

TMK Consulting Engineers

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Civil • Structural • Environmental
Geotechnical • Mechanical • Electrical
Fire • Hydraulics • Lifts • Green ESD

Berri Office: 25 Vaughan Terrace, Berri SA 5343

**STRUCTURAL CALCULATIONS AND
DETAILS
(SR1)**

Builder / Agent:	ITHINK DESIGN PTY LTD	Job Number:	1710168
Owner:	NIATRON 10 PTY LTD	Date:	8/10/2018
Project:	PROPOSED RESIDENTIAL DEVELOPMENT	Order No.	419R
Project Location:	419 REGENCY ROAD, PROSPECT SA		

The Calculations and Details enclosed give specific recommendations for the above mentioned building / structure. These must be read in conjunction with all listed attachments. Changes to the design or construction must not be made without further written advice from the Engineer. A full copy of this document is to be forwarded to all future owner(s).

This report is valid for a period of 24 months, based on current standards, regulations, etc.

ATTACHMENTS: CRCS, FC1-FC2, SC1-SC214.

SITE INSPECTIONS:

1. Refer to CR1 – Construction and Footing Report Recommendations.

NOTE: 1. *These inspections will incur additional fees.*
2. *We require 24 hours notice when booking inspections.*

ADDITIONAL NOTES/REQUIREMENTS:

1. Refer to Structural Drawings for details.

For and on behalf of
TMK Consulting Engineers

TONY GARREFFA
Senior Associate / Team Leader

TMK Consulting Engineers

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**CONDITIONS FOR THE USE OF STRUCTURAL
CALCULATIONS AND DETAILS
(CRCS)****1. GENERAL**

- 1.1 These Structural Calculations and Details (hereinafter named the "Report") give specific recommendations for the particular building described in this report. This Report must be read in conjunction with all listed attachments. Changes to the design or construction must not be made without further written advice from the Engineer.
- 1.2 The Owner and all contractors will comply in all respects and at all times with all terms, conditions and recommendations contained in, or attached to, this Report.
- 1.2.1 It is essential that the Owner reads the entire report carefully as it contains important information, relating not only to the construction, but also to obligations and liabilities.
- 1.2.2 If the Owner requires different details to that recommended, our office must be notified prior to the commencement of construction, and advice will be given accordingly.
- 1.2.3 If there are any aspects of the Report that are not understood, please contact the Engineer.
- 1.3 The Engineer may (and the Owner hereby authorizes the Engineer to):
- 1.3.1 Issue instructions (including an instruction to cease construction) on behalf of the Owner to any person engaged in the construction of the building, or any part thereof, to ensure construction of the building in accordance with this Report and any modification thereof. If any modification as aforesaid may be likely to result in additional construction costs exceeding \$3,500.00 (plus GST), the Engineer may issue an instruction to cease construction in order to obtain the approval of the Owner for such modification.
- 1.3.2 Make such modifications to the Report as the Engineer may deem necessary during the course of construction.
- 1.4 The Owner shall be responsible for, and indemnify the Engineer against, all and any costs and charges and all claims and demands made for any additional costs incurred by reason of any act, requirement or instruction of the Engineer made or given pursuant to Clause 1.3.
- 1.5 The Engineer shall not be liable for any defect in or damage to the building / construction caused by or contributed to by any breach of the terms, conditions and recommendations committed, permitted or allowed by the Owner.
- 1.6 Where more than one person is named as the Owner, all these terms, conditions and recommendations shall bind all such persons jointly and each such person severally, and any instruction or information given to the Engineer by any one such person shall be deemed to be given by all other such persons.

2. TERMS OF ENGAGEMENT

- 2.1 All work will be carried out in accordance with TMK's standard '*Terms and Conditions of Engagement for Consulting Services*'.



Ref.: 1710168
Date: 08-Oct-18
Design: RR
Page: FC1

REACTIVE SOIL MOVEMENT CALCULATIONS

Sub-number :

These calculations comply with the requirements of AS 2870—2011 and "Special Provisions for the Design of Residential Slabs and Footings for South Australian Conditions", February 2013.

Boreholes : 1 to 4

The values of the Differential Mound Movement y_m for the Centre Heave (C/H) and Edge Heave (E/H) conditions are intended for use in the Walsh Method of Analysis and comply with Clause F2 of AS 2870—2011.

Depth of Design Suction Change : 4.0 metres Table 2.4
Depth of the Cracked Zone : 3.0 metres Clause 2.3.2
Include the effects of trees : N (Y=Yes, N=No)

Summary of soil profile parameters used in calculations

	HOLE 1		HOLE 2		HOLE 3		HOLE 4	
Fill < 5 yrs **	metres		metres		metres		metres	
Cut < 2 yrs **	metres		metres		metres		metres	
Bedrock **	metres		metres		metres		metres	
Water Table **	metres		metres		metres		metres	
Horizons	Depth	lps	Depth	lps	Depth	lps	Depth	lps
1	0.35	0.005	0.30	0.005	0.40	0.005		
2	1.30	0.010	1.60	0.010	1.70	0.010		
3	2.40	0.020	2.20	0.020	2.00	0.020		
4	3.20	0.030	3.00	0.030	3.30	0.030		
5	5.10	0.060	5.01	0.060	5.20	0.060		
6	5.65	0.050	5.80	0.050	5.70	0.050		
7	6.00	0.035	6.00	0.035	6.00	0.035		

** Leave blank if this parameter does not influence the design

Summary of tree effect parameters (to AS 2870—2011 Appendix H)

This design EXCLUDES TREE EFFECTS			
HT =	n/a	Design height of row of trees (m)	
D_t =	n/a	Distance to row of trees (m)	
D_t / HT =	n/a	Tree Factor	Appendix H
S_v =	n/a	Surface Value of ΔpF	Table 2.4
Δu_{base} =	n/a	due the effect of the row of trees	
and H_t =	n/a	metres	Appendix H

Summary of calculated surface movement values

	HOLE 1	HOLE 2	HOLE 3	HOLE 4
Characteristic Surface Movement, y_s (mm)	34.5	31.3	34.3	n/a
Surface Movement due to the Effects of Trees, y_t (mm)	n/a	n/a	n/a	n/a

The maximum value of y_s is 35 mm Site Classification : M-D (Table 2.3)

=>For design, use: C/H: y_m = 25 mm Appendix F2
E/H: y_m = 18 mm Appendix F2
=>For design, use: y_t = n/a mm Appendix H

Notes / Comments :

In the case where trees have been found on the site or tree planting is planned in the vicinity of the proposed works, this design attempts to account for their effects by allowing for a vertical soil movement greater than would be expected to occur as a result of normal seasonal moisture movements beneath and adjacent to the footing. However, due to the complexity of tree root geometry, variable moisture extraction by the tree and the difficulty in predicting future tree growth, a precise design for the effects of trees is outside current knowledge.

The Owner must understand that, although appropriate precautions have been taken in this design for the effects of trees, some tree-induced movement in the structure must be accepted.

REACTIVE SOIL MOVEMENT CALCULATIONS

Sub-number :

Boreholes :

These calculations comply with the requirements of AS 2870—2011 and "Special Provisions for the Design of Residential Slabs and Footings for South Australian Conditions", February 2013.

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	HOLE 1		HOLE 2		HOLE 3		HOLE 4	
Fill < 5 yrs **	metres		metres		metres		metres	
Cut < 2 yrs **	metres		metres		metres		metres	
Bedrock **	metres		metres		metres		metres	
Water Table **	metres		metres		metres		metres	
Horizons	Depth	lps	Depth	lps	Depth	lps	Depth	lps
1	0.30	0.010	0.60	0.010	0.70	0.010		
2	1.40	0.020	1.20	0.020	1.00	0.020		
3	2.20	0.030	2.00	0.030	2.30	0.030		
4	4.10	0.060	4.01	0.060	4.20	0.060		
5	4.65	0.050	4.80	0.050	4.70	0.050		
6	5.00	0.035	5.00	0.035	5.00	0.035		
7								

** Leave blank if this parameter does not influence the design

Summary of tree effect parameters (to AS 2870—2011 Appendix H)

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S_v =	n/a	Surface Value of ΔpF	Table 2.4
Δu_{base} =	n/a	due the effect of the row of trees	
and H_t =	n/a	metres	Appendix H

Summary of calculated surface movement values

	HOLE 1	HOLE 2	HOLE 3	HOLE 4
Characteristic Surface Movement, y_s (mm)	72.1	74.1	69.4	n/a
Surface Movement due to the Effects of Trees, y_t (mm)	n/a	n/a	n/a	n/a

The maximum value of y_s is 74 mm Site Classification : H2-D (Table 2.3)

=>For design, use: C/H: y_m = 52 mm Appendix F2

E/H: y_m = 37 mm Appendix F2

=>For design, use: y_t = n/a mm Appendix H

Notes / Comments :

In the case where trees have been found on the site or tree planting is planned in the vicinity of the proposed works, this design attempts to account for their effects by allowing for a vertical soil movement greater than would be expected to occur as a result of normal seasonal moisture movements beneath and adjacent to the footing. However, due to the complexity of tree root geometry, variable moisture extraction by the tree and the difficulty in predicting future tree growth, a precise design for the effects of trees is outside current knowledge.

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**STRUCTURAL CALCULATIONS
(SC1)**

Builder / Agent:	THINK DESIGN STUDIO PTY LTD	Job Number:	1710168
Owner:	NIATRON 10 PTY LTD	Date:	8/10/2018
Project:	PROPOSED RESIDENTIAL DEVELOPMENT		
Project Location:	419 REGENCY ROAD, PROSPECT SA		

GENERAL NOTES:

- These calculations are to be read in conjunction with the associated Architectural Drawings, Footing Construction Report, Structural Drawings and / or Details.
- All work to comply with relevant Australian Standards including but not limited to:
 - AS/NZ 1170 - Structural design actions
 - AS 1554 - Structural steel welding
 - AS 2327 - Composite structures
 - AS 3600 - Concrete structures
 - AS 3610 - Formwork for concrete
 - AS 3700 - Masonry structures
 - AS 4100 - Steel structures
 - AS/NZS 4600 - Cold formed steel structures
 - AS 1163 - Structural steel hollow sections
 - AS 1657 - Fixed platforms, walkways, stairways and ladders
 - AS/NZ 2312 - Guide to the protection of structural steel against atmospheric corrosion by the use of protective coatings
 - AS 3850 - Tilt up concrete construction
 - AS 4678 - Earth retaining structures



Ref.: 1710168
Date: 03-Sep-18
Design: KWA
Page: SC 2

WIND SPEED CALCULATION

These calculations comply with the requirements of AS/NZS 1170.2:2011 - Wind Actions (Amendments 1 & 2) and the Building Code of Australia (BCA Volume 1).

Site : 419 REGENCY ROAD PROSPECT

Description : 4 Storey Residence

DESIGN WIND SPEEDS

Importance Level :	2	▼
R_{strength} :	500 years	
$R_{\text{serviceability}}$:	20 years	
Wind region :	A	▼
Terrain category :	3	▼
Reference height, z :	14 metres	
Terrain/height, $M_{z,\text{cat}}$:	0.88	
Direction, M_d :	1.00	
Shielding, M_s :	1.00	
Topography, M_t :	1.00	
$V_{R,\text{strength}}$:	45 m/s	
$V_{R,\text{serviceability}}$:	37 m/s	
=> $V_{\text{des},\text{strength}}$:	39.5 m/s	
=> $V_{\text{des},\text{serviceability}}$:	32.5 m/s	

BCA Vol.1 - 2013 Table B1.2a
BCA Vol.1 - 2013 Table B1.2b

Figure 3.1
Clause 4.2.1

Table 4.1

Table 3.2

Clause 4.3

Clause 4.4

Table 3.1

Clause 2.3

→ For Designs to AS 4055-2012 Wind loads for housing (Table 2.2) use Wind Class N2

Notes on comparing results from AS/NZS 1170.2:2011 with AS 4055 - 2012 (Refer to Appendix A3 of AS 4055 - 2012)

AS 4055 - 2012 Wind Classifications were derived from a range of design scenarios evaluated using AS/NZS 1170.2:2011 - Wind Actions (incl. Amendments 1 & 2), in which the following criteria were applied:

1. The annual probability of exceedance - 1/500 (approximately equivalent to $R_{\text{strength}} = 500$ years);
2. A factor of 0.95 on (strength) wind speed accounted for various effects unique to housing;
3. A 5% margin was allowed on the wind speed for assigning N and C classes;
4. Average roof height was taken as 6.5 metres;
5. In AS 4055 - 2012, $M_{z,\text{cat}}$ was derived from AS/NZS 1170.2:2011 using a reference height of 6.5 metres;
6. The topographic multiplier, M_t , was derived from the hill shape multiplier defined in Table A2 of AS 4055 - 2012, except that the **separation zone at the crest** (AS/NZS 1170.2:2011, Figure 4.4) was not included in AS 4055 - 2012.

Consequently, although the **Design Gust Speeds** $V_{h,u}$ and $V_{h,s}$ for N and C class sites for housing do not exactly correspond to the values of $V_{\text{des},\text{strength}}$ and $V_{\text{des},\text{serviceability}}$ from AS/NZS 1170.2:2011, the correlation shown above between the **Design Wind Speeds** determined from AS/NZS 1170.2:2011 and the **N and C Wind Classifications** determined from AS 4055 - 2012 is acceptable for design purposes.

Calculation of wind pressure for roof. (uplift)

$$V_{des. strength} = 29.5 \text{ m/s}$$

$$V_{des. serviceability} = 32.5 \text{ m/s}$$

Wind pressure

$$P = (0.5 P_{air}) [V_{des.0}]^2 C_{fig} C_{dyn}$$

$$P_u = 0.94 \text{ kpa}$$

$$P_s = 0.64 \text{ kpa}$$

$$C_{pn} = 1.2$$

$$\therefore q_u = 0.94 \times 1.2 = \underline{1.13 \text{ kpa}}$$

$$q_s = 0.64 \times 1.2 = \underline{0.77 \text{ kpa}}$$

uplift pressure (net)

$$P_u^* = q_u - 0.9 G = 1.13 - 0.9 \times 0.4 = \underline{0.77 \text{ kpa}}$$

$$P_s^* = q_s - G = 0.77 - 0.4 = \underline{0.37 \text{ kpa}}$$

Purlin RP1.

Spacing 1200 mm ; Span = 5.7 m. (double span and cantilever 1.4 m)

Loads.

$$DL = 0.4 \times 1.2 = 0.48 \text{ kN/m.}$$

$$LL = 0.25 \times 1.2 = 0.3 \text{ kN/m.}$$

$$WL = 1.13 \times 1.2 = 1.36 \text{ kN/m.}$$

$$W_{ult} \downarrow = 0.48 \times 1.2 + 0.3 \times 1.5 \\ = 1.03 \text{ kN/m.}$$

$$W_{ult} \uparrow = 1.36 - 0.9 \times 0.48 \\ = 0.93 \text{ kN/m.}$$

$$W_{serv} \downarrow = 0.48 + 0.7 \times 0.3 = 0.69 \text{ kN/m.}$$

From design table

try $\geq 150 \times 12$ - 1 row bridging

$$\phi W_L = 1.58 \text{ kN/m} > \text{req (OK)}$$

$$\phi W_U = 1.58 \text{ kN/m} > \text{req (OK)}$$

$$\text{expected d-l. deflection ratio} = \frac{L}{150 \times 1.58 / 0.48}$$

$$\text{expected serv. deflection ratio} = \frac{L}{494 \text{ (OK)}} \\ = \frac{L}{150 \times 1.58 / 0.69} = \frac{L}{343 \text{ (OK)}}$$

$$\geq 150 \times 12 @ 1200 \text{ mm ; 1 row bridging (OK)}$$

Checking for cantilever, 1.4 m

Try $\geq 200 \times 19$ with back span ~ 6000 , Strength (OK)

Deflection at cantilever end for 1.5 m. (avg $\frac{0.77 + 1.47}{2} = 1.12$)

$$\text{deflection ratio ; } dL = \frac{L}{150 \times 1.12 / 0.48} \\ = \frac{L}{350 \text{ (OK)}}$$

Checked :

Date :/...../.....

Try With $\geq 200 \times 15$

$$\text{deflection for 1.5 m cant.} \Rightarrow \frac{0.60 + 1.12}{2} = 0.86.$$

$$\therefore \text{deflection ratio; } dL = \frac{L}{150 + \frac{0.86}{0.48}} = \frac{L}{268} \quad \text{OK for cant.}$$

$$W_{\text{new}} = \frac{L}{150 + \frac{0.86}{0.69}} = \frac{L}{187} \quad \text{OK for cant.}$$

\therefore Adopt $\geq 200 \times 15 @ 1200 \text{ c/c}$

Check for requirement of bridging
Capacity for double lapped span (5700)

No bridging? $\downarrow W \downarrow = 2.1 \text{ kN/m} > \text{req. OK}$

$\uparrow W \uparrow = 2.62 \text{ kN/m} > \text{req. OK}$

OG RPI - Adopt $\geq 200 \times 15 @ 1200 \text{ c/c}$
no bridging

RP2.

Check as single span 4.7m.

loads. $w_{ult \downarrow} = 1.03 \text{ kN/m}$

$w_{ult \uparrow} = 0.93 \text{ kN/m}$

$w_{sew \downarrow} = 0.69 \text{ kN/m}$

try $\geq 150 \times 12$, 1 row bridging

$\phi w_{\downarrow} = 1.02 \text{ kN/m} > \text{req.}$

$\phi w_{\uparrow} = 1.42 \text{ kN/m} > \text{req.}$

deflection ratio;

$$dL \Rightarrow \frac{L}{150 \times \frac{1.11}{0.48}} = \frac{L}{346} \quad \text{not good.}$$

Try with exact span 4.7m.

from table

$$dL \Rightarrow \frac{L}{150 \times \frac{1.18}{0.48}} = \frac{L}{368} \quad (OK)$$

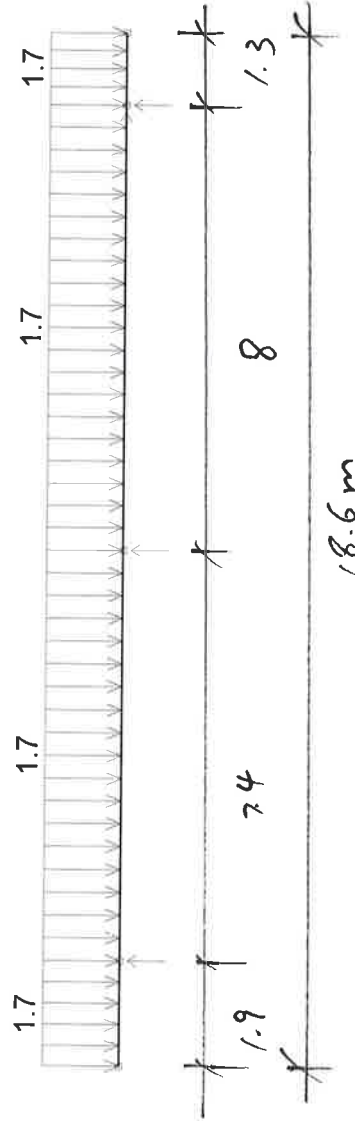
$$w_{sew} \Rightarrow \frac{L}{150 \times \frac{1.18}{0.69}} = \frac{L}{256} \quad (OK)$$

Adopt $\geq 150 \times 12 @ 1200 \text{ c/c}$; 1 row bridging

Load Cases:
— 1 P DL

R.R1 230 PFC

DL



$$DL: \text{Roof} = 0.4 \text{ kpa} \times 4.2 \text{ m} = 1.7 \text{ kN/m}$$

$$LL: \text{Roof} = 0.25 \text{ kpa} \times 4.2 \text{ m} = 1.1 \text{ kN/m}$$

$$W_u = 1.13 \text{ kpa} \times 4.2 \text{ m} = 4.75 \text{ kN/m}$$

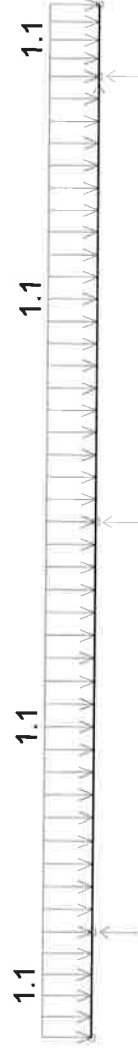
$$\text{theta: } 270 \text{ phi: } 0 W_s = 0.77 \text{ kpa} \times 4.2 \text{ m} = 3.24 \text{ kN/m DL}$$

Y ↑
Z → X

Load Cases:

— 2 P LL

LL



Y
Z
X

theta: 270 phi: 0

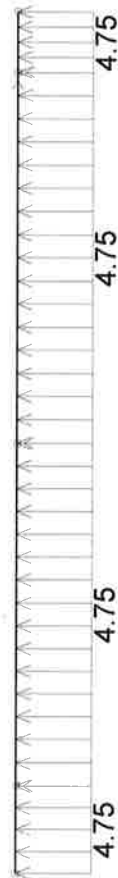
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LL

K:\2017\10\17\10168\Structural Drawings and Calcs\Design Calculations and Details\ROOF BEAM\R1

1710168
SC8

Load Cases:
—— 5 P WIND ULMIMATE



Y
Z
X

theta: 270 phi: 0

WIND UPLIFT ULMIMATE

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Job: R1

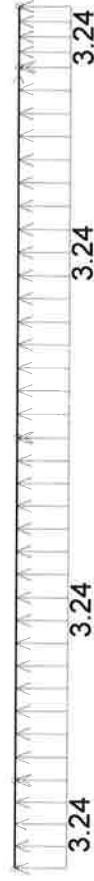
R.R1

1710168

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Load Cases:

— 6 P WIND SERVICEABILITY



Y
↑
Z → X

theta: 270 phi: 0

WIND UPLIFT SERVICEABILITY

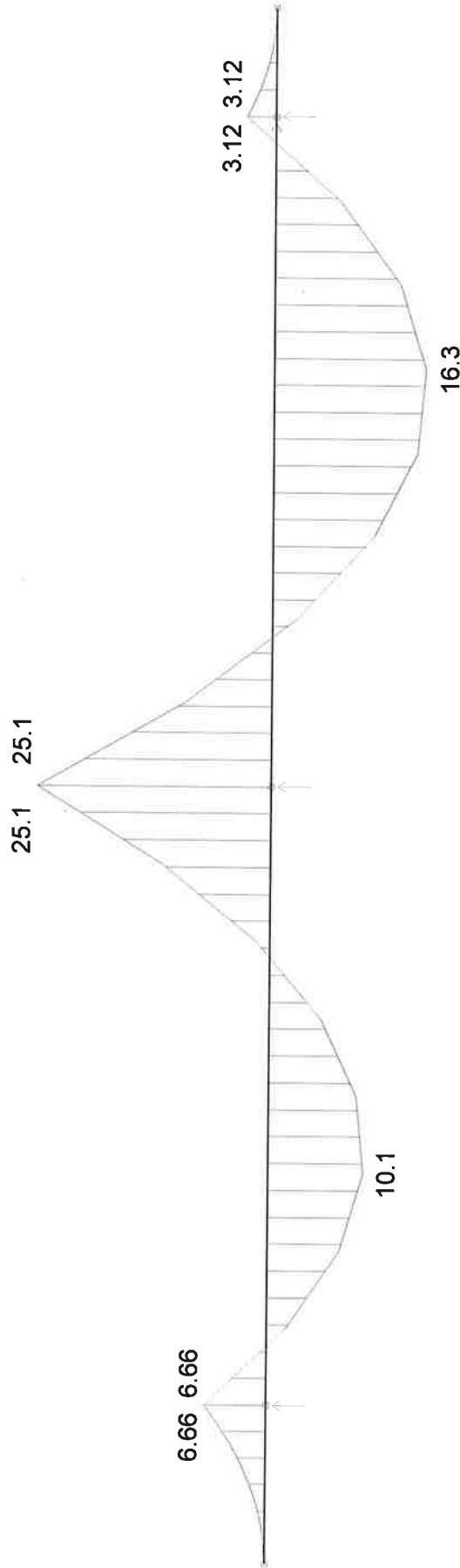
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1710168
SC10

Load Cases:
 — 3 C 1.2DL+1.5LL

R.R1 Z30PFC $l_e = 1.9 \times 2 = 3.8m$ Cantilever
 $l_e = 1.2m$ mid-span



Z30PFC $\phi M_b = 31.9 kNm$ for $l_e = 4m$ $\phi M_b > 6.66$ o.k.
 $\phi M_b = 59.1 kNm$ for $l_e = 1.5m$ $\phi M_b > 25.1 kNm$ o.k.

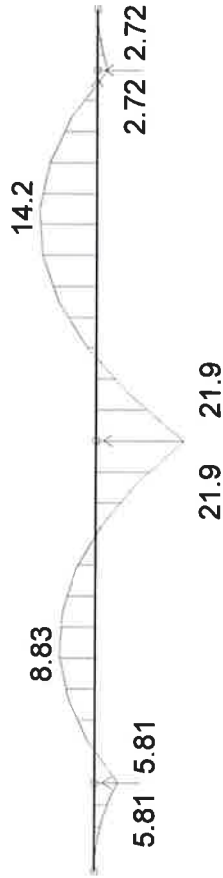
Y
 Z
 X

theta: 270 phi: 0

1.2DL+1.5LL MOMENT

Bending Moment, Mz

Load Cases:
7 C WIND ULTIMATE-O,9DL



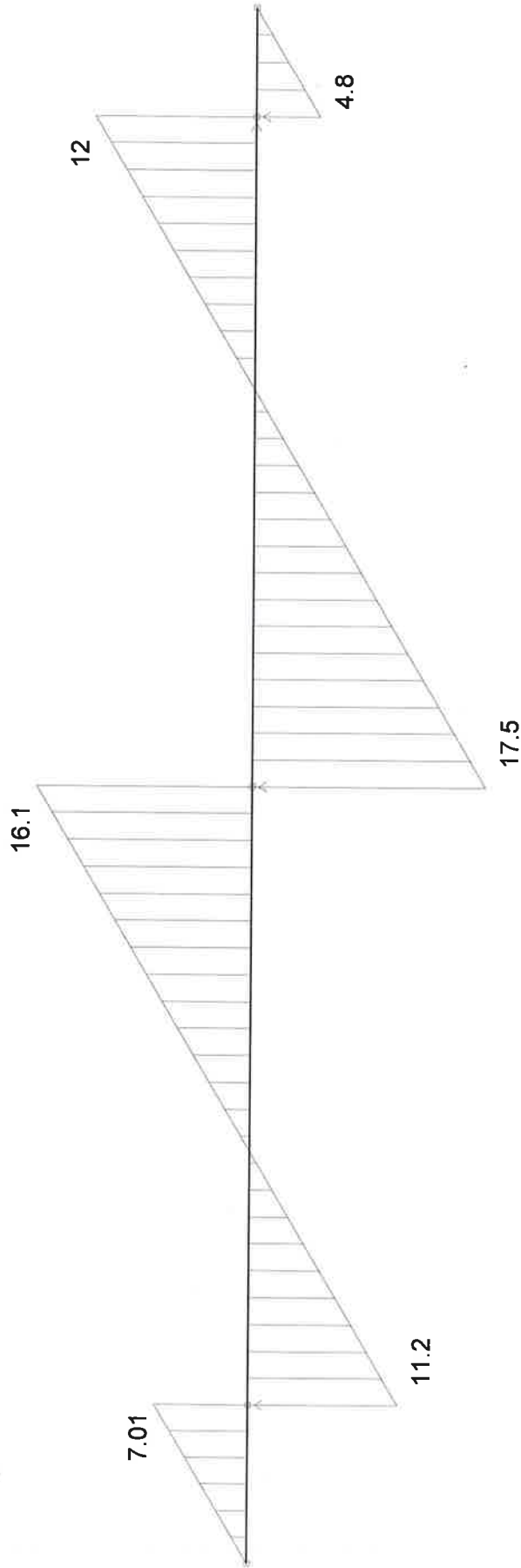
Y
Z
X

theta: 270 phi: 0

WIND UPLIFT ULTIMATE MOMENT

Bending Moment, Mz

Load Cases:
—— 3 C 1.2DL+1.5LL



Y
Z
X

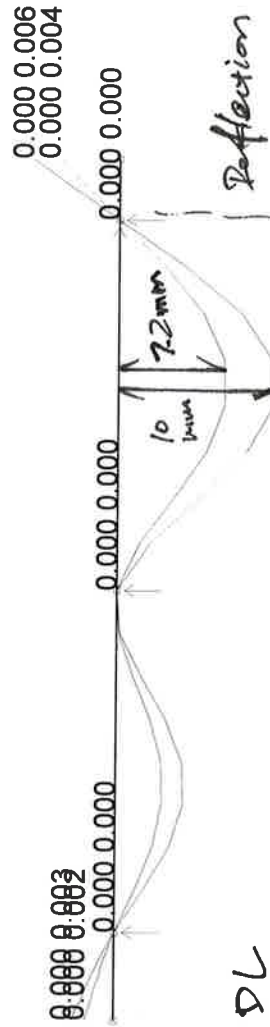
theta: 270 phi: 0

1.2DL+1.5LL SHEAR

Shear Force, Fy

Load Cases:

- 1 P DL
- 4 C DL+0.7LL



Deflection under DL

$$\Delta_{\text{limit}} = \frac{8000}{360} = 22 \text{ mm} \text{ use } 12 \text{ mm} > 7.2 \text{ mm o.k.}$$

$$\Delta_{\text{limit}} = \frac{1300}{180} = 7.2 \text{ mm} > 4 \text{ mm o.k.}$$

Deflection DL+0.7LL

$$\Delta_{\text{limit}} = 18 \text{ mm} > 10 \text{ mm o.k.}$$

$$\Delta_{\text{limit}} = \frac{1300}{150} = 8.6 \text{ mm} > 6 \text{ mm o.k.}$$

Y
Z
X

theta: 270 phi: 0

DEFLECTION UNDER DL, DL+0.7LL

Displaced Shape

— 8 C WIND SERVICEABILITY -DL



theta: 270 phi: 0

WIND UPLIFT SERVICEABILITY DEFLECTION

Displaced Shape

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Job: R1

R.R1

1710168

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6 Sep 2018

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INPUT/ANALYSIS REPORT

Job: R1

Title: R.R1

1710168

Type: Plane frame

Date: 6 Sep 2018

Time: 3:58 PM

Nodes 5
 Members 4
 Spring supports 0
 Sections 1
 Materials 1
 Primary load cases 2
 Combination load cases 2

Analysis: Linear elastic

LOAD CASES

Case	Type	Analysis	Title
1	P	L	DL
2	P	L	LL
3	C	L	1.2DL+1.5LL
4	C	L	DL+0.7LL

Analysis Types:

S - Skipped (not analysed)

L - Linear

N - Non-linear

NODE COORDINATES

Node	X m	Y m	Z m	Restraint
101	0.000	0.000	0.000	000000
102	1.900	0.000	0.000	010000
103	9.300	0.000	0.000	010000
104	17.300	0.000	0.000	111000
105	18.600	0.000	0.000	000000

MEMBER DEFINITION

Member	A	B	C	Prop	Matl	Rel-A	Rel-B	Length m
101	101	102	Y	10	1	000000	000000	1.900
102	102	103	Y	10	1	000000	000000	7.400
103	103	104	Y	10	1	000000	000000	8.000
104	104	105	Y	10	1	000000	000000	1.300

LIBRARY SECTIONS

Section	Library	Name	Axis	Comment
10	asw	230PFC	Y	All_spans

SECTION PROPERTIES

Section	Ax m2	Ay m2	Az m2	J m4	Iy m4	Iz m4	fact
10	3.200E-03	0.000E+00	0.000E+00	1.080E-07	1.760E-06	2.680E-05	

MATERIAL PROPERTIES

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

TABLE OF QUANTITIES

MATERIAL 1

Section	Name	Length m	Mass tonne	Comment
10	230PFC	18.600	0.467	All_spans
		18.600	0.467	

CONDITION NUMBER

Maximum condition number: 1.114E+01 at node: 105 DOFN: 6

CASE 1: DL

TMK Consulting Engineers

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Member Loads

Member	Form	T A S	F1	X1	F2	X2
101	UNIF	FY GL	-1.700			
102	UNIF	FY GL	-1.700			
103	UNIF	FY GL	-1.700			
104	UNIF	FY GL	-1.700			

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-31.620	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-294.066

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging

Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 1: DL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	0.65	0.00	0.00	0.00	-0.12
3	0.76	0.00	1.29	0.00	0.00	0.00	-0.49
4	1.14	0.00	1.94	0.00	0.00	0.00	-1.10
5	1.52	0.00	2.58	0.00	0.00	0.00	-1.96
6	1.90	0.00	3.23	0.00	0.00	0.00	-3.07

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0019	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0016	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0012	0.0000	0.0001	0.0000
4	1.1	0.0000	0.0009	0.0000	0.0001	0.0000
5	1.5	0.0000	0.0005	0.0000	0.0001	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-5.14	0.00	0.00	0.00	-3.07
2	1.48	0.00	-2.63	0.00	0.00	0.00	2.68
3	2.96	0.00	-0.11	0.00	0.00	0.00	4.71
4	4.44	0.00	2.40	0.00	0.00	0.00	3.02
5	5.92	0.00	4.92	0.00	0.00	0.00	-2.40
6	7.40	0.00	7.44	0.00	0.00	0.00	-11.55

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.5	0.0000	-0.0021	0.0000	-0.0021	0.0000
3	3.0	0.0000	-0.0032	0.0000	-0.0032	0.0000
4	4.4	0.0000	-0.0025	0.0000	-0.0025	0.0000
5	5.9	0.0000	-0.0007	0.0000	-0.0007	0.0000
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 103: Nodes 103 - 104 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-8.06	0.00	0.00	0.00	-11.55
2	1.60	0.00	-5.34	0.00	0.00	0.00	-0.82
3	3.20	0.00	-2.62	0.00	0.00	0.00	5.55
4	4.80	0.00	0.10	0.00	0.00	0.00	7.58
5	6.40	0.00	2.82	0.00	0.00	0.00	5.25
6	8.00	0.00	5.54	0.00	0.00	0.00	-1.44

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.6	0.0000	-0.0029	0.0000	-0.0029	0.0000
3	3.2	0.0000	-0.0063	0.0000	-0.0063	0.0000
4	4.8	0.0000	-0.0073	0.0000	-0.0073	0.0000
5	6.4	0.0000	-0.0048	0.0000	-0.0048	0.0000
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 104: Nodes 104 - 105 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-2.21	0.00	0.00	0.00	-1.44
2	0.26	0.00	-1.77	0.00	0.00	0.00	-0.92
3	0.52	0.00	-1.33	0.00	0.00	0.00	-0.52
4	0.78	0.00	-0.88	0.00	0.00	0.00	-0.23
5	1.04	0.00	-0.44	0.00	0.00	0.00	-0.06
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3	0.0000	0.0008	0.0000	0.0000	0.0000
3	0.5	0.0000	0.0016	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0024	0.0000	0.0000	0.0000
5	1.0	0.0000	0.0032	0.0000	0.0000	0.0000
6	1.3	0.0000	0.0040	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 1: DL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	8.37	0.00	0.00	0.00	0.00
103	0.00	15.50	0.00	0.00	0.00	0.00
104	0.00	7.75	0.00	0.00	0.00	0.00
SUM:	0.00	31.62	0.00 (all nodes)			

Max. residual: 2.864E-14 at DOFN: 10

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

CASE 2: LL

Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-1.100			
102	UNIF	FY	GL		-1.100			
103	UNIF	FY	GL		-1.100			
104	UNIF	FY	GL		-1.100			

Sum of Applied Loads (Global Axes):

FX: 0.000 FY: -20.460 FZ: 0.000

Moments about the global origin:

MX: 0.000 MY: 0.000 MZ: -190.278

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging

Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 2: LL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	0.42	0.00	0.00	0.00	-0.08
3	0.76	0.00	0.84	0.00	0.00	0.00	-0.32
4	1.14	0.00	1.25	0.00	0.00	0.00	-0.71
5	1.52	0.00	1.67	0.00	0.00	0.00	-1.27
6	1.90	0.00	2.09	0.00	0.00	0.00	-1.99

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0012	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0010	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0008	0.0000	0.0000	0.0000
4	1.1	0.0000	0.0006	0.0000	0.0001	0.0000
5	1.5	0.0000	0.0003	0.0000	0.0000	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-3.33	0.00	0.00	0.00	-1.99
2	1.48	0.00	-1.70	0.00	0.00	0.00	1.74
3	2.96	0.00	-0.07	0.00	0.00	0.00	3.05

4	4.44	0.00	1.56	0.00	0.00	0.00	1.95
5	5.92	0.00	3.18	0.00	0.00	0.00	-1.56
6	7.40	0.00	4.81	0.00	0.00	0.00	-7.47
Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000	
2	1.5	0.0000	-0.0013	0.0000	-0.0013	0.0000	
3	3.0	0.0000	-0.0021	0.0000	-0.0021	0.0000	
4	4.4	0.0000	-0.0016	0.0000	-0.0016	0.0000	
5	5.9	0.0000	-0.0004	0.0000	-0.0004	0.0000	
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000	

MEMBER 103: Nodes 103 - 104 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-5.22	0.00	0.00	0.00	-7.47
2	1.60	0.00	-3.46	0.00	0.00	0.00	-0.53
3	3.20	0.00	-1.70	0.00	0.00	0.00	3.59
4	4.80	0.00	0.06	0.00	0.00	0.00	4.90
5	6.40	0.00	1.82	0.00	0.00	0.00	3.39
6	8.00	0.00	3.58	0.00	0.00	0.00	-0.93

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000	
2	1.6	0.0000	-0.0019	0.0000	-0.0019	0.0000	
3	3.2	0.0000	-0.0041	0.0000	-0.0041	0.0000	
4	4.8	0.0000	-0.0047	0.0000	-0.0047	0.0000	
5	6.4	0.0000	-0.0031	0.0000	-0.0031	0.0000	
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000	

MEMBER 104: Nodes 104 - 105 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-1.43	0.00	0.00	0.00	-0.93
2	0.26	0.00	-1.14	0.00	0.00	0.00	-0.59
3	0.52	0.00	-0.86	0.00	0.00	0.00	-0.33
4	0.78	0.00	-0.57	0.00	0.00	0.00	-0.15
5	1.04	0.00	-0.29	0.00	0.00	0.00	-0.04
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000	
2	0.3	0.0000	0.0005	0.0000	0.0000	0.0000	
3	0.5	0.0000	0.0011	0.0000	0.0000	0.0000	
4	0.8	0.0000	0.0016	0.0000	0.0000	0.0000	
5	1.0	0.0000	0.0021	0.0000	0.0000	0.0000	
6	1.3	0.0000	0.0026	0.0000	0.0000	0.0000	

SUPPORT REACTIONS

CASE 2: LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	5.42	0.00	0.00	0.00	0.00
103	0.00	10.03	0.00	0.00	0.00	0.00
104	0.00	5.01	0.00	0.00	0.00	0.00

SUM: 0.00 20.46 0.00 (all nodes)

Max. residual: -7.633E-15 at DOFN: 11

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

CASE 3: 1.2DL+1.5LL

Load Combinations

Case	Factor	
1	1.200	DL
2	1.500	LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-68.634	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-638.296

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 3: 1.2DL+1.5LL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	1.40	0.00	0.00	0.00	-0.27
3	0.76	0.00	2.80	0.00	0.00	0.00	-1.07
4	1.14	0.00	4.21	0.00	0.00	0.00	-2.40
5	1.52	0.00	5.61	0.00	0.00	0.00	-4.26
6	1.90	0.00	7.01	0.00	0.00	0.00	-6.66

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0042	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0034	0.0000	0.0001	0.0000
3	0.8	0.0000	0.0026	0.0000	0.0001	0.0000
4	1.1	0.0000	0.0018	0.0000	0.0002	0.0000
5	1.5	0.0000	0.0010	0.0000	0.0001	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-11.17	0.00	0.00	0.00	-6.66
2	1.48	0.00	-5.71	0.00	0.00	0.00	5.82
3	2.96	0.00	-0.24	0.00	0.00	0.00	10.23
4	4.44	0.00	5.22	0.00	0.00	0.00	6.55
5	5.92	0.00	10.68	0.00	0.00	0.00	-5.22
6	7.40	0.00	16.14	0.00	0.00	0.00	-25.06

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.5	0.0000	-0.0045	0.0000	-0.0045	0.0000
3	3.0	0.0000	-0.0069	0.0000	-0.0069	0.0000
4	4.4	0.0000	-0.0054	0.0000	-0.0054	0.0000
5	5.9	0.0000	-0.0015	0.0000	-0.0015	0.0000
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 103: Nodes 103 - 104 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-17.50	0.00	0.00	0.00	-25.06
2	1.60	0.00	-11.60	0.00	0.00	0.00	-1.78
3	3.20	0.00	-5.69	0.00	0.00	0.00	12.05
4	4.80	0.00	0.21	0.00	0.00	0.00	16.44
5	6.40	0.00	6.11	0.00	0.00	0.00	11.39
6	8.00	0.00	12.02	0.00	0.00	0.00	-3.12

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.6	0.0000	-0.0063	0.0000	-0.0063	0.0000
3	3.2	0.0000	-0.0137	0.0000	-0.0137	0.0000
4	4.8	0.0000	-0.0158	0.0000	-0.0158	0.0000
5	6.4	0.0000	-0.0104	0.0000	-0.0104	0.0000
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 104: Nodes 104 - 105 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-4.80	0.00	0.00	0.00	-3.12
2	0.26	0.00	-3.84	0.00	0.00	0.00	-2.00
3	0.52	0.00	-2.88	0.00	0.00	0.00	-1.12
4	0.78	0.00	-1.92	0.00	0.00	0.00	-0.50
5	1.04	0.00	-0.96	0.00	0.00	0.00	-0.12
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3	0.0000	0.0018	0.0000	0.0000	0.0000
3	0.5	0.0000	0.0035	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0053	0.0000	0.0000	0.0000
5	1.0	0.0000	0.0070	0.0000	0.0000	0.0000
6	1.3	0.0000	0.0087	0.0000	0.0000	0.0000

TMK Consulting Engineers

Job: R1

R.R1

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SUPPORT REACTIONS

CASE 3: 1.2DL+1.5LL

Node	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
102	0.00	18.18	0.00	0.00	0.00	0.00
103	0.00	33.64	0.00	0.00	0.00	0.00
104	0.00	16.81	0.00	0.00	0.00	0.00
SUM:	0.00	68.63	0.00	(all nodes)		

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

CASE 4: DL+0.7LL

Load Combinations

Case	Factor	
1	1.000	DL
2	0.700	LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-45.942	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-427.261

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 4: DL+0.7LL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	0.94	0.00	0.00	0.00	-0.18
3	0.76	0.00	1.88	0.00	0.00	0.00	-0.71
4	1.14	0.00	2.82	0.00	0.00	0.00	-1.61
5	1.52	0.00	3.75	0.00	0.00	0.00	-2.85
6	1.90	0.00	4.69	0.00	0.00	0.00	-4.46

Point	Offset m	X-glob m	Y-glob m	Z-glob m	y-local m	z-local m
1	0.0	0.0000	0.0028	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0023	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0018	0.0000	0.0001	0.0000
4	1.1	0.0000	0.0012	0.0000	0.0001	0.0000
5	1.5	0.0000	0.0007	0.0000	0.0001	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	0.00	-7.47	0.00	0.00	0.00	-4.46
2	1.48	0.00	-3.82	0.00	0.00	0.00	3.90
3	2.96	0.00	-0.16	0.00	0.00	0.00	6.85
4	4.44	0.00	3.49	0.00	0.00	0.00	4.38
5	5.92	0.00	7.15	0.00	0.00	0.00	-3.49
6	7.40	0.00	10.80	0.00	0.00	0.00	-16.78

Point	Offset m	X-glob m	Y-glob m	Z-glob m	y-local m	z-local m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.5	0.0000	-0.0030	0.0000	-0.0030	0.0000
3	3.0	0.0000	-0.0046	0.0000	-0.0046	0.0000
4	4.4	0.0000	-0.0036	0.0000	-0.0036	0.0000
5	5.9	0.0000	-0.0010	0.0000	-0.0010	0.0000
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 103: Nodes 103 - 104 Section 10: 230PFC Y

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	0.00	-11.72	0.00	0.00	0.00	-16.78
2	1.60	0.00	-7.76	0.00	0.00	0.00	-1.19
3	3.20	0.00	-3.81	0.00	0.00	0.00	8.07
4	4.80	0.00	0.14	0.00	0.00	0.00	11.01
5	6.40	0.00	4.09	0.00	0.00	0.00	7.62
6	8.00	0.00	8.04	0.00	0.00	0.00	-2.09

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Job: R1

R.R1

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3:59 PM

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.6	0.0000	-0.0042	0.0000	-0.0042	0.0000
3	3.2	0.0000	-0.0092	0.0000	-0.0092	0.0000
4	4.8	0.0000	-0.0106	0.0000	-0.0106	0.0000
5	6.4	0.0000	-0.0070	0.0000	-0.0070	0.0000
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 104: Nodes 104 - 105 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-3.21	0.00	0.00	0.00	-2.09
2	0.26	0.00	-2.57	0.00	0.00	0.00	-1.34
3	0.52	0.00	-1.93	0.00	0.00	0.00	-0.75
4	0.78	0.00	-1.28	0.00	0.00	0.00	-0.33
5	1.04	0.00	-0.64	0.00	0.00	0.00	-0.08
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3	0.0000	0.0012	0.0000	0.0000	0.0000
3	0.5	0.0000	0.0024	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0035	0.0000	0.0000	0.0000
5	1.0	0.0000	0.0047	0.0000	0.0000	0.0000
6	1.3	0.0000	0.0058	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 4: DL+0.7LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	12.17	0.00	0.00	0.00	0.00
103	0.00	22.52	0.00	0.00	0.00	0.00
104	0.00	11.25	0.00	0.00	0.00	0.00

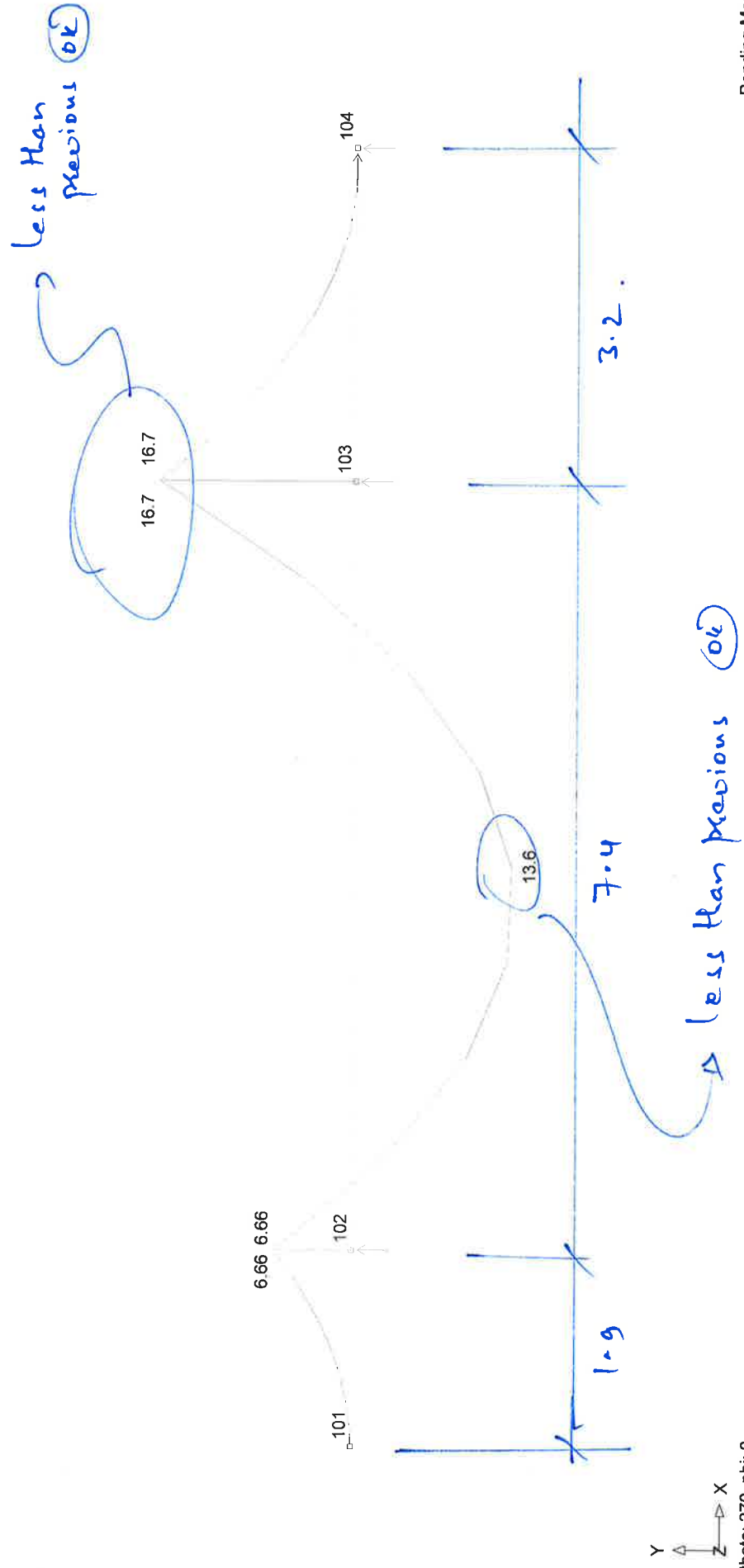
SUM: 0.00 45.94 0.00 (all nodes)

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

Roof Beam R21 - revised span because of full height panels.
 Loading - same as before

Adopted 230 PFC OK for strength.

