.60 kPa x	1.20 m =		-0.72 kN/m		
Ws(dwn)	0.27 kPa x	1.20 m =		0.32 kN/m		
K_c =		1.0 (1 effective surfaces)				AS 1170.2 Cl. 5.4.3
C_{fig}(up) =		-0.9				AS 1170.2 Tab. 5.3(A)
C_{fig}(dwn) =		0.4 (incl. internal pressures, $C_{p,i}$ = -0.2)				AS 1170.2 Tab. 5.3(A)
	ULS 1 -		1.2G+1.5Q =	1.22 kN/m		
	ULS 2 -		0.9G+Wu(up) =	-0.49 kN/m		
	ULS 3 -		1.2G+Wu(dwn) =	1.24 kN/m		
	SLS 1 -		G+0.7Q =	0.85 kN/m		
	SLS 2 -		G+Ws(up) =	-0.08 kN/m		
	SLS 3 -		G+Ws(dwn) =	0.96 kN/m		
			w*down =	1.24 kN/m		
			w*up =	-0.49 kN/m		
			w*s =	0.96 kN/m		
			M* =	22.3 kNm		
			=	-8.8 kNm (up)		
			V* =	7.4 kN (max)		
OUTPUT *- As per LYSAGHT Users guide*						
Strength			w(down) =	2.48 kN/m	> w*down	***0.50 OK***
			w(up) =	-2.02 kN/m	> w*up	***0.24 OK***
Deflection			I_x =	23.1 x 10^6 mm^4		
			Under w*s, δ =	56.1 mm	up	
			L /	214	> 150	***OK***
			Under G (only), δ =	37.3 mm		
			L /	321	> 300	***OK***

Reference: 2018-7161
Project: 97 King William Street, Kent Town **Checked by:** BR
Designer: JT Date: 6/06/2018 **Index:** R - 2

CALCULATIONS			REF./COMMENT
i) Check purlins can act as struts			
LOADING	Raking =	89 kN	
	Length of roof =	40.4 m	
	Wind load =	2.19 kN/m	
	N_c* =	2.63 kN	
	Effective length, L_e =	4.0 m	
OUTPUT			
Refer to "Cold Steel" output	Governing load factor =	1.61 > 1	***OK***
ii) Check additional loads from A/C units			
LOADING	Condensor unit (G) =	500 kg	Approx.
	1.2G =	5.9 kN	
	location =	4.0 m from LHS	
	M* =	23.5 kNm	
	Total M* =	45.9 kNm	
OUTPUT	ϕ =	0.9	
	fsy =	450 Mpa	
	Ze =	125 x 10^3 mm^3	
	ϕM_s =	50.6 kNm > M*	***0.91 OK***



Adopt: Z30030 - 2 row of bridging

CALCULATIONS	REF./COMMENT

b) Roof beam - RB1

INPUT

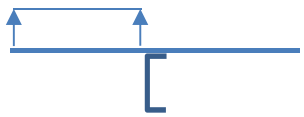
Member: 300 PFC

Span, L =	12.0	m (simply supported)
Loadwidth =	2.0	m (dwn)
=	1.0	m (up)
Self weight =	40.1	kg/m

LOADING

Dead, G	0.45	kPa x	2.0	m =	0.90	kN/m	Roof
Live, Q	0.25	kPa x	2.0	m =	0.50	kN/m	
Wu(up)	-1.48	kPa x	1.0	m =	-1.48	kN/m	
Ws(up)	-1.00	kPa x	1.0	m =	-1.00	kN/m	
C_{fig}(up) =		-1.5					AS 1170.2 Tab. D8

Upwards wind pressure from overhang



ULS 1 -	1.2G+1.5Q =	2.30	kN/m
ULS 2 -	0.9G+Wu(up) =	-0.32	kN/m
SLS 1 -	G+0.7Q =	1.64	kN/m
SLS 2 -	G+Ws(up) =	0.29	kN/m

	ULS 1	ULS 2	G	Q
M* (kNm)	41.4	-5.7	-	-
V* (kN)	13.8	-1.9	7.8	3.0
	(CRITICAL)			

OUTPUT

Strength

k_r =	1.0			
L_e =	1.2	m		
E =	200000	MPa		
ϕ =	0.9			
I_x =	72.4	x 10^6 mm^4		
ϕM_s =	152	kNm	> M*	***0.27 OK***

Deflection

Under G+0.7Q, δ =	31	mm		
L/	392		> 300	***OK***

Adopt: 300 PFC

CALCULATIONS	REF./COMMENT

12) ROOF BRACING

Design roof bracing to transfer lateral loads through the roof framing into the external precast panels.

INPUT

Member:	20mm rod - RB1	
Length, L =	15	m (approx.)
Yield strength, f_{sy} =	250	MPa (assumed min.)
Diameter, d =	20	mm
Area of steel, A_{st} =	314	mm^2

LOADING

From earlier analysis it was deemed that earthquake loading governed lateral loads at the base of the structure, however due to the lightweight nature of the steel framed roof is was found that wind loads governed lateral loading on the roof. The worst case wind loads are parallel with the short direction of the structure.

Raking force at roof, F_R =	89	kN	*From previous*
# of braces resisting force, n =	8		
Raking force per brace, F =	11	kN	
Angle of bracing, θ =	39.6	degrees	
Tensile force in brace, N^*_t =	14	kN	

OUTPUT

Check tensile capacity of rod as per AS 4100

$\varphi N_t = \varphi f_{sy} A_{st}$

ϕ =	0.9			Tab. 3.4
ϕN_t =	71	kN	$> N^*_t$	***0.20 OK***

Check bolted shear connection of rod as per ASI Design Capacity Tables

Try:	M20 8.8/S Bolt			
ϕV_{fn} =	92.6	kN		
No. Of bolts =	1			
Total, ϕV_{fn} =	92.6	kN	$> N^*_t$	***0.16 OK***

Adopt: 20mm diameter rod bracing. Min. yield strength to be 250 MPa.