Envelope for Moment Mz
 — Maximum
 — Minimum
 Enveloped Cases:
 3 C 1.2DL+1.5LL

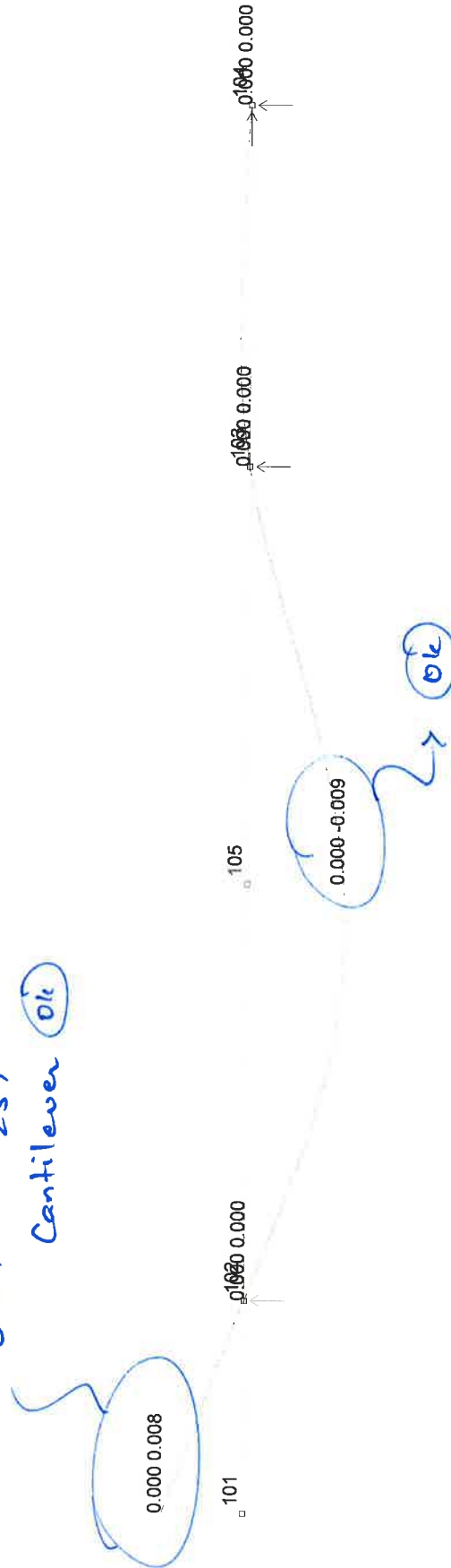


RR1 - Check for deflection. (D.L.)

7 Oct 2018
 11:33 AM

Load Cases:
 — 1 P DL

8mm ie $\frac{L}{237}$ for
 Cantilever **OK**



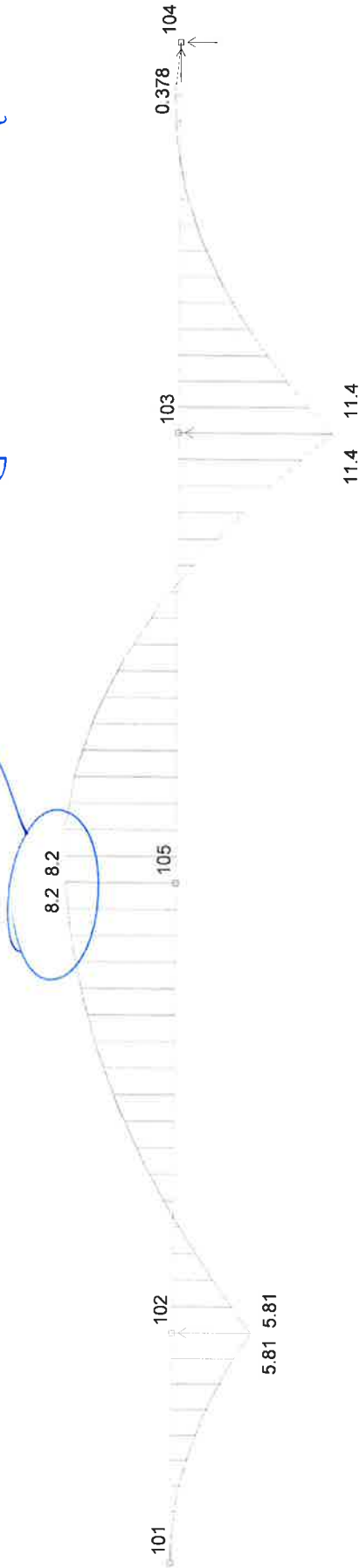
Y
 Z
 theta: 270 phi: 0

Under wind load. — 230 pFe.

7 Oct 2018
 12:30 PM

Load Cases:
 — 6 C .9DL+WLult

$\phi M = 1810 \text{ kNm} > \text{req.}$
 (OK)
 Fly brace not required.



Y
 Z
 X
 theta: 270 phi: 0

Bending Moment, Mz

Microstran [V8.11r]

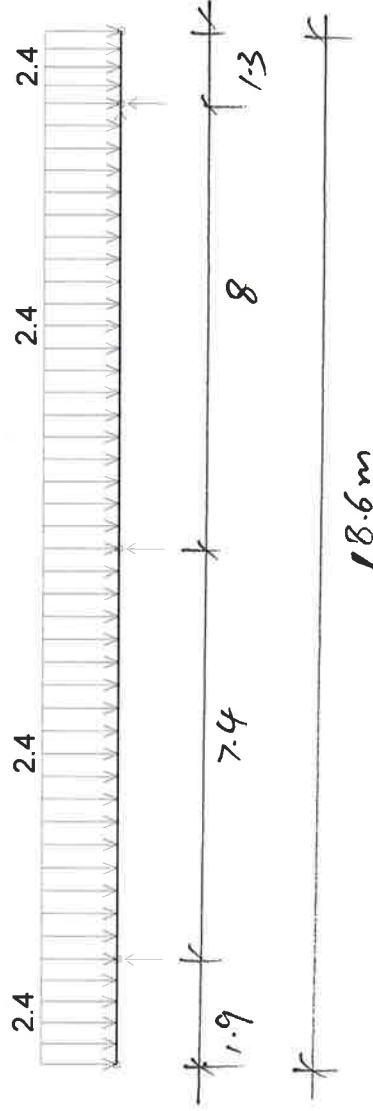
K:\2017\10\1710168\Structural Drawings and Calcs\Design Calculations and Details\ROOF BEAMR1 revised span

1710168
 SC25

Load Cases:
1 PDL

R.R2 . 250UB 37.3

→ 250UB26 adequate with fly brace
@ 3600% — refer to cal's at back.



DL: Roof = $0.4 \times 6 \text{ m} = 2.4 \text{ kN/m}$

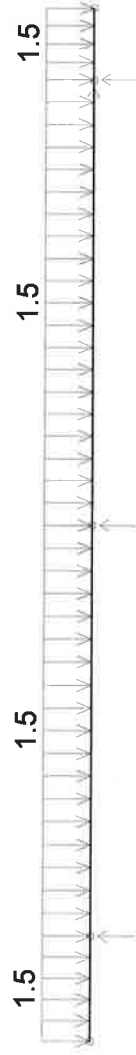
LL: Roof = $0.75 \times 6 \text{ m} = 1.5 \text{ kN/m}$

WL = $1.13 \times 6 = 6.78 \text{ kN/m}$
theta: 270 phi: 0

Y ↑
Z → X

DL

Load Cases:
—— 2 P LL

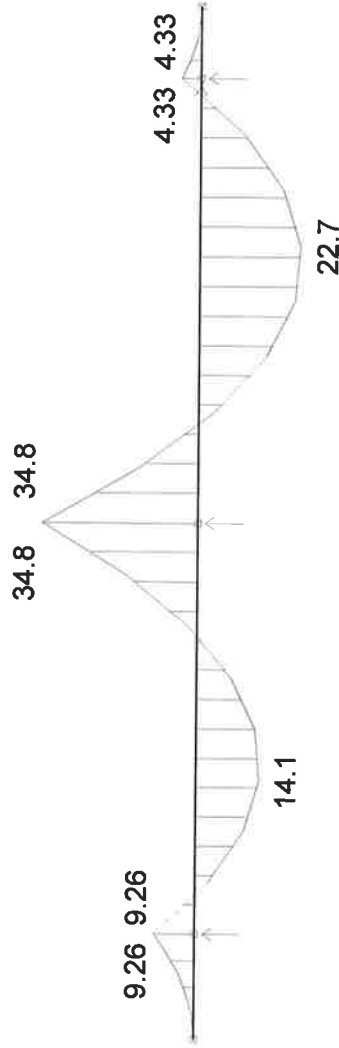


Y
Δ
Z → X

theta: 270 phi: 0

LL

Load Cases:
 — 3 C 1.2DL+1.5LL



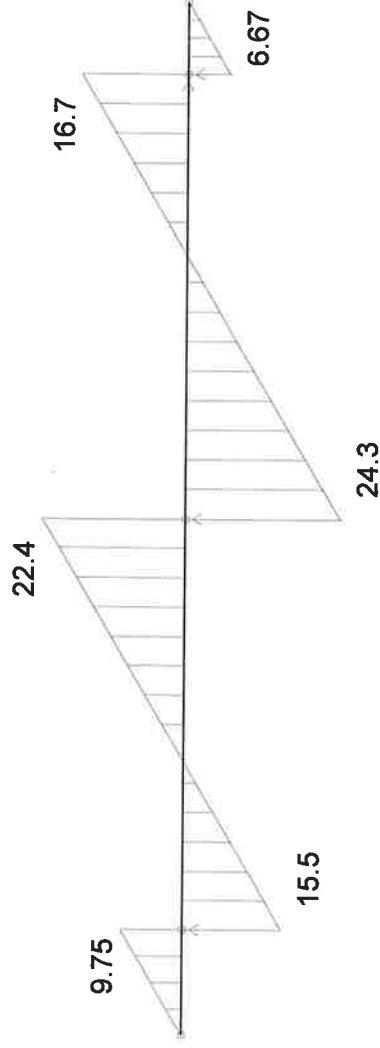
Y
 Z
 X

theta: 270 phi: 0

1.2DL+1.5LL

Bending Moment, Mz

Load Cases:
—— 3 C 1.2DL+1.5LL



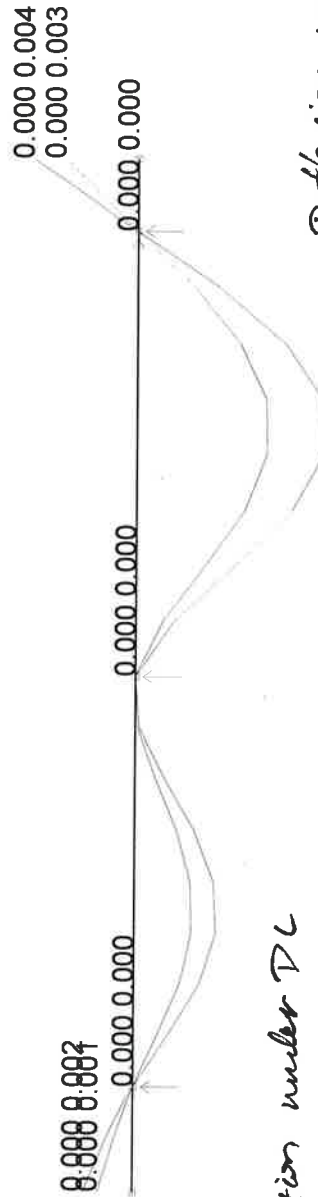
Y
Z
X

theta: 270 phi: 0

1.2DL+1.5LL SHEAR

Shear Force, Fy

Load Cases:
— 1 P DL
— 4 C DL+0.7LL



Deflection under DL

In span $\Delta_{limit} = \frac{8000}{360} = 22mm > 5mm$ o.k.

Center $\Delta_{limit} = \frac{1300}{350} = 3.7mm > 3mm$ o.k.

Deflection under DL+0.7LL

$\Delta_{limit} = \frac{8000}{350} = 22.8mm > 7mm$ o.k.

$\Delta_{limit} = \frac{1300}{200} = 6.5mm > 4mm$ o.k.

Y
Z
X

theta: 270 phi: 0

DEFLECTION DL, DL+0.7LL

Displaced Shape

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R.R2
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INPUT/ANALYSIS REPORT

Job: R2
Title: R.R2
1710168
Type: Plane frame
Date: 4 Sep 2018
Time: 09:06 AM

Nodes	5
Members	4
Spring supports	0
Sections	1
Materials	1
Primary load cases	2
Combination load cases	2

Analysis: Linear elastic

LOAD CASES

Case	Type	Analysis	Title
1	P	L	DL
2	P	L	LL
3	C	L	1.2DL+1.5LL
4	C	L	DL+0.7LL

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

NODE COORDINATES

Node	X m	Y m	Z m	Restraint
101	0.000	0.000	0.000	000000
102	1.900	0.000	0.000	010000
103	9.300	0.000	0.000	010000
104	17.300	0.000	0.000	111000
105	18.600	0.000	0.000	000000

MEMBER DEFINITION

Member	A	B	C	Prop	Matl	Rel-A	Rel-B	Length m
101	101	102	Y	10	1	000000	000000	1.900
102	102	103	Y	10	1	000000	000000	7.400
103	103	104	Y	10	1	000000	000000	8.000
104	104	105	Y	10	1	000000	000000	1.300

LIBRARY SECTIONS

Section	Library	Name	Axis	Comment
10	asw	250UB37.3	Y	All_spans

SECTION PROPERTIES

Section	Ax m2	Ay m2	Az m2	J m4	Iy m4	Iz m4	fact
10	4.750E-03	0.000E+00	0.000E+00	1.580E-07	5.660E-06	5.570E-05	

MATERIAL PROPERTIES

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

TABLE OF QUANTITIES

MATERIAL		1			
Section	Name	Length m	Mass tonne	Comment	
10	250UB37.3	18.600	0.694	All_spans	
		18.600	0.694		

CONDITION NUMBER

Maximum condition number: 1.114E+01 at node: 105 DOFN: 6
CASE 1: DL

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Job: R2

R.R2

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Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-2.400			
102	UNIF	FY	GL		-2.400			
103	UNIF	FY	GL		-2.400			
104	UNIF	FY	GL		-2.400			

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-44.640	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-415.152

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
 Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 1: DL

MEMBER 101: Nodes 101 - 102 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	0.91	0.00	0.00	0.00	-0.17
3	0.76	0.00	1.82	0.00	0.00	0.00	-0.69
4	1.14	0.00	2.74	0.00	0.00	0.00	-1.56
5	1.52	0.00	3.65	0.00	0.00	0.00	-2.77
6	1.90	0.00	4.56	0.00	0.00	0.00	-4.33

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0013	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0011	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0008	0.0000	0.0000	0.0000
4	1.1	0.0000	0.0006	0.0000	0.0001	0.0000
5	1.5	0.0000	0.0003	0.0000	0.0000	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-7.26	0.00	0.00	0.00	-4.33
2	1.48	0.00	-3.71	0.00	0.00	0.00	3.79
3	2.96	0.00	-0.16	0.00	0.00	0.00	6.65
4	4.44	0.00	3.39	0.00	0.00	0.00	4.26
5	5.92	0.00	6.95	0.00	0.00	0.00	-3.39
6	7.40	0.00	10.50	0.00	0.00	0.00	-16.30

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.5	0.0000	-0.0014	0.0000	-0.0014	0.0000
3	3.0	0.0000	-0.0022	0.0000	-0.0022	0.0000
4	4.4	0.0000	-0.0017	0.0000	-0.0017	0.0000
5	5.9	0.0000	-0.0005	0.0000	-0.0005	0.0000
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 103: Nodes 103 - 104 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-11.38	0.00	0.00	0.00	-16.30
2	1.60	0.00	-7.54	0.00	0.00	0.00	-1.16
3	3.20	0.00	-3.70	0.00	0.00	0.00	7.84
4	4.80	0.00	0.14	0.00	0.00	0.00	10.70
5	6.40	0.00	3.98	0.00	0.00	0.00	7.41
6	8.00	0.00	7.82	0.00	0.00	0.00	-2.03

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.6	0.0000	-0.0020	0.0000	-0.0020	0.0000
3	3.2	0.0000	-0.0043	0.0000	-0.0043	0.0000
4	4.8	0.0000	-0.0050	0.0000	-0.0050	0.0000
5	6.4	0.0000	-0.0033	0.0000	-0.0033	0.0000
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 104: Nodes 104 - 105 Section 10: 250UB37.3 Y

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Job: R2

R.R2

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Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-3.12	0.00	0.00	0.00	-2.03
2	0.26	0.00	-2.50	0.00	0.00	0.00	-1.30
3	0.52	0.00	-1.87	0.00	0.00	0.00	-0.73
4	0.78	0.00	-1.25	0.00	0.00	0.00	-0.32
5	1.04	0.00	-0.62	0.00	0.00	0.00	-0.08
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3	0.0000	0.0006	0.0000	0.0000	0.0000
3	0.5	0.0000	0.0011	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0016	0.0000	0.0000	0.0000
5	1.0	0.0000	0.0022	0.0000	0.0000	0.0000
6	1.3	0.0000	0.0027	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 1: DL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	11.82	0.00	0.00	0.00	0.00
103	0.00	21.88	0.00	0.00	0.00	0.00
104	0.00	10.94	0.00	0.00	0.00	0.00

SUM: 0.00 44.64 0.00 (all nodes)

Max. residual: 6.328E-15 at DOFN: 11

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

CASE 2: LL

Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-1.500			
102	UNIF	FY	GL		-1.500			
103	UNIF	FY	GL		-1.500			
104	UNIF	FY	GL		-1.500			

Sum of Applied Loads (Global Axes):

FX: 0.000 FY: -27.900 FZ: 0.000

Moments about the global origin:

MX: 0.000 MY: 0.000 MZ: -259.470

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging

Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 2: LL

MEMBER 101: Nodes 101 - 102 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	0.57	0.00	0.00	0.00	-0.11
3	0.76	0.00	1.14	0.00	0.00	0.00	-0.43
4	1.14	0.00	1.71	0.00	0.00	0.00	-0.97
5	1.52	0.00	2.28	0.00	0.00	0.00	-1.73
6	1.90	0.00	2.85	0.00	0.00	0.00	-2.71

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0008	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0007	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0005	0.0000	0.0000	0.0000
4	1.1	0.0000	0.0004	0.0000	0.0000	0.0000
5	1.5	0.0000	0.0002	0.0000	0.0000	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-4.54	0.00	0.00	0.00	-2.71
2	1.48	0.00	-2.32	0.00	0.00	0.00	2.37
3	2.96	0.00	-0.10	0.00	0.00	0.00	4.16

4	4.44	0.00	2.12	0.00	0.00	0.00	2.66
5	5.92	0.00	4.34	0.00	0.00	0.00	-2.12
6	7.40	0.00	6.56	0.00	0.00	0.00	-10.19
Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000	
2	1.5	0.0000	-0.0009	0.0000	-0.0009	0.0000	
3	3.0	0.0000	-0.0014	0.0000	-0.0014	0.0000	
4	4.4	0.0000	-0.0011	0.0000	-0.0011	0.0000	
5	5.9	0.0000	-0.0003	0.0000	-0.0003	0.0000	
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000	

MEMBER 103: Nodes 103 - 104 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-7.12	0.00	0.00	0.00	-10.19
2	1.60	0.00	-4.72	0.00	0.00	0.00	-0.72
3	3.20	0.00	-2.32	0.00	0.00	0.00	4.90
4	4.80	0.00	0.08	0.00	0.00	0.00	6.68
5	6.40	0.00	2.48	0.00	0.00	0.00	4.63
6	8.00	0.00	4.88	0.00	0.00	0.00	-1.27

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000	
2	1.6	0.0000	-0.0012	0.0000	-0.0012	0.0000	
3	3.2	0.0000	-0.0027	0.0000	-0.0027	0.0000	
4	4.8	0.0000	-0.0031	0.0000	-0.0031	0.0000	
5	6.4	0.0000	-0.0020	0.0000	-0.0020	0.0000	
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000	

MEMBER 104: Nodes 104 - 105 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-1.95	0.00	0.00	0.00	-1.27
2	0.26	0.00	-1.56	0.00	0.00	0.00	-0.81
3	0.52	0.00	-1.17	0.00	0.00	0.00	-0.46
4	0.78	0.00	-0.78	0.00	0.00	0.00	-0.20
5	1.04	0.00	-0.39	0.00	0.00	0.00	-0.05
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000	
2	0.3	0.0000	0.0003	0.0000	0.0000	0.0000	
3	0.5	0.0000	0.0007	0.0000	0.0000	0.0000	
4	0.8	0.0000	0.0010	0.0000	0.0000	0.0000	
5	1.0	0.0000	0.0014	0.0000	0.0000	0.0000	
6	1.3	0.0000	0.0017	0.0000	0.0000	0.0000	

SUPPORT REACTIONS

CASE 2: LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	7.39	0.00	0.00	0.00	0.00
103	0.00	13.68	0.00	0.00	0.00	0.00
104	0.00	6.83	0.00	0.00	0.00	0.00

SUM: 0.00 27.90 0.00 (all nodes)

Max. residual: 4.302E-15 at DOFN: 11

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

CASE 3: 1.2DL+1.5LL

Load Combinations

Case	Factor
1	1.200 DL
2	1.500 LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-95.418	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-887.387

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension	Shear - End A sagging
Torque - Right-hand twist	Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 3: 1.2DL+1.5LL

MEMBER 101: Nodes 101 - 102 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	1.95	0.00	0.00	0.00	-0.37
3	0.76	0.00	3.90	0.00	0.00	0.00	-1.48
4	1.14	0.00	5.85	0.00	0.00	0.00	-3.33
5	1.52	0.00	7.80	0.00	0.00	0.00	-5.93
6	1.90	0.00	9.75	0.00	0.00	0.00	-9.26

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0028	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0023	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0018	0.0000	0.0001	0.0000
4	1.1	0.0000	0.0012	0.0000	0.0001	0.0000
5	1.5	0.0000	0.0007	0.0000	0.0001	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-15.52	0.00	0.00	0.00	-9.26
2	1.48	0.00	-7.93	0.00	0.00	0.00	8.10
3	2.96	0.00	-0.34	0.00	0.00	0.00	14.22
4	4.44	0.00	7.25	0.00	0.00	0.00	9.10
5	5.92	0.00	14.85	0.00	0.00	0.00	-7.25
6	7.40	0.00	22.44	0.00	0.00	0.00	-34.84

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.5	0.0000	-0.0030	0.0000	-0.0030	0.0000
3	3.0	0.0000	-0.0046	0.0000	-0.0046	0.0000
4	4.4	0.0000	-0.0036	0.0000	-0.0036	0.0000
5	5.9	0.0000	-0.0010	0.0000	-0.0010	0.0000
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 103: Nodes 103 - 104 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-24.33	0.00	0.00	0.00	-34.84
2	1.60	0.00	-16.13	0.00	0.00	0.00	-2.48
3	3.20	0.00	-7.92	0.00	0.00	0.00	16.76
4	4.80	0.00	0.29	0.00	0.00	0.00	22.86
5	6.40	0.00	8.50	0.00	0.00	0.00	15.83
6	8.00	0.00	16.71	0.00	0.00	0.00	-4.33

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.6	0.0000	-0.0042	0.0000	-0.0042	0.0000
3	3.2	0.0000	-0.0092	0.0000	-0.0092	0.0000
4	4.8	0.0000	-0.0106	0.0000	-0.0106	0.0000
5	6.4	0.0000	-0.0070	0.0000	-0.0070	0.0000
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 104: Nodes 104 - 105 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-6.67	0.00	0.00	0.00	-4.33
2	0.26	0.00	-5.34	0.00	0.00	0.00	-2.77
3	0.52	0.00	-4.00	0.00	0.00	0.00	-1.56
4	0.78	0.00	-2.67	0.00	0.00	0.00	-0.69
5	1.04	0.00	-1.33	0.00	0.00	0.00	-0.17
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3	0.0000	0.0012	0.0000	0.0000	0.0000
3	0.5	0.0000	0.0024	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0035	0.0000	0.0000	0.0000
5	1.0	0.0000	0.0047	0.0000	0.0000	0.0000
6	1.3	0.0000	0.0058	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 3: 1.2DL+1.5LL

Node	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
102	0.00	25.27	0.00	0.00	0.00	0.00
103	0.00	46.77	0.00	0.00	0.00	0.00
104	0.00	23.38	0.00	0.00	0.00	0.00
SUM:	0.00	95.42	0.00 (all nodes)			

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

CASE 4: DL+0.7LL

Load Combinations

Case	Factor
1	1.000 DL
2	0.700 LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-64.170	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-596.781

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 4: DL+0.7LL

MEMBER 101: Nodes 101 - 102 Section 10: 250UB37.3 Y

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.38	0.00	1.31	0.00	0.00	0.00	-0.25
3	0.76	0.00	2.62	0.00	0.00	0.00	-1.00
4	1.14	0.00	3.93	0.00	0.00	0.00	-2.24
5	1.52	0.00	5.24	0.00	0.00	0.00	-3.99
6	1.90	0.00	6.55	0.00	0.00	0.00	-6.23

Point	Offset m	X-glob m	Y-glob m	Z-glob m	y-local m	z-local m
1	0.0	0.0000	0.0019	0.0000	0.0000	0.0000
2	0.4	0.0000	0.0015	0.0000	0.0000	0.0000
3	0.8	0.0000	0.0012	0.0000	0.0001	0.0000
4	1.1	0.0000	0.0008	0.0000	0.0001	0.0000
5	1.5	0.0000	0.0004	0.0000	0.0001	0.0000
6	1.9	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 250UB37.3 Y

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	0.00	-10.44	0.00	0.00	0.00	-6.23
2	1.48	0.00	-5.33	0.00	0.00	0.00	5.45
3	2.96	0.00	-0.23	0.00	0.00	0.00	9.56
4	4.44	0.00	4.88	0.00	0.00	0.00	6.12
5	5.92	0.00	9.98	0.00	0.00	0.00	-4.88
6	7.40	0.00	15.09	0.00	0.00	0.00	-23.43

Point	Offset m	X-glob m	Y-glob m	Z-glob m	y-local m	z-local m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.5	0.0000	-0.0020	0.0000	-0.0020	0.0000
3	3.0	0.0000	-0.0031	0.0000	-0.0031	0.0000
4	4.4	0.0000	-0.0024	0.0000	-0.0024	0.0000
5	5.9	0.0000	-0.0007	0.0000	-0.0007	0.0000
6	7.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 103: Nodes 103 - 104 Section 10: 250UB37.3 Y

Point	Offset m	Axial kN	Shear-y kN	Shear-z kN	Torque kNm	Moment-y kNm	Moment-z kNm
1	0.00	0.00	-16.36	0.00	0.00	0.00	-23.43
2	1.60	0.00	-10.84	0.00	0.00	0.00	-1.66
3	3.20	0.00	-5.32	0.00	0.00	0.00	11.27
4	4.80	0.00	0.20	0.00	0.00	0.00	15.37
5	6.40	0.00	5.72	0.00	0.00	0.00	10.65
6	8.00	0.00	11.24	0.00	0.00	0.00	-2.92

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.6	0.0000	-0.0028	0.0000	-0.0028	0.0000
3	3.2	0.0000	-0.0062	0.0000	-0.0062	0.0000
4	4.8	0.0000	-0.0071	0.0000	-0.0071	0.0000
5	6.4	0.0000	-0.0047	0.0000	-0.0047	0.0000
6	8.0	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 104: Nodes 104 - 105 Section 10: 250UB37.3 Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-4.49	0.00	0.00	0.00	-2.92
2	0.26	0.00	-3.59	0.00	0.00	0.00	-1.87
3	0.52	0.00	-2.69	0.00	0.00	0.00	-1.05
4	0.78	0.00	-1.79	0.00	0.00	0.00	-0.47
5	1.04	0.00	-0.90	0.00	0.00	0.00	-0.12
6	1.30	0.00	0.00	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.3	0.0000	0.0008	0.0000	0.0000	0.0000
3	0.5	0.0000	0.0016	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0024	0.0000	0.0000	0.0000
5	1.0	0.0000	0.0031	0.0000	0.0000	0.0000
6	1.3	0.0000	0.0039	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 4: DL+0.7LL

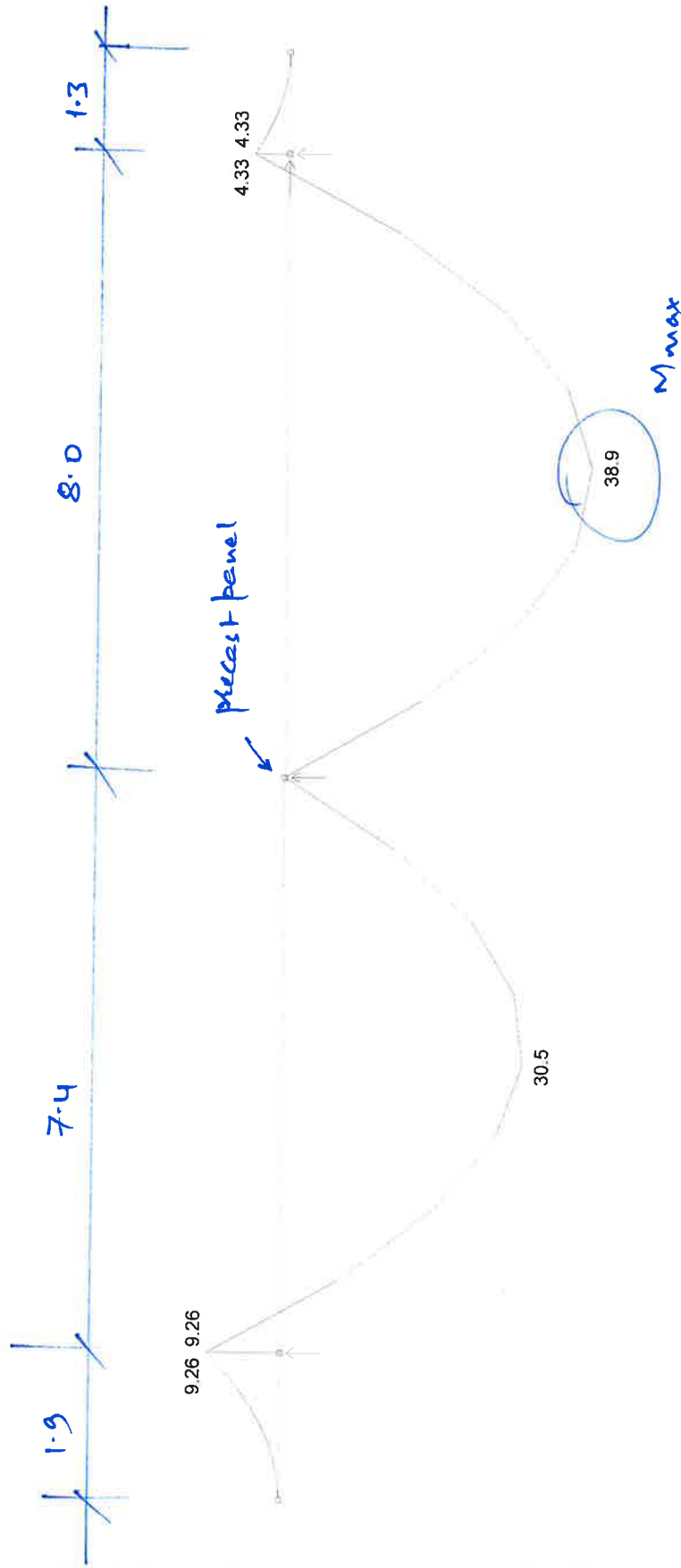
Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	17.00	0.00	0.00	0.00	0.00
103	0.00	31.45	0.00	0.00	0.00	0.00
104	0.00	15.72	0.00	0.00	0.00	0.00

SUM: 0.00 64.17 0.00 (all nodes)

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

Roof beam R.R2 - revised span condition. - Using 250 UB26.
* Loading same as before.

Envelope for Moment Mz
Maximum
Minimum
Enveloped Cases:
3 C 1.2DL+1.5LL



Adopt 250 UB26

Restrain at 1200% by purlin
00 $\phi M = 68.3 \text{ kNm} > \text{req'd}$

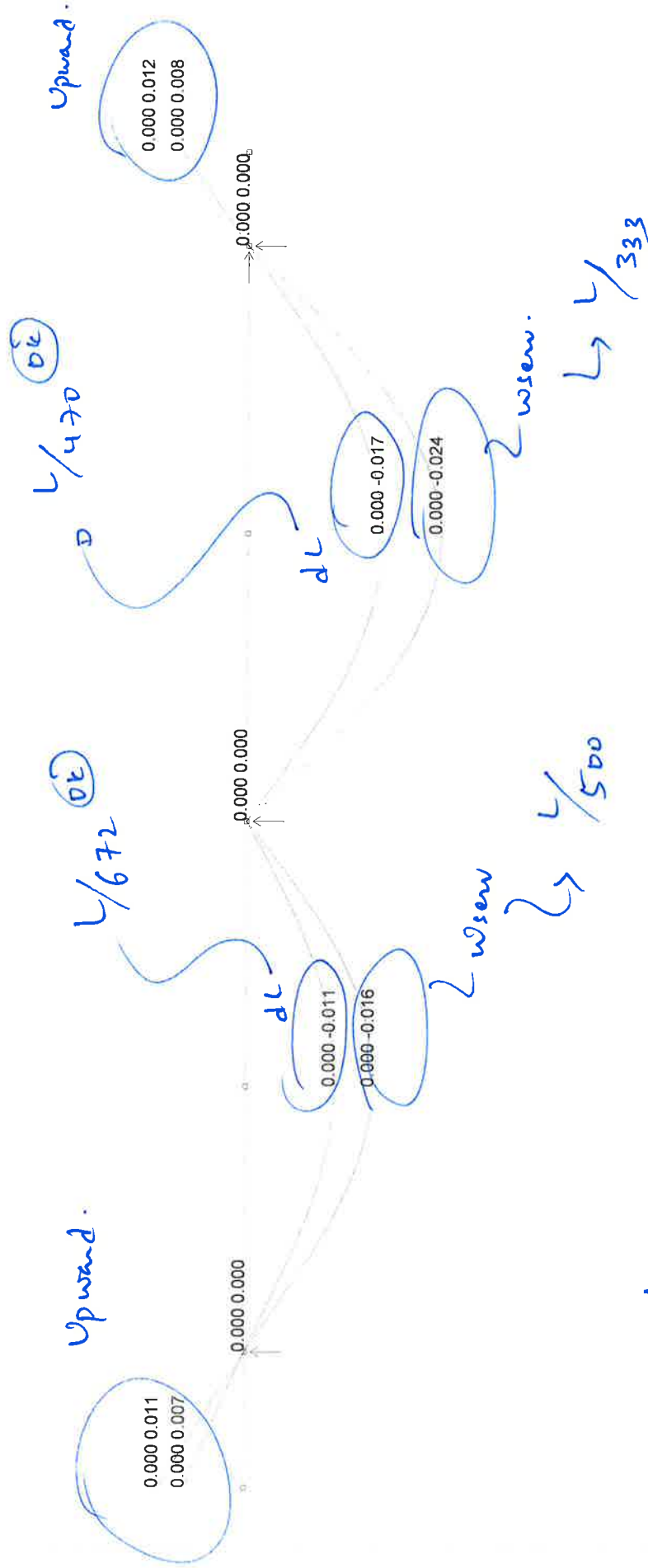
Y
Z
X
theta: 270 phi: 0

Bending Moment, Mz

Deflection check \rightarrow 250 v 26.

7 Oct 2018
 11:56 AM

Load Cases:
 — 1 P DL
 — 4 C DL+0.7LL



Adopt 250 v 26

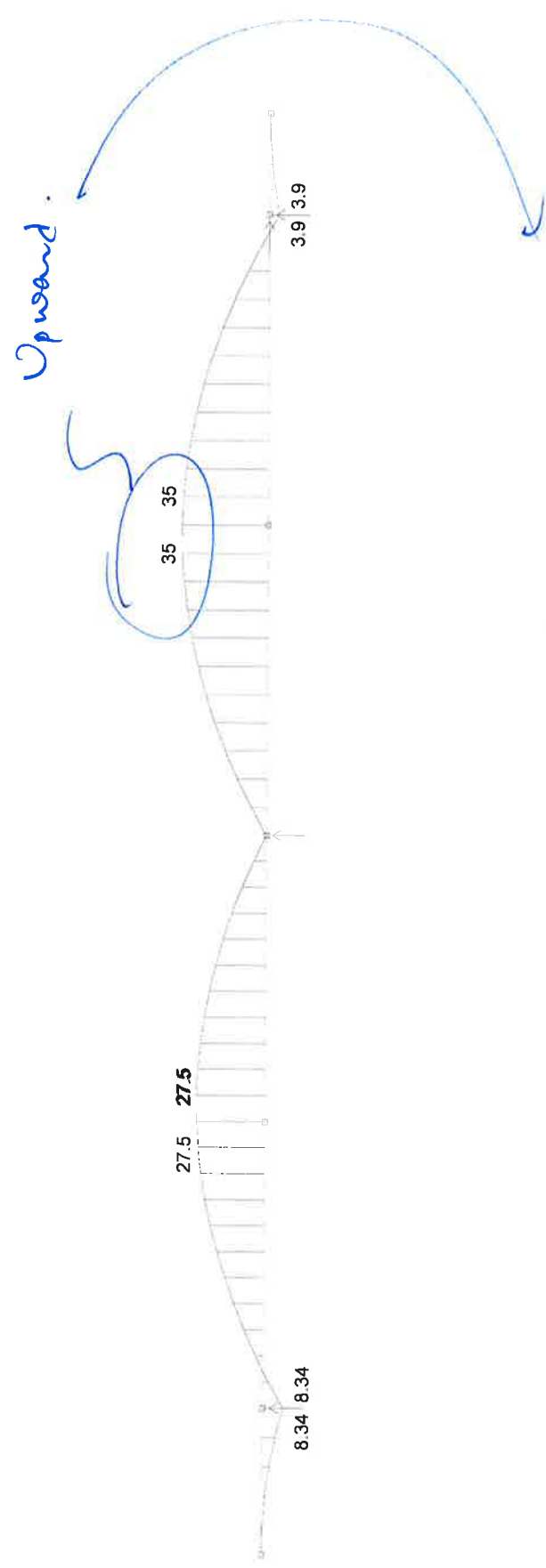
Y
 Z
 theta: 270 phi: 0

Displaced Shape

Under wind load → 250 UB 26.

7 Oct 2018
 12:15 PM

Load Cases:
 — 6 C .9DL+WL ult



Capacity of 250 UB 26
 if fly brace at every
 third purlin i.e. 3600 mm.
 $\phi M = 43.1 \text{ kNm}$
 $= 48.7 \text{ kNm}$

Adopt 250 UB 26 with fly brace @
 every third purlin 3600 mm.

Y
 Z
 theta: 270 phi: 0

Bending Moment, Mz

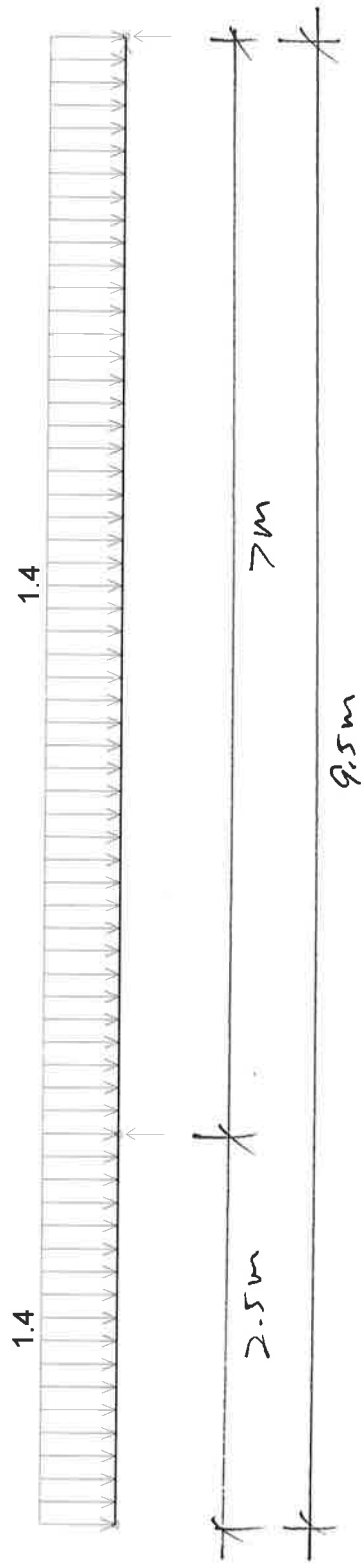
Microtran [V8.11r]

K:\2017\10\1710168\Structural Drawings and Calcs\Design Calculations and Details\ROOF BEAM\R2 revised span

1710168
 SC40

Load Cases:
 — 1 P DL

R.R3 230 PFC



$$D.L: \text{Roof} = 0.4 \text{ kpa} \times 3.5 \text{ m} = 1.4 \text{ kN/m}$$

$$L.L: \text{Roof} = 0.25 \text{ kpa} \times 3.5 \text{ m} = 0.9 \text{ kN/m}$$

Y
 Z → X

theta: 270 phi: 0

DL

Load Cases:
—— 2 P LL



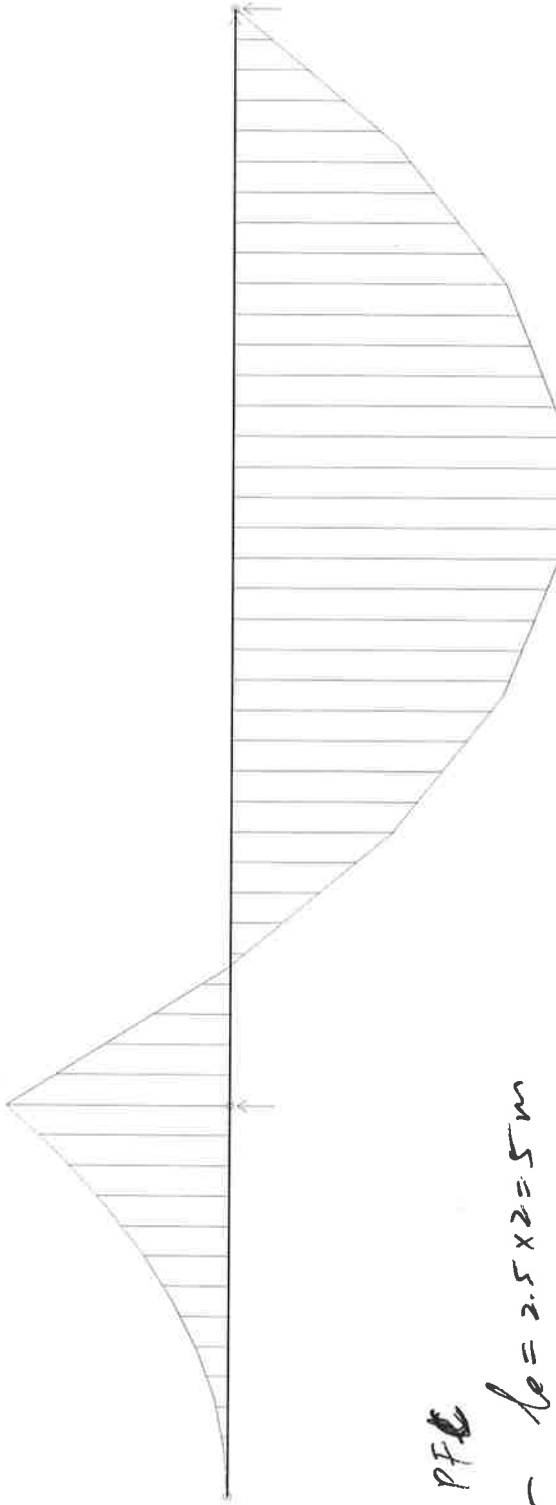
Y
Z
X

theta: 270 phi: 0

LL

Load Cases:
 — 3 C 1.2DL+1.5LL

9.47 9.47



230 PFC
 Cantilever $l_e = 2.5 \times 2 = 5m$

$$\phi M_b = 26.4 kNm > 9.47 kNm \text{ o.k.}$$

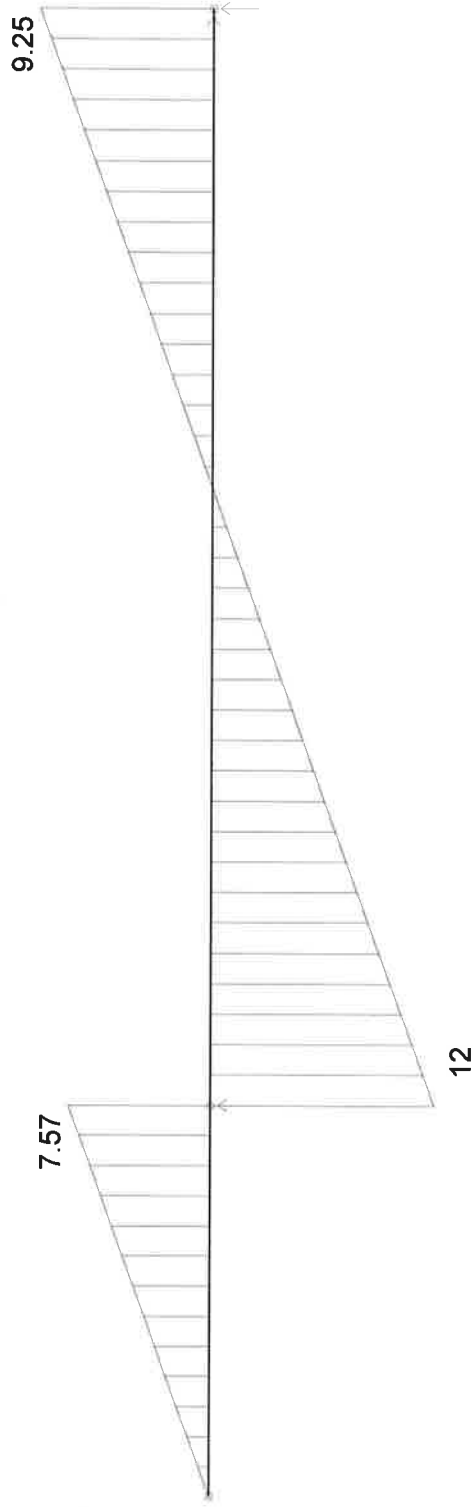
Y
 Z → X

theta: 270 phi: 0

1.2DL+1.5LL MOMENT

Bending Moment, Mz

Load Cases:
—— 3 C 1.2DL+1.5LL



Y
Z
X

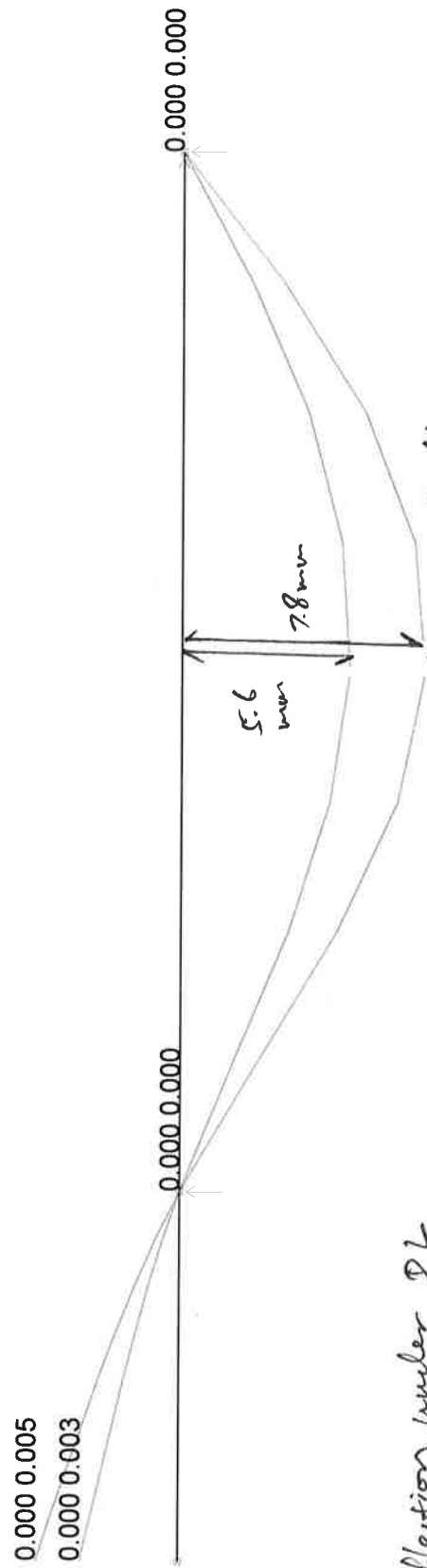
theta: 270 phi: 0

1.2DL+1.5LL SHEAR

Shear Force, Fy

Load Cases:

- 1 PDL
— 4 C DL+0.7LL



Deflection under DL

In spec $\Delta \text{ limit} = \frac{7000}{360} = 19.4 \mu\text{m} > 5.6 \mu\text{m o.f.}$

$$\Delta_{\text{limit}} = \frac{2500}{2500} = 10 \text{ mm} \quad \Delta_{\text{mean}} = 0.1 \text{ k}$$

$$\Delta_{\text{limit}} = \frac{2500}{250} = 10 \text{ min} > 5 \text{ min OK}$$

X
↑
Z

theta: 270 phi: 0

Displaced Shape

DEFLECTION DL, DL+0.7LL

Microstran [V8.11r]

K:\2017\10\17\10168\Structural Drawings and Calcs\Design Calculations and Details\ROOF BEAM\IR3

1710168
, SC45

TMK Consulting Engineers
Job: R3
R.R3
1710168

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4 Sep 2018
9:21 AM

INPUT/ANALYSIS REPORT

Job: R3
Title: R.R3
1710168
Type: Plane frame
Date: 4 Sep 2018
Time: 09:21 AM

Nodes	3
Members	2
Spring supports	0
Sections	1
Materials	1
Primary load cases	2
Combination load cases	2

Analysis: Linear elastic

LOAD CASES

Case	Type	Analysis	Title
1	P	L	DL
2	P	L	LL
3	C	L	1.2DL+1.5LL
4	C	L	DL+0.7LL

Analysis Types:

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

NODE COORDINATES

Node	X m	Y m	Z m	Restraint
101	0.000	0.000	0.000	000000
102	2.500	0.000	0.000	010000
103	9.500	0.000	0.000	111000

MEMBER DEFINITION

Member	A	B	C	Prop	Matl	Rel-A	Rel-B	Length m
101	101	102	Y	10	1	000000	000000	2.500
102	102	103	Y	10	1	000000	000000	7.000

LIBRARY SECTIONS

Section	Library	Name	Axis	Comment
10	asw	230PFC	Y	All_spans

SECTION PROPERTIES

Section	Ax m2	Ay m2	Az m2	J m4	Iy m4	Iz m4	fact
10	3.200E-03	0.000E+00	0.000E+00	1.080E-07	1.760E-06	2.680E-05	

MATERIAL PROPERTIES

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

TABLE OF QUANTITIES

Section	Name	Length m	Mass tonne	Comment
10	230PFC	9.500	0.239	All_spans
		9.500	0.239	

CONDITION NUMBER

Maximum condition number: 4.000E+00 at node: 101 DOFN: 6

CASE 1: DL

Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-1.400			
102	UNIF	FY	GL		-1.400			

TMK Consulting Engineers

Job: R3

R.R3

1710168

Page 2 of 5

4 Sep 2018

9:21 AM

Sum of Applied Loads (Global Axes):

FX: 0.000 FY: -13.300 FZ: 0.000

Moments about the global origin:

MX: 0.000 MY: 0.000 MZ: -63.175

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging

Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 1: DL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.50	0.00	0.70	0.00	0.00	0.00	-0.17
3	1.00	0.00	1.40	0.00	0.00	0.00	-0.70
4	1.50	0.00	2.10	0.00	0.00	0.00	-1.57
5	2.00	0.00	2.80	0.00	0.00	0.00	-2.80
6	2.50	0.00	3.50	0.00	0.00	0.00	-4.38

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0033	0.0000	0.0000	0.0000
2	0.5	0.0000	0.0027	0.0000	0.0001	0.0000
3	1.0	0.0000	0.0021	0.0000	0.0002	0.0000
4	1.5	0.0000	0.0015	0.0000	0.0002	0.0000
5	2.0	0.0000	0.0008	0.0000	0.0002	0.0000
6	2.5	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-5.53	0.00	0.00	0.00	-4.38
2	1.40	0.00	-3.57	0.00	0.00	0.00	1.99
3	2.80	0.00	-1.61	0.00	0.00	0.00	5.61
4	4.20	0.00	0.35	0.00	0.00	0.00	6.48
5	5.60	0.00	2.31	0.00	0.00	0.00	4.61
6	7.00	0.00	4.27	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.4	0.0000	-0.0029	0.0000	-0.0029	0.0000
3	2.8	0.0000	-0.0052	0.0000	-0.0052	0.0000
4	4.2	0.0000	-0.0055	0.0000	-0.0055	0.0000
5	5.6	0.0000	-0.0036	0.0000	-0.0036	0.0000
6	7.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 1: DL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	9.02	0.00	0.00	0.00	0.00
103	0.00	4.28	0.00	0.00	0.00	0.00

SUM: 0.00 13.30 0.00 (all nodes)

Max. residual: -2.665E-15 at DOFN: 2

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

CASE 2: LL

Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-0.900			
102	UNIF	FY	GL		-0.900			

Sum of Applied Loads (Global Axes):

FX: 0.000 FY: -8.550 FZ: 0.000

Moments about the global origin:

MX: 0.000 MY: 0.000 MZ: -40.612

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 2: LL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.50	0.00	0.45	0.00	0.00	0.00	-0.11
3	1.00	0.00	0.90	0.00	0.00	0.00	-0.45
4	1.50	0.00	1.35	0.00	0.00	0.00	-1.01
5	2.00	0.00	1.80	0.00	0.00	0.00	-1.80
6	2.50	0.00	2.25	0.00	0.00	0.00	-2.81

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0021	0.0000	0.0000	0.0000
2	0.5	0.0000	0.0017	0.0000	0.0001	0.0000
3	1.0	0.0000	0.0014	0.0000	0.0001	0.0000
4	1.5	0.0000	0.0010	0.0000	0.0001	0.0000
5	2.0	0.0000	0.0005	0.0000	0.0001	0.0000
6	2.5	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-3.55	0.00	0.00	0.00	-2.81
2	1.40	0.00	-2.29	0.00	0.00	0.00	1.28
3	2.80	0.00	-1.03	0.00	0.00	0.00	3.60
4	4.20	0.00	0.23	0.00	0.00	0.00	4.17
5	5.60	0.00	1.49	0.00	0.00	0.00	2.97
6	7.00	0.00	2.75	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.4	0.0000	-0.0019	0.0000	-0.0019	0.0000
3	2.8	0.0000	-0.0034	0.0000	-0.0034	0.0000
4	4.2	0.0000	-0.0036	0.0000	-0.0036	0.0000
5	5.6	0.0000	-0.0023	0.0000	-0.0023	0.0000
6	7.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 2: LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	5.80	0.00	0.00	0.00	0.00
103	0.00	2.75	0.00	0.00	0.00	0.00
SUM:	0.00	8.55	0.00 (all nodes)			

Max. residual: -8.882E-16 at DOFN: 6

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

CASE 3: 1.2DL+1.5LL

Load Combinations

Case	Factor
1	1.200 DL
2	1.500 LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-28.785	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-136.729

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 3: 1.2DL+1.5LL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.50	0.00	1.51	0.00	0.00	0.00	-0.38
3	1.00	0.00	3.03	0.00	0.00	0.00	-1.51
4	1.50	0.00	4.55	0.00	0.00	0.00	-3.41
5	2.00	0.00	6.06	0.00	0.00	0.00	-6.06
6	2.50	0.00	7.57	0.00	0.00	0.00	-9.47

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0071	0.0000	0.0000	0.0000
2	0.5	0.0000	0.0059	0.0000	0.0002	0.0000
3	1.0	0.0000	0.0046	0.0000	0.0003	0.0000
4	1.5	0.0000	0.0033	0.0000	0.0004	0.0000
5	2.0	0.0000	0.0018	0.0000	0.0004	0.0000
6	2.5	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-11.96	0.00	0.00	0.00	-9.47
2	1.40	0.00	-7.72	0.00	0.00	0.00	4.30
3	2.80	0.00	-3.47	0.00	0.00	0.00	12.14
4	4.20	0.00	0.77	0.00	0.00	0.00	14.03
5	5.60	0.00	5.01	0.00	0.00	0.00	9.98
6	7.00	0.00	9.25	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.4	0.0000	-0.0063	0.0000	-0.0063	0.0000
3	2.8	0.0000	-0.0113	0.0000	-0.0113	0.0000
4	4.2	0.0000	-0.0120	0.0000	-0.0120	0.0000
5	5.6	0.0000	-0.0077	0.0000	-0.0077	0.0000
6	7.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 3: 1.2DL+1.5LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	19.53	0.00	0.00	0.00	0.00
103	0.00	9.25	0.00	0.00	0.00	0.00

SUM: 0.00 28.78 0.00 (all nodes)

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

CASE 4: DL+0.7LL

Load Combinations

Case	Factor
1	1.000 DL
2	0.700 LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-19.285	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-91.604

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension	Shear - End A sagging
Torque - Right-hand twist	Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 4: DL+0.7LL

MEMBER 101: Nodes 101 - 102 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.50	0.00	1.01	0.00	0.00	0.00	-0.25
3	1.00	0.00	2.03	0.00	0.00	0.00	-1.01

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4	1.50	0.00	3.04	0.00	0.00	0.00	-2.28
5	2.00	0.00	4.06	0.00	0.00	0.00	-4.06
6	2.50	0.00	5.07	0.00	0.00	0.00	-6.34

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0048	0.0000	0.0000	0.0000
2	0.5	0.0000	0.0039	0.0000	0.0001	0.0000
3	1.0	0.0000	0.0031	0.0000	0.0002	0.0000
4	1.5	0.0000	0.0022	0.0000	0.0003	0.0000
5	2.0	0.0000	0.0012	0.0000	0.0002	0.0000
6	2.5	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 230PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-8.01	0.00	0.00	0.00	-6.34
2	1.40	0.00	-5.17	0.00	0.00	0.00	2.88
3	2.80	0.00	-2.33	0.00	0.00	0.00	8.13
4	4.20	0.00	0.51	0.00	0.00	0.00	9.40
5	5.60	0.00	3.36	0.00	0.00	0.00	6.69
6	7.00	0.00	6.20	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	1.4	0.0000	-0.0042	0.0000	-0.0042	0.0000
3	2.8	0.0000	-0.0076	0.0000	-0.0076	0.0000
4	4.2	0.0000	-0.0080	0.0000	-0.0080	0.0000
5	5.6	0.0000	-0.0052	0.0000	-0.0052	0.0000
6	7.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

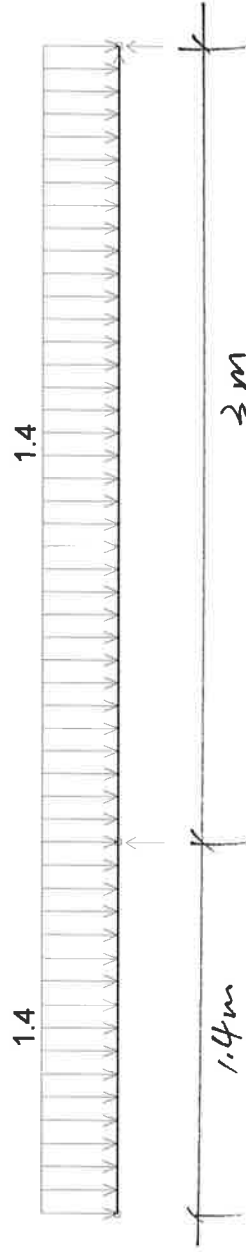
CASE 4: DL+0.7LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	13.09	0.00	0.00	0.00	0.00
103	0.00	6.20	0.00	0.00	0.00	0.00
SUM:	0.00	19.28	0.00 (all nodes)			

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

Load Cases:
— 1 P DL

R.R4 & R.R6 150 PFC



$$DL: \text{Roof} = 0.4 \text{ kPa} \times 3.5 \text{ m} = 1.4 \text{ kN/m}$$

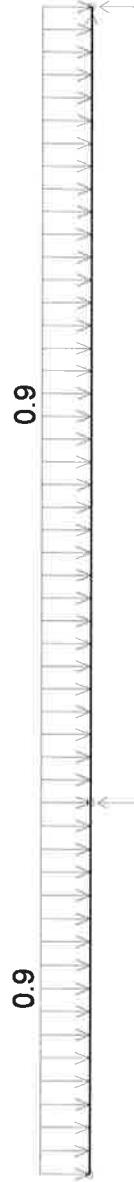
$$LL: \text{Roof} = 0.25 \text{ kPa} \times 3.5 \text{ m} = 0.9 \text{ kN/m}$$

Y
Z
X

theta: 270 phi: 0

DL

Load Cases:
—— 2 P LL

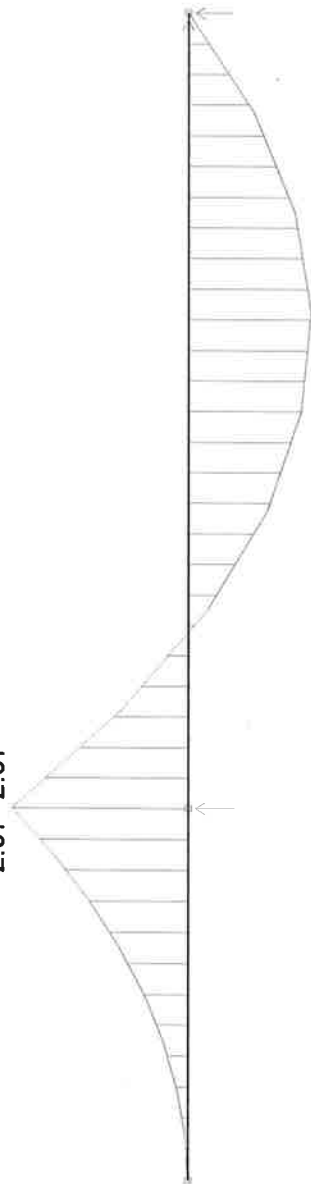


Y
Z
X
theta: 270 phi: 0

LL

Load Cases:
 — 3 C 1.2DL+1.5LL

2.97 2.97



2.08

150 PFC Cantilever $P_e = 1.4 \times 2 = 2.8$ m $\phi M_b = 2.6$ kNm > 2.97 kNm o.k

Y
 Z
 X

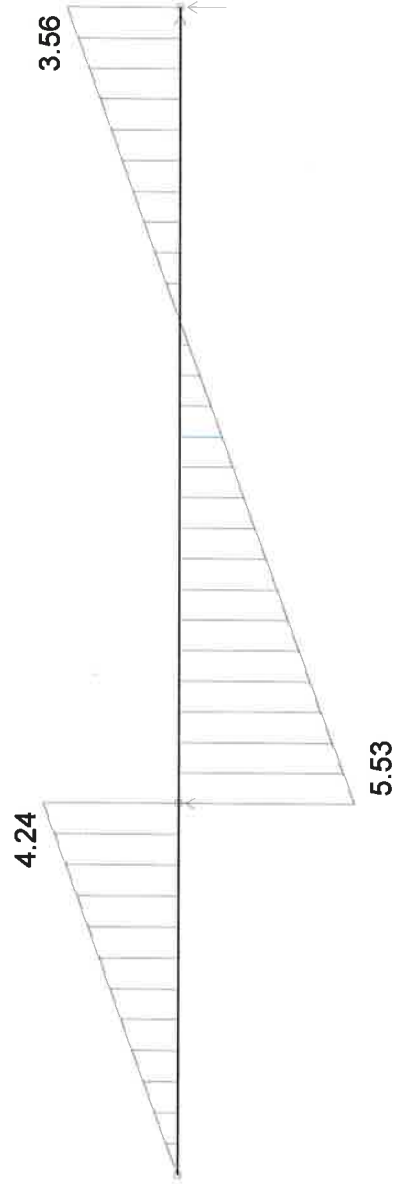
theta: 270 phi: 0

1.2DL+1.5LL MOMENT

Bending Moment, Mz

Load Cases:

3 C 1.2DL+1.5LL



Y
Z
X

theta: 270 phi: 0

1.2DL+1.5LL SHEAR

Shear Force, Fy

Load Cases:

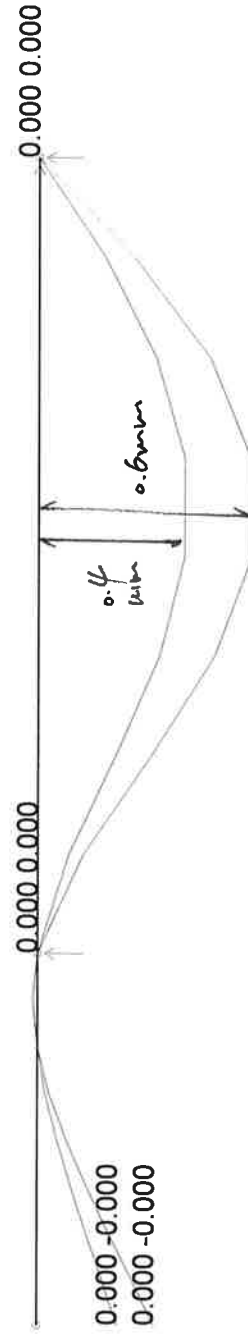
— 1 P DL

— 4 C DL+0.7LL

Y
Z X

theta: 270 phi: 0

Microtran [V8.11r]



DEFLECTION UNDER DL, DL+0.7LL

Displaced Shape

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SC55

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R.R6

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INPUT/ANALYSIS REPORT

Job: R6

Title: R.R6
1710168

Type: Plane frame

Date: 4 Sep 2018

Time: 09:34 AM

Nodes 3
 Members 2
 Spring supports 0
 Sections 1
 Materials 1
 Primary load cases 2
 Combination load cases 2

Analysis: Linear elastic

LOAD CASES

Case	Type	Analysis	Title
1	P	L	DL
2	P	L	LL
3	C	L	1.2DL+1.5LL
4	C	L	DL+0.7LL

Analysis Types:

S - Skipped (not analysed)

L - Linear

N - Non-linear

NODE COORDINATES

Node	X m	Y m	Z m	Restraint
101	0.000	0.000	0.000	000000
102	1.400	0.000	0.000	010000
103	4.400	0.000	0.000	111000

MEMBER DEFINITION

Member	A	B	C	Prop	Matl	Rel-A	Rel-B	Length m
101	101	102	Y	10	1	000000	000000	1.400
102	102	103	Y	10	1	000000	000000	3.000

LIBRARY SECTIONS

Section	Library	Name	Axis	Comment
10	asw	150PFC	Y	All_spans

SECTION PROPERTIES

Section	Ax m2	Ay m2	Az m2	J m4	Iy m4	Iz m4	fact
10	2.250E-03	0.000E+00	0.000E+00	5.490E-08	1.290E-06	8.340E-06	

MATERIAL PROPERTIES

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

TABLE OF QUANTITIES

MATERIAL 1

Section	Name	Length m	Mass tonne	Comment
10	150PFC	4.400	0.078	All_spans
		4.400	0.078	

CONDITION NUMBER

Maximum condition number: 4.000E+00 at node: 101 DOFN: 6

CASE 1: DL

Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-1.400			
102	UNIF	FY	GL		-1.400			

Sum of Applied Loads (Global Axes):

FX: 0.000 FY: -6.160 FZ: 0.000
Moments about the global origin:
MX: 0.000 MY: 0.000 MZ: -13.552

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 1: DL

MEMBER 101: Nodes 101 - 102 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.28	0.00	0.39	0.00	0.00	0.00	-0.05
3	0.56	0.00	0.78	0.00	0.00	0.00	-0.22
4	0.84	0.00	1.18	0.00	0.00	0.00	-0.49
5	1.12	0.00	1.57	0.00	0.00	0.00	-0.88
6	1.40	0.00	1.96	0.00	0.00	0.00	-1.37

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	-0.0002	0.0000	0.0000	0.0000
2	0.3	0.0000	-0.0002	0.0000	0.0000	0.0000
3	0.6	0.0000	-0.0001	0.0000	0.0001	0.0000
4	0.8	0.0000	0.0000	0.0000	0.0001	0.0000
5	1.1	0.0000	0.0000	0.0000	0.0001	0.0000
6	1.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-2.56	0.00	0.00	0.00	-1.37
2	0.60	0.00	-1.72	0.00	0.00	0.00	-0.09
3	1.20	0.00	-0.88	0.00	0.00	0.00	0.69
4	1.80	0.00	-0.04	0.00	0.00	0.00	0.96
5	2.40	0.00	0.80	0.00	0.00	0.00	0.73
6	3.00	0.00	1.64	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.6	0.0000	-0.0002	0.0000	-0.0002	0.0000
3	1.2	0.0000	-0.0004	0.0000	-0.0004	0.0000
4	1.8	0.0000	-0.0004	0.0000	-0.0004	0.0000
5	2.4	0.0000	-0.0003	0.0000	-0.0003	0.0000
6	3.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 1: DL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	4.52	0.00	0.00	0.00	0.00
103	0.00	1.64	0.00	0.00	0.00	0.00
SUM:	0.00	6.16	0.00	(all nodes)		

Max. residual: -1.110E-16 at DOFN: 2

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

CASE 2: LL

Member Loads

Member	Form	T	A	S	F1	X1	F2	X2
101	UNIF	FY	GL		-0.900			
102	UNIF	FY	GL		-0.900			

Sum of Applied Loads (Global Axes):

FX: 0.000 FY: -3.960 FZ: 0.000
Moments about the global origin:
MX: 0.000 MY: 0.000 MZ: -8.712

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
 Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 2: LL

MEMBER 101: Nodes 101 - 102 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.28	0.00	0.25	0.00	0.00	0.00	-0.04
3	0.56	0.00	0.50	0.00	0.00	0.00	-0.14
4	0.84	0.00	0.76	0.00	0.00	0.00	-0.32
5	1.12	0.00	1.01	0.00	0.00	0.00	-0.56
6	1.40	0.00	1.26	0.00	0.00	0.00	-0.88

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	-0.0001	0.0000	0.0000	0.0000
2	0.3	0.0000	-0.0001	0.0000	0.0000	0.0000
3	0.6	0.0000	-0.0001	0.0000	0.0000	0.0000
4	0.8	0.0000	0.0000	0.0000	0.0000	0.0000
5	1.1	0.0000	0.0000	0.0000	0.0000	0.0000
6	1.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-1.64	0.00	0.00	0.00	-0.88
2	0.60	0.00	-1.10	0.00	0.00	0.00	-0.06
3	1.20	0.00	-0.56	0.00	0.00	0.00	0.44
4	1.80	0.00	-0.02	0.00	0.00	0.00	0.62
5	2.40	0.00	0.52	0.00	0.00	0.00	0.47
6	3.00	0.00	1.06	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.6	0.0000	-0.0001	0.0000	-0.0001	0.0000
3	1.2	0.0000	-0.0002	0.0000	-0.0002	0.0000
4	1.8	0.0000	-0.0003	0.0000	-0.0003	0.0000
5	2.4	0.0000	-0.0002	0.0000	-0.0002	0.0000
6	3.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 2: LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	2.90	0.00	0.00	0.00	0.00
103	0.00	1.06	0.00	0.00	0.00	0.00
SUM:	0.00	3.96	0.00	(all nodes)		

Max. residual: -1.110E-16 at DOFN: 6

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

CASE 3: 1.2DL+1.5LL

Load Combinations

Case	Factor
1	1.200 DL
2	1.500 LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-13.332	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-29.330

SIGN CONVENTION

Positive Forces (Member Axes):

Axial - Tension Shear - End A sagging
 Torque - Right-hand twist Moment - Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 3: 1.2DL+1.5LL

MEMBER 101: Nodes 101 - 102 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.28	0.00	0.85	0.00	0.00	0.00	-0.12
3	0.56	0.00	1.70	0.00	0.00	0.00	-0.48
4	0.84	0.00	2.55	0.00	0.00	0.00	-1.07
5	1.12	0.00	3.39	0.00	0.00	0.00	-1.90
6	1.40	0.00	4.24	0.00	0.00	0.00	-2.97

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	-0.0005	0.0000	0.0000	0.0000
2	0.3	0.0000	-0.0003	0.0000	0.0001	0.0000
3	0.6	0.0000	-0.0002	0.0000	0.0001	0.0000
4	0.8	0.0000	-0.0001	0.0000	0.0001	0.0000
5	1.1	0.0000	0.0000	0.0000	0.0001	0.0000
6	1.4	0.0000	0.0000	0.0000	0.0000	0.0000

MEMBER 102: Nodes 102 - 103 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-5.53	0.00	0.00	0.00	-2.97
2	0.60	0.00	-3.72	0.00	0.00	0.00	-0.19
3	1.20	0.00	-1.90	0.00	0.00	0.00	1.49
4	1.80	0.00	-0.08	0.00	0.00	0.00	2.08
5	2.40	0.00	1.74	0.00	0.00	0.00	1.59
6	3.00	0.00	3.56	0.00	0.00	0.00	0.00

Point	Offset	X-glob	Y-glob	Z-glob	y-local	z-local
	m	m	m	m	m	m
1	0.0	0.0000	0.0000	0.0000	0.0000	0.0000
2	0.6	0.0000	-0.0004	0.0000	-0.0004	0.0000
3	1.2	0.0000	-0.0008	0.0000	-0.0008	0.0000
4	1.8	0.0000	-0.0009	0.0000	-0.0009	0.0000
5	2.4	0.0000	-0.0006	0.0000	-0.0006	0.0000
6	3.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 3: 1.2DL+1.5LL

Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	9.78	0.00	0.00	0.00	0.00
103	0.00	3.56	0.00	0.00	0.00	0.00
SUM:	0.00	13.33	0.00 (all nodes)			

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

CASE 4: DL+0.7LL

Load Combinations

Case	Factor	
1	1.000	DL
2	0.700	LL

Sum of Applied Loads (Global Axes):

FX:	0.000	FY:	-8.932	FZ:	0.000
Moments about the global origin:					
MX:	0.000	MY:	0.000	MZ:	-19.650

SIGN CONVENTION

Positive Forces (Member Axes):

Axial	- Tension	Shear	- End A sagging
Torque	- Right-hand twist	Moment	- Sagging

Deflections:

Global deflections are absolute.

Local deflections are relative to chord joining displaced end nodes.

MEMBER FORCES AND DEFLECTIONS

CASE 4: DL+0.7LL

MEMBER 101: Nodes 101 - 102 Section 10: 150PFC Y

Point	Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	m	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0.28	0.00	0.57	0.00	0.00	0.00	-0.08
3	0.56	0.00	1.14	0.00	0.00	0.00	-0.32

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4	0.84	0.00	1.71	0.00	0.00	0.00	-0.72
5	1.12	0.00	2.27	0.00	0.00	0.00	-1.27
6	1.40	0.00	2.84	0.00	0.00	0.00	-1.99
Point Offset	X-glob	Y-glob	Z-glob	y-local	z-local		
	m	m	m	m	m		
1	0.0	0.0000	-0.0003	0.0000	0.0000	0.0000	
2	0.3	0.0000	-0.0002	0.0000	0.0000	0.0000	
3	0.6	0.0000	-0.0001	0.0000	0.0001	0.0000	
4	0.8	0.0000	0.0000	0.0000	0.0001	0.0000	
5	1.1	0.0000	0.0000	0.0000	0.0001	0.0000	
6	1.4	0.0000	0.0000	0.0000	0.0000	0.0000	

MEMBER 102: Nodes 102 - 103 Section 10: 150PFC Y

Point Offset	Axial	Shear-y	Shear-z	Torque	Moment-y	Moment-z
	kN	kN	kN	kNm	kNm	kNm
1	0.00	0.00	-3.71	0.00	0.00	-1.99
2	0.60	0.00	-2.49	0.00	0.00	-0.13
3	1.20	0.00	-1.27	0.00	0.00	1.00
4	1.80	0.00	-0.05	0.00	0.00	1.40
5	2.40	0.00	1.16	0.00	0.00	1.06
6	3.00	0.00	2.38	0.00	0.00	0.00
Point Offset	X-glob	Y-glob	Z-glob	y-local	z-local	
	m	m	m	m	m	
1	0.0	0.0000	0.0000	0.0000	0.0000	
2	0.6	0.0000	-0.0002	0.0000	-0.0002	0.0000
3	1.2	0.0000	-0.0005	0.0000	-0.0005	0.0000
4	1.8	0.0000	-0.0006	0.0000	-0.0006	0.0000
5	2.4	0.0000	-0.0004	0.0000	-0.0004	0.0000
6	3.0	0.0000	0.0000	0.0000	0.0000	0.0000

SUPPORT REACTIONS

CASE 4: DL+0.7LL

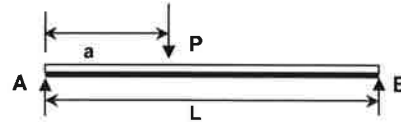
Node	Force-X	Force-Y	Force-Z	Moment-X	Moment-Y	Moment-Z
	kN	kN	kN	kNm	kNm	kNm
102	0.00	6.55	0.00	0.00	0.00	0.00
103	0.00	2.38	0.00	0.00	0.00	0.00
SUM:	0.00	8.93	0.00	(all nodes)		

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

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member R.R5 Design span, L = 7400 mm
Effective length of lateral restraint, l_e = 1200 mm
Distance, a, from LH Support to Point Load = 4000 mm



Load Type	UDL	Width	
1a. Uniform DL	0.40 kPa	x 1.80 m =	0.72 kN/m
wall load			0.75 kN/m
Self Weight			0.25 kN/m
		Total	1.72 kN/m
1b. Point DL R.CB1		Total	3.10 kN
2a. Uniform LL	0.25 kPa	x 1.80 m =	0.45 kN/m
		Total	0.00 kN/m
2b. Point LL R.CB1		Total	1.30 kN
3a. Uniform WL (see note) Other WL	-1.13 kPa	x 1.80 m =	-2.03 kN/m
		Total	0.00 kN/m
3b. Point WL wind		Total	-5.60 kN

STRENGTH DESIGN Combination of Load Types **

- ☒ DL + LL
- ☐ DL + LL + WL
- ☐ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	20
2. Dead Load + 0.7 * Live Load	250	28
3. Wind Load Only	200	20

Bending Moments and Stiffness (Refer to Combination of Load Types for Member Strength Calculations)		
$W^* = 2.74$ kN/m	$P^* = 5.67$ kN	$M^* = 29.04$ kNm
$R_A^* = 12.74$ kN	$R_B^* = 13.20$ kN	$I_{req,DL} = 23.3E+6$ mm ⁴
$W_{s,DL+0.7LL} = 2.03$ kN/m	$P_{s,DL+0.7LL} = 4.01$ kN	$I_{req,DL+0.7LL} = 20.2E+6$ mm ⁴
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 000.0E+0$ mm ⁴

← Governs

Trial Section

Trial beam size :	200 UB 25.4
Depth of section, d =	203 mm
Flange width, b_f =	133 mm
Flange thickness, t_f =	7.8 mm
Web thickness, t_w =	5.8 mm
Section area, A_g =	3230 mm ²
$I_x = 23.6E+6$ mm ⁴	∴ OK
$Z_x = 232E+3$ mm ³	
$S_x = 260E+3$ mm ³	
$r_x = 85.4$ mm	
$I_y = 3.06E+6$ mm ⁴	
$Z_y = 46.1E+3$ mm ³	
$S_y = 70.9E+3$ mm ³	
$r_y = 30.8$ mm	
$J = 62.7E+3$ mm ⁴	
$I_w = 29.2E+9$ mm ⁶	
Flange, f_y =	320 MPa
Web, f_y =	320 MPa
$k_f = 1.00$	
$Z_{ex} = 259E+3$ mm ³	
$Z_{ey} = 68.8E+3$ mm ³	
Compactness =	N

Material Properties

E =	200E+3 MPa
G =	76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$

Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{ex})$

$M_{sx} =$	82.9 kNm (Clause 5.2.1)
$\phi M_{sx} =$	74.6 kNm > 29.04 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$

Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m =$	0.00	AS 4100 Table 5.6.1
$\alpha_m =$	1.13	
$k_t =$	1.00	Table 5.6.3(1)
$k_l =$	1.40	Table 5.6.3(2)
$k_r =$	1.00	Table 5.6.3(3)
$l_e =$	1680 mm	
$M_o =$	232.4 kNm (Eq. 6.5.1.1(3))	
$\alpha_s =$	0.85	Eq. 5.6.1.1(2)
$M_{bx} =$	79.3 kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} =$	71.4 kNm > 29.04 kNm ∴ OK	

ADOPT

200 UB 25.4

Restrain top flange laterally at 1200 mm centres (maximum)

ALTERNATIVELY ADOPT 230 PFC: $I_x = 26.8 \times 10^6$ mm⁴ & $\phi M_{bx} = 59.1$ kNm



Ref.: 1710168

Date: 07-Oct-18

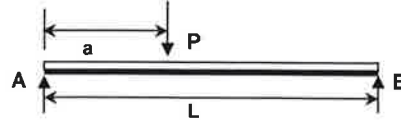
Design: RR

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STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member R.R5 Design span, $L = 7400$ mm
Effective length of lateral restraint, $l_e = 4000$ mm
Distance, a , from LH Support to Point Load = 4000 mm



Load Type	UDL	Width	
1a. Uniform DL	0.40 kPa	x 1.80 m =	0.72 kN/m
wall load			0.75 kN/m
Self Weight			0.25 kN/m
		Total	1.72 kN/m
1b. Point DL R CB1		Total	3.10 kN
2a. Uniform LL	0.25 kPa	x 1.80 m =	0.45 kN/m
		Total	0.00 kN/m
		Total	0.45 kN/m
2b. Point LL R CB1		Total	1.30 kN
3a. Uniform WL (see note) Other WL	-1.13 kPa	x 1.80 m =	-2.03 kN/m
		Total	0.00 kN/m
		Total	-2.03 kN/m
3b. Point WL wind		Total	-5.60 kN

**STRENGTH DESIGN
Combination of
Load Types ****

- ☐ DL + LL
☐ DL + LL + WL
☒ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	20
2. Dead Load + 0.7 * Live Load	250	28
3. Wind Load Only	200	20

Bending Moments and Stiffness

(Refer to Combination of Load Types for Member Strength Calculations)

$W^* = -0.49$ kN/m	$P^* = -2.81$ kN	$M^* = -8.47$ kNm	
$R_A^* = -3.09$ kN	$R_B^* = -3.32$ kN	$I_{req,DL} = 23.3E+6$ mm ⁴	← Governs
$W_{s,DL+0.7LL} = 2.03$ kN/m	$P_{s,DL+0.7LL} = 4.01$ kN	$I_{req,DL+0.7LL} = 20.2E+6$ mm ⁴	
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 000.0E+0$ mm ⁴	

Trial Section

Trial beam size : 200 UB 25.4

Depth of section, $d =$	203 mm	
Flange width, $b_f =$	133 mm	
Flange thickness, $t_f =$	7.8 mm	
Web thickness, $t_w =$	5.8 mm	
Section area, $A_g =$	3230 mm ²	
$I_x =$	23.6E+6 mm ⁴	∴ OK
$Z_x =$	232E+3 mm ³	
$S_x =$	260E+3 mm ³	
$r_x =$	85.4 mm	
$I_y =$	3.06E+6 mm ⁴	
$Z_y =$	46.1E+3 mm ³	
$S_y =$	70.9E+3 mm ³	
$r_y =$	30.8 mm	
$J =$	62.7E+3 mm ⁴	
$I_w =$	29.2E+9 mm ⁶	
Flange, $f_y =$	320 MPa	
Web, $f_y =$	320 MPa	
$k_f =$	1.00	
$Z_{ex} =$	259E+3 mm ³	
$Z_{ey} =$	68.8E+3 mm ³	
Compactness =	N	

Material Properties

$E =$	200E+3 MPa
$G =$	76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$ Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{ex})$

$M_{sx} =$	82.9 kNm (Clause 5.2.1)
$\phi M_{sx} =$	74.6 kNm > 8.47 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$ Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m =$	0.00	AS 4100 Table 5.6.1
$\alpha_m =$	1.13	
$k_t =$	1.00	Table 5.6.3(1)
$k_l =$	1.00	Table 5.6.3(2)
$k_r =$	1.00	Table 5.6.3(3)
$l_e =$	4000 mm	
$M_o =$	56.4 kNm (Eq. 6.5.1.1(3))	
$\alpha_s =$	0.48	Eq. 5.6.1.1(2)
$M_{bx} =$	45.1 kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} =$	40.6 kNm > 8.47 kNm ∴ OK	

ADOPT

200 UB 25.4

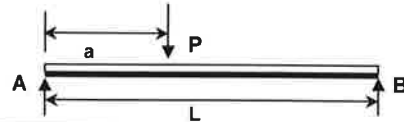
Restrain bottom flange laterally at 4000 mm centres (maximum)

→ A1F 230 PFC
8 M = 31.9 kNm > req.
OK

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member R.R6 Design span, $L = 4700$ mm
Effective length of lateral restraint, $l_e = 1200$ mm
Distance, a , from LH Support to Point Load = 0 mm



Load Type	UDL	Width	
1a. Uniform DL	0.40 kPa	x 3.60 m =	1.44 kN/m
wall load			0.75 kN/m
Self Weight			0.14 kN/m
		Total	2.33 kN/m
1b. Point DL	Description	Total	0.00 kN
2a. Uniform LL	0.25 kPa	x 3.60 m =	0.90 kN/m
		Total	0.90 kN/m
2b. Point LL	Description	Total	0.00 kN
3a. Uniform WL (see note)	-1.13 kPa	x 3.60 m =	-4.07 kN/m
Other WL		Total	-4.07 kN/m
3b. Point WL		Total	0.00 kN

STRENGTH DESIGN Combination of Load Types **

- ☒ DL + LL
☐ DL + LL + WL
☐ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness

(Refer to Combination of Load Types for Member Strength Calculations)

$W^* = 4.14$ kN/m	$P^* = 0.00$ kN	$M^* = 11.44$ kNm
$R_A^* = 9.74$ kN	$R_B^* = 9.74$ kN	$I_{req,DL} = 6.2E+6$ mm ⁴
$W_{s,DL+0.7LL} = 2.96$ kN/m	$P_{s,DL+0.7LL} = 0.00$ kN	$I_{req,DL+0.7LL} = 5.2E+6$ mm ⁴
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 0.00E+0$ mm ⁴

← Governs

Trial Section

Trial beam size : 150 UB 14

Depth of section, $d = 150$ mm	
Flange width, $b_f = 75$ mm	
Flange thickness, $t_f = 7$ mm	
Web thickness, $t_w = 5$ mm	
Section area, $A_g = 1780$ mm ²	
$I_x = 6.7E+6$ mm ⁴	∴ OK
$Z_x = 89E+3$ mm ³	
$S_x = 102E+3$ mm ³	
$r_x = 61.1$ mm	
$I_y = 495.00E+3$ mm ⁴	
$Z_y = 13.2E+3$ mm ³	
$S_y = 20.8E+3$ mm ³	
$r_y = 16.6$ mm	
$J = 28.1E+3$ mm ⁴	
$I_w = 2.5E+9$ mm ⁶	
Flange, $f_y = 320$ MPa	
Web, $f_y = 320$ MPa	
$k_f = 1.00$	
$Z_{ex} = 102E+3$ mm ³	
$Z_{ey} = 19.8E+3$ mm ³	
Compactness = C	

Material Properties

$E = 200E+3$ MPa
$G = 76.9E+3$ MPa

Section Capacity -- $M^* \leq \phi M_{sx}$

Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{sx})$

$M_{sx} = 32.6$ kNm (Clause 5.2.1)
$\phi M_{sx} = 29.4$ kNm > 11.44 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$

Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m = 0.00$	AS 4100 Table 5.6.1
$\alpha_m = 1.13$	
$k_t = 1.00$	Table 5.6.3(1)
$k_l = 1.40$	Table 5.6.3(2)
$k_r = 1.00$	Table 5.6.3(3)
$l_e = 1680$ mm	
$M_o = 36.9$ kNm (Eq. 6.5.1.1(3))	
$\alpha_s = 0.64$	Eq. 5.6.1.1(2)
$M_{bx} = 23.5$ kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} = 21.1$ kNm > 11.44 kNm ∴ OK	

ADOPT

150 UB 14

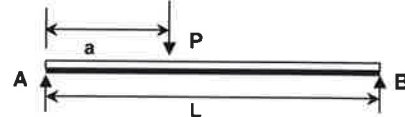
Restrain top flange laterally at 1200 mm centres (maximum)

→ Alt. Use 150 PF2

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member R.R6 Design span, $L = 4700$ mm
Effective length of lateral restraint, $l_e = 4700$ mm
Distance, a , from LH Support to Point Load = 0 mm



Load Type	UDL	Width	
1a. Uniform DL	0.40 kPa	x 3.60 m =	1.44 kN/m
wall load			0.75 kN/m
Self Weight			0.14 kN/m
		Total	2.33 kN/m
1b. Point DL	Description	Total	0.00 kN
2a. Uniform LL	0.25 kPa	x 3.60 m =	0.90 kN/m
		Total	0.00 kN/m
2b. Point LL	Description	Total	0.00 kN
3a. Uniform WL (see note)	-1.13 kPa	x 3.60 m =	-4.07 kN/m
Other WL		Total	0.00 kN/m
3b. Point WL		Total	0.00 kN

STRENGTH DESIGN Combination of Load Types **

- ☐ DL + LL
☐ DL + LL + WL
☒ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness

(Refer to Combination of Load Types for Member Strength Calculations)

$W^* = -1.97$ kN/m	$P^* = 0.00$ kN	$M^* = -5.45$ kNm
$R_A^* = -4.64$ kN	$R_B^* = -4.64$ kN	$I_{req,DL} = 6.2E+6$ mm ⁴
$W_{s,DL+0.7LL} = 2.96$ kN/m	$P_{s,DL+0.7LL} = 0.00$ kN	$I_{req,DL+0.7LL} = 5.2E+6$ mm ⁴
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 0.00E+0$ mm ⁴

← Governs

Trial Section

Trial beam size :	150 UB 14
Depth of section, $d =$	150 mm
Flange width, $b_f =$	75 mm
Flange thickness, $t_f =$	7 mm
Web thickness, $t_w =$	5 mm
Section area, $A_g =$	1780 mm ²
$I_x =$	6.7E+6 mm ⁴
$Z_x =$	89E+3 mm ³
$S_x =$	102E+3 mm ³
$r_x =$	61.1 mm
$I_y =$	495.00E+3 mm ⁴
$Z_y =$	13.2E+3 mm ³
$S_y =$	20.8E+3 mm ³
$r_y =$	16.6 mm
$J =$	28.1E+3 mm ⁴
$I_w =$	2.5E+9 mm ⁶
Flange, $f_y =$	320 MPa
Web, $f_y =$	320 MPa
$k_f =$	1.00
$Z_{ex} =$	102E+3 mm ³
$Z_{ey} =$	19.8E+3 mm ³
Compactness =	C

∴ OK

Material Properties

$E =$	200E+3 MPa
$G =$	76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$

Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{ex})$

$M_{sx} =$	32.6 kNm (Clause 5.2.1)
$\phi M_{sx} =$	29.4 kNm > 5.45 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$

Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m =$	0.00	AS 4100 Table 5.6.1
$\alpha_m =$	1.13	
$k_t =$	1.00	Table 5.6.3(1)
$k_l =$	1.00	Table 5.6.3(2)
$k_r =$	1.00	Table 5.6.3(3)
$l_e =$	4700 mm	
$M_o =$	10.3 kNm (Eq. 6.5.1.1(3))	
$\alpha_s =$	0.26	Eq. 5.6.1.1(2)
$M_{bx} =$	9.8 kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} =$	8.8 kNm > 5.45 kNm ∴ OK	

ADOPT

150 UB 14

Restrain bottom flange laterally at 4700 mm centres (maximum)

→ Alt. Use 150 PFC

R.Rg

loading width = 0.75

$$d.l. = 0.4 + 0.75 + 0.5$$

$$= 0.8 \text{ kN/m}$$

$$ll = 0.25 + 0.75$$

$$= 0.2 \text{ kN/m}$$

$$wl = 1.13 + 0.75 = 0.85 \text{ kN/m}$$

$$\frac{P}{1.2} \quad 3.2 \quad \frac{1}{2}$$

$$M_{max.} \rightarrow 1.2DL + 0.5LL$$

$$- 1 \text{ kNm}; + 1.2 \text{ kNm}$$

$$\rightarrow 0.9DL + WL$$

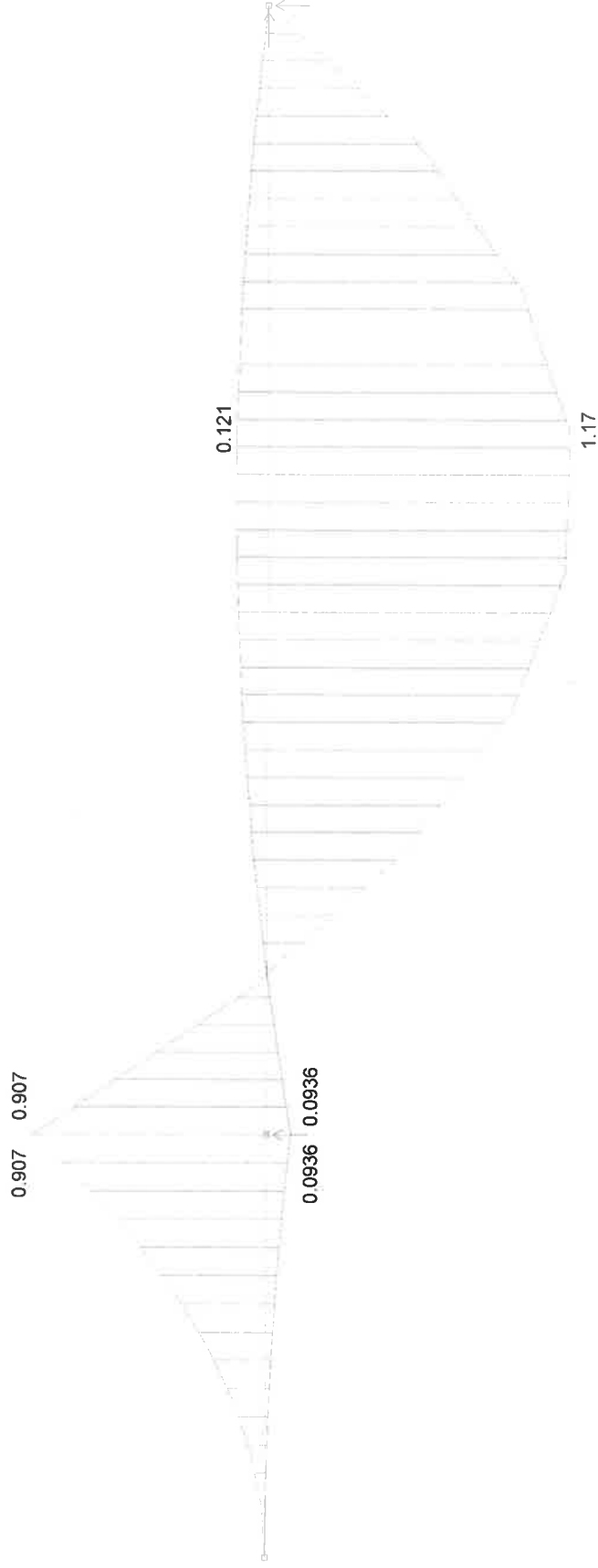
expected d.l. ~~to~~ using 100 PFC = 1mm up.

Adopt 100 PFC.

Category beam 2CA2

Use 100 PFC

Load Cases:
— 3 C 1.2DL+1.5LL
— 6 C .9DL + WL ult



Bending Moment, Mz

Load Cases:
—— 1 P DL



Y
Z
X

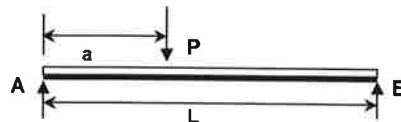
theta: 270 phi: 0

Displaced Shape

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member R.CB1 Design span, L = 5500 mm
Effective length of lateral restraint, l_e = 3000 mm
Distance, a, from LH Support to Point Load = 3000 mm



Load Type	UDL	Width		
1a. Uniform DL	0.40 kPa	x 0.60 m =	0.24 kN/m	
			0.00 kN/m	
Self Weight			0.16 kN/m	
			0.40 kN/m	
1b. Point DL				3.60 kN
2a. Uniform LL	0.25 kPa	x 0.60 m =	0.15 kN/m	
			0.00 kN/m	
			0.15 kN/m	
2b. Point LL				2.25 kN
3a. Uniform WL (see note)	-1.13 kPa	x 0.60 m =	-0.68 kN/m	
Other WL			0.00 kN/m	
			-0.68 kN/m	
3b. Point WL Wind ult (1.13x9)				-10.20 kN

STRENGTH DESIGN Combination of Load Types **

- ☐ DL + LL
☐ DL + LL + WL
☒ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness	(Refer to Combination of Load Types for Member Strength Calculations)		
$W^* = 0.70$ kN/m	$P^* = 7.70$ kN	$M^* = 13.13$ kNm	
$R_A^* = 5.43$ kN	$R_B^* = 6.13$ kN	$I_{req,DL} = 7.1E+6$ mm ⁴	
$W_{s,DL+0.7LL} = 0.50$ kN/m	$P_{s,DL+0.7LL} = 5.18$ kN	$I_{req,DL+0.7LL} = 6.6E+6$ mm ⁴	
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 0.00E+0$ mm ⁴	

← **Governs**

Trial Section

Trial beam size :	180 UB 16.1	
Depth of section, d =	173 mm	
Flange width, b_f =	90 mm	
Flange thickness, t_f =	7 mm	
Web thickness, t_w =	4.5 mm	
Section area, A_g =	2040 mm ²	
$I_x = 10.6E+6$ mm ⁴		∴ OK
$Z_x = 123E+3$ mm ³		
$S_x = 138E+3$ mm ³		
$r_x = 72$ mm		
$I_y = 853.00E+3$ mm ⁴		
$Z_y = 19.0E+3$ mm ³		
$S_y = 29.4E+3$ mm ³		
$r_y = 20.4$ mm		
$J = 31.5E+3$ mm ⁴		
$I_w = 5.9E+9$ mm ⁶		
Flange, f_y =	320 MPa	
Web, f_y =	320 MPa	
$k_f = 1.00$		
$Z_{ex} = 138E+3$ mm ³		
$Z_{ey} = 28.4E+3$ mm ³		
Compactness =	C	

Material Properties

E =	200E+3 MPa
G =	76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$

Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{ex})$

$M_{sx} =$	44.2 kNm (Clause 5.2.1)
$\phi M_{sx} =$	39.7 kNm > 13.13 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$

Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m =$	0.00	AS 4100 Table 5.6.1
$\alpha_m =$	1.13	
$k_1 =$	1.00	Table 5.6.3(1)
$k_2 =$	1.00	Table 5.6.3(2)
$k_r =$	1.00	Table 5.6.3(3)
$l_e =$	3000 mm	
$M_o =$	26.4 kNm (Eq. 6.5.1.1(3))	
$\alpha_s =$	0.44	Eq. 5.6.1.1(2)
$M_{bx} =$	22.0 kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} =$	19.8 kNm > 13.13 kNm ∴ OK	

ADOPT

180 UB 16.1

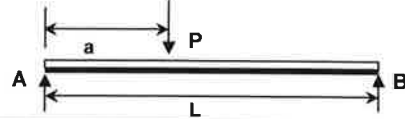
Restrain top flange laterally at 3000 mm centres (maximum)

Alt. Use 180 PFL

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member R.CB1 Design span, $L = 5500$ mm
Effective length of lateral restraint, $I_e = 3000$ mm
Distance, a , from LH Support to Point Load = 3000 mm



Load Type	UDL	Width	
1a. Uniform DL	0.40 kPa	x 0.60 m =	0.24 kN/m
			0.00 kN/m
Self Weight			0.16 kN/m
		Total	0.40 kN/m
1b. Point DL		Total	3.60 kN
2a. Uniform LL	0.25 kPa	x 0.60 m =	0.15 kN/m
			0.00 kN/m
		Total	0.15 kN/m
2b. Point LL		Total	2.25 kN
3a. Uniform WL (see note)	-1.13 kPa	x 0.60 m =	-0.68 kN/m
Other WL			0.00 kN/m
		Total	-0.68 kN/m
3b. Point WL Wind ult (1.13x9)		Total	-10.20 kN

STRENGTH DESIGN Combination of Load Types **

- ☒ DL + LL
☐ DL + LL + WL
☐ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness

(Refer to Combination of Load Types for Member Strength Calculations)

$W^* = 0.70$ kN/m	$P^* = 7.70$ kN	$M^* = 13.13$ kNm	
$R_A^* = 5.43$ kN	$R_B^* = 6.13$ kN	$I_{req,DL} = 7.1E+6$ mm ⁴	← Governs
$W_{s,DL+0.7LL} = 0.50$ kN/m	$P_{s,DL+0.7LL} = 5.18$ kN	$I_{req,DL+0.7LL} = 6.6E+6$ mm ⁴	
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 0.00E+0$ mm ⁴	

Trial Section

Trial beam size :	180 UB 16.1	
Depth of section, $d =$	173 mm	
Flange width, $b_f =$	90 mm	
Flange thickness, $t_f =$	7 mm	
Web thickness, $t_w =$	4.5 mm	
Section area, $A_g =$	2040 mm ²	
$I_x =$	10.6E+6 mm ⁴	∴ OK
$Z_x =$	123E+3 mm ³	
$S_x =$	138E+3 mm ³	
$r_x =$	72 mm	
$I_y =$	853.00E+3 mm ⁴	
$Z_y =$	19.0E+3 mm ³	
$S_y =$	29.4E+3 mm ³	
$r_y =$	20.4 mm	
$J =$	31.5E+3 mm ⁴	
$I_w =$	5.9E+9 mm ⁶	
Flange, $f_y =$	320 MPa	
Web, $f_y =$	320 MPa	
$k_f =$	1.00	
$Z_{ex} =$	138E+3 mm ³	
$Z_{ey} =$	28.4E+3 mm ³	
Compactness =	C	

Material Properties

$E =$	200E+3 MPa
$G =$	76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$

Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{sx})$

$M_{sx} =$	44.2 kNm (Clause 5.2.1)
$\phi M_{sx} =$	39.7 kNm > 13.13 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$

Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m =$	0.00	AS 4100 Table 5.6.1
$\alpha_m =$	1.13	
$k_t =$	1.00	Table 5.6.3(1)
$k_l =$	1.00	Table 5.6.3(2)
$k_r =$	1.00	Table 5.6.3(3)
$I_e =$	3000 mm	
$M_o =$	26.4 kNm (Eq. 6.5.1.1(3))	
$\alpha_s =$	0.44	Eq. 5.6.1.1(2)
$M_{bx} =$	22.0 kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} =$	19.8 kNm > 13.13 kNm ∴ OK	

ADOPT

180 UB 16.1

Restrain top flange laterally at 3000 mm centres (maximum)

Alt. Use 180 PFC

RR7.

Span = 5 m.

Loading width = 0.75 m.

Line load.

$$dL = 0.4 * 0.75 = 0.3 \text{ kN/m}$$

$$U = 0.25 * 0.75 = 0.2 \text{ kN/m}$$

$$WL = 1.13 * 0.75 = 0.85 \text{ kN/m}$$

$$W_{ult \downarrow} = 0.66 \text{ kN/m}$$

$$W_{ult \uparrow} = 0.85 - 0.4 * 0.3$$

$$= 0.58 \text{ kN/m}$$

$$W_{sewd} = 0.44 \text{ kN/m}$$

Adopt C200x15 ; $\phi_{wd} \& \phi_{wt} \gg \text{req.}$
RR8

Cantilever 1.5 m

Back span 3.2 m.

Try $\angle 150 \times 12$ — no bracing
From design table: strength (ok)

deflection ratio ω for back span of 4 m.

$$\text{for } dL = \frac{L}{150 * \frac{(0.91 + 1.59)}{2(0.48)}} = \frac{L}{226} \quad \text{(ok) for cant.}$$

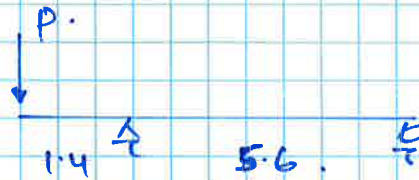
$$W_{sewd} = \frac{L}{150 * \frac{(0.91 + 1.59)}{2(0.69)}} = \frac{L}{157} \quad \text{(ok) for cant.}$$

Adopt C 150x10

Checked :

Date : .../.../...

RCB2



Reaction from RR7.

$$R_{DL} = 0.75 \text{ kN}$$

$$R_{LL} = 0.5 \text{ kN}$$

$$R_{ult} = 0.75 \times 1.2 + 0.5 \times 1.5$$

$$= 1.65 \text{ kN}$$

$$M_{max} = 1.4 \times 1.65 = 2.31 \text{ kNm}$$

Adopt 125 PPL.

Columns.

3C1. →

Reaction from RR4, RR6 and RR1 & RR2; Critical one.

Critical either Reaction from RR1 OR RR2

$$N_{RR1} = 33.5 \text{ kN}$$

$$N_{RR2} = 28 \text{ kN}$$

Design ->

 $N_{RR1} \rightarrow$ take 50 kN. ; Left = 3.8m
Adopt 89 x 3.5 SHS → refer to calcs on
Spread sheet.

3C2 →

Reaction from RR3

$$N_{RR3} = 20 \text{ kN} ; \text{ Left} = 3.8 \text{ m}$$

Adopt 100 x 6 SHS to limit deflection of beam

Checked :

Date :/...../.....



Ref.: 1710168
Date: 07-Oct-18
Design: RR
Page: SC72

COLUMN DESIGN - SHS SECTIONS - PINNED TOP & PINNED BASE

Column 3C1

These calculations comply with the requirements of AS 4100 - 1998 Steel Structures.

$N^* = 50.0$ kN vertical compression load (strength factored load)
 $l_h = 3800$ mm column height
 $k_e = 1.00$ effective length factor (Clause 4.6.3)
 $e = 100$ mm applied load eccentricity at the top of the column
 $\Rightarrow M^* = 5.00$ kNm

Column material yield stress

☒ 350 MPa ☐ 450 MPa

Trial column size : 89x89x3.5 SHS (C350)

$A_n = A_g =$	1150 mm ²	$f_y =$	350 MPa	$k_f =$	1.0
$Z_e =$	36.5E+3 mm ³	$r_x = r_y =$	34.5 mm	$S_x =$	36.5E+3 mm ³
$I_x = I_y =$	1.4E+6 mm ⁴	$J =$	2.2E+6 mm ⁴	$b/t =$	22.4

Check member capacity

Capacity factors

$\phi_b = 0.9$ Table 3.4 - bending $\phi_c = 0.9$ Table 3.4 - compression
For cold-formed (non-stress relieved) SHS, $\alpha_b = -0.5$ Table 6.3.3(2)

(a) Nominal section capacity in compression

Clause 6.1

$N_s = k_f \cdot A_n \cdot f_y = 402.5$ kN
 $\phi_c \cdot N_s = 362.3$ kN
> 50 kN Required : OK

(b) Nominal member capacity in compression

Clause 6.3

$\lambda_n =$	130.3	$\alpha_a =$	14.40	$\lambda =$	123.13
$\eta =$	0.357	$\xi =$	0.863	$\alpha_c =$	0.405

Clause 6.3.3

$N_c = \alpha_c \cdot N_s \leq N_s = 162.8$ kN
 $\phi_c \cdot N_c = 146.6$ kN
> 50 kN Required : OK

(c) Nominal section capacity for combined bending and compression

$M_{sx} = f_y \cdot Z_e = 12.8$ kNm Clause 5.2.1
 $M_{rx} = 12.8$ kNm Clause 8.3.2(a) and (b)
 $\phi_b \cdot M_{rx} = 11.5$ kNm
> 5 kNm Required : OK

(d) Nominal member capacity for combined bending and compression

$M_i = 8.4$ kNm Clause 8.4.2.2
 $\phi_b \cdot M_i = 7.6$ kNm
> 5 kNm Required : OK

\Rightarrow Column is Satisfactory in Combined Bending and Compression

ADOPT 89x89x3.5 SHS (C350)

Type recommendation in
these two lines



Ref.: 1710168
Date: 07-Oct-18
Design: RR
Page: SC73

COLUMN DESIGN - SHS SECTIONS - PINNED TOP & PINNED BASE

Column 3C2

These calculations comply with the requirements of AS 4100 - 1998 Steel Structures.

$N^* = 30.0$ kN vertical compression load (strength factored load)
 $l_h = 3800$ mm column height
 $k_e = 1.00$ effective length factor (Clause 4.6.3)
 $e = 100$ mm applied load eccentricity at the top of the column
 $\Rightarrow M^* = 3.00$ kNm

Column material yield stress

☒ 350 MPa ☐ 450 MPa

Trial column size :

89x89x3.5 SHS (C350)

$A_n = A_g = 1150$ mm ²	$f_y = 350$ MPa	$k_f = 1.0$
$Z_e = 36.5E+3$ mm ³	$r_x = r_y = 34.5$ mm	$S_x = 36.5E+3$ mm ³
$I_x = I_y = 1.4E+6$ mm ⁴	$J = 2.2E+6$ mm ⁴	$b/t = 22.4$

Check member capacity

Capacity factors

$\phi_b = 0.9$ Table 3.4 - bending $\phi_c = 0.9$ Table 3.4 - compression
For cold-formed (non-stress relieved) SHS, $\alpha_b = -0.5$ Table 6.3.3(2)

(a) Nominal section capacity in compression

Clause 6.1

$N_s = k_f \cdot A_n \cdot f_y = 402.5$ kN
 $\phi_c \cdot N_s = 362.3$ kN

> 30 kN Required : OK

(b) Nominal member capacity in compression

Clause 6.3

$\lambda_n = 130.3$	$\alpha_a = 14.40$	$\lambda = 123.13$
$\eta = 0.357$	$\xi = 0.863$	$\alpha_c = 0.405$

Clause 6.3.3

$N_c = \alpha_c \cdot N_s \leq N_s = 162.8$ kN
 $\phi_c \cdot N_c = 146.6$ kN

> 30 kN Required : OK

(c) Nominal section capacity for combined bending and compression

$M_{sx} = f_y \cdot Z_e = 12.8$ kNm
 $M_{rx} = 12.8$ kNm
 $\phi_b \cdot M_{rx} = 11.5$ kNm

Clause 5.2.1

Clause 8.3.2(a) and (b)

> 3 kNm Required : OK

(d) Nominal member capacity for combined bending and compression

$M_i = 10.2$ kNm
 $\phi_b \cdot M_i = 9.1$ kNm

Clause 8.4.2.2

> 3 kNm Required : OK

\Rightarrow Column is Satisfactory in Combined Bending and Compression

ADOPT 89x89x3.5 SHS (C350)

Type recommendation in these two lines

Adopt 100x6 SHS to limit deflection of beam over

Delta Core - 3rd floor.

Maximum span = 7.8m.

Loadings :- topping slab - 80mm
live load - 2 kPa.

(DC1)

From design table

Delta core 200 ΔC 200.12 (0.3)

(0.12)

with out topping

With 60mm topping still ΔC 200.12 (0.3) (0.12)

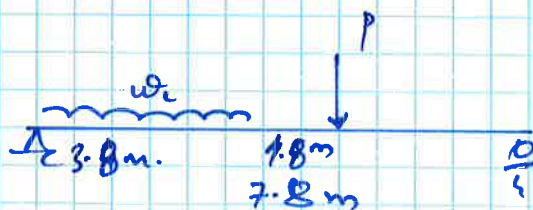
Because of Additional loads from non-load bearing walls and finishes adopt ΔC 200.12 (0.5)

Delta core taking line load from precast panel and roof over

[DC2] (Delta core 250)

UDL from topping = 0.08×24
= 1.92 kPa
~ 2 kPa.

L.L. = 2 kPa.



line load from wall over (w_t)

ht = 4.5m, thickness = 0.15m.

roof = 0.5 kPa, loading width = $\frac{5.5}{2} + 1.5 = 4.25$ m.

∴ SdL = $0.15 \times 4.5 \times 24 + 0.5 \times 4.25 = 18.325$ kN/m.

SLL = $0.25 \times 4.25 = 1.06$ kN/m

P ⇒ loading area = $4.25 \times 4.5 = 19.125$ m² ~ 19 m².

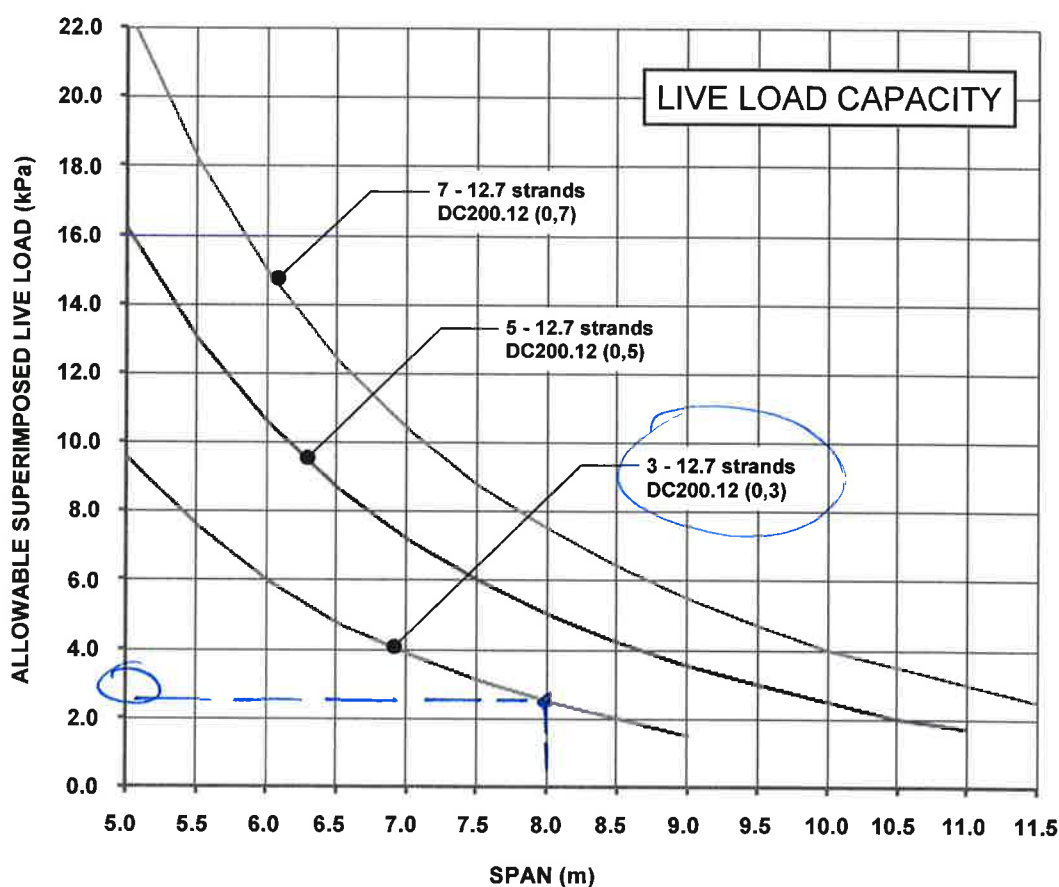
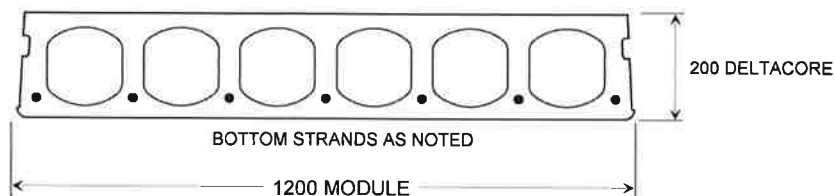
PdL = $0.5 \times 12 + 0.15 \times 4.5 \times 24 \times 1.2 = 26$ kN, $P_u = 4.25 \times \frac{4.5}{2} \times 0.25 = 2.4$

Checked :

Date : .../.../...

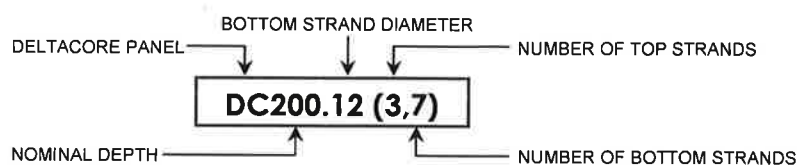
Deltacore 200

DC200 Live Load Capacity



NOTES:

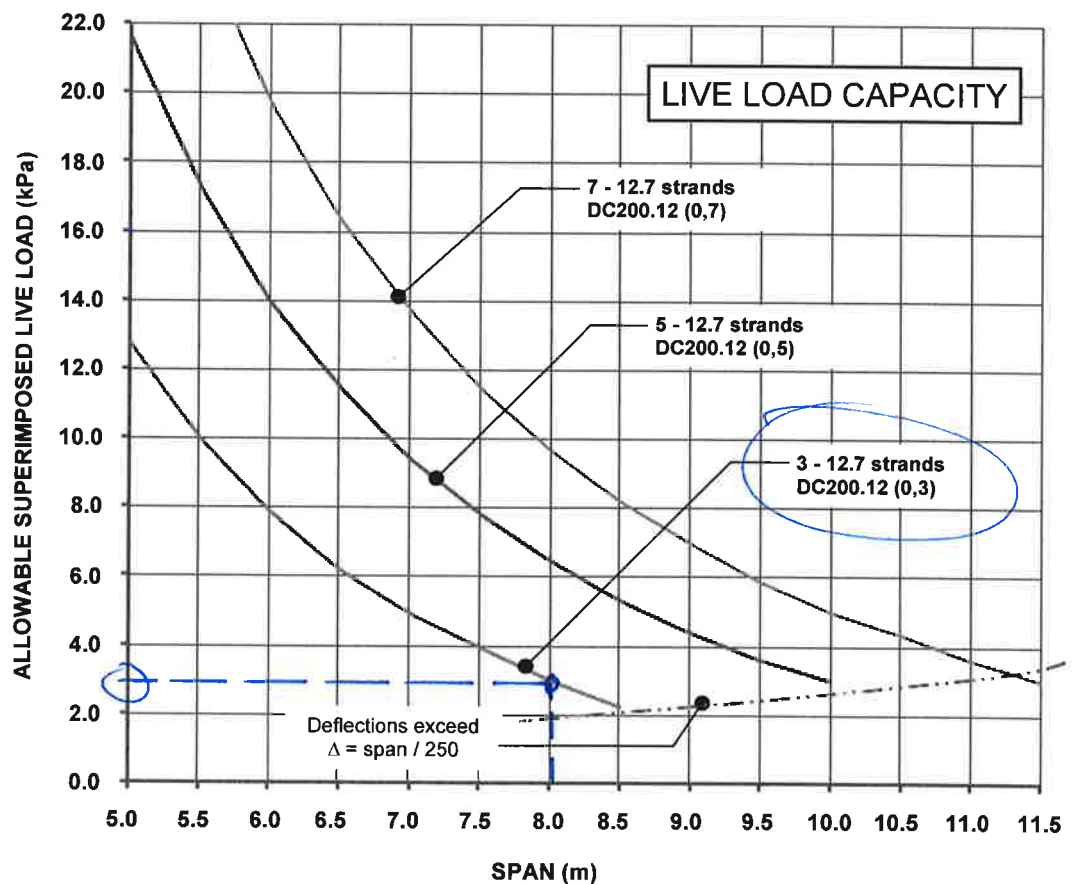
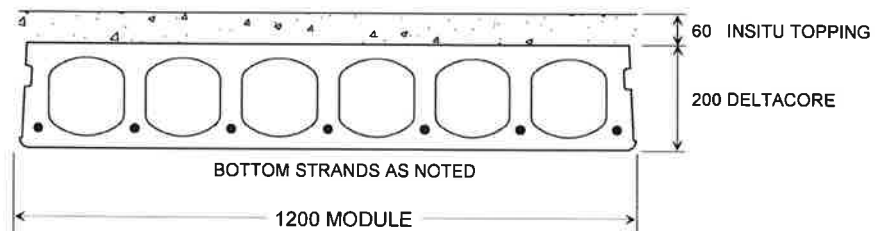
1. Concrete Compressive Strength: Deltacore Planks - 60 MPa
2. All strands are stressed to 70% U.T.S.
3. Cover to bottom strands 40mm



www.deltacorporation.com.au

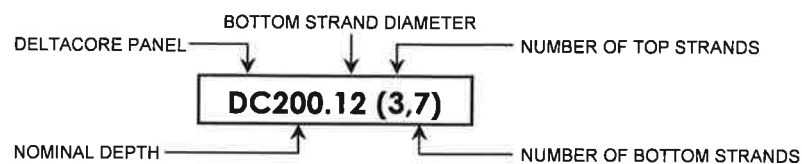
Deltacore 200

DC200 Topped Live Load Capacity



NOTES:

1. Concrete Compressive Strength: Deltacore Planks - 60 MPa; Insitu Topping Slab - 32MPa
2. All strands are stressed to 70% U.T.S.
3. Cover to bottom strands 40mm



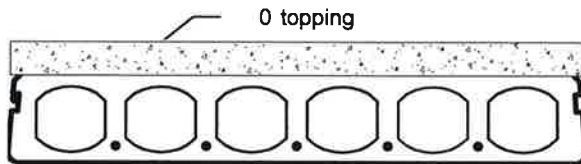
www.deltacorporation.com.au

Client	NIATRON 10 PTY LTD	Job no.	1710168	Sheet	1	of	6
Project	Proposed Residential Revelopment	Calcs by	RR	Date	13/07/2018		
Subject	Third floor	Checked by	RR	Date	13/07/2018		

DESIGN OF DELTACORE PLANKS - Sheet 1

1.0 DELTACORE PLANK DETAILS

Plank Type : DC 200



Concrete Strength - In-situ topping : 32 MPa
 - Deltacore plank : 60 MPa

Prestressing - Top : 0 - 12.7 dia Strands
 - Bottom : 5 - 12.7 dia Strands

Initial Prestress - Top strands : 70% UTS
 - Bottom strands : 70% UTS

SPAN = 7.800 m

Cover to strands - Top strands : 47 mm
 - Bottom strands : 40 mm

Section Properties :

• Deltacore Plank : (un-topped)	Area	A =	$125 \times 10^3 \text{ mm}^2$
	Moment of Inertia	I =	$655 \times 10^6 \text{ mm}^4$
	Section Modulus - Top	$Z_t =$	$6.4 \times 10^6 \text{ mm}^3$
	- Bottom	$Z_b =$	$6.5 \times 10^6 \text{ mm}^3$

• Composite Section :		<u>Gross Section</u>	<u>Cracked Section</u>
Area	A =	$125 \times 10^3 \text{ mm}^2$	$A_{cr} = 96 \times 10^3 \text{ mm}^2$
Moment of Inertia	I =	$655 \times 10^6 \text{ mm}^4$	$I_g = 655 \times 10^6 \text{ mm}^4$
Section Modulus - Top	$Z_t =$	$6.4 \times 10^6 \text{ mm}^3$	$I_{cr} = 641 \times 10^6 \text{ mm}^4$
- Bottom	$Z_b =$	$6.5 \times 10^6 \text{ mm}^3$	$I_{eff} = 655 \times 10^6 \text{ mm}^4$
			(DL + SDL + Ψ_L LL)

2.0 APPLIED LOADS

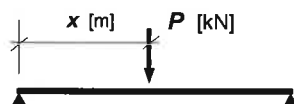


Uniformly distributed

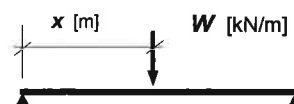
$$w_{DL} = 2.6 \text{ kPa}$$

$$w_{SDL} = 2.0 \text{ kPa}$$

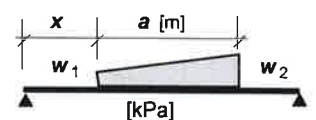
$$w_{LL} = 2.0 \text{ kPa}$$



Point Loads



Transverse Line Loads

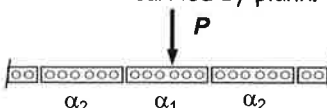


Trapezoidal Loads

Notes :

For point loads -

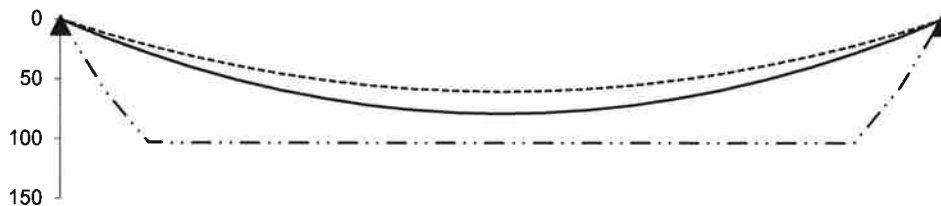
α_1 = Proportion of load
 carried by plank.



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DESIGN OF DELTACORE PLANKS - Sheet 2

3.0 BENDING MOMENTS



Maximum Moments :

$$M_{serv} = 60 \text{ kN-m}$$

$$M_{ult}^* = 79 \text{ kN-m}$$

$$\phi M_r = 103 \text{ kN-m}$$

BENDING MOMENT DIAGRAM

At Transfer of Prestress :

Required concrete strength : $f_{cp} = 15 \text{ MPa}$ (cf. $f'_{cp} = 36 \text{ MPa}$)

Extreme top fibre stress : $f_{ct} = -0.48 \text{ MPa}$ (cf. $f'_{ct} = -1.50 \text{ MPa}$)

Construction (during pouring of topping slab) :

Bending Moments :

Dead Loads - $M_{DL} = 24 \text{ kN-m}$

Live Loads - $M_{LL} = 9 \text{ kN-m}$

($M_{uo.min} = 101 \text{ kN-m}$)

$$\Rightarrow \begin{aligned} M_{ult}^* &= 42 \text{ kN-m} & (\text{cf. } \phi M_r &= 103 \text{ kN-m} \quad (0.41 \phi M_r)) \\ M_{serv} &= 33 \text{ kN-m} & \Delta f_p &= -24 \text{ MPa} \end{aligned}$$

Composite Section Analysis :

Bending Moments :

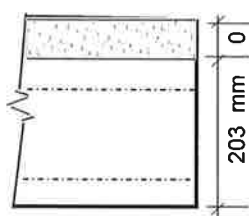
Dead Loads - $M_{DL} = 24 \text{ kN-m}$

Superimposed DL - $M_{SDL} = 18 \text{ kN-m}$

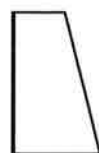
Live Loads - $M_{LL} = 18 \text{ kN-m}$

($M_{uo.min} = 101 \text{ kN-m}$)

$$\Rightarrow \begin{aligned} M_{ult}^* &= 79 \text{ kN-m} & (\text{cf. } \phi M_r &= 103 \text{ kN-m} \quad (0.76 \phi M_r)) \\ M_{serv} &= 60 \text{ kN-m} & \Delta f_p &= 4 \text{ MPa} \end{aligned}$$



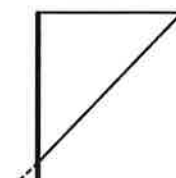
3.3 MPa



5.5 MPa

Deltacore Plank
(During construction)

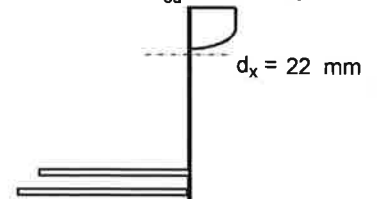
4.9 MPa



Cracked

Topped Plank
(Service Stresses)

$f_{cu} = 51.0 \text{ MPa}$



$f_p = 1725 \text{ MPa}$
($\epsilon_p = 2.33\%$)

Ultimate Limit State

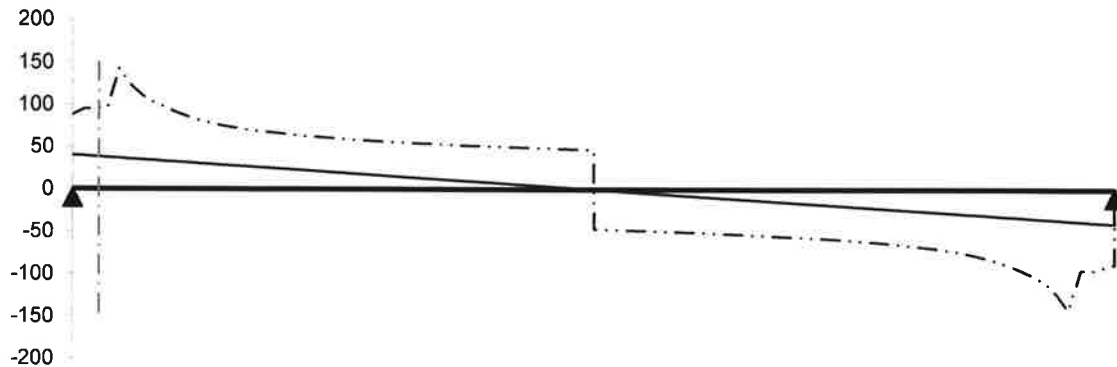
Concrete Stresses :

Service Limit State

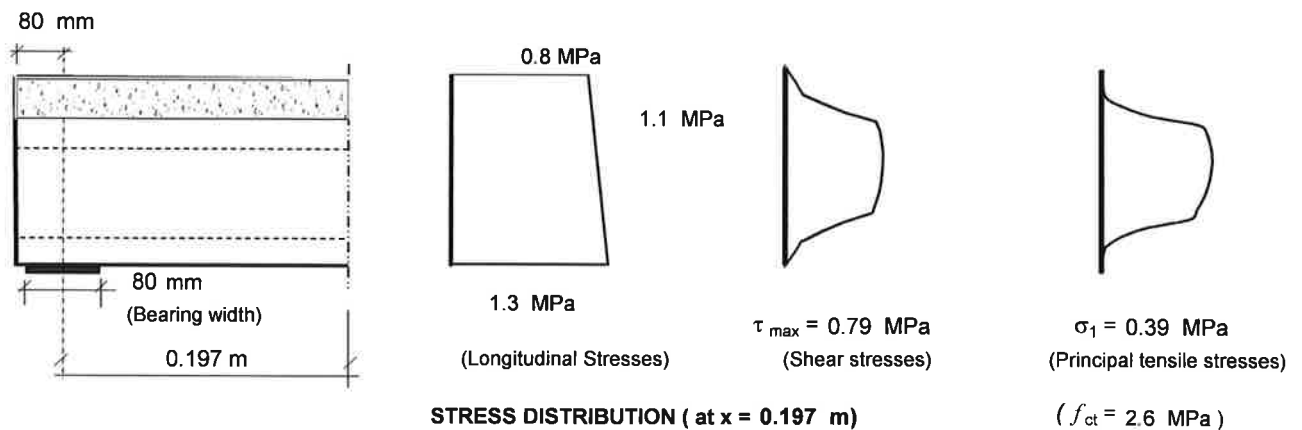
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DESIGN OF DELTACORE PLANKS - Sheet 3

4.0 SHEAR FORCES



SHEAR FORCE DIAGRAM



STRESS DISTRIBUTION (at x = 0.197 m)

Ultimate Shear Capacity :

At 0.197 m from support :

(a) Flexure-shear cracking :

$$M_o = 54 \text{ kN-m} \quad (\sigma_{cp,f} = 8.4 \text{ MPa})$$

$$V_o = 270 \text{ kN}$$

$$V_c = 314 \text{ kN}$$

(b) Web-shear cracking :

$$V_t = 137 \text{ kN}$$

$$\Rightarrow \phi V_{uc} = 96 \text{ kN}$$

Ultimate Applied Shear Force :

$$\text{Dead Loads} - V_{DL} = 11.5 \text{ kN}$$

$$\text{Superimposed DL} - V_{SDL} = 8.9 \text{ kN}$$

$$\text{Live Loads} - V_{LL} = 8.9 \text{ kN}$$

$$\Rightarrow V_{ult} = 38.3 \text{ kN} \quad (0.40 \phi V_{uc})$$

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DESIGN OF DELTACORE PLANKS - Sheet 4

5.0 DEFLECTIONS

◆ Deflections at Transfer of Prestress

Upward camber due to prestressing	- 12 mm
Deflections due to plank selfweight	7 mm
Total	- 5 mm

◆ At Pouring of Topping Slab

Deflection of plank (incl prestress)	- 10 mm
Deflection due to topping selfwt	0 mm
Total	- 10 mm

◆ Deflections of Composite Section

(30 years after pouring topping slab)

Total Dead Load deflections	- 14 mm
Deflections due to differential shrinkage	0 mm
Additional deflections due to SDL	7 mm
Additional deflections due to :	
Short term Live Loading	3 mm ($\Psi_s = 0.7$)
Long term Live Loading	3 mm ($\Psi_l = 0.4$)
Total	- 3 mm (Span / 2322)

Incremental long term deflections:

After application of SDL & Live loads	6 mm (Span / 1232)
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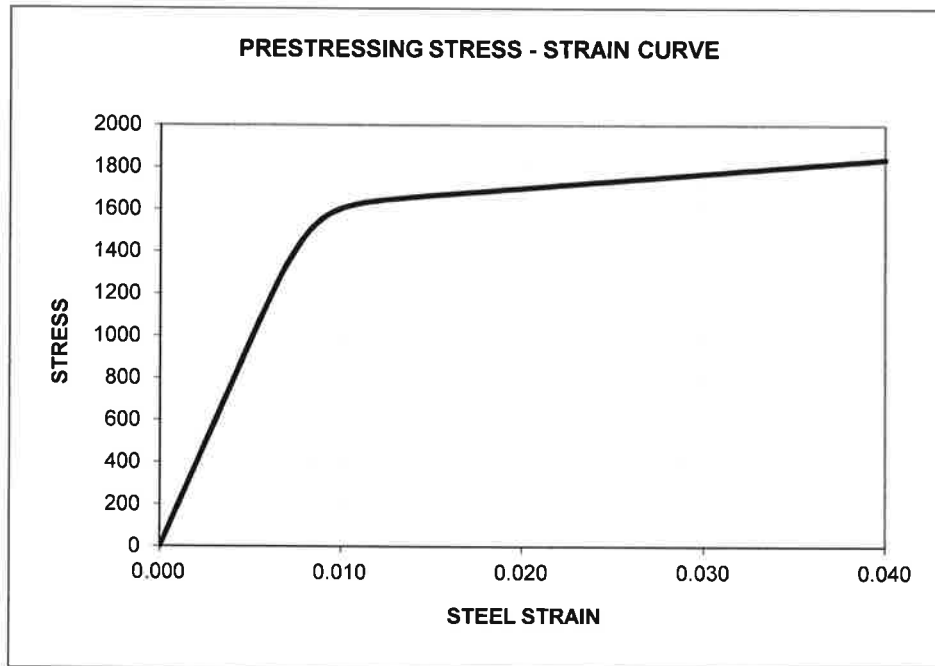
6.0 NATURAL FREQUENCY

Vibration design live load :	$w_{LL} = 0.5 \text{ kPa}$
Natural Frequency :	$n = 6.8 \text{ Hz}$

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DESIGN OF DELTACORE PLANKS - Sheet 5

7.0 SUMMARY OF MATERIAL PROPERTIES



Tendon Properties:

Strands	F_p	F_{py}
9.3 dia.	1860 MPa	1580 MPa
12.7 dia.	1840 MPa	1565 MPa
15.2 dia.	1750 MPa	1488 MPa

$$E_p = 195\,000 \text{ MPa}$$

TABLE 1 - Stress - Strain Curve for 12.7 dia Strands (Bottom Strands)

Summary of Prestressing Forces :

Force / Strand	Bottom Strands	Second Layer	Top Strands
Jacking Force :	129 kN	0 kN	0 kN
Forces at Transfer :	125 kN	0 kN	0 kN
Long Term Forces :	103 kN	0 kN	0 kN
(Long term Losses)	(20.0%)		

Concrete Properties:

◆ Deltacore Plank :

Concrete Strength : $f'_c = 60 \text{ MPa}$

Basic Creep Coefficient : $\phi_{cc,b} = 2.0$

Basic Drying Shrinkage : $\epsilon_{csd,b} = 900 \times 10^{-6}$

◆ Topping Slab :

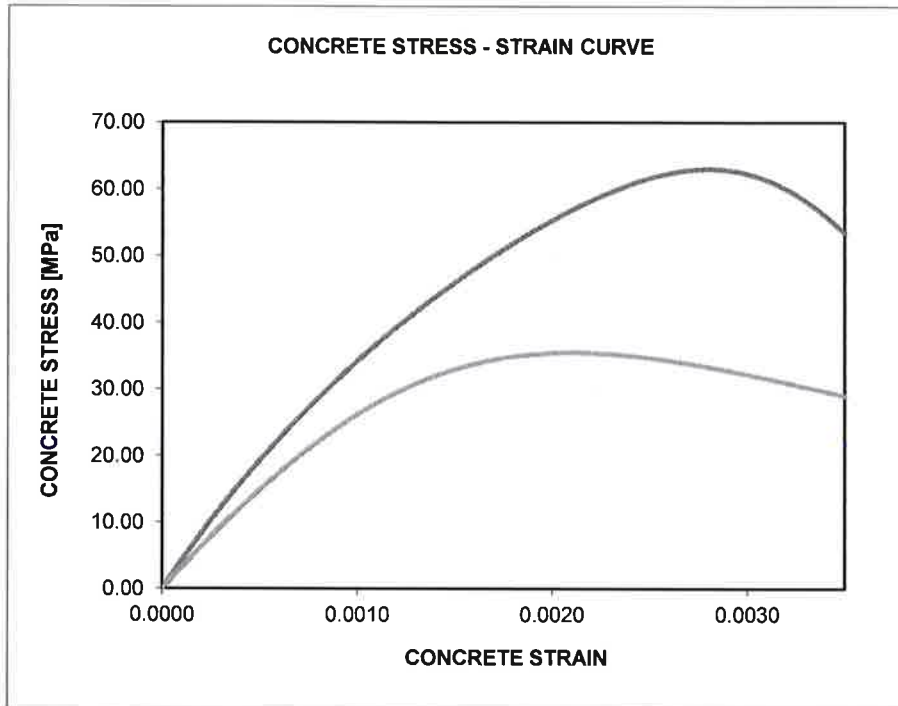
Concrete Strength : $f'_c = 32 \text{ MPa}$

Basic Creep Coefficient : $\phi_{cc,b} = 3.4$

Basic Drying Shrinkage : $\epsilon_{csd,b} = 900 \times 10^{-6}$

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DESIGN OF DELTACORE PLANKS - Sheet 6



Deltacore plank :

$$f'_c = 60 \text{ MPa}$$

In-situ topping :

$$f'_c = 32 \text{ MPa}$$

Limiting concrete strain:

$$\epsilon_c = 0.003$$

TABLE 2 - Concrete Stress - Strain Curve

Time Dependent Concrete Properties of Deltacore Panel	Concrete Strength	Elastic Modulus	Shrinkage Strain	Design Creep Factor ($1 + \phi_{cc} / \alpha$)			
	f_{cm}	E_{cj}	ϵ_{cs}	Swf.	DL _{add}	SDL	LL
At transfer of Prestress :	36 MPa	32 120 MPa	66×10^{-6}	-	-	-	-
At pouring of topping slab :	58 MPa	37 890 MPa	349×10^{-6}	2.42	-	-	-
At application of Applied Loads :	61 MPa	38 390 MPa	559×10^{-6}	3.13	1.35	-	-
30 years after pouring topping slab:	61 MPa	38 470 MPa	674×10^{-6}	3.57	1.88	1.52	1.52

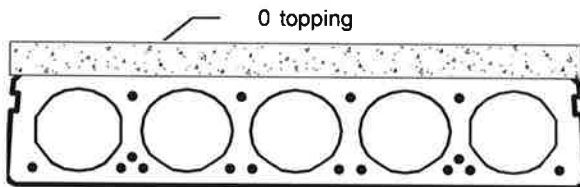
TABLE 3 - Time Dependent Concrete Properties for Deltacore Panel

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DESIGN OF DELTACORE PLANKS - Sheet 1

1.0 DELTACORE PLANK DETAILS

Plank Type : DC 250



Concrete Strength - In-situ topping : 32 MPa
 - Deltacore plank : 60 MPa

Prestressing - Top : 4 - 12.7 dia Strands
 - Bottom : 12 - 12.7 dia Strands

Initial Prestress - Top strands : 70% UTS
 - Bottom strands : 70% UTS

SPAN = 7.800 m

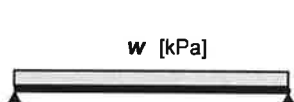
Cover to strands - Top strands : 37 mm
 - Bottom strands : 40 mm

Section Properties :

• Deltacore Plank : (un-topped)	Area	$A = 160 \times 10^3 \text{ mm}^2$
	Moment of Inertia	$I = 1305 \times 10^6 \text{ mm}^4$
	Section Modulus - Top	$Z_t = 10.1 \times 10^6 \text{ mm}^3$
	- Bottom	$Z_b = 10.6 \times 10^6 \text{ mm}^3$

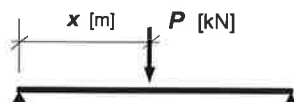
• Composite Section :		<u>Gross Section</u>	<u>Cracked Section</u>
	Area	$A = 160 \times 10^3 \text{ mm}^2$	$A_{cr} = 93 \times 10^3 \text{ mm}^2$
	Moment of Inertia	$I = 1305 \times 10^6 \text{ mm}^4$	$I_g = 1305 \times 10^6 \text{ mm}^4$
	Section Modulus - Top	$Z_t = 10.1 \times 10^6 \text{ mm}^3$	$I_{cr} = 303 \times 10^6 \text{ mm}^4$
	- Bottom	$Z_b = 10.6 \times 10^6 \text{ mm}^3$	$I_{eff} = 1233 \times 10^6 \text{ mm}^4$
			(DL + SDL + Ψ LL, LL)

2.0 APPLIED LOADS



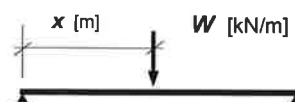
Uniformly distributed

$w_{DL} = 3.3 \text{ kPa}$
 $w_{SDL} = 2.0 \text{ kPa}$
 $w_{LL} = 2.0 \text{ kPa}$

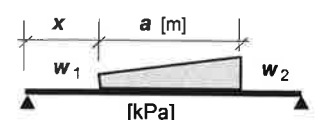


Point Loads

	P	x	α_1
SDL	26.0	4.800	100%
LL	2.4	4.800	100%



Transverse Line Loads



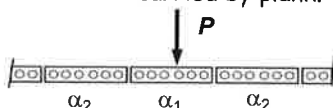
Trapezoidal Loads

	$w1$	x	$w2$	a
SDL	19.0	0.000	19.0	3.800
LL	1.0	0.000	1.0	3.800

Notes :

For point loads -

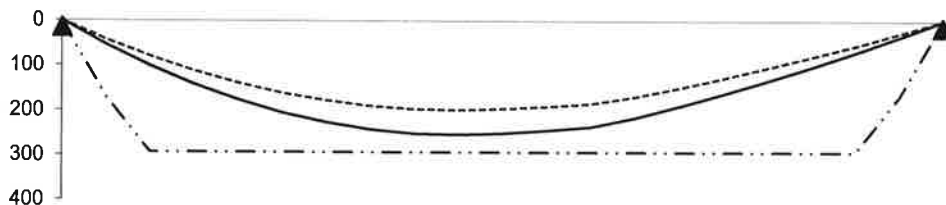
α_1 = Proportion of load
 carried by plank.



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DESIGN OF DELTACORE PLANKS - Sheet 2

3.0 BENDING MOMENTS



Maximum Moments :

$$M_{serv} = 199 \text{ kN-m}$$

$$M_{ult}^* = 254 \text{ kN-m}$$

$$\phi M_r = 293 \text{ kN-m}$$

BENDING MOMENT DIAGRAM

◆ At Transfer of Prestress :

Required concrete strength : $f_{cp} = 27 \text{ MPa}$ (cf. $f'_{cp} = 36 \text{ MPa}$)

Extreme top fibre stress : $f_{ct} = 5.23 \text{ MPa}$ (cf. $f'_{ct} = -3.00 \text{ MPa}$)

◆ Construction (during pouring of topping slab) :

Bending Moments :

Dead Loads - $M_{DL} = 30 \text{ kN-m}$

Live Loads - $M_{LL} = 9 \text{ kN-m}$

($M_{uo,min} = 224 \text{ kN-m}$)

$$\Rightarrow \begin{aligned} M_{ult}^* &= 50 \text{ kN-m} & (\text{cf. } \phi M_r &= 293 \text{ kN-m} \quad (0.17 \phi M_r)) \\ M_{serv} &= 40 \text{ kN-m} & \Delta f_p &= -52 \text{ MPa} \end{aligned}$$

◆ Composite Section Analysis :

Bending Moments :

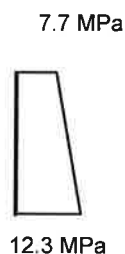
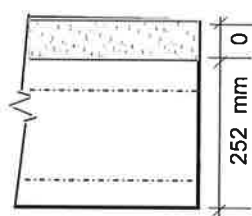
Dead Loads - $M_{DL} = 30 \text{ kN-m}$

Superimposed DL - $M_{SDL} = 143 \text{ kN-m}$

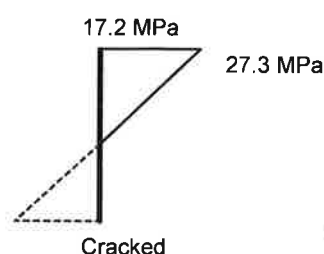
Live Loads - $M_{LL} = 26 \text{ kN-m}$

($M_{uo,min} = 224 \text{ kN-m}$)

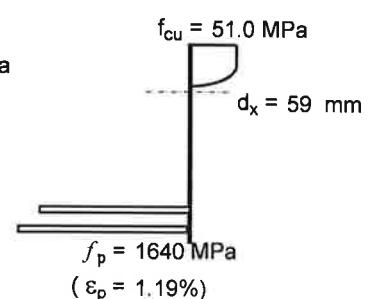
$$\Rightarrow \begin{aligned} M_{ult}^* &= 254 \text{ kN-m} & (\text{cf. } \phi M_r &= 293 \text{ kN-m} \quad (0.87 \phi M_r)) \\ M_{serv} &= 199 \text{ kN-m} & \Delta f_p &= 89 \text{ MPa} \end{aligned}$$



Deltacore Plank
(During construction)



Topped Plank
(Service Stresses)



Ultimate Limit State

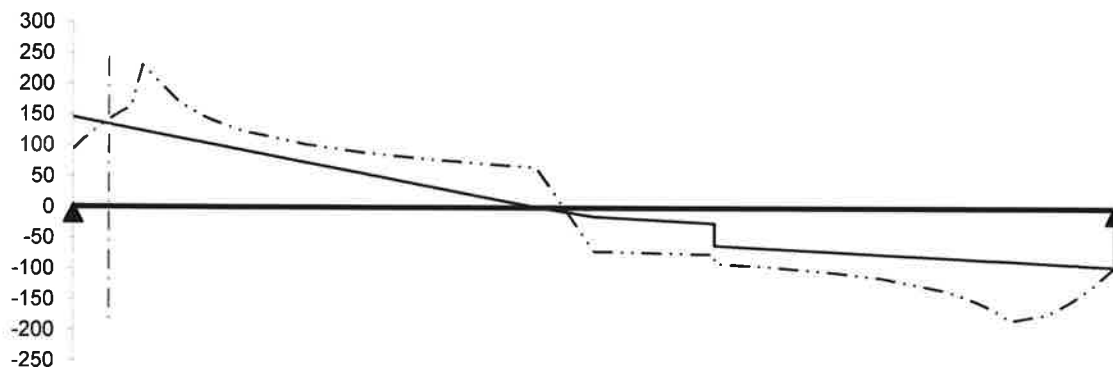
Concrete Stresses :

Service Limit State

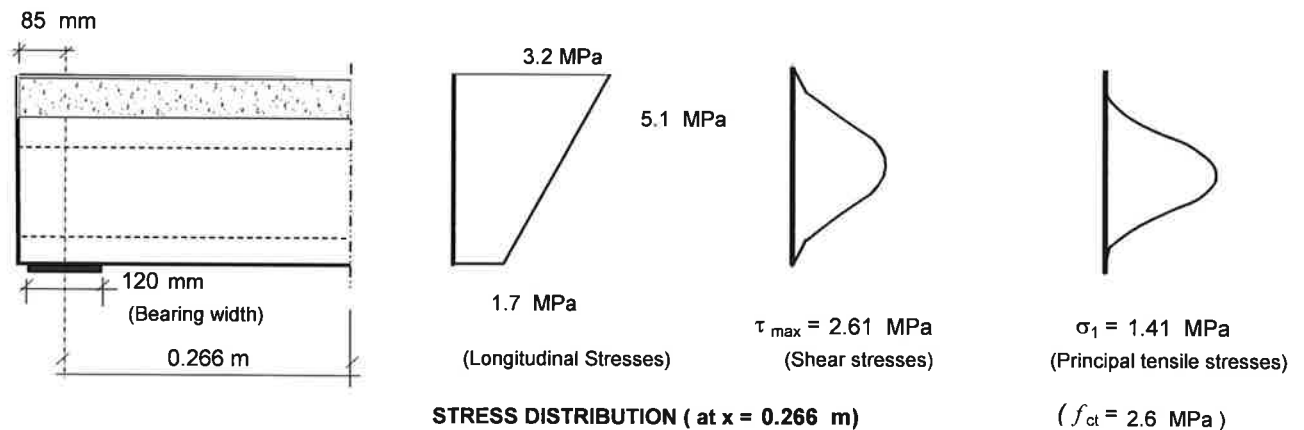
Client	NIATRON 10 PTY LTD	Job no.	1710168	Sheet	3 of 6
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DESIGN OF DELTACORE PLANKS - Sheet 3

4.0 SHEAR FORCES



SHEAR FORCE DIAGRAM



Ultimate Shear Capacity :

At 0.266 m from support :

(a) Flexure-shear cracking :

$$M_o = 138 \text{ kN-m} \quad (\sigma_{cp,r} = 13.0 \text{ MPa})$$

$$V_o = 498 \text{ kN}$$

$$V_c = 566 \text{ kN}$$

(b) Web-shear cracking :

$$V_t = 202 \text{ kN}$$

$$\Rightarrow \phi V_{uc} = 141 \text{ kN}$$

Ultimate Applied Shear Force :

$$\text{Dead Loads} - V_{DL} = 14.6 \text{ kN}$$

$$\text{Superimposed DL} - V_{SDL} = 78.2 \text{ kN}$$

$$\text{Live Loads} - V_{LL} = 12.8 \text{ kN}$$

$$\Rightarrow V_{ult} = 134.4 \text{ kN} \quad (0.95 \phi V_{uc})$$

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DESIGN OF DELTACORE PLANKS - Sheet 4

5.0 DEFLECTIONS

◆ Deflections at Transfer of Prestress

Upward camber due to prestressing	- 11 mm
Deflections due to plank selfweight	5 mm
Total	- 7 mm

◆ At Pouring of Topping Slab

Deflection of plank (incl prestress)	- 11 mm
Deflection due to topping selfwt	0 mm
Total	- 11 mm

◆ Deflections of Composite Section

(30 years after pouring topping slab)

Total Dead Load deflections	- 14 mm	
Deflections due to differential shrinkage	0 mm	
Additional deflections due to SDL	26 mm	
Additional deflections due to :		
Short term Live Loading	4 mm	($\Psi_s = 0.7$)
Long term Live Loading	2 mm	($\Psi_l = 0.4$)
Total	16 mm	(Span / 486)

Incremental long term deflections:

After application of SDL & Live loads	27 mm	(Span / 291)
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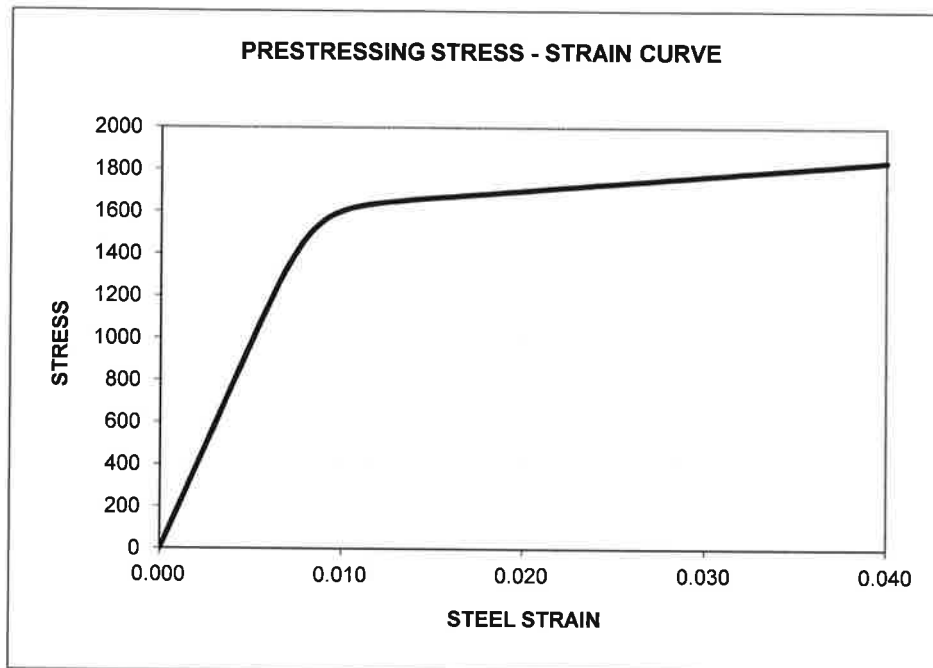
6.0 NATURAL FREQUENCY

Vibration design live load :	$w_{LL} = 0.5 \text{ kPa}$
Natural Frequency :	$n = 8.6 \text{ Hz}$

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DESIGN OF DELTACORE PLANKS - Sheet 5

7.0 SUMMARY OF MATERIAL PROPERTIES



Tendon Properties:

Strands	F_p	F_{py}
9.3 dia.	1860 MPa	1580 MPa
12.7 dia.	1840 MPa	1565 MPa
15.2 dia.	1750 MPa	1488 MPa

$$E_p = 195\,000 \text{ MPa}$$

TABLE 1 - Stress - Strain Curve for 12.7 dia Strands (Bottom Strands)

Summary of Prestressing Forces :

Force / Strand	Bottom Strands	Second Layer	Top Strands
Jacking Force :	129 kN	129 kN	129 kN
Forces at Transfer :	120 kN	121 kN	123 kN
Long Term Forces :	90 kN	90 kN	88 kN
(Long term Losses)	(30.6%)	(29.9%)	(31.5%)

Concrete Properties:

◆ Deltacore Plank :

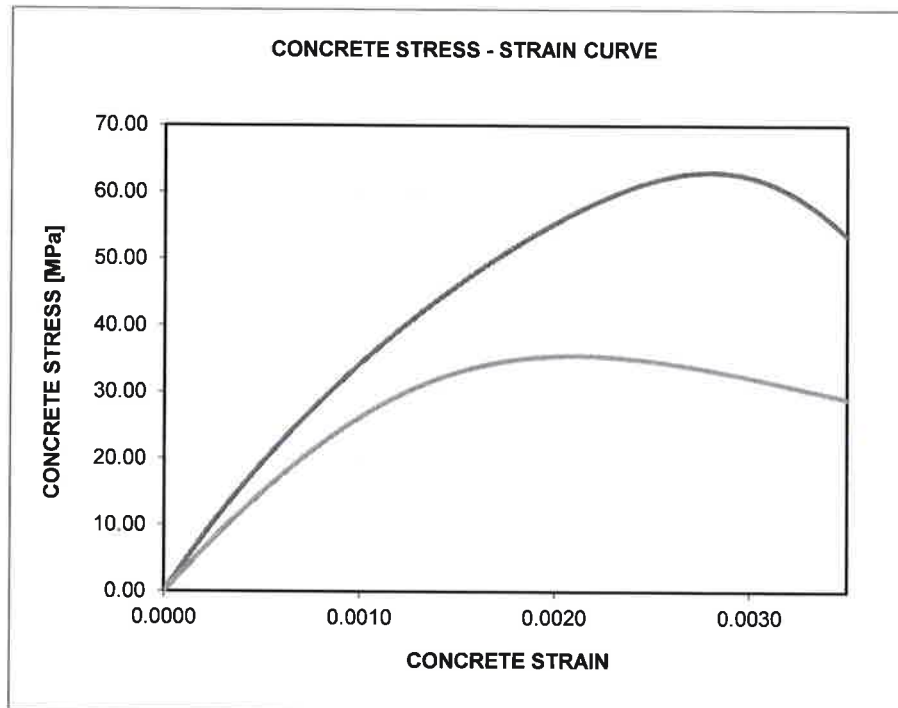
Concrete Strength : $f'_c = 60 \text{ MPa}$
 Basic Creep Coefficient : $\phi_{cc,b} = 2.0$
 Basic Drying Shrinkage : $\epsilon_{csd,b} = 900 \times 10^{-6}$

◆ Topping Slab :

Concrete Strength : $f'_c = 32 \text{ MPa}$
 Basic Creep Coefficient : $\phi_{cc,b} = 3.4$
 Basic Drying Shrinkage : $\epsilon_{csd,b} = 900 \times 10^{-6}$

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DESIGN OF DELTACORE PLANKS - Sheet 6



Deltacore plank :

$$f'_c = 60 \text{ MPa}$$

In-situ topping :

$$f'_c = 32 \text{ MPa}$$

Limiting concrete strain:

$$\epsilon_c = 0.003$$

TABLE 2 - Concrete Stress - Strain Curve

Time Dependent Concrete Properties of Deltacore Panel	Concrete Strength	Elastic Modulus	Shrinkage Strain	Design Creep Factor ($1 + \phi_{cc} / \alpha$)			
	f'_{cm}	E_{cj}	ϵ_{cs}	Swf.	DL _{add}	SDL	LL
At transfer of Prestress :	36 MPa	32 120 MPa	53×10^{-6}	-	-	-	-
At pouring of topping slab :	58 MPa	37 890 MPa	310×10^{-6}	2.06	-	-	-
At application of Applied Loads :	61 MPa	38 390 MPa	511×10^{-6}	2.48	1.32	-	-
30 years after pouring topping slab:	61 MPa	38 470 MPa	641×10^{-6}	2.80	1.86	1.47	1.46

TABLE 3 - Time Dependent Concrete Properties for Deltacore Panel

1710168
sc89

\\tmk7\jobs\2017\10\1710168\Structural Drawings and Calcs\Design Calculations and Details\RR\Preliminary\Third floor cantilever balcony slab.rpf

RAPT - Version: 6.5.16.0

Reinforced And Post-Tensioned Concrete Analysis & Design Package

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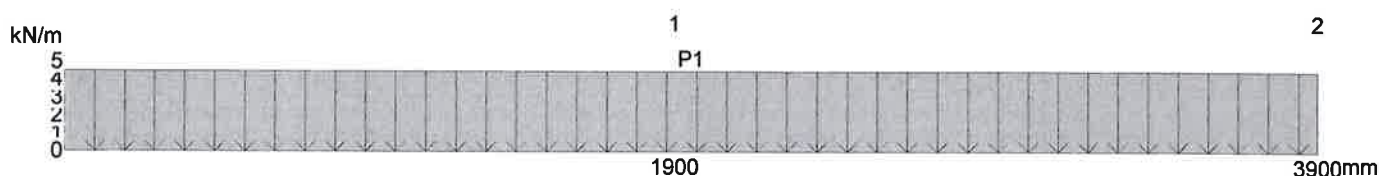
Cantilever Slab

180 thick N12@ 200¢
each way top & bottom

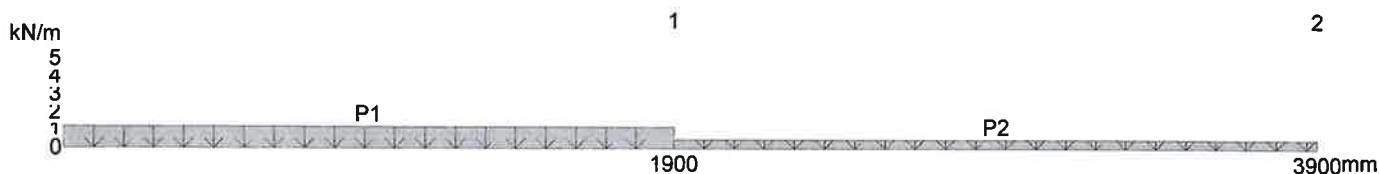
Licensee
TMK Consulting Engineers
Level 6
100 Pirie Street
Adelaide SA 5000
11169065160718WPN3

Input

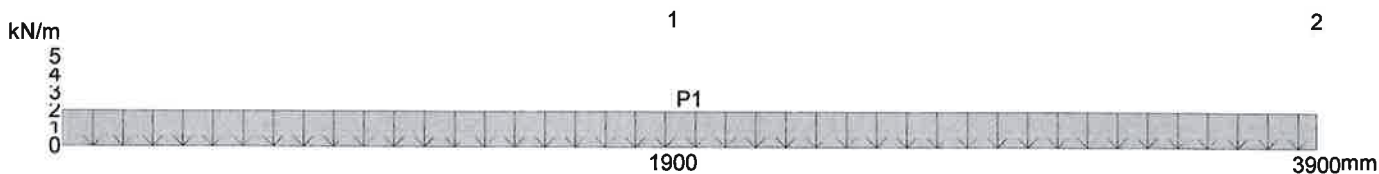
Load Case 1 : 1. Self Weight



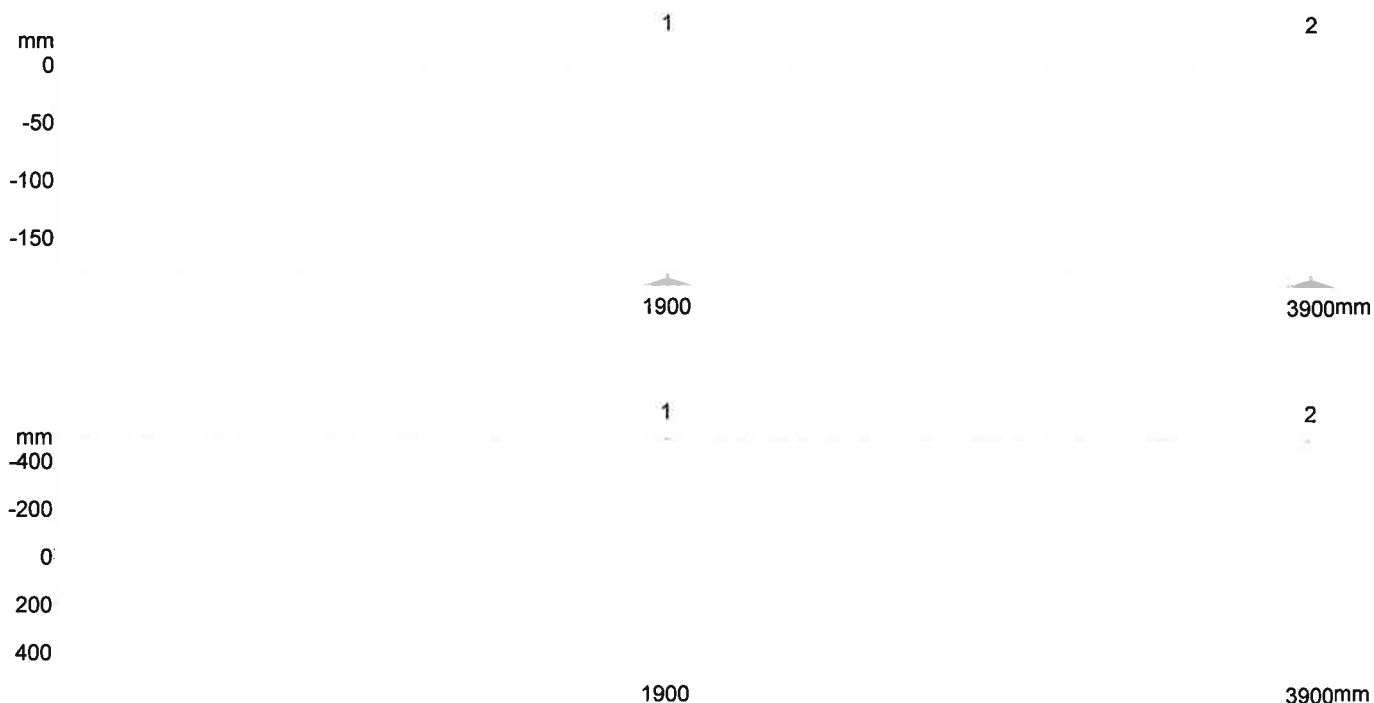
Load Case 2 : 2. Extra Dead Load



Load Case 3 : 3. Live Load



Elevation view



Warnings

Input

No errors or warnings were found.

Output

Warning: Flat slabs - slabs which rely on flexural action through the slab or slab thickenings such as drop panels or band beams to discrete column supports, require concentration of the top reinforcement and tendons over supports within a defined width from the column face to facilitate moment transfer to the column. This width varies and the amount of reinforcement varies for different design codes. In the major codes, the requirements are specified in AS3600 cl 9.1.2, Eurocode 1992 - 1 - 1 2004 cl 9.4.1, ACI318-14 cl 8.4.2.3 and BS8110 cl 3.7.3.1. For example AS3600 requires that 25% of total negative moment for the entire slab panel be resisted by reinforcement within a width D either side of the column. RAPT cannot define this concentration as it does not know the transverse spacing of the tendons. It is the designer's responsibility to ensure that these clauses are complied with.

Bending Moments

Load Cases

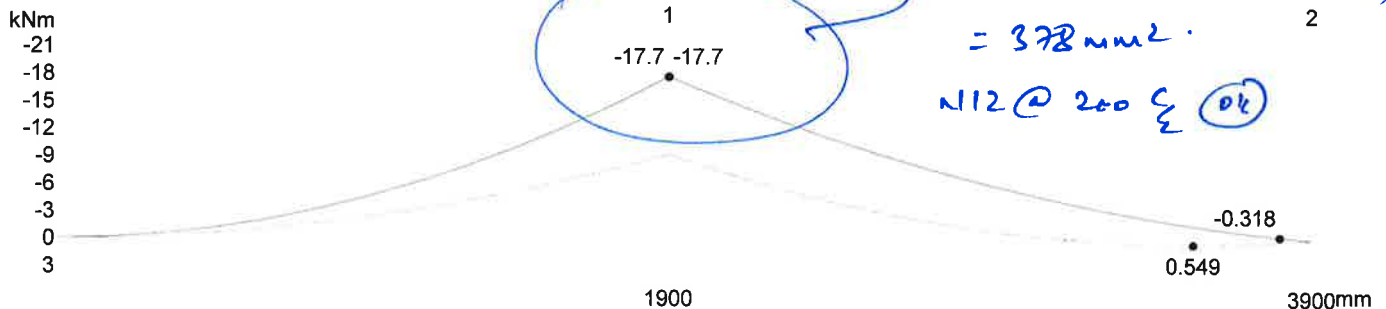
Column Actions

Col No. 1		Self Weight	Extra Dead Load	Live Load
Moment Above	kNm	-0	-0	-0
Moment Below	kNm	-0	-0	-0
Reaction	kN	17.11	3.86	7.6
Elastic Rotation	##	-2.6e-4	0	-1.16e-4
Elastic Axial Shortening	mm	0	0	0

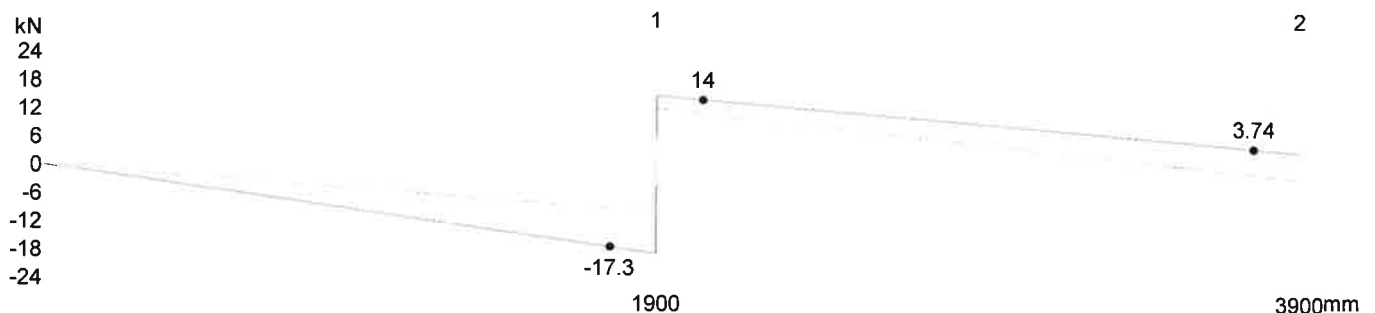
Col No. 2		Self Weight	Extra Dead Load	Live Load
Moment Above	kNm	-0	-0	-0
Moment Below	kNm	-0	-0	-0
Reaction	kN	0.44	-0.58	0.2
Elastic Rotation	##	0	0	0
Elastic Axial Shortening	mm	0	0	0

Load Combinations

Ultimate Flexure



Moment Moment 1 Moment 2

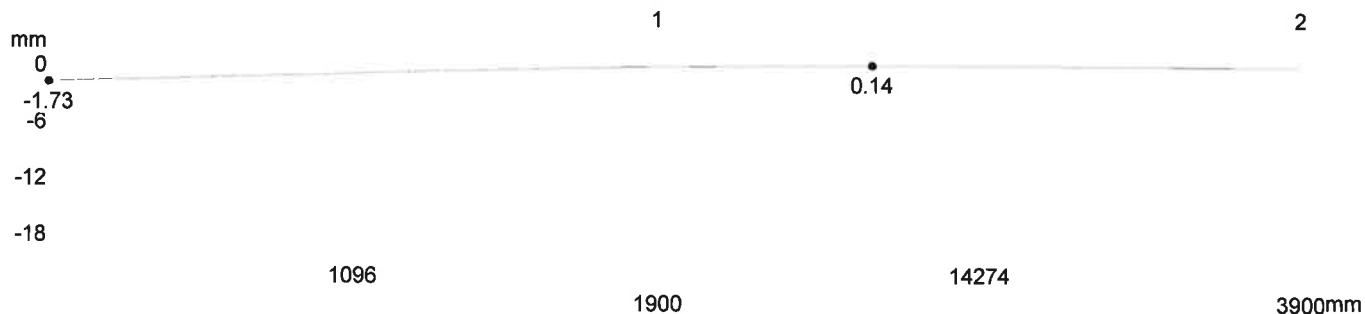


Shear Shear 1 Shear 2

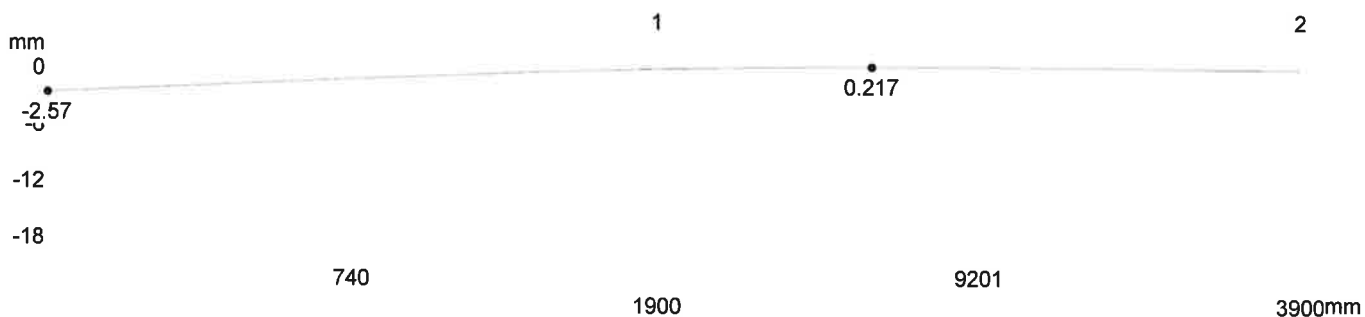
Deflections

All Spans Loaded

Transfer

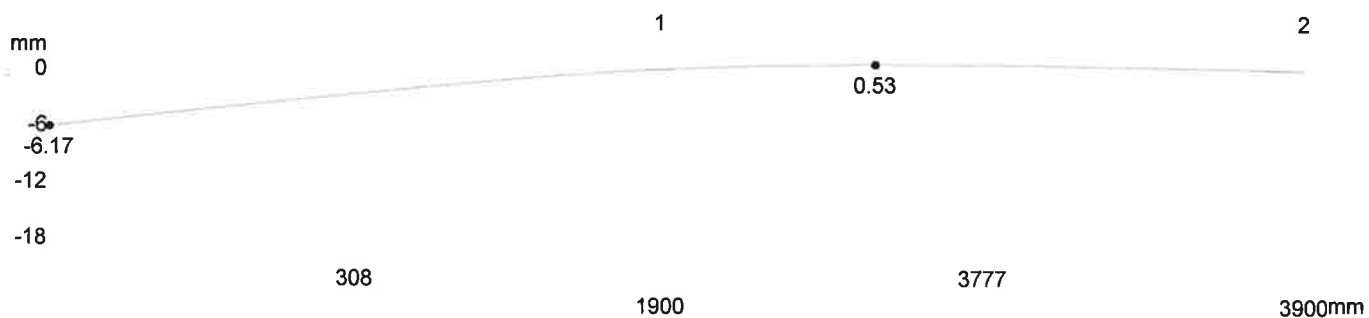


Short Term



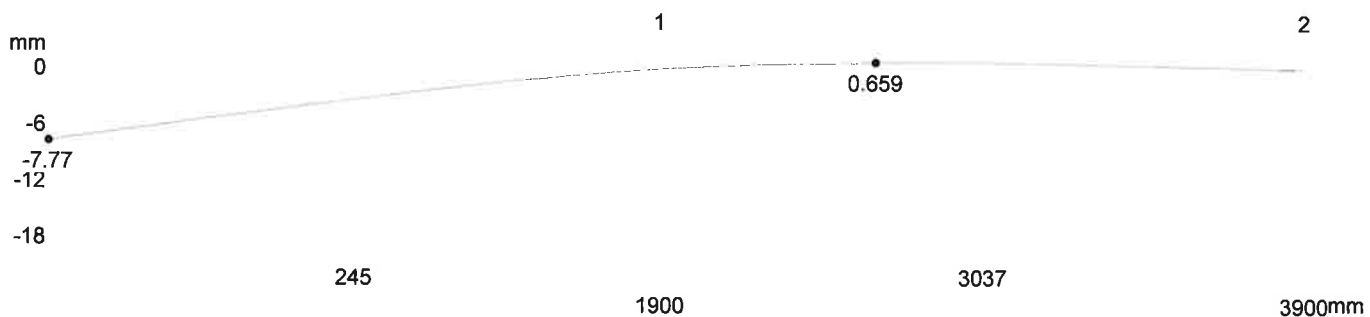
Even Spans Loaded Odd Spans Loaded All Spans Loaded

Incremental



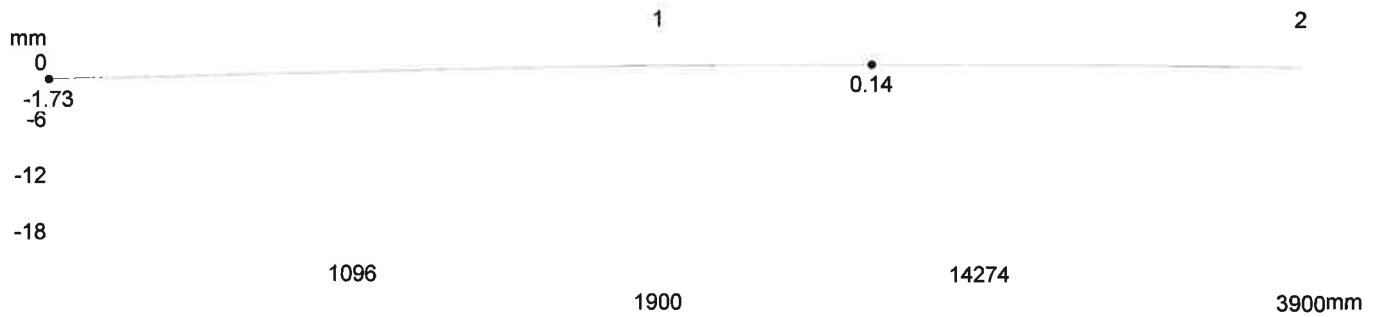
Even Spans Loaded Odd Spans Loaded All Spans Loaded

Total Long Term

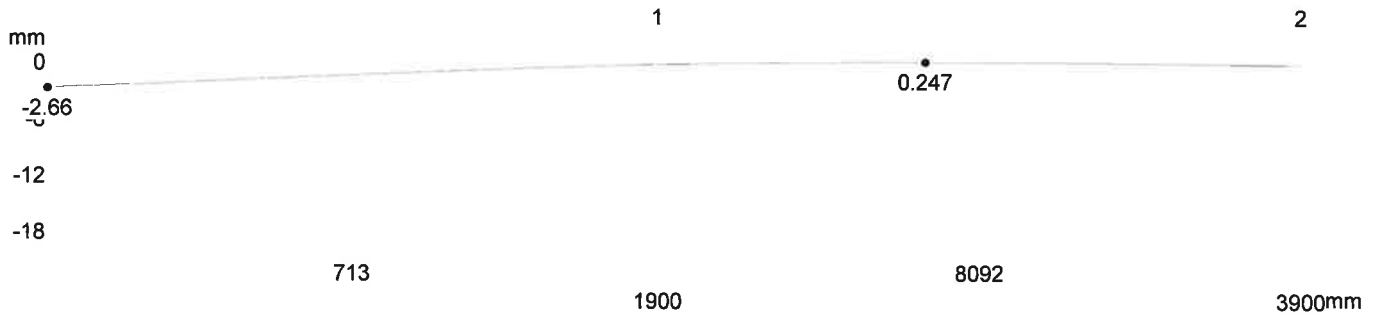


Even Spans Loaded Odd Spans Loaded All Spans Loaded

Even Spans Loaded Transfer

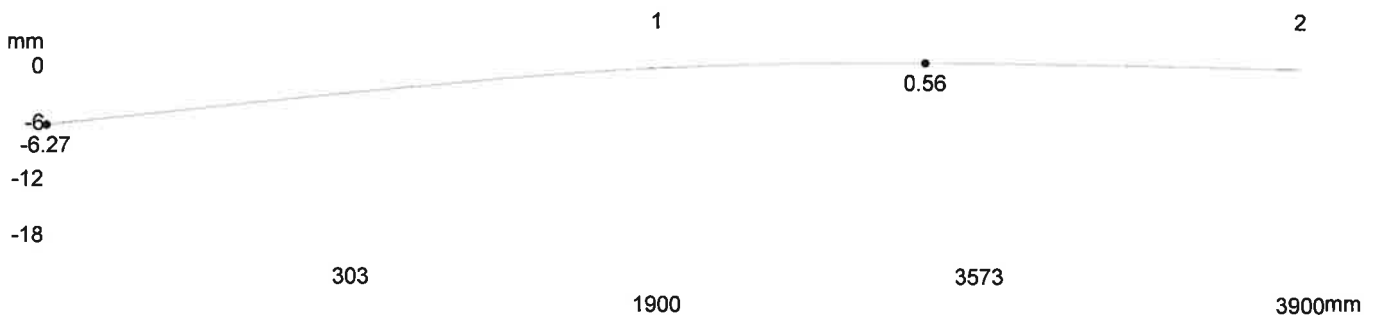


Short Term



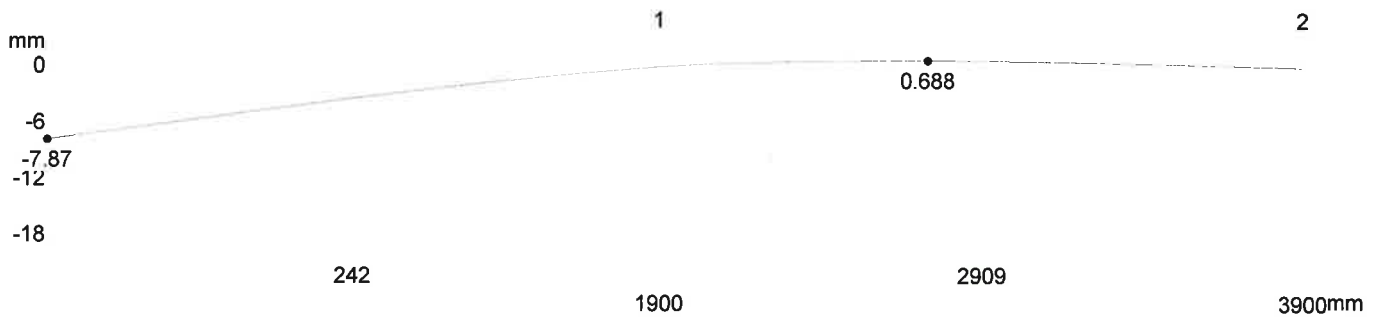
All Spans Loaded Odd Spans Loaded **Even Spans Loaded**

Incremental



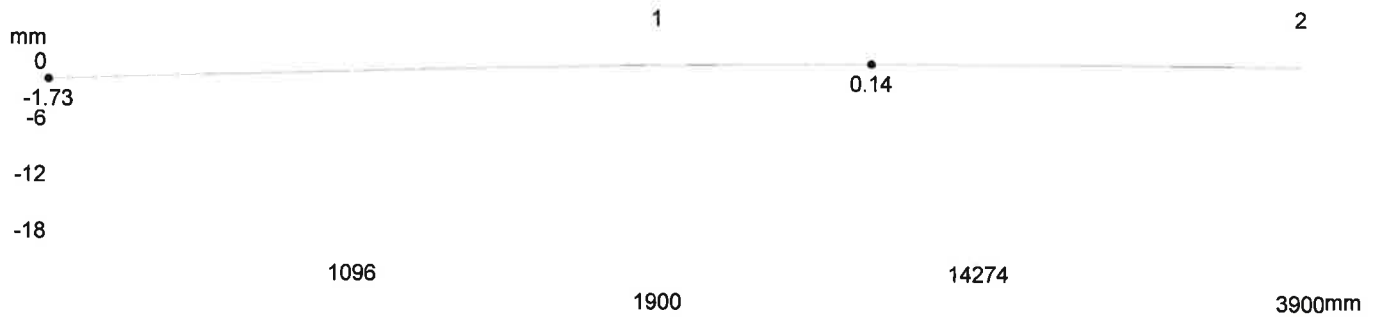
All Spans Loaded Odd Spans Loaded **Even Spans Loaded**

Total Long Term

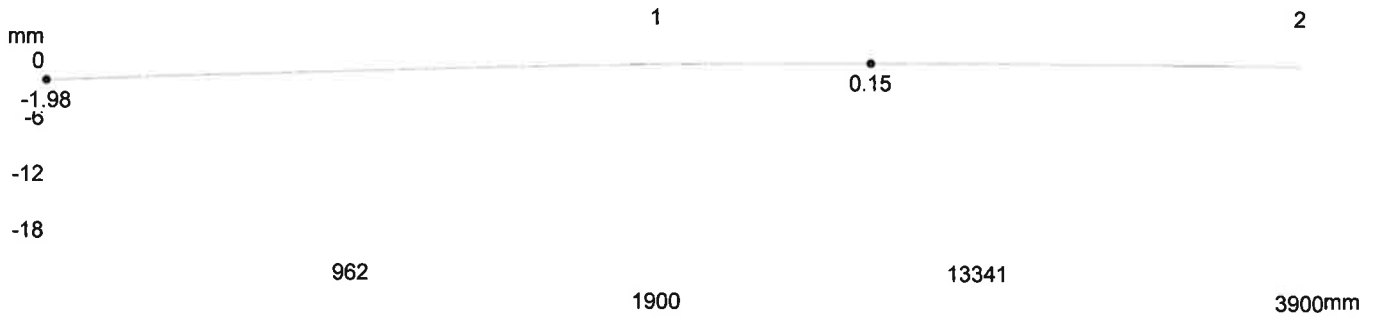


All Spans Loaded Odd Spans Loaded **Even Spans Loaded**

Odd Spans Loaded
Transfer

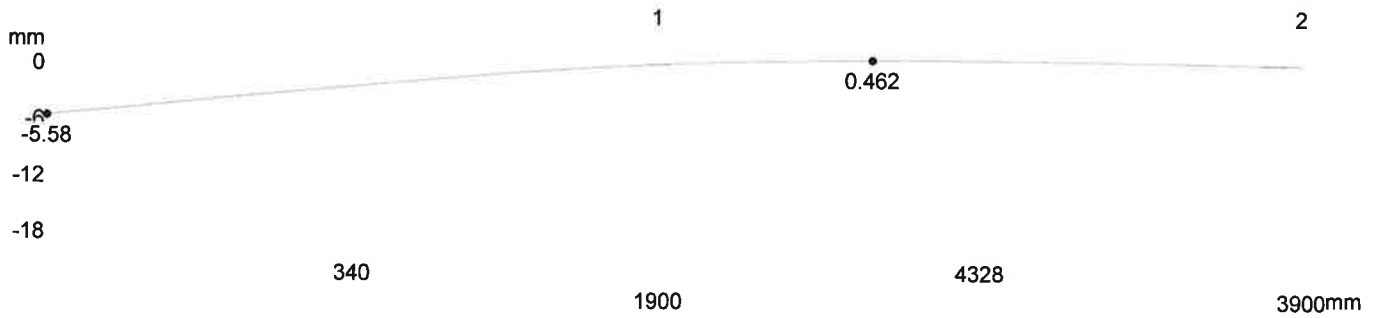


Short Term



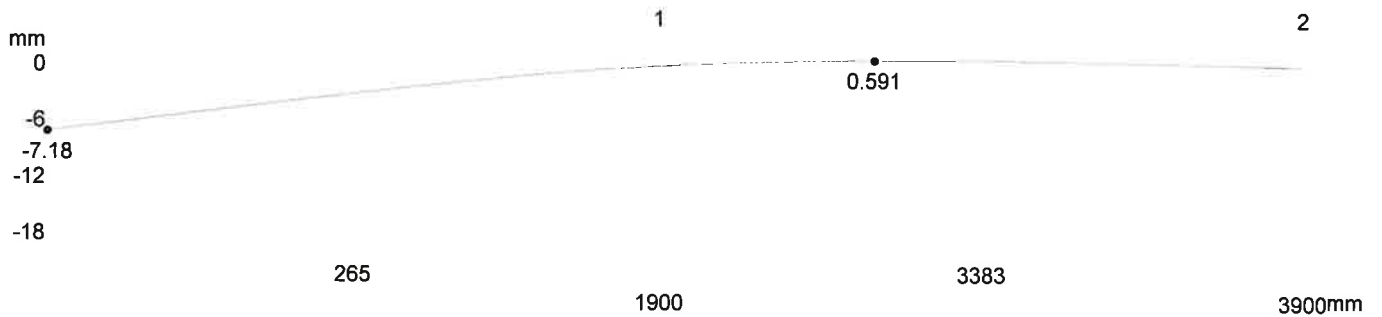
All Spans Loaded Even Spans Loaded **Odd Spans Loaded**

Incremental



All Spans Loaded Even Spans Loaded **Odd Spans Loaded**

Total Long Term



All Spans Loaded Even Spans Loaded **Odd Spans Loaded**

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SC94

\\tmk7\jobs\2017\10\1710168\Structural Drawings and Calcs\Design Calculations and Details\RR\Preliminary\Third floor balcony slab.rpf

RAPT - Version: 6.5.16.0

Reinforced And Post-Tensioned Concrete Analysis & Design Package

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Balcony Slab

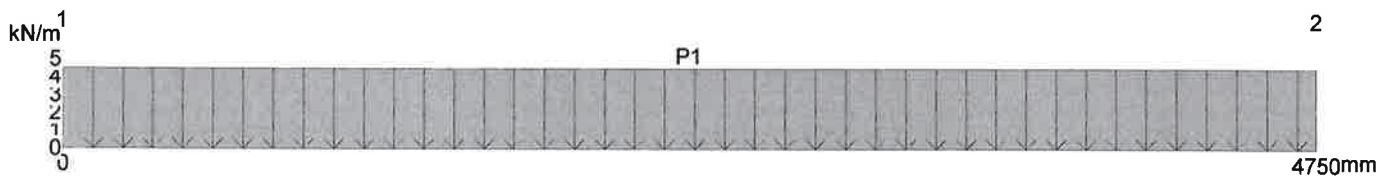
180 thick

N12 @ 200 ϕ each way
top and bottom.

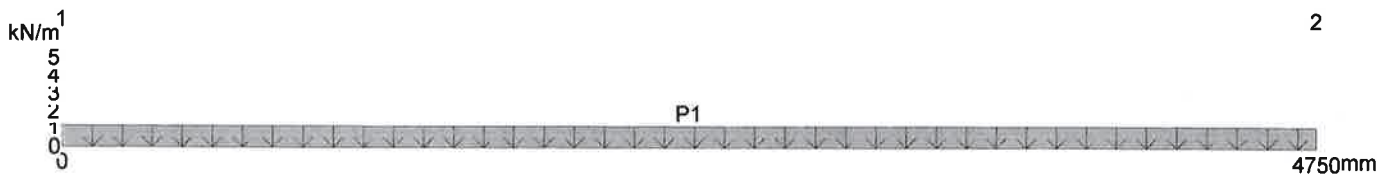
Licensee
TMK Consulting Engineers
Level 6
100 Pirie Street
Adelaide SA 5000
11169065160718WPN3

Input

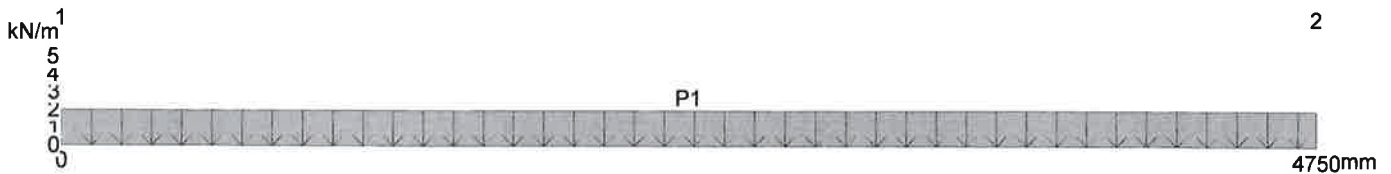
Load Case 1 : 1. Self Weight



Load Case 2 : 2. Extra Dead Load



Load Case 3 : 3. Live Load



Elevation view



Warnings

Input

No errors or warnings were found.

Output

Warning: Flat slabs - slabs which rely on flexural action through the slab or slab thickenings such as drop panels or band beams to discrete column supports, require concentration of the top reinforcement and tendons over supports within a defined width from the column face to facilitate moment transfer to the column. This width varies and the amount of reinforcement varies for different design codes. In the major codes, the requirements are specified in AS3600 cl 9.1.2, Eurocode 1992 - 1 - 1 2004 cl 9.4.1, ACI318-14 cl 8.4.2.3 and BS8110 cl 3.7.3.1. For example AS3600 requires that 25% of total negative moment for the entire slab panel be resisted by reinforcement within a width D either side of the column. RAPT cannot define this concentration as it does not know the transverse spacing of the tendons. It is the designer's responsibility to ensure that these clauses are complied with.

Bending Moments

Load Cases

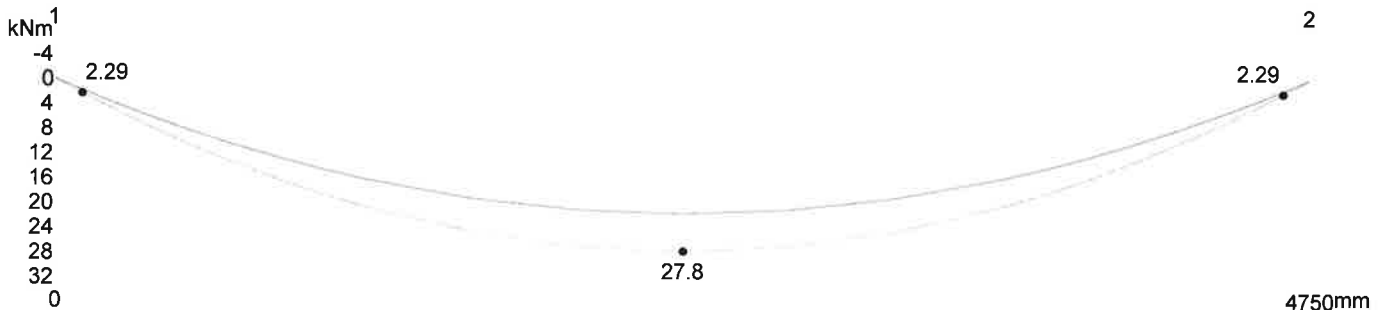
Column Actions

Col No. 1		Self Weight	Extra Dead Load	Live Load
Moment Above	kNm	-0	-0	-0
Moment Below	kNm	-0	-0	-0
Reaction	kN	10.69	2.85	4.75
Elastic Rotation	##	1.3e-3	3.47e-4	5.79e-4
Elastic Axial Shortening	mm	0	0	0

Col No. 2		Self Weight	Extra Dead Load	Live Load
Moment Above	kNm	-0	-0	-0
Moment Below	kNm	-0	-0	-0
Reaction	kN	10.69	2.85	4.75
Elastic Rotation	##	-1.3e-3	-3.47e-4	-5.79e-4
Elastic Axial Shortening	mm	0	0	0

Load Combinations

Ultimate Flexure



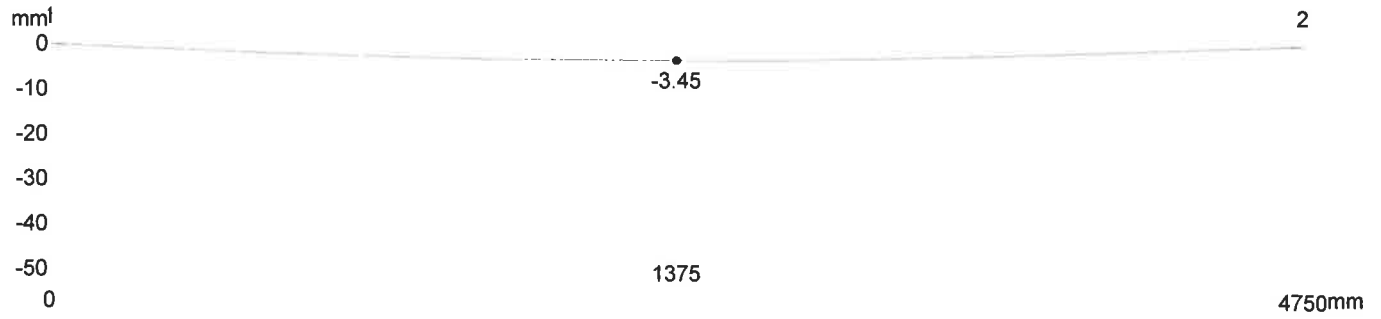
Moment Moment 1 Moment 2



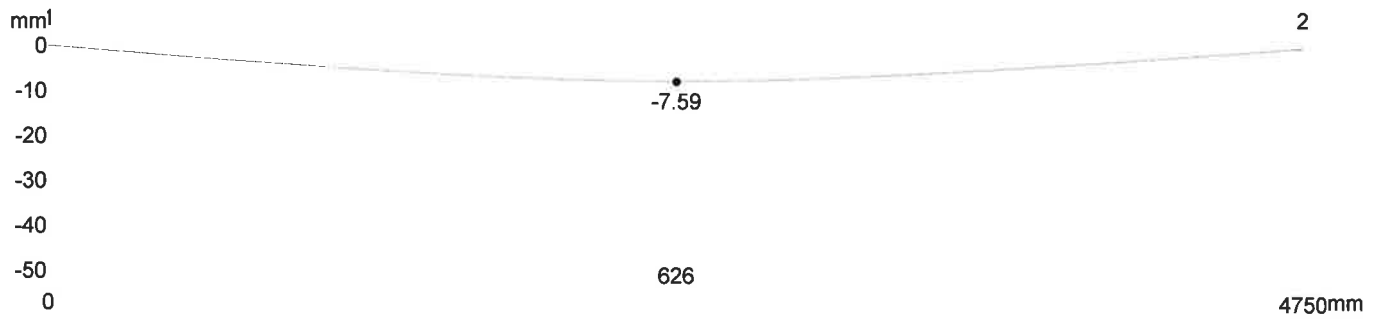
Shear Shear 1 Shear 2

Deflections

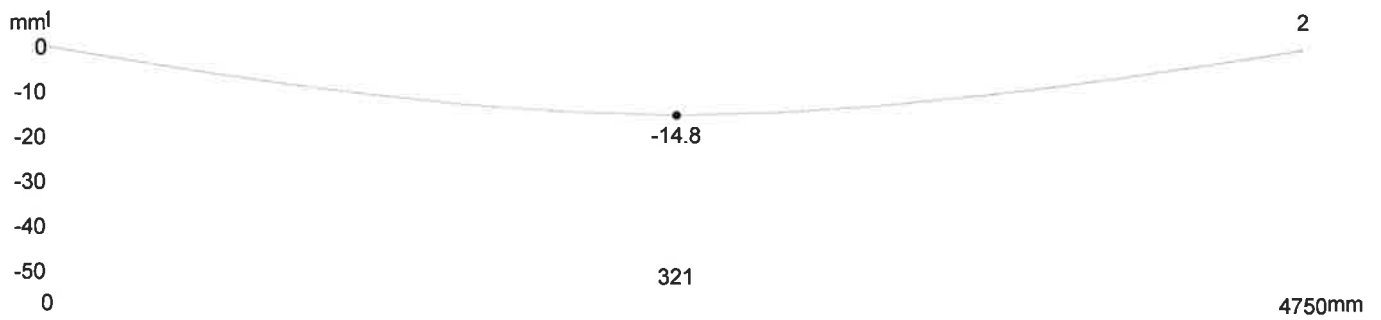
All Spans Loaded Transfer



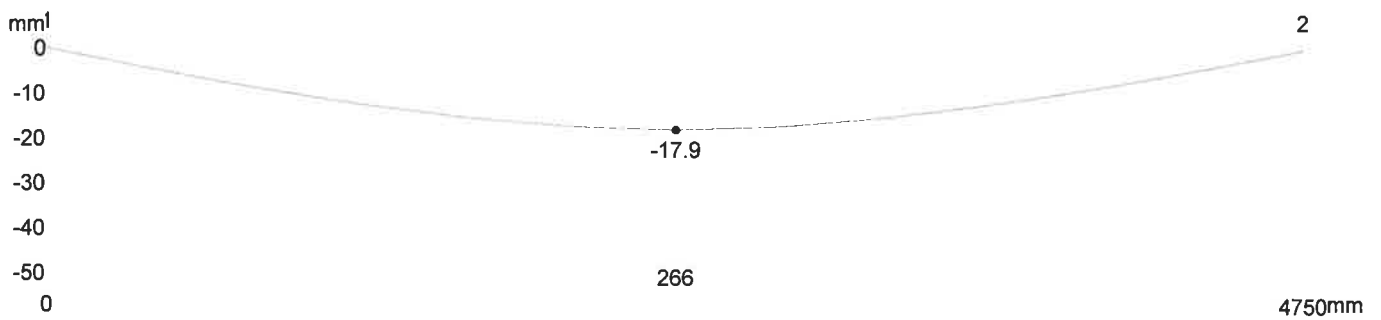
Short Term



Incremental

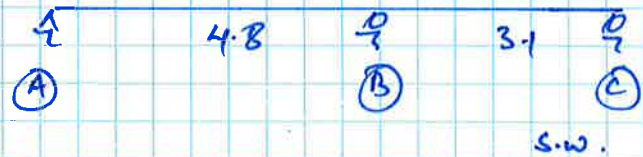


Total Long Term



Floor beam 3FB1.

loading width = $\frac{2}{2} = 1\text{m}$.
take 1.2m because of adjacent topping slab.



$$DL = 0.18 \times 24 \times 1.2 + 1 \times 1.2 = 6.4 \text{ kN/m} + 0.6 = 7 \text{ kN/m}$$

$$LL = 2 \times 1.2 = 2.4 \text{ kN/m}$$

Analysis from microstran.

expected d.l. deflection with 200 UB 22 = 7mm.

ie $\frac{L}{685}$ (OK)

$\Rightarrow w_{serv} = 8\text{mm}$ (OK)

$$M_{max} = \begin{aligned} +ve &= 22.4 \text{ kNm} \\ -ve &= 26.7 \text{ kNm} \end{aligned}$$

200 UB 22 (OK)

$$\begin{aligned} \text{Reaction on } N^* \Rightarrow A &= 24 \text{ kN} \\ B &= 62 \text{ kN} \\ \& C &= 10 \text{ kN} \end{aligned}$$

Column @ B. (2C1)

$$N^* = 62 \text{ kN} \sim \text{say } 80 \text{ kN} \& M^* = 80 \times 0.05 = 4 \text{ kNm}$$

left = 3.2m.

Adopt 83x6 SHS \rightarrow refer to cal's on spread sheet

Checked :

Date :/..../..

Envelope for Moment Mz

Maximum

Minimum

Envelope Cases:

3 C 1.2d+1.5l

26.7 26.7

4.15

22.4

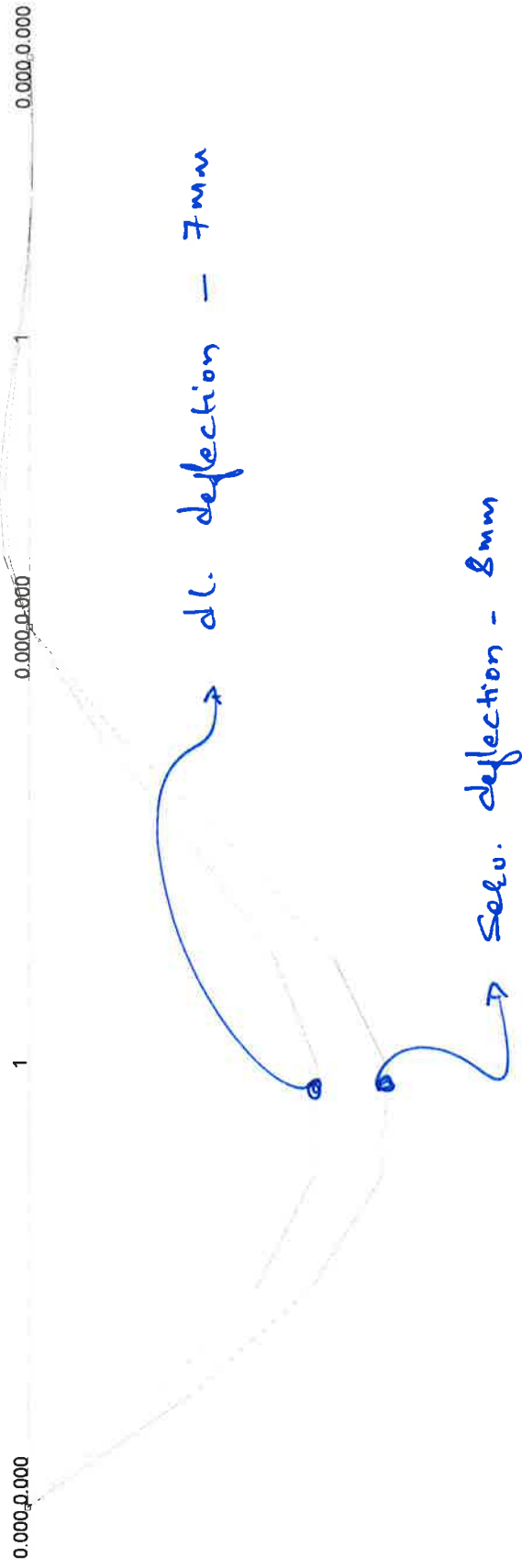
Y
Z
X

theta: 270 phi: 0

Bending Moment, Mz

Load Cases:
—— 1 P dl
—— 4 C dl+.7ll

Sections:
—— 1 200UB22.3 Y



Y
Z
X
theta: 270 phi: 0

Displaced Shape



Ref.: 1710168

Date: 13-Jul-18

Design: RR

Page: sc100

COLUMN DESIGN - SHS SECTIONS - PINNED TOP & PINNED BASE

Column 2c1.

These calculations comply with the requirements of AS 4100 - 1998 Steel Structures.

$N^* =$	80.0 kN	vertical compression load (strength factored load)
$l_h =$	3200 mm	column height
$k_e =$	1.00	effective length factor (Clause 4.6.3)
$e =$	50 mm	applied load eccentricity at the top of the column
$\Rightarrow M^* =$	4.00 kNm	

Column material yield stress

☒ 350 MPa ☐ 450 MPa

Trial column size : 89x89x3.5 SHS (C350)

$A_n = A_g =$	1150 mm ²	$f_y =$	350 MPa	$k_f =$	1.0
$Z_e =$	36.5E+3 mm ³	$r_x = r_y =$	34.5 mm	$S_x =$	36.5E+3 mm ³
$I_x = I_y =$	1.4E+6 mm ⁴	$J =$	2.2E+6 mm ⁴	$b/t =$	22.4

Check member capacity

Capacity factors

$\phi_b =$ 0.9 Table 3.4 - bending $\phi_c =$ 0.9 Table 3.4 - compression

For cold-formed (non-stress relieved) SHS, $\alpha_b =$ -0.5 Table 6.3.3(2)

(a) Nominal section capacity in compression

Clause 6.1

$$N_s = k_f \cdot A_n \cdot f_y = 402.5 \text{ kN}$$

$$\phi_c \cdot N_s = 362.3 \text{ kN}$$

> 80 kN Required :OK

(b) Nominal member capacity in compression

Clause 6.3

$\lambda_n =$	109.7	$\alpha_a =$	16.28	$\lambda =$	101.61
$\eta =$	0.287	$\xi =$	1.005	$\alpha_c =$	0.530

Clause 6.3.3

$$N_c = \alpha_c \cdot N_s \leq N_s = 213.4 \text{ kN}$$

$$\phi_c \cdot N_c = 192.1 \text{ kN}$$

> 80 kN Required :OK

(c) Nominal section capacity for combined bending and compression

$$M_{sx} = f_y \cdot Z_e = 12.8 \text{ kNm}$$

Clause 5.2.1

$$M_{rx} = 11.7 \text{ kNm}$$

Clause 8.3.2(a) and (b)

$$\phi_b \cdot M_{rx} = 10.6 \text{ kNm}$$

> 4 kNm Required :OK

(d) Nominal member capacity for combined bending and compression

$$M_i = 7.5 \text{ kNm}$$

Clause 8.4.2.2

$$\phi_b \cdot M_i = 6.7 \text{ kNm}$$

> 4 kNm Required :OK

\Rightarrow Column is Satisfactory in Combined Bending and Compression

ADOPT 89x89x3.5 SHS (C350)

Type recommendation in these two lines



Ref.: 1710168

Date: 13-Jul-18

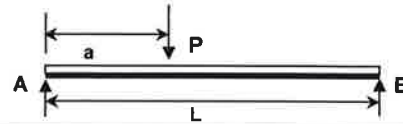
Design: RR

Page: SC101

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member 3FB2 Design span, L = 4800 mm
Effective length of lateral restraint, l_e = 1000 mm
Distance, a, from LH Support to Point Load = 0 mm



Load Type	UDL	Width	
1a. Uniform DL	0.40 kPa	x 0.00 m =	0.00 kN/m
From RAPT			21.00 kN/m
Self Weight			0.31 kN/m
		Total	21.31 kN/m
1b. Point DL	Description	Total	0.00 kN
2a. Uniform LL	0.25 kPa	x 0.00 m =	0.00 kN/m
From RAPT			8.00 kN/m
		Total	8.00 kN/m
2b. Point LL	Description	Total	0.00 kN
3a. Uniform WL (see note)	0.00 kPa	x 0.00 m =	0.00 kN/m
Other WL			0.00 kN/m
		Total	0.00 kN/m
3b. Point WL	Description	Total	0.00 kN

STRENGTH DESIGN
Combination of Load Types **

- ☒ DL + LL
☐ DL + LL + WL
☐ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness	(Refer to Combination of Load Types for Member Strength Calculations)		
$W^* = 37.58$ kN/m	$P^* = 0.00$ kN	$M^* = 108.22$ kNm	
$R_A^* = 90.18$ kN	$R_B^* = 90.18$ kN	$I_{req,DL} = 61.4E+6$ mm ⁴	← Governs
$W_{s,DL+0.7LL} = 26.91$ kN/m	$P_{s,DL+0.7LL} = 0.00$ kN	$I_{req,DL+0.7LL} = 51.7E+6$ mm ⁴	
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 000.0E+0$ mm ⁴	

Trial Section

Trial beam size :	310 UB 32
Depth of section, d =	298 mm
Flange width, b_f =	149 mm
Flange thickness, t_f =	8 mm
Web thickness, t_w =	5.5 mm
Section area, A_g =	4080 mm ²
$I_x = 63.2E+6$ mm ⁴	∴ OK
$Z_x = 424E+3$ mm ³	
$S_x = 475E+3$ mm ³	
$r_x = 124$ mm	
$I_y = 4.42E+6$ mm ⁴	
$Z_y = 59.3E+3$ mm ³	
$S_y = 91.8E+3$ mm ³	
$r_y = 32.9$ mm	
J = 86.5E+3 mm ⁴	
$I_w = 92.9E+9$ mm ⁶	
Flange, f_y =	320 MPa
Web, f_y =	320 MPa
$k_f = 0.92$	
$Z_{ex} = 467E+3$ mm ³	
$Z_{ey} = 86.9E+3$ mm ³	
Compactness =	N

Material Properties

E = 200E+3 MPa
G = 76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$ Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{ex})$

$M_{sx} = 149.4$ kNm (Clause 5.2.1)
 $\phi M_{sx} = 134.5$ kNm > 108.22 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$ Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m = 0.00$
 $\alpha_m = 1.13$ AS 4100 Table 5.6.1
 $k_t = 1.00$ Table 5.6.3(1)
 $k_l = 1.40$ Table 5.6.3(2)
 $k_r = 1.00$ Table 5.6.3(3)
 $l_e = 1400$ mm
 $M_o = 667.9$ kNm (Eq. 6.5.1.1(3))
 $\alpha_s = 0.91$ Eq. 5.6.1.1(2)
 $M_{bx} = 154.3$ kNm (Eq. 5.6.1.1(1))
 $\phi M_{bx} = 134.5$ kNm > 108.22 kNm ∴ OK

ADOPT

310 UB 32

Restrain top flange laterally at 1000 mm centres (maximum)

Alternately Adopt 300 PFL
 $M = 144$ kNm > Req (OK)
Available = 72.4 x 106 mm⁴.



Ref.: 1710168

Date: 07-Oct-18

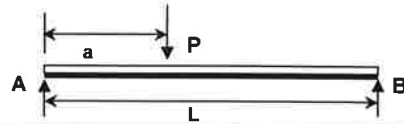
Design: RR

Page: 5C102

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 — 1998 Steel Structures

Member 3FB3 Design span, $L = 2000$ mm
Effective length of lateral restraint, $I_e = 2000$ mm
Distance, a , from LH Support to Point Load = 0 mm



Load Type	UDL	Width		
1a. Uniform DL	6.06 kPa	x 3.90 m =	23.63	kN/m
Description			0.00	kN/m
Self Weight			0.18	kN/m
		Total	23.81	kN/m
1b. Point DL			0.00	kN
Description		Total	0.00	kN
2a. Uniform LL	2.00 kPa	x 3.90 m =	7.80	kN/m
Description			0.00	kN/m
		Total	7.80	kN/m
2b. Point LL			0.00	kN
Description		Total	0.00	kN
3a. Uniform WL (see note)	0.00 kPa	x 0.00 m =	0.00	kN/m
Other WL			0.00	kN/m
		Total	0.00	kN/m
3b. Point WL			0.00	kN
Description		Total	0.00	kN

STRENGTH DESIGN
Combination of Load Types **

- ☒ DL + LL
☐ DL + LL + WL
☐ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness

(Refer to Combination of Load Types for Member Strength Calculations)

$W^* = 40.27$ kN/m	$P^* = 0.00$ kN	$M^* = 20.14$ kNm	
$R_A^* = 40.27$ kN	$R_B^* = 40.27$ kN	$I_{req,DL} = 4.5E+6$ mm ⁴	← Governs
$W_{s,DL+0.7LL} = 29.27$ kN/m	$P_{s,DL+0.7LL} = 0.00$ kN	$I_{req,DL+0.7LL} = 3.8E+6$ mm ⁴	
$W_{s,WL} = 0.00$ kN/m	$P_{s,WL} = 0.00$ kN	$I_{req,WL} = 0.00E+0$ mm ⁴	

Trial Section

Trial beam size : 180 UB 18.1

Depth of section, $d = 175$ mm	
Flange width, $b_f = 90$ mm	
Flange thickness, $t_f = 8$ mm	
Web thickness, $t_w = 5$ mm	
Section area, $A_g = 2300$ mm ²	
$I_x = 12.1E+6$ mm ⁴	∴ OK
$Z_x = 139E+3$ mm ³	
$S_x = 157E+3$ mm ³	
$r_x = 72.6$ mm	
$I_y = 975.00E+3$ mm ⁴	
$Z_y = 21.7E+3$ mm ³	
$S_y = 33.7E+3$ mm ³	
$r_y = 20.6$ mm	
$J = 44.8E+3$ mm ⁴	
$I_w = 6.8E+9$ mm ⁶	
Flange, $f_y = 320$ MPa	
Web, $f_y = 320$ MPa	
$k_f = 1.00$	
$Z_{ex} = 157E+3$ mm ³	
$Z_{ey} = 32.5E+3$ mm ³	
Compactness = C	

Material Properties

$E = 200E+3$ MPa
$G = 76.9E+3$ MPa

Section Capacity -- $M^* \leq \phi M_{sx}$ Check $M^* \leq \phi M_{sx} (= \phi f_y Z_{ex})$

$M_{sx} = 50.2$ kNm (Clause 5.2.1)
$\phi M_{sx} = 45.2$ kNm > 20.14 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$ Check $M^* \leq \phi M_{bx} (= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx})$

$\beta_m = 0.00$	AS 4100 Table 5.6.1
$\alpha_m = 1.13$	
$k_t = 1.00$	Table 5.6.3(1)
$k_l = 1.40$	Table 5.6.3(2)
$k_r = 1.00$	Table 5.6.3(3)
$I_e = 2800$ mm	
$M_o = 35.6$ kNm (Eq. 6.5.1.1(3))	
$\alpha_s = 0.49$	Eq. 5.6.1.1(2)
$M_{bx} = 28.0$ kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} = 25.2$ kNm > 20.14 kNm ∴ OK	

ADOPT

180 UB 18.1

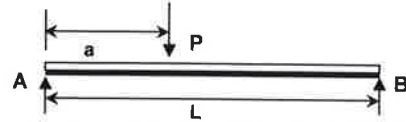
Restrain top flange laterally at 2000 mm centres (maximum)

Alt Use 200PFC

STEEL BEAM DESIGN

These calculations comply with the requirements of AS4100 – 1998 Steel Structures

Member 3FB4 Design span, L = 4000 mm
Effective length of lateral restraint, l_e = 1000 mm
Distance, a, from LH Support to Point Load = 0 mm



Load Type	UDL	Width		
1a. Uniform DL	6.06 kPa	x 4.70 m =	28.48 kN/m	
concrete wall ((.15x24)x3.6)			12.96 kN/m	
Self Weight			0.31 kN/m	
		Total	41.76 kN/m	
1b. Point DL	Description	Total	0.00 kN	
2a. Uniform LL	2.00 kPa	x 4.70 m =	9.40 kN/m	
Description		Total	0.00 kN/m	
		Total	9.40 kN/m	
2b. Point LL	Description	Total	0.00 kN	
3a. Uniform WL (see note)	0.00 kPa	x 0.00 m =	0.00 kN/m	
Other WL		Total	0.00 kN/m	
		Total	0.00 kN/m	
3b. Point WL	Description	Total	0.00 kN	

STRENGTH DESIGN Combination of Load Types **

- ☒ DL + LL
- ☐ DL + LL + WL
- ☐ DL + WL

*Note: For WL, 'positive' loads act downward and 'negative' loads act upward

** AS/NZS 1170.0:2002

Deflection Limits	Span:Deflection Ratio	Maximum Deflection (mm)
1. Dead Load	360	12
2. Dead Load + 0.7 * Live Load	250	18
3. Wind Load Only	200	20

Bending Moments and Stiffness

(Refer to Combination of Load Types for Member Strength Calculations)

$W^* =$ 64.21 kN/m	$P^* =$ 0.00 kN	$M^* =$ 128.41 kNm	
$R_A^* =$ 128.41 kN	$R_B^* =$ 128.41 kN	$I_{req,DL} =$ 62.6E+6 mm ⁴	← Governs
$W_{s,DL+0.7LL} =$ 48.34 kN/m	$P_{s,DL+0.7LL} =$ 0.00 kN	$I_{req,DL+0.7LL} =$ 50.3E+6 mm ⁴	
$W_{s,WL} =$ 0.00 kN/m	$P_{s,WL} =$ 0.00 kN	$I_{req,WL} =$ 000.0E+0 mm ⁴	

Trial Section

Trial beam size :	310 UB 32	
Depth of section, d =	298 mm	
Flange width, b_f =	149 mm	
Flange thickness, t_f =	8 mm	
Web thickness, t_w =	5.5 mm	
Section area, A_g =	4080 mm ²	
$I_x =$ 63.2E+6 mm ⁴		∴ OK
$Z_x =$ 424E+3 mm ³		
$S_x =$ 475E+3 mm ³		
$r_x =$ 124 mm		
$I_y =$ 4.42E+6 mm ⁴		
$Z_y =$ 59.3E+3 mm ³		
$S_y =$ 91.8E+3 mm ³		
$r_y =$ 32.9 mm		
J = 86.5E+3 mm ⁴		
$I_w =$ 92.9E+9 mm ⁶		
Flange, f_y =	320 MPa	
Web, f_y =	320 MPa	
$k_f =$ 0.92		
$Z_{ex} =$ 467E+3 mm ³		
$Z_{ey} =$ 86.9E+3 mm ³		
Compactness =	N	

Material Properties

E =	200E+3 MPa
G =	76.9E+3 MPa

Section Capacity -- $M^* \leq \phi M_{sx}$

Check $M^* \leq \phi M_{sx}$ ($= \phi f_y Z_{ex}$)

$M_{sx} =$	149.4 kNm (Clause 5.2.1)
$\phi M_{sx} =$	134.5 kNm > 128.41 kNm ∴ OK

Member Capacity -- $M^* \leq \phi M_{bx}$

Check $M^* \leq \phi M_{bx}$ ($= \phi \alpha_m \alpha_s M_{sx} \leq \phi M_{sx}$)

$\beta_m =$	0.00	AS 4100 Table 5.6.1
$\alpha_m =$	1.13	
$k_1 =$	1.00	Table 5.6.3(1)
$k_2 =$	1.40	Table 5.6.3(2)
$k_r =$	1.00	Table 5.6.3(3)
$l_e =$	1400 mm	
$M_o =$	667.9 kNm (Eq. 6.5.1.1(3))	
$\alpha_s =$	0.91	Eq. 5.6.1.1(2)
$M_{bx} =$	154.3 kNm (Eq. 5.6.1.1(1))	
$\phi M_{bx} =$	134.5 kNm > 128.41 kNm ∴ OK	

ADOPT

310 UB 32

Restrain top flange laterally at 1000 mm centres (maximum)

Alt use 300 PFC.

$M = 144$ kN top restrain
 $I = 72.4 \times 10^6$ mm⁴
Available

Third Floor. (Carry beam 3CB1 & Column 2C2)

Load on (A)

floor beam 3FB1.

$$DL = 7 \text{ kN/m}$$

$$U = 2.4 \text{ kN/m}$$

Load on B.

floor beam 3FB2

$$DL = 22 \text{ kN/m}$$

$$U = 7.6 \text{ kN/m}$$

Analysis from microstran.

$$M_{\text{max on 3FB2}} = 10.6 \text{ kNm. Use 300 PFC}$$

$$\text{Expected d.l. deflection} = 12 \text{ mm i.e. } \frac{L}{400} \text{ (OK) (Total)}$$

Adopt 300 PFC.

$$\text{serv. deflection} = 16 \text{ mm i.e. } \frac{L}{300} \text{ (OK)}$$

$$M_{\text{max on 3CB1}} = 11.6 \text{ kNm. Use 300 PFC.}$$

$$\text{Expected d.l. deflection} = 4 \text{ mm i.e. } \frac{L}{250} \text{ on cantilever (OK)}$$

Adopt 300 PFC

$$\text{serv. deflection} = 5 \text{ mm i.e. } \frac{L}{200} \text{ (OK)}$$

Reactions: at (2)

$$R_{DL} = 133 \text{ kN.}$$

$$R_{UL} = 49 \text{ kN.}$$

Column 2C2.

$$N_k = 233.1 \text{ kN.}$$

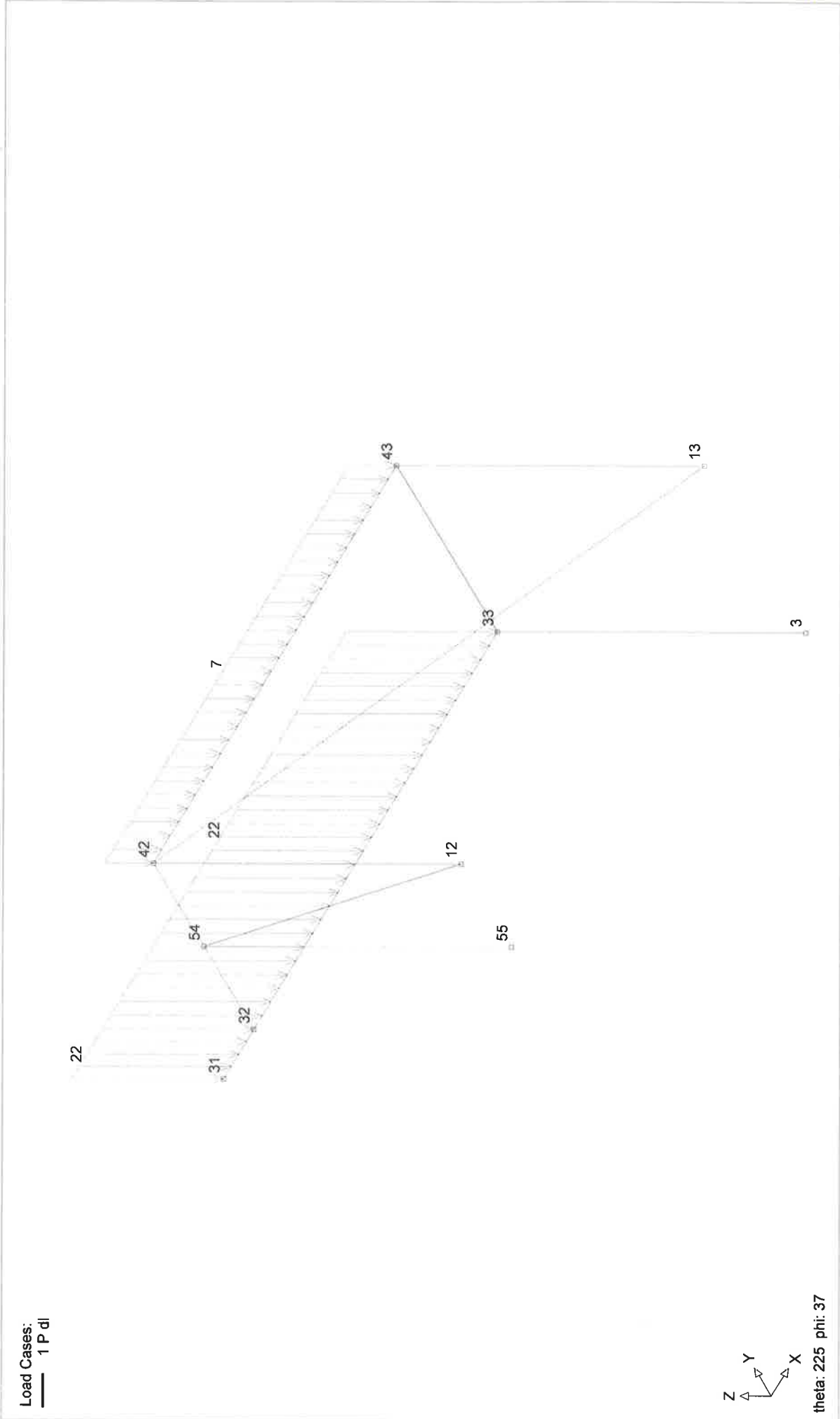
take 250 kN.

Design from spreadsheet

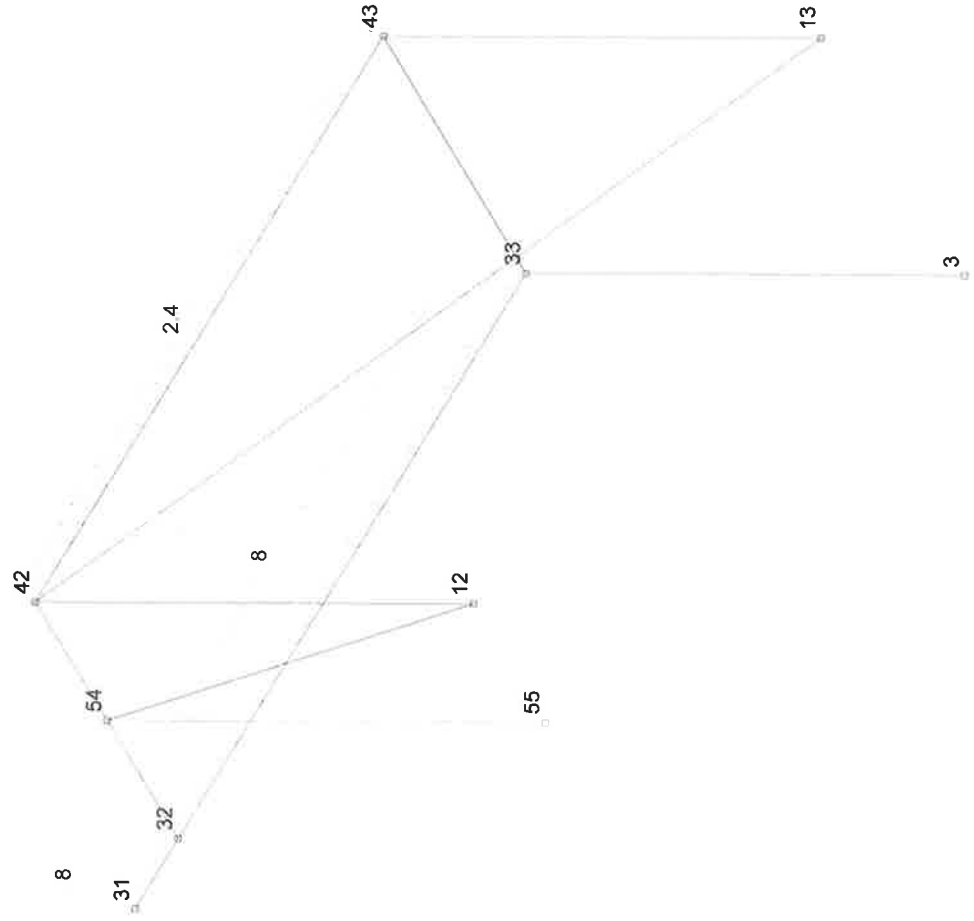
Adopt 100x5 SHS

Checked :

Date :/..../.....



Load Cases:
 2 P II



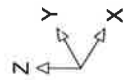
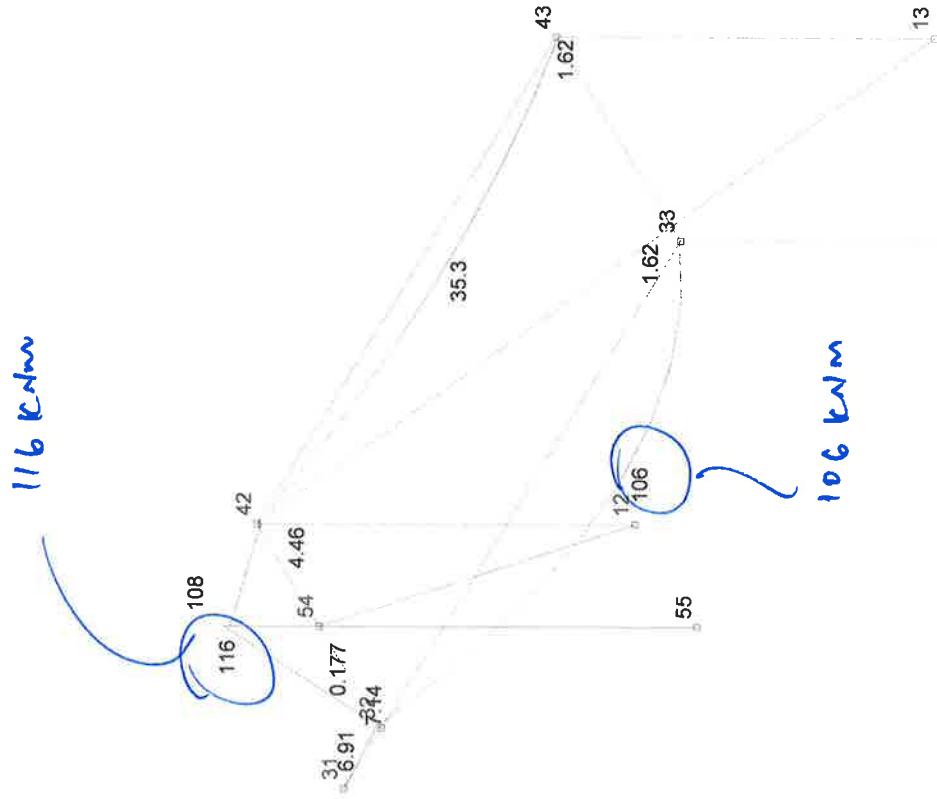
theta: 225 phi: 37

Microtran [V8.11r]

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1710168
 SC106

Envelope for Moment Mz
 — Maximum
 — Minimum
 Enveloped Cases:
 3 C1.2dl+1.5ll



theta: 225 phi: 37

Bending Moment, Mz

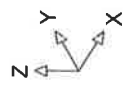
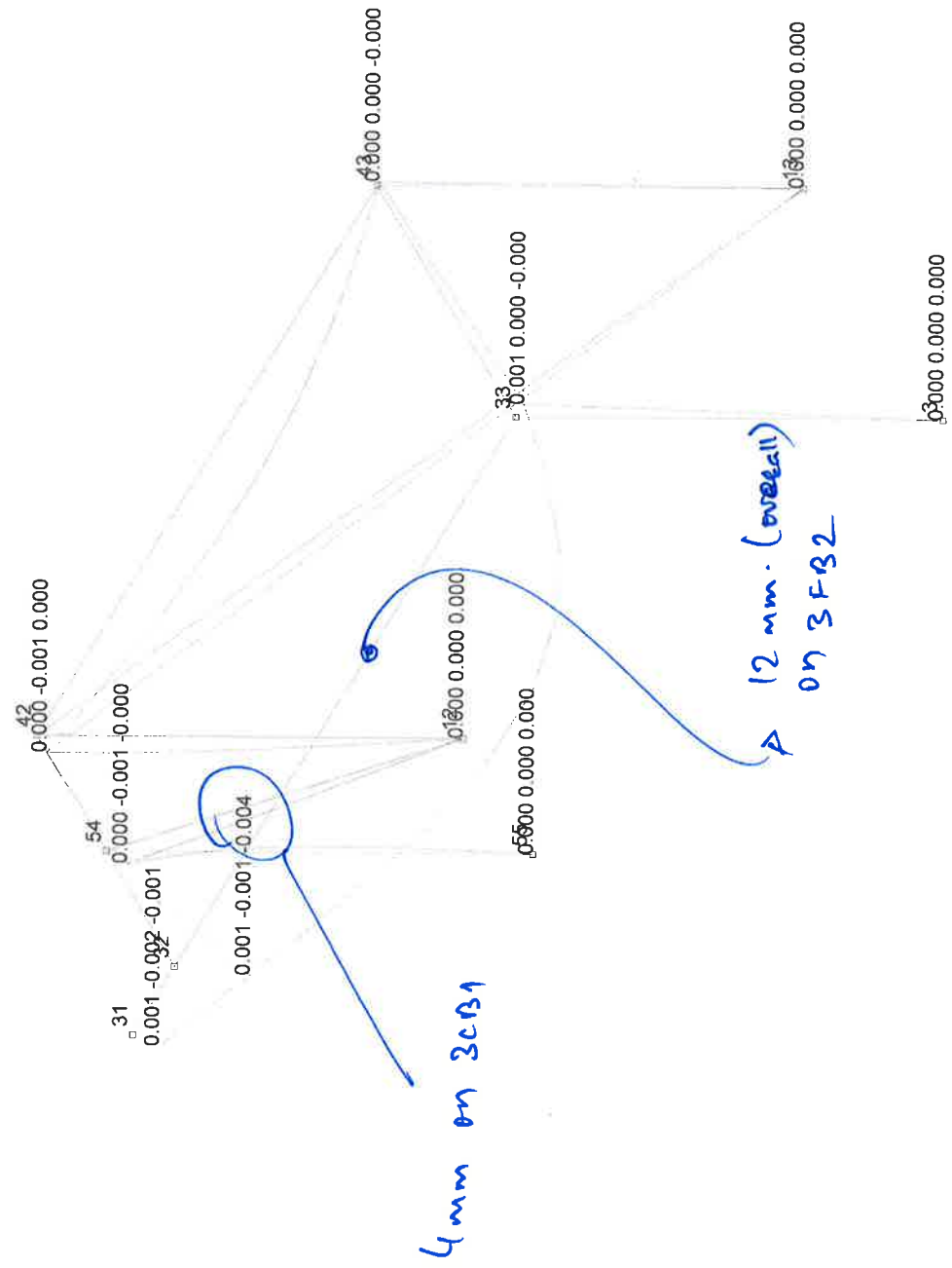
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Microstran [V8.11]

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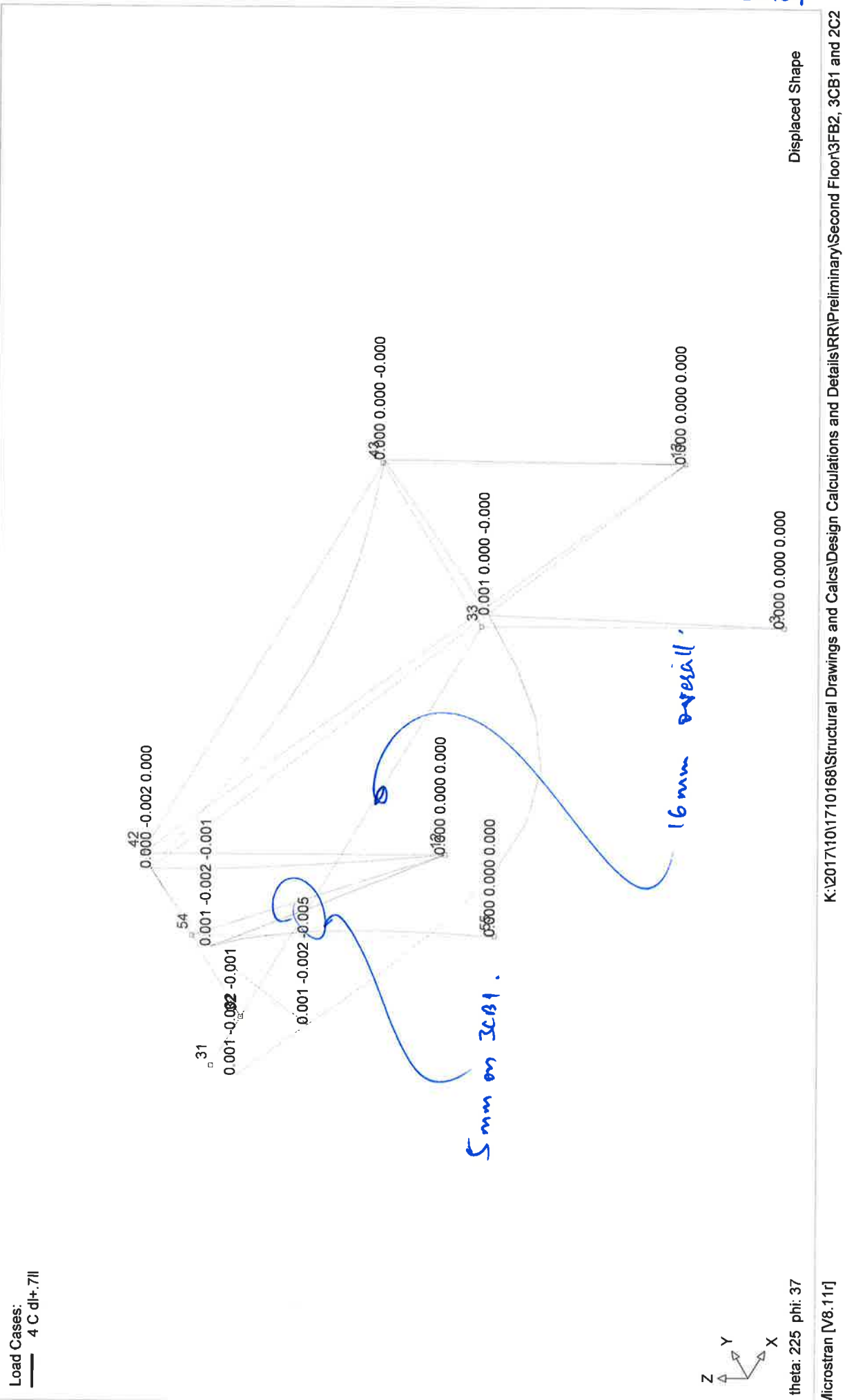
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 SC107

Load Cases:
 — 1 P dl

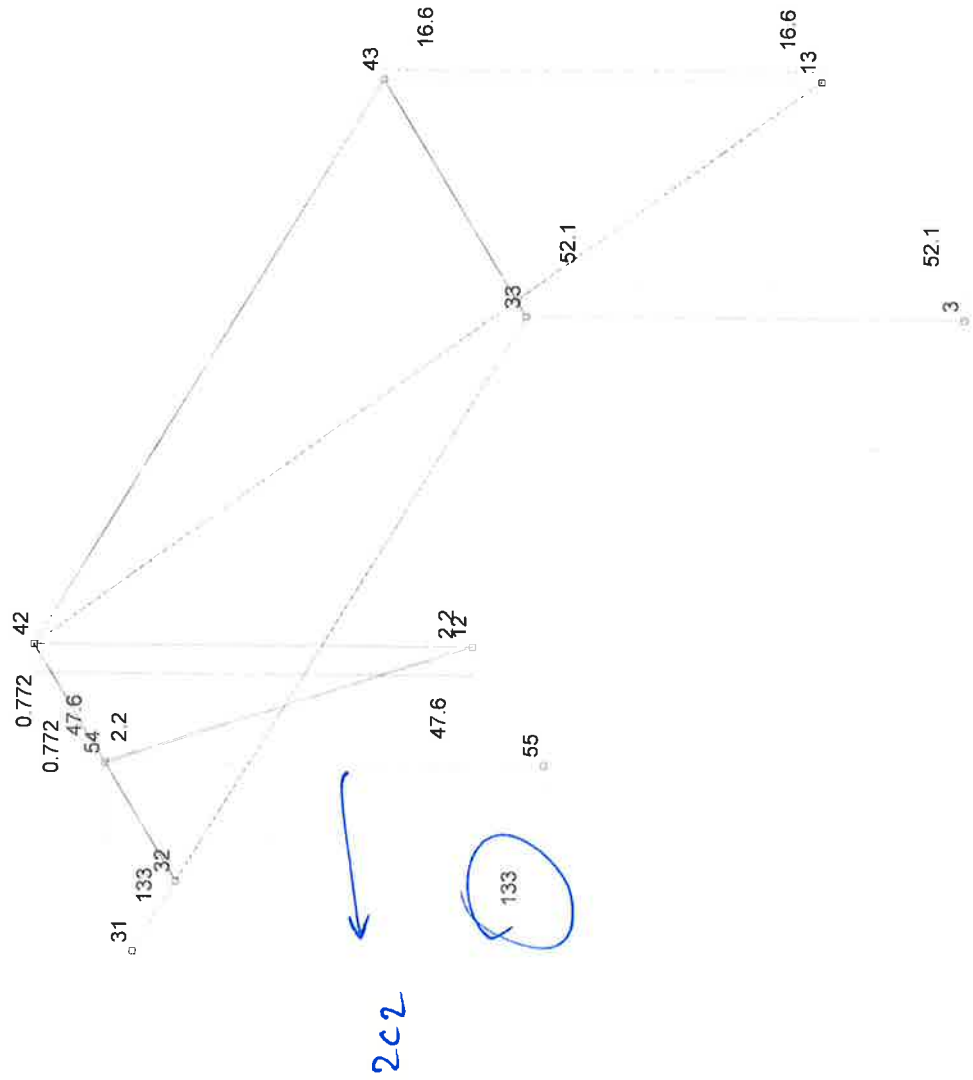


theta: 225 phi: 37

Displaced Shape



Load Cases:
 — 1 P dl



Z
 Y
 X

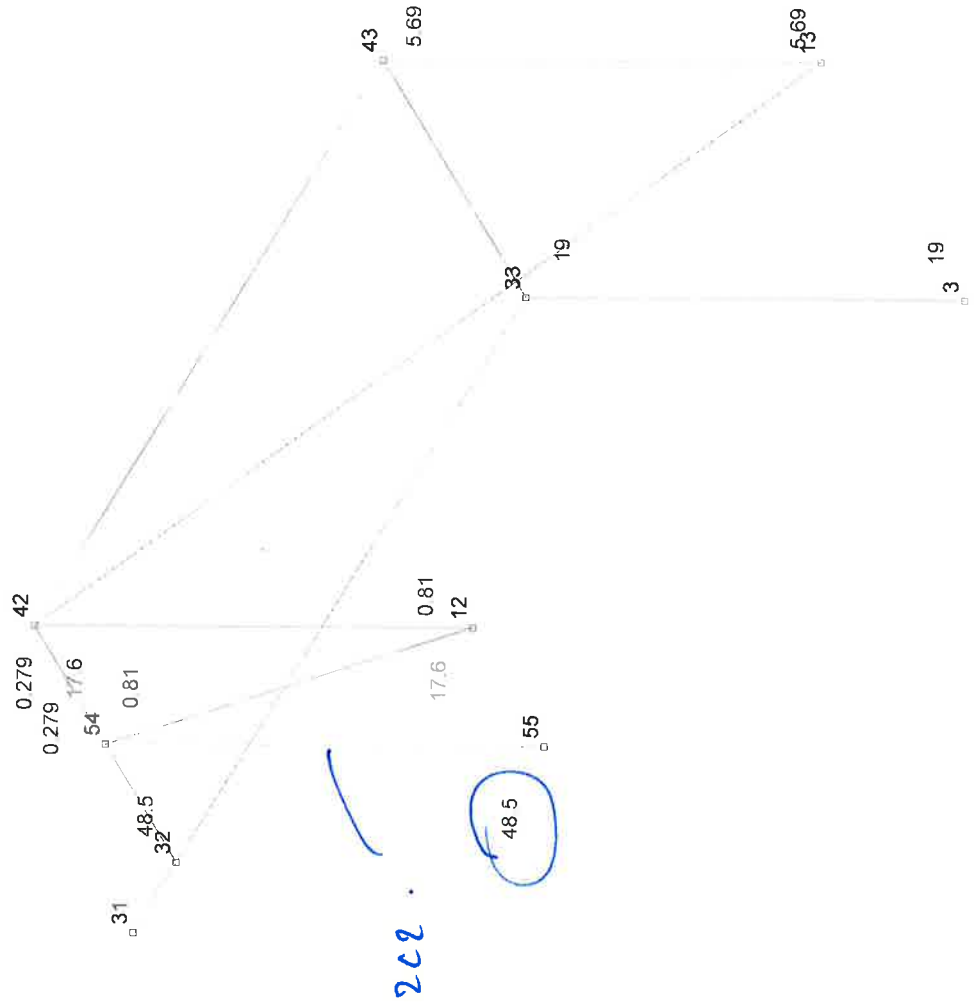
theta: 225 phi: 37

Axial Force, Fx

Microtran [V8.11r]

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Load Cases:
 2 P II



theta: 225 phi: 37

Microstran [V8.11r]

Axial Force, Fx

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Ref.: 1710168
Date: 07-Oct-18
Design: RR
Page: SC112

COLUMN DESIGN - SHS SECTIONS - PINNED TOP & PINNED BASE

Column 2C2.

These calculations comply with the requirements of AS 4100 - 1998 Steel Structures.

$N^* = 250.0$ kN vertical compression load (strength factored load)
 $l_h = 3000$ mm column height
 $k_e = 1.00$ effective length factor (Clause 4.6.3)
 $e = 50$ mm applied load eccentricity at the top of the column
 $\Rightarrow M^* = 12.50$ kNm

Column material yield stress

☒ 350 MPa ☐ 450 MPa

Trial column size :

100x100x9.0 SHS (C350)

$A_n = A_g = 3000$ mm ²	$f_y = 350$ MPa	$k_f = 1.0$
$Z_e = 98.6E+3$ mm ³	$r_x = r_y = 36.1$ mm	$S_x = 98.6E+3$ mm ³
$I_x = I_y = 3.9E+6$ mm ⁴	$J = 7.0E+6$ mm ⁴	$b/t = 8.1$

Check member capacity

Capacity factors

$\phi_b = 0.9$ Table 3.4 - bending $\phi_c = 0.9$ Table 3.4 - compression

For cold-formed (non-stress relieved) SHS, $\alpha_b = -0.5$ Table 6.3.3(2)

(a) Nominal section capacity in compression

Clause 6.1

$$N_s = k_f \cdot A_n \cdot f_y = 1050 \text{ kN}$$

$$\phi_c \cdot N_s = 945.0 \text{ kN}$$

> 250 kN Required ::OK

(b) Nominal member capacity in compression

Clause 6.3

$\lambda_n = 98.3$	$\alpha_a = 17.44$	$\lambda = 89.61$
$\eta = 0.248$	$\xi = 1.130$	$\alpha_c = 0.613$

Clause 6.3.3

$$N_c = \alpha_c \cdot N_s \leq N_s = 643.4 \text{ kN}$$

$$\phi_c \cdot N_c = 579.0 \text{ kN}$$

> 250 kN Required ::OK

(c) Nominal section capacity for combined bending and compression

$$M_{sx} = f_y \cdot Z_e = 34.5 \text{ kNm}$$

Clause 5.2.1

$$M_{rx} = 29.9 \text{ kNm}$$

Clause 8.3.2(a) and (b)

$$\phi_b \cdot M_{rx} = 27.0 \text{ kNm}$$

> 12.5 kNm Required ::OK

(d) Nominal member capacity for combined bending and compression

$$M_i = 19.6 \text{ kNm}$$

Clause 8.4.2.2

$$\phi_b \cdot M_i = 17.6 \text{ kNm}$$

> 12.5 kNm Required ::OK

\Rightarrow Column is Satisfactory in Combined Bending and Compression

ADOPT 100x100x9.0 SHS (C350)

Type recommendation in these two lines

Project Name:

Results

Lysaght Bondek Design Software Version 2.0

419 Regency RoadBondek Slab - 3rd Floor**Design Output**

Parameter	Notation	Spans		
		Single	End	Interior
Slab thickness	D (mm)	150		
Top (negative) reinforcement over supports:	A's (mm ²)	Not applicable		
Pattern of negative reinforcement		Not applicable		
Concrete cover	c (mm)	30		
Transverse (shrinkage/temperature effects) reinforcement, additional for D500L, total for D500N	Ashr (mm ²)	110		
Fire reinforcement (additional to shrinkage and negative reinforcement)	Afire (mm ²)	0		
Bottom tensile (positive) reinforcement (additional to Bondek sheeting)	A+,mid (mm ²)	0		
Number of temporary props		0		

Input parameters

Type of Buildings	Steel Frame	Negative Reinforcement Diameter	-
Span Configuration	Single Spans	Negative Reinforcement Grade	D500N
Continuous Spans	-		
Exposure Classification	A2	D	150
Ll/Ls	-		
Deflection Limits of Composite Slabs	Total <L/250	Bondek sheeting	0.75 mm
	NO		
L,eff, mm	1500	Q live load	3 kPa
Formwork Deflection Limits	Visual quality important	Gsdl superimposed dead load	0.5 kPa
		M weight of stacked materials	
Formwork sheets continue over number of spans	Single span	construction stage 1	4 kPa
		ψs	0.7
Crack control for shrinkage and temperature effects	Moderate	ψl	0.4
		Fire Design	Not required
Crack control for flexure	-	Fire Resistance Periods	0min
f _c	32 MPa		
Shrinkage Reinforcement Grade	D500L	Fire Reinforcement Options	-
Mesh or transverse bar diameter, mm	SL82		
		Positive and Fire Reinforcement bar diameter, mm	12 mm
Mesh longitudinal bar diameter, mm	-	Environment for shrinkage	Other
		Cover to top reinforcement, mm	30
Mesh longitudinal bar spacing, mm	-	Support width, mm	100

Second Floor

DELTCORE Floor:

No line load from load bearing wall

∴ Take DC1 → DC200.12 (0.5)

Refer to previous calculation i.e. third floor.

OFF FORM Slab:

Same as third floor.

180 thick with $N12 @ 200\frac{1}{2}$ each way top and bottom.

Bonded Slab:

150 thick → same as 2nd floor.

Floor beams; 2FB1, 2FB3 and 2FB4 → Same as 3rd floor.

Carry beam 2CB1 → Same as 3CB1 of third floor

Column reactions same as above

$$\text{ie } R_{DL} = 133 \text{ kN}$$

$$R_{LL} = 49 \text{ kN}$$

$$\text{For Column 1C2} \rightarrow N^* = (133 \times 1.2 + 49 \times 1.5) \times 2 \leftarrow \text{two floors}$$

$$= 466 \text{ kN} \sim 500 \text{ kN.}$$

Comparing this reaction with Reaction from 2CB2 for critical design

Checked :

Date :/..../..

Floor beam 2FB1.

P - Reaction from column
over

$$P_{DL} = 36 \text{ kN}, P_{LL} = 13 \text{ kN}$$

Line load.

$$dL = 7 \text{ kN/m}$$

$$LL = 2.4 \text{ kN/m}$$

Analysis from microstran.

$$\text{Max } +ve = 27.4 \text{ kNm}$$

$$-ve = 41.1 \text{ kNm}$$

try 200 vs 22

$$M_{+ve} = 52 \text{ kNm} > \text{req. (OK)}$$

$$M_{-ve} = 52 \text{ kNm} > \text{req. (OK)}$$

expected deflection $dL = 6 \text{ mm}$ i.e. $L/650$ (OK)

$w_{serv} = 8 \text{ mm}$ i.e. $L/487$ (OK)

Adopt 200 vs 22.

Reaction

	dL	LL
R_A	15 kN	6 kN
R_B	69 kN	25 kN
R_C	9 kN	3 kN

Column design.

$$1 \text{ Cl} \rightarrow N^* = 69 \times 1.2 + 25 \times 1.5 = 120.3 \text{ kN}$$

Design from spread sheet \rightarrow Adopt 100 x 5 SMC

Load Cases:
— 1 P dl

Sections:
— 1 200UB22.3 Y



Y
Z
X
theta: 270 phi: 0

Load Cases:
2 P II

Sections:
1 200UB22.3 Y



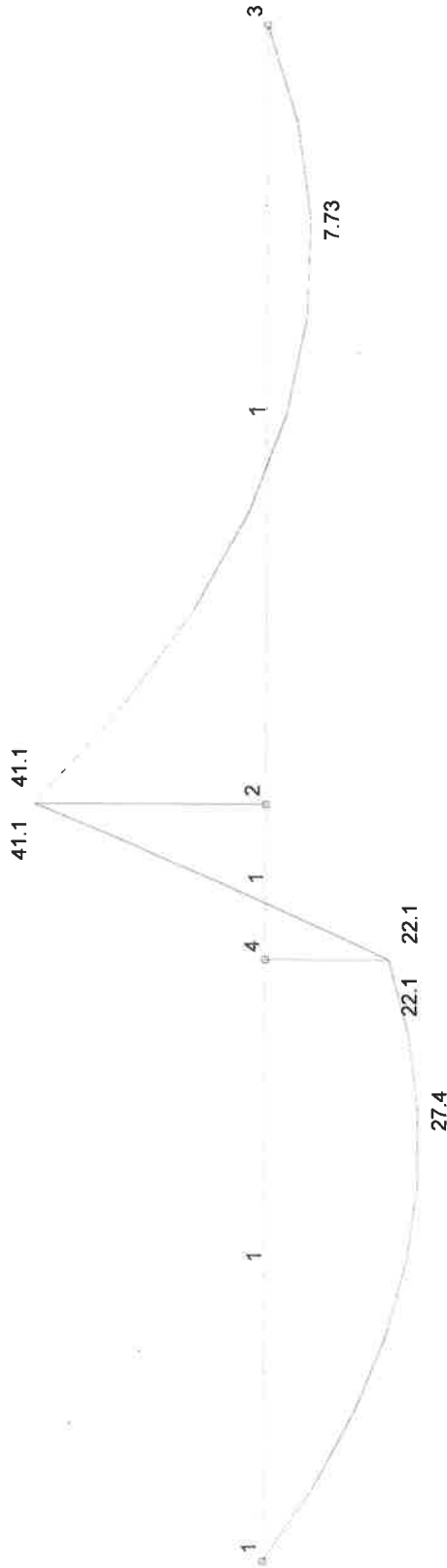
Y
Z
X
theta: 270 phi: 0

Envelope for Moment Mz

--- Maximum
--- Minimum

Enveloped Cases:
3 C 1.2d+1.5l

Sections:
--- 1 200UB22.3 Y

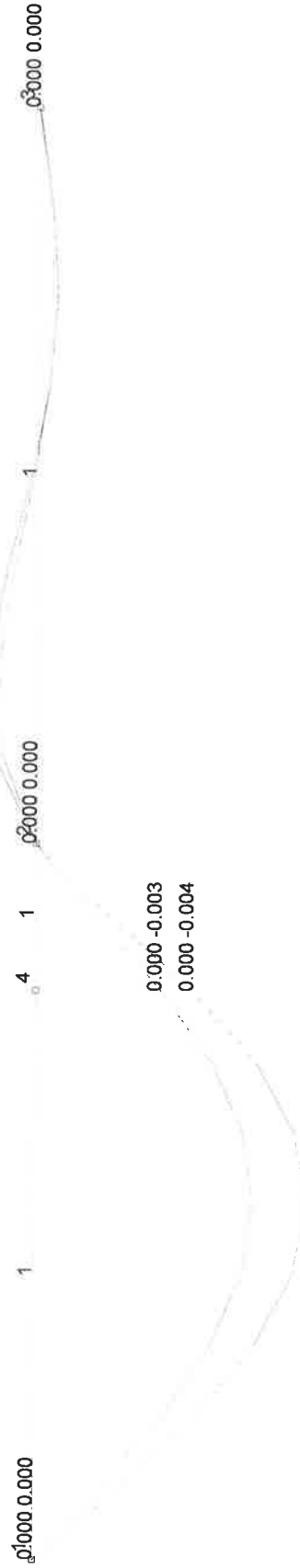


Y
Z
X
theta: 270 phi: 0

Bending Moment, Mz

— 1 PdI
— 4 Cdl+.7II

— 1 200UB22.3 Y



theta: 270 phi: 0

Displaced Shape



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Design: RR

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COLUMN DESIGN - SHS SECTIONS - PINNED TOP & PINNED BASE

Column 1C1.

These calculations comply with the requirements of AS 4100 - 1998 Steel Structures.

$N^* =$	140.0 kN	vertical compression load (strength factored load)
$l_h =$	3000 mm	column height
$k_e =$	1.00	effective length factor (Clause 4.6.3)
$e =$	50 mm	applied load eccentricity at the top of the column
$\Rightarrow M^* =$	7.00 kNm	

Column material yield stress

☒ 350 MPa ☐ 450 MPa

Trial column size :

100x100x5.0 SHS (C350)

$A_n = A_g =$	1810 mm ²	$f_y =$	350 MPa	$k_f =$	1.0
$Z_e =$	63.5E+3 mm ³	$r_x = r_y =$	38.3 mm	$S_x =$	63.5E+3 mm ³
$I_x = I_y =$	2.7E+6 mm ⁴	$J =$	4.4E+6 mm ⁴	$b/t =$	17.0

Check member capacity

Capacity factors

$\phi_b =$ 0.9 Table 3.4 - bending $\phi_c =$ 0.9 Table 3.4 - compression

For cold-formed (non-stress relieved) SHS, $\alpha_b =$ -0.5 Table 6.3.3(2)

(a) Nominal section capacity in compression

Clause 6.1

$$N_s = k_f \cdot A_n \cdot f_y = 633.5 \text{ kN}$$

$$\phi_c \cdot N_s = 570.2 \text{ kN}$$

> 140 kN Required :: OK

(b) Nominal member capacity in compression

Clause 6.3

$\lambda_n =$	92.7	$\alpha_a =$	18.03	$\lambda =$	83.66
$\eta =$	0.229	$\xi =$	1.211	$\alpha_c =$	0.655

Clause 6.3.3

$$N_c = \alpha_c \cdot N_s \leq N_s = 414.9 \text{ kN}$$

$$\phi_c \cdot N_c = 373.4 \text{ kN}$$

> 140 kN Required :: OK

(c) Nominal section capacity for combined bending and compression

$$M_{sx} = f_y \cdot Z_e = 22.2 \text{ kNm}$$

Clause 5.2.1

$$M_{rx} = 19.8 \text{ kNm}$$

Clause 8.3.2(a) and (b)

$$\phi_b \cdot M_{rx} = 17.8 \text{ kNm}$$

> 7 kNm Required :: OK

(d) Nominal member capacity for combined bending and compression

$$M_i = 13.9 \text{ kNm}$$

Clause 8.4.2.2

$$\phi_b \cdot M_i = 12.5 \text{ kNm}$$

> 7 kNm Required :: OK

\Rightarrow Column is Satisfactory in Combined Bending and Compression

ADOPT 100x100x5.0 SHS (C350)

Type recommendation in these two lines

Second floor (Carry beam 2CB2, 2FB1 and Column 1C2)

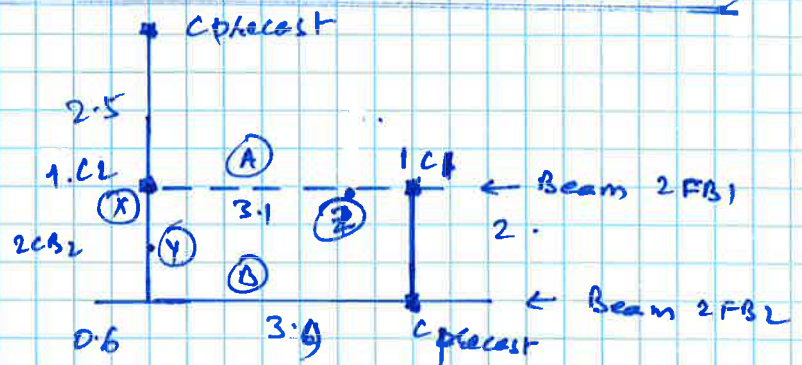
Load on beam (A)

Floor beam 2F1.

Line load \Rightarrow

$$dL = 7 \text{ kN/m}$$

$$LL = 2.4 \text{ kN/m}$$



Load on beam (B)

$$dL = 17.11 + 3.86 = 20.97 \text{ kN/m} \sim 22 \text{ kN/m}$$

$$LL = 7.6 \text{ kN/m}$$

Point load at (Y)

$$P_{dL} = 138 \text{ kN}$$

$$P_{uL} = 49 \text{ kN}$$

Point load at (Z)

$$P_{dL} = 36 \text{ kN}$$

$$P_{uL} = 13 \text{ kN}$$

Analysis from microstran.

2CB2 \rightarrow $M_{max} = 432 \text{ kNm}$. Adopt - 310 UC 118
for deflection limit use 310 UC 152
2FB2: $M_{max} 69$ - Use 300 PFC.

For 2CB2

expected dL deflection = 10 mm

i.e. $\frac{L}{200}$

expected serv. deflection = 13 mm

i.e. $\frac{L}{153}$ (DL)

$$2CB2 = 310 \text{ UC } 137$$

$$2FB2 = 300 \text{ PFC}$$

Reaction at (X)

$$P_{dL} = 301 \text{ kN}$$

$$P_{uL} = 110 \text{ kN}$$

Column design from spread sheet (1C2)

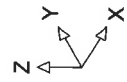
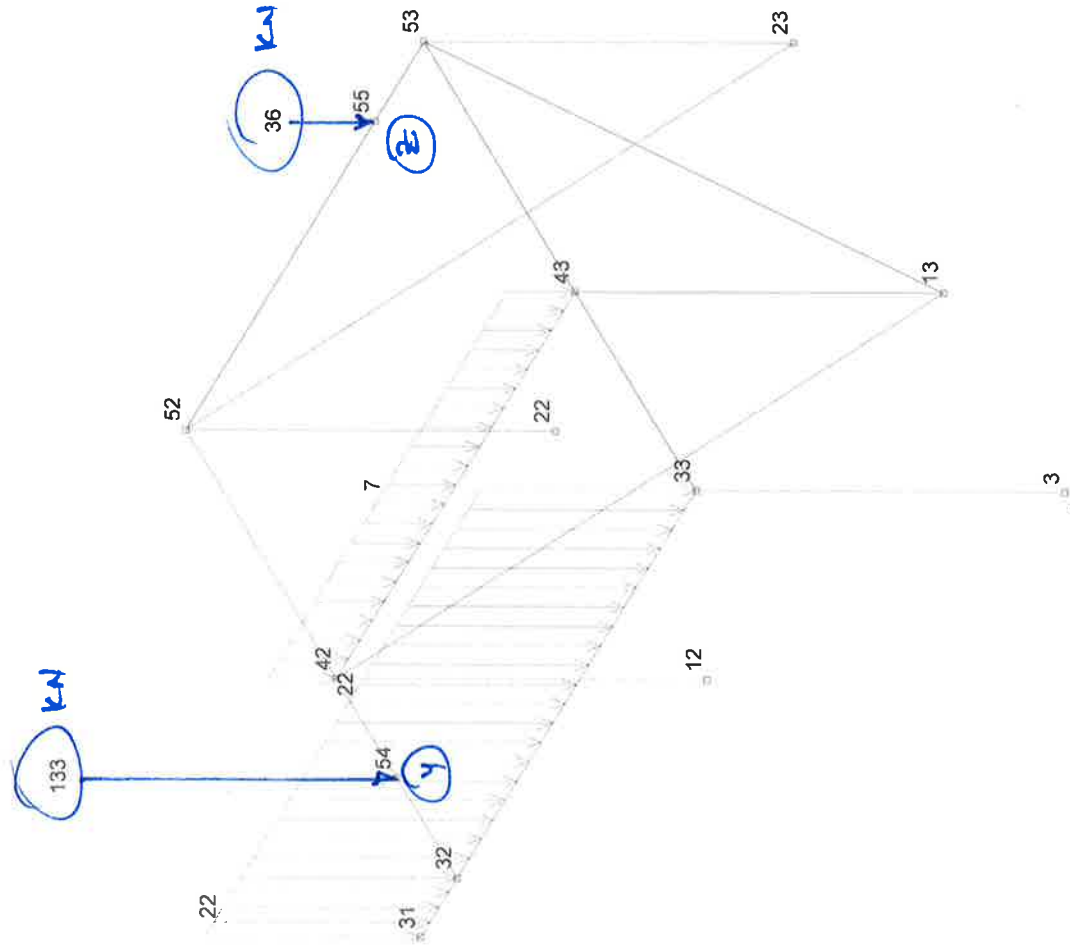
$$N^* = 526.2 \text{ kNm} \sim 550 \text{ kN}, L_{eff} = 3 \text{ m}$$

Adopt 150 x 9 SHS minimum.

Checked :

Date :

Load Cases:
 — 1 P dl



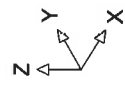
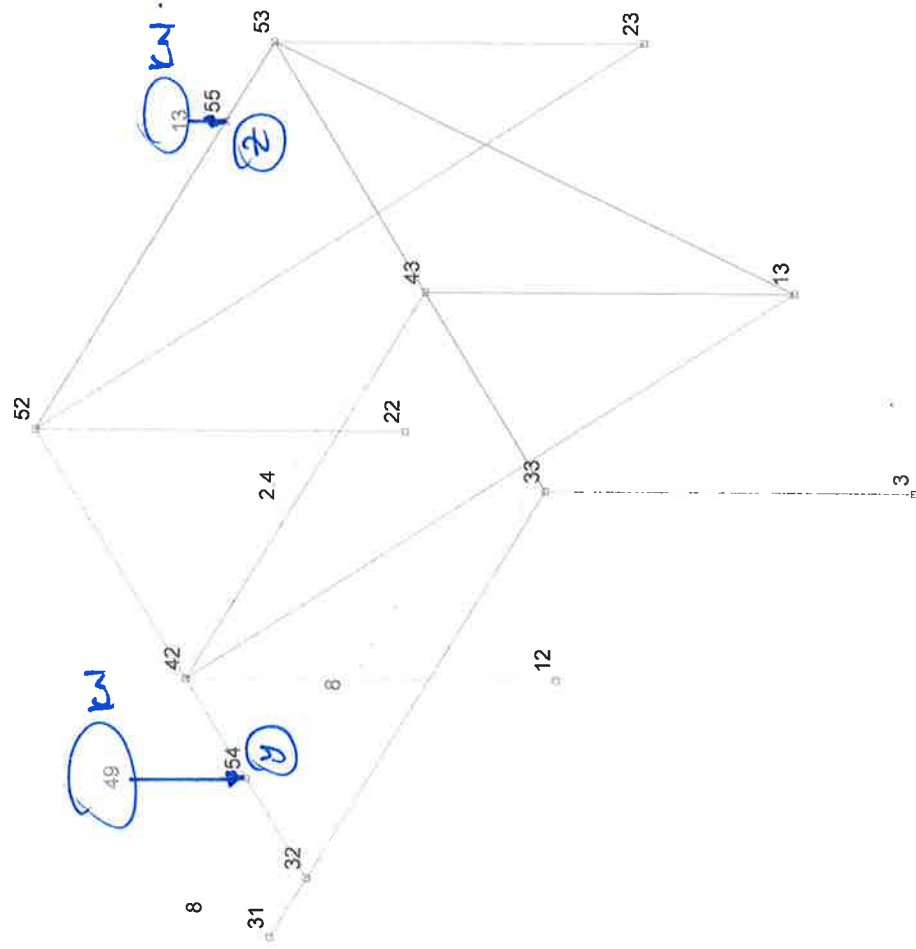
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Microtran [V8.11r]

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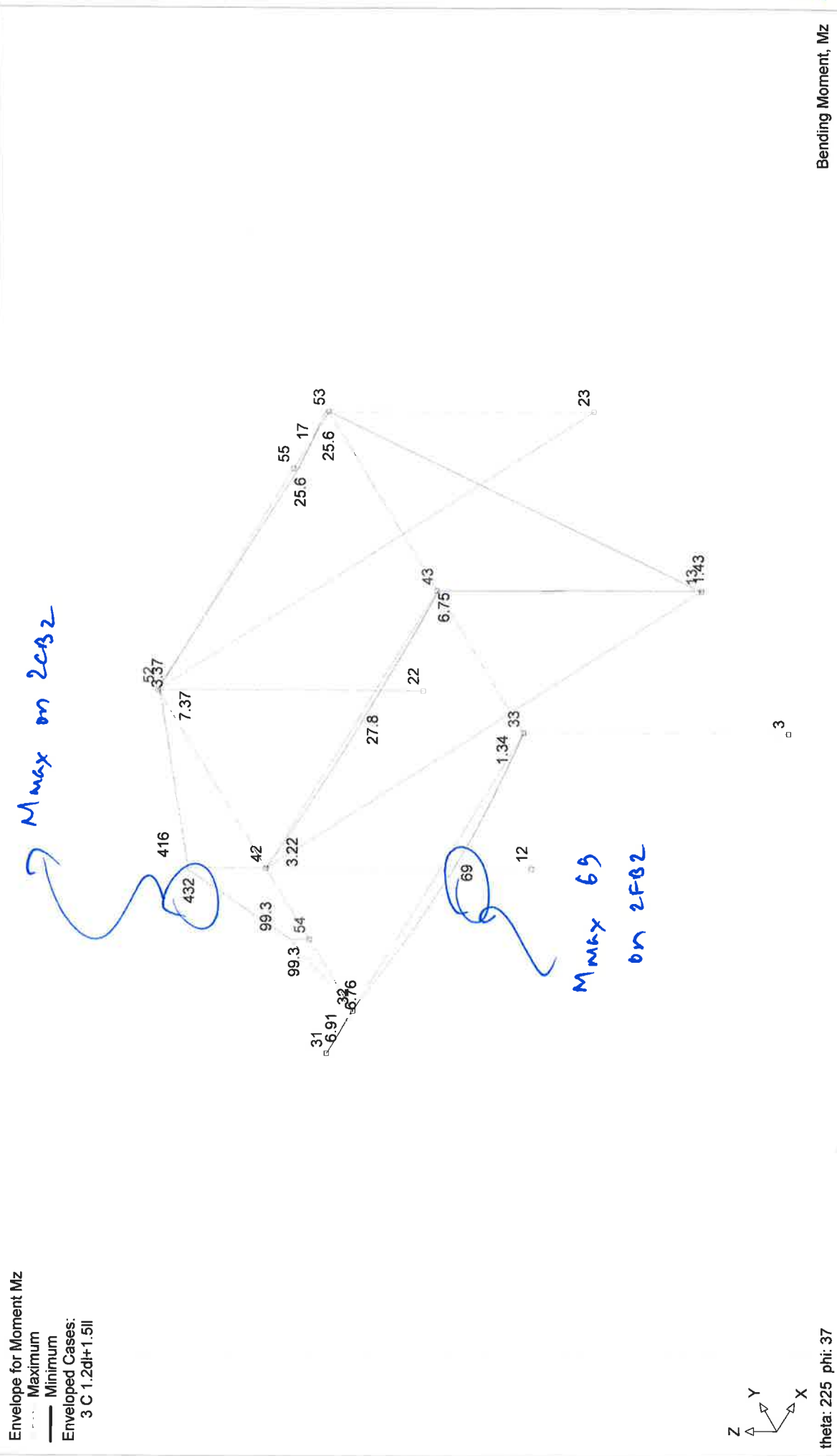
Load Cases:
 2 P II



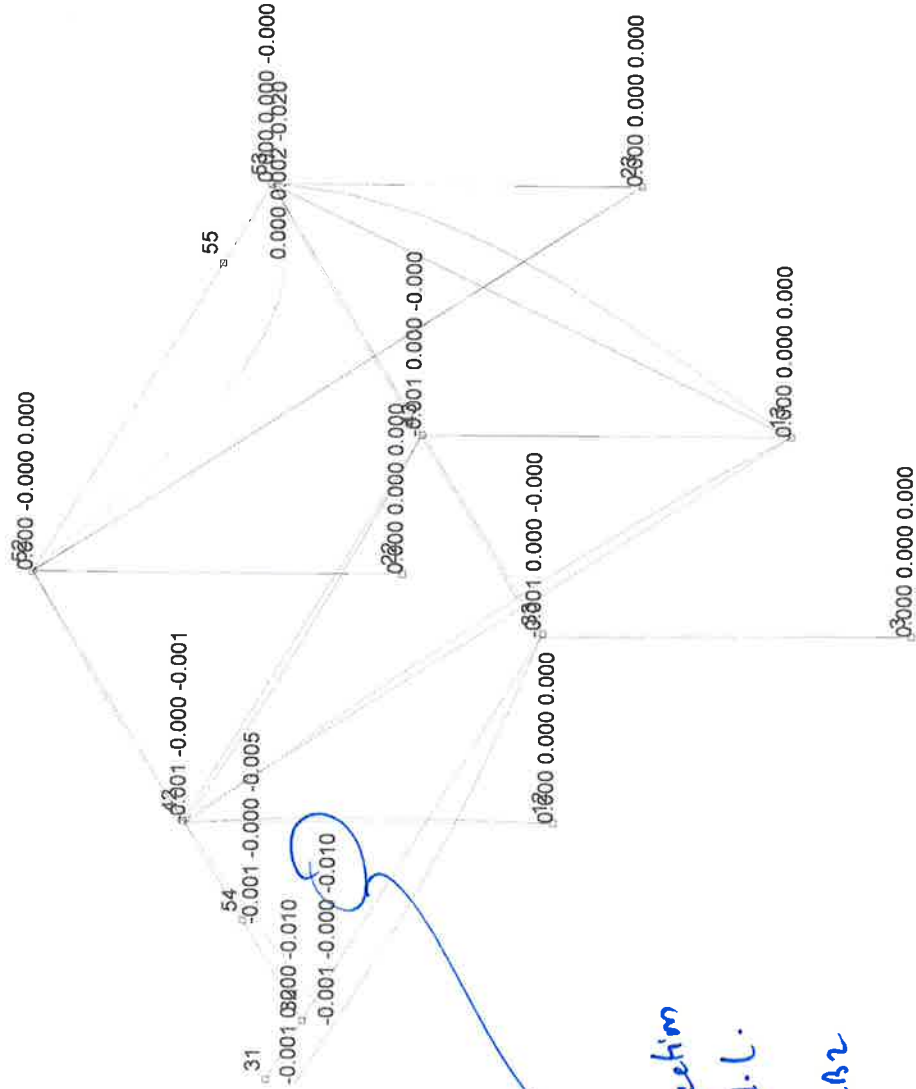
theta: 225 phi: 37

Microstran [V8.11r]

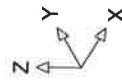
K:\2017\101710168\Structural Drawings and Calcs\Design Calculations and Details\RR\Preliminary\Second Floor\2FB2, 2CB1 and 1C2



Load Cases:
 — 1 P dl



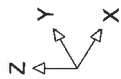
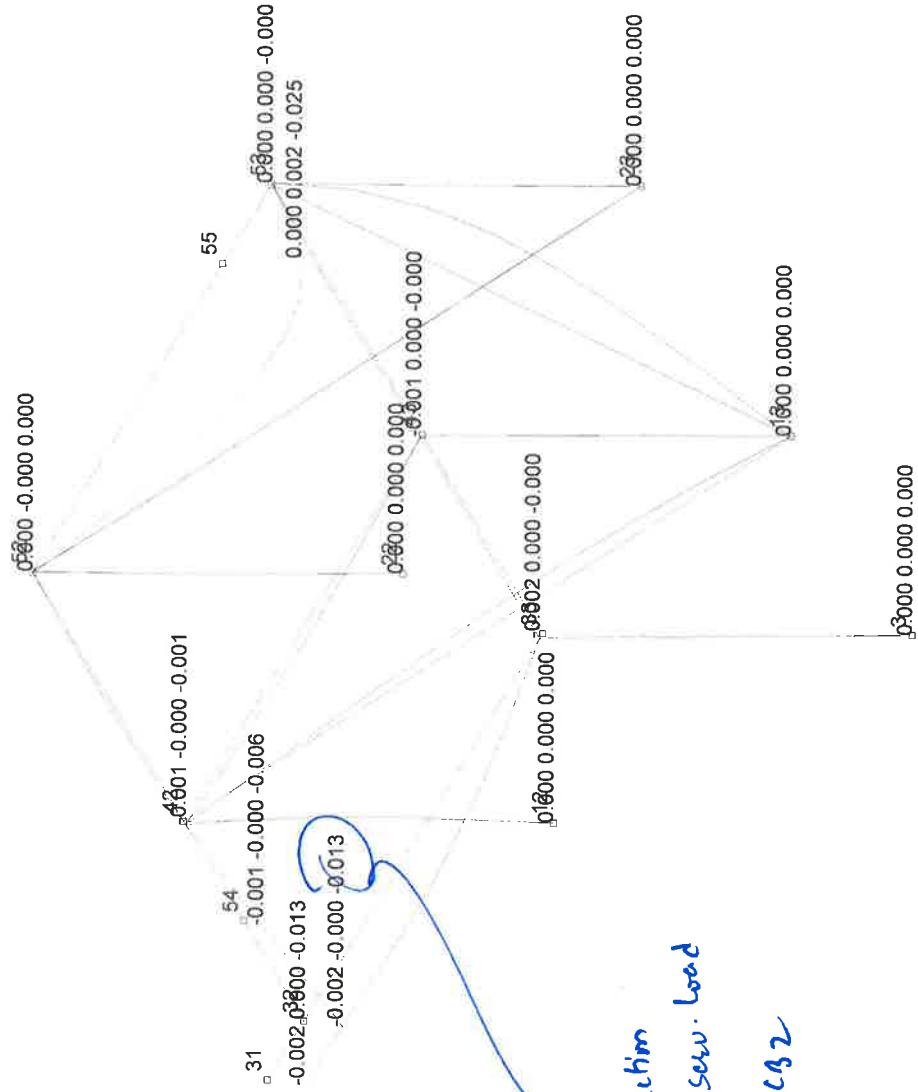
10 mm deflection
 under d.l.
 on 2CB2



theta: 225 phi: 37

Displaced Shape

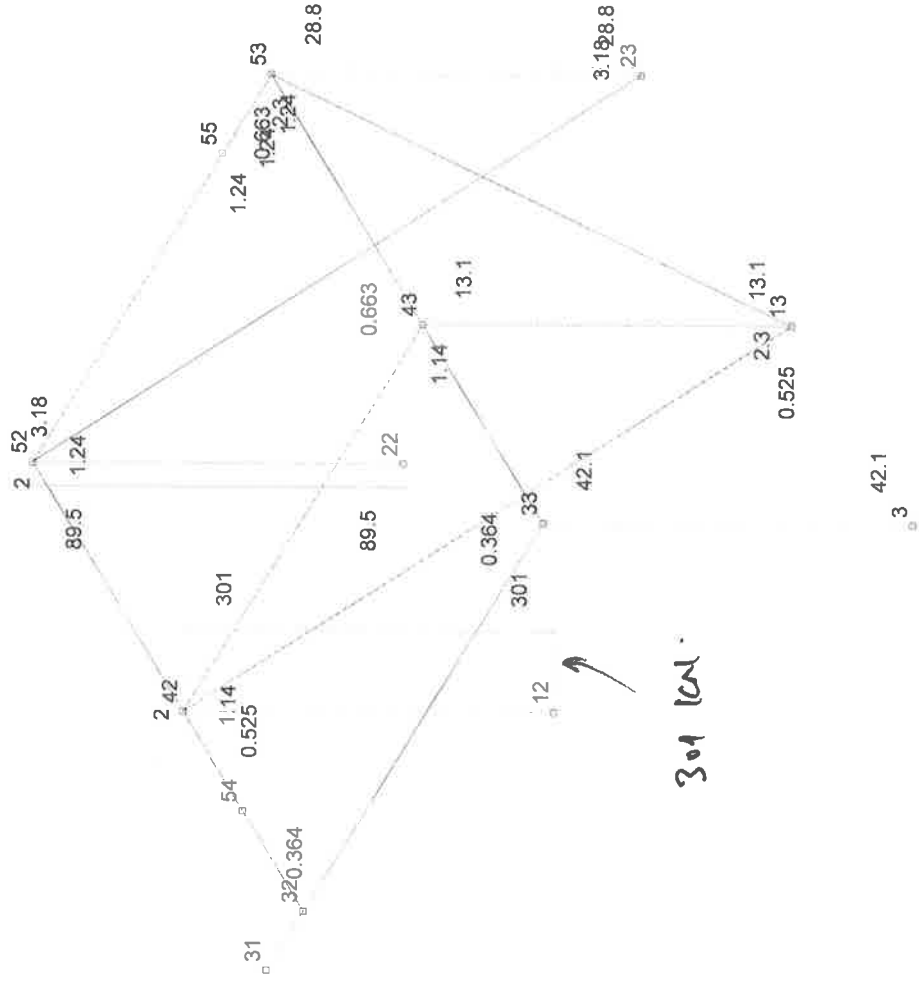
Load Cases:
 — 4 C dlt+.7ll



theta: 225 phi: 37

Displaced Shape

Load Cases:
— 1 P dl



Z
Y
X

theta: 225 phi: 37

Axial Force, Fx

[illegible]

theta: 225 phi: 37

Axial Force, F_x

Microstran [V8.11r]

K:\2017\10\17\10168\Structural Drawings and Calcs\Design Calculations and Details\RR\Preliminary\Second Floor\2FB2, 2CB1 and 1C2

1710168
SC128



Ref.: 1710168
Date: 07-Oct-18
Design: RR
Page: SC129

COLUMN DESIGN - SHS SECTIONS - PINNED TOP & PINNED BASE

Column 1C2

These calculations comply with the requirements of AS 4100 - 1998 Steel Structures.

$N^* = 550.0$ kN vertical compression load (strength factored load)
 $l_h = 3000$ mm column height
 $k_e = 1.00$ effective length factor (Clause 4.6.3)
 $e = 75$ mm applied load eccentricity at the top of the column
 $\Rightarrow M^* = 41.25$ kNm

Column material yield stress

☒ 350 MPa ☐ 450 MPa

Trial column size : 150x150x9.0 SHS (C350)

$A_n = A_g = 4800$ mm ²	$f_y = 350$ MPa	$k_f = 1.0$
$Z_e = 248.0E+3$ mm ³	$r_x = r_y = 56.6$ mm	$S_x = 248.0E+3$ mm ³
$I_x = I_y = 15.4E+6$ mm ⁴	$J = 26.1E+6$ mm ⁴	$b/t = 13.7$

Check member capacity

Capacity factors

$\phi_b = 0.9$ Table 3.4 - bending $\phi_c = 0.9$ Table 3.4 - compression
For cold-formed (non-stress relieved) SHS, $\alpha_b = -0.5$ Table 6.3.3(2)

(a) Nominal section capacity in compression

Clause 6.1

$N_s = k_f A_n f_y = 1680$ kN
 $\phi_c N_s = 1512.0$ kN

> 550 kN Required ::OK

(b) Nominal member capacity in compression

Clause 6.3

$\lambda_n = 62.7$	$\alpha_a = 20.57$	$\lambda = 52.43$
$\eta = 0.127$	$\xi = 2.160$	$\alpha_c = 0.849$

Clause 6.3.3

$N_c = \alpha_c N_s \leq N_s = 1425.9$ kN
 $\phi_c N_c = 1283.3$ kN

> 550 kN Required ::OK

(c) Nominal section capacity for combined bending and compression

$M_{sx} = f_y Z_e = 86.8$ kNm
 $M_{rx} = 65.2$ kNm
 $\phi_b M_{rx} = 58.6$ kNm

Clause 5.2.1

Clause 8.3.2(a) and (b)

> 41.25 kNm Required ::OK

(d) Nominal member capacity for combined bending and compression

$M_i = 49.6$ kNm
 $\phi_b M_i = 44.6$ kNm

Clause 8.4.2.2

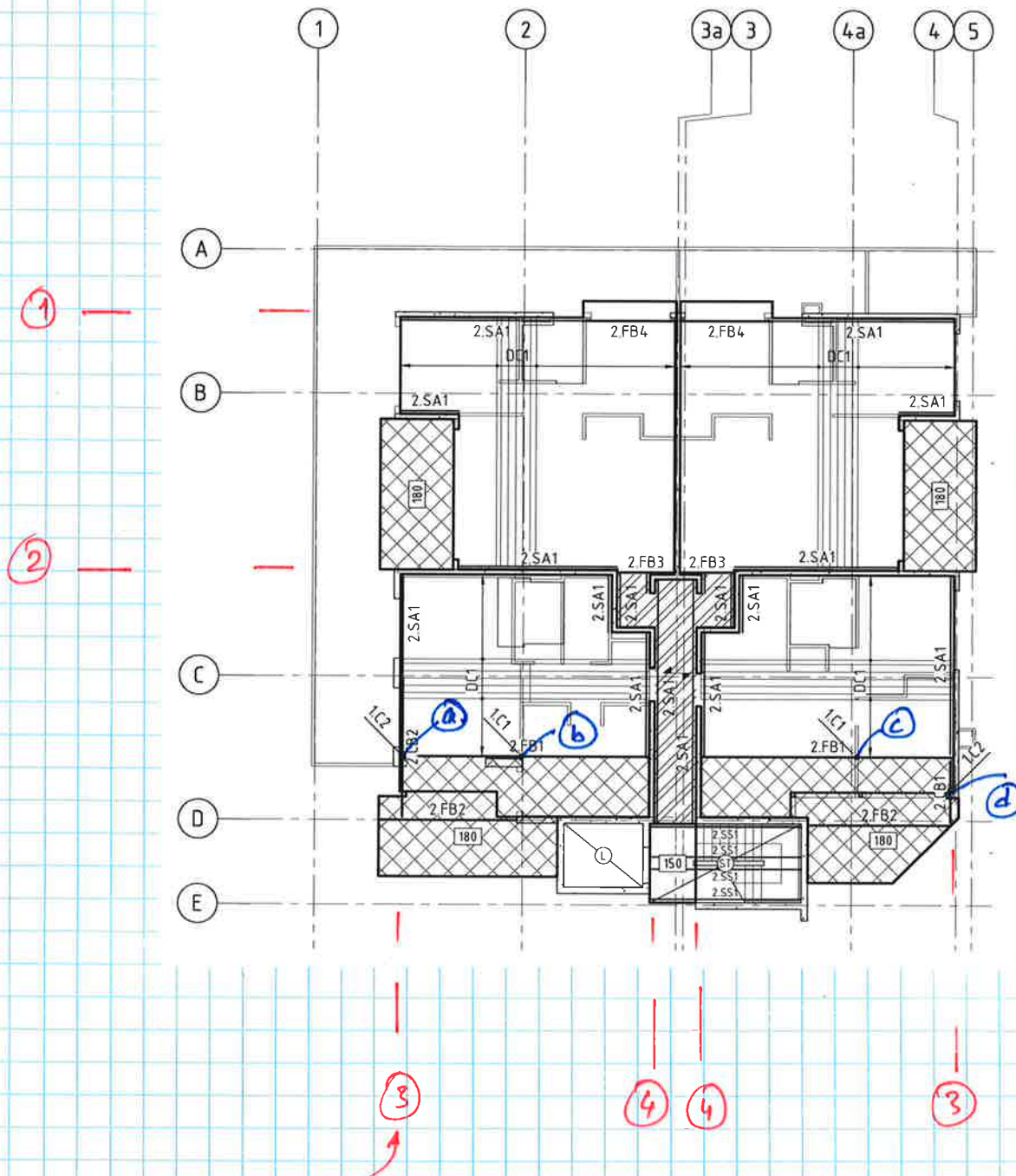
> 41.25 kNm Required ::OK

\Rightarrow Column is Satisfactory in Combined Bending and Compression

ADOPT 150x150x9.0 SHS (C350)

Type recommendation in
these two lines

loads on slab (1st floor)



additional load
because of wall panel
thickness
(d.l.)

Add Point loads from Columns.

Checked :

Date :/..../..

Line and point load on flat slab.

$$\text{Delta Core} - 200 \text{ thick} = 2.6 \text{ kN/m}^2$$

$$\text{Delta Core} - 250 \text{ thick} = 3.34 \text{ kN/m}^2$$

$$\text{Topping slab} = 80 \text{ mm ie } 24 \times 0.08 = 1.92 \text{ kN/m}^2$$

$$\text{Ceiling / nominal services / carpet finish} = 0.5 \text{ kPa}$$

$$\text{light wt wall load} = 0.3 \text{ kPa.}$$

$$\underline{2.72 \text{ kPa.}}$$

$$\therefore \text{for } 200 \text{ thick delta core } dL = 5.32 \text{ kPa.}$$

$$\text{for } 250 \text{ thick delta core } dL = 6.06 \text{ kPa}$$

$$\text{for Roof} = 0.5 \text{ kPa.}$$

$$\text{Live loads} \quad \begin{array}{l} \text{Floor} \quad - \quad 2 \text{ kPa} \\ \text{Roofs} \quad - \quad 0.25 \text{ kPa.} \end{array}$$

$$\text{Wall} \Rightarrow (0.15 \times 24 + 0.3) = 3.9 \text{ kN/m} \quad (\text{per meter width/ht})$$

Calculating loads.

Along ① Loading width = 4 m.

$$D.L. \Rightarrow 3^{\text{rd}} \text{ Floor} \Rightarrow 250 \text{ thick} \rightarrow 6.06 \times 4 = 24.24 \text{ kN/m}$$

$$2^{\text{nd}} \text{ Floor} \Rightarrow 200 \text{ thick} \rightarrow 5.32 \times 4 = 21.28 \text{ kN/m}$$

$$1^{\text{st}} \text{ Floor} \Rightarrow \text{Self calculated by SAFE}$$

$$\text{Roof} \Rightarrow 0.5 \times 4 = 2 \text{ kN/m.}$$

$$\text{Wall} = 10 \text{ m high (3 level)} = 3.9 \times 10 = 39 \text{ kN/m}$$

$$\therefore \text{Total } dL = 86.52 \text{ kN/m.}$$

$$L.L. \Rightarrow 3^{\text{rd}}, 2^{\text{nd}} = (2 \times 4) \times 2 = 16 \text{ kN/m.}$$

$$1^{\text{st}} \text{ floor} - \text{varying } 2 \text{ \& } 5 \text{ kPa.}$$

$$\text{Roof} = 0.25 \times 4 = 1 \text{ kN/m.}$$

$$\therefore \text{Total L.L.} = 17 \text{ kN/m.}$$

Checked :

Date :/..../..

Along ② → Adopt same as ①

Along ③ → Adopt same as ①

Along ④ →

$$\text{loading width} = 9.4/2 = 4.7 \text{ m}$$

$$\text{D.L.} \Rightarrow \text{line load} = 5.32 \times 4.7 \times 2 + 0.5 \times 4.7 = 52.35 \text{ kN/m}$$

$$\text{wall} = 39 \text{ kN/m}$$

$$\text{total d.l.} = 91.35 \text{ kN/m} \sim 92 \text{ kN/m}$$

$$\text{LL} \Rightarrow (2 \times 4.7) \times 2 + 0.25 \times 4.7 = 20 \text{ kN/m}$$

Point loads from Column →

Result from microstran

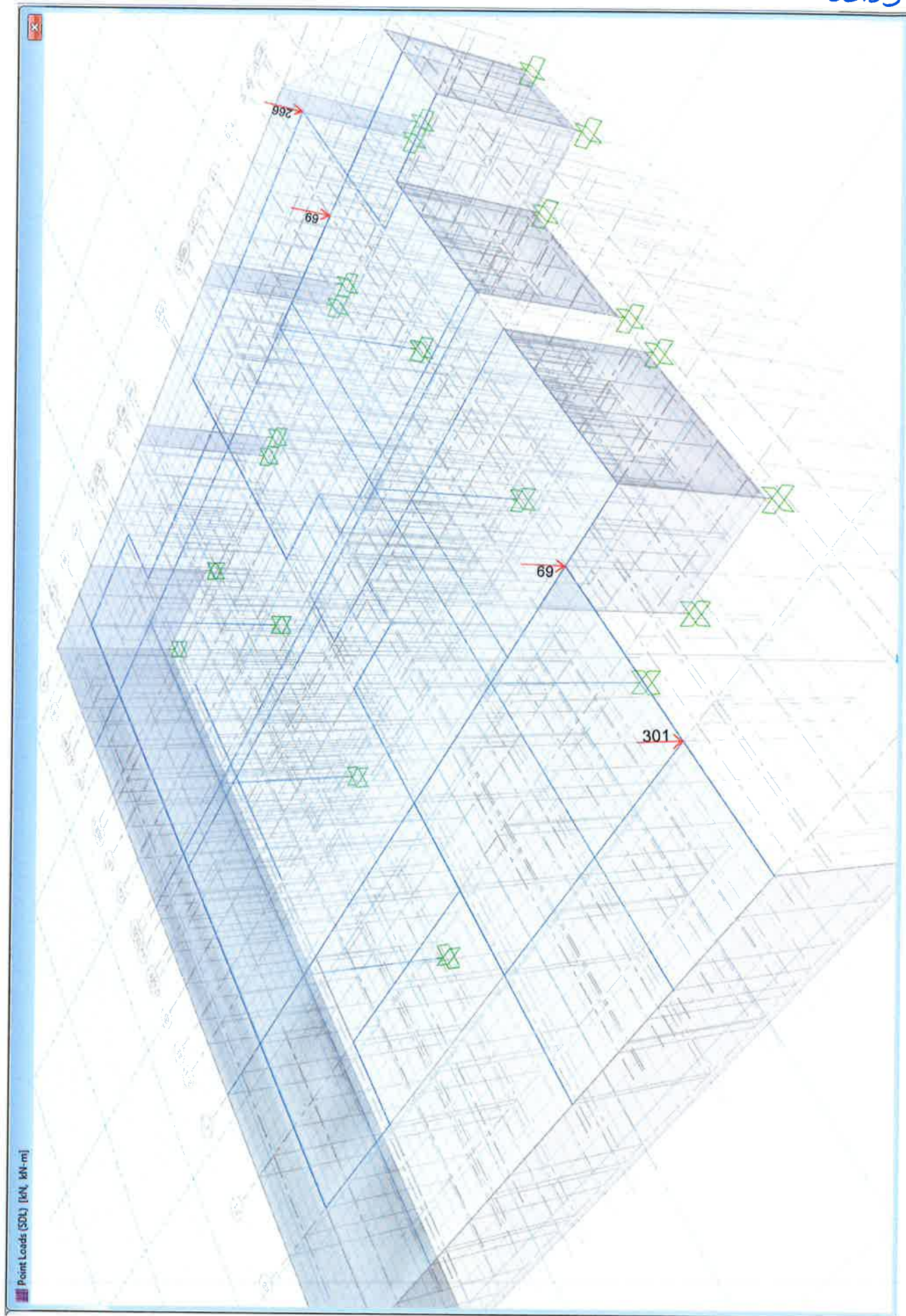
$$\text{DL} \Rightarrow \begin{aligned} P_a &= 301 \text{ kN} \\ P_b &= 69 \text{ kN} \\ P_c &= 69 \text{ kN} \\ P_d &= 266 \text{ kN} \end{aligned}$$

$$\text{LL} \Rightarrow \begin{aligned} P_a &= 110 \text{ kN} \\ P_b &= 25 \text{ kN} \\ P_c &= 25 \text{ kN} \\ P_d &= 97 \text{ kN} \end{aligned}$$

Design beams and slab from SAFE

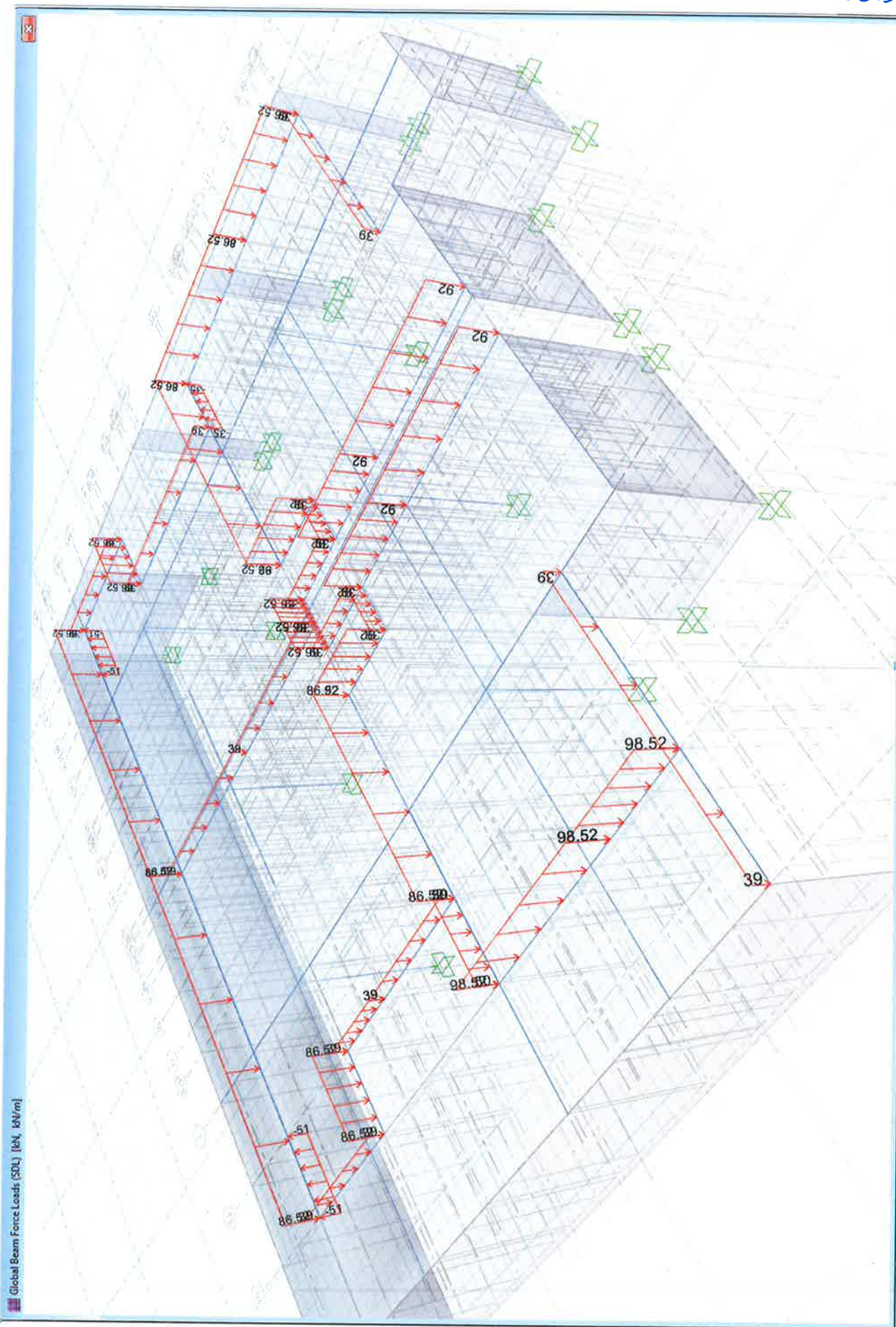
Uniformly distributed L.L. ⇒ varying 2 kPa, 4 kPa & 5 kPa

First Floor Slab

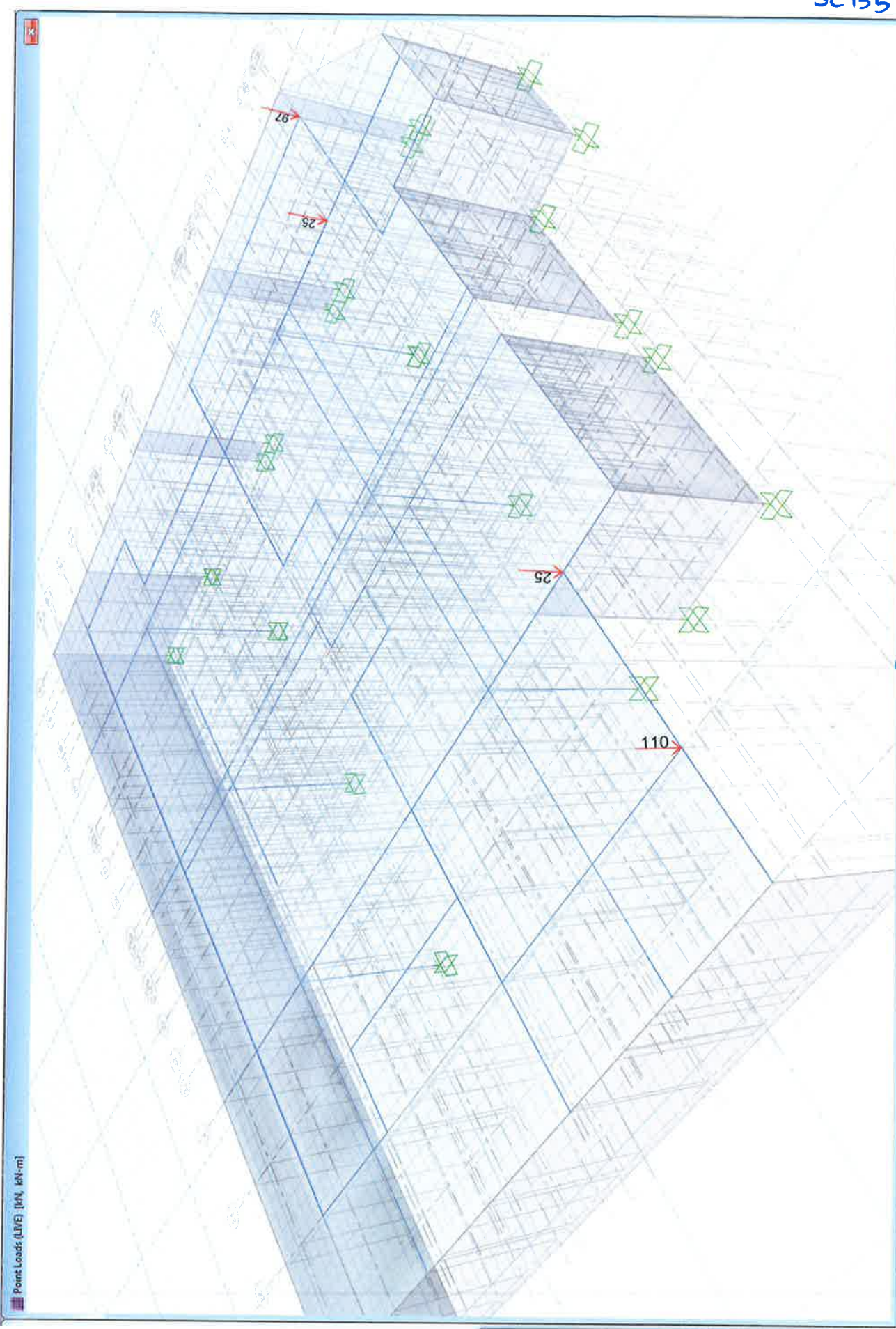
Super Imposed Dead Load
Point Loads

Super Imposed Dead Load
Line Loads

Global Beam Force Loads (SDL) [kN, kN/m]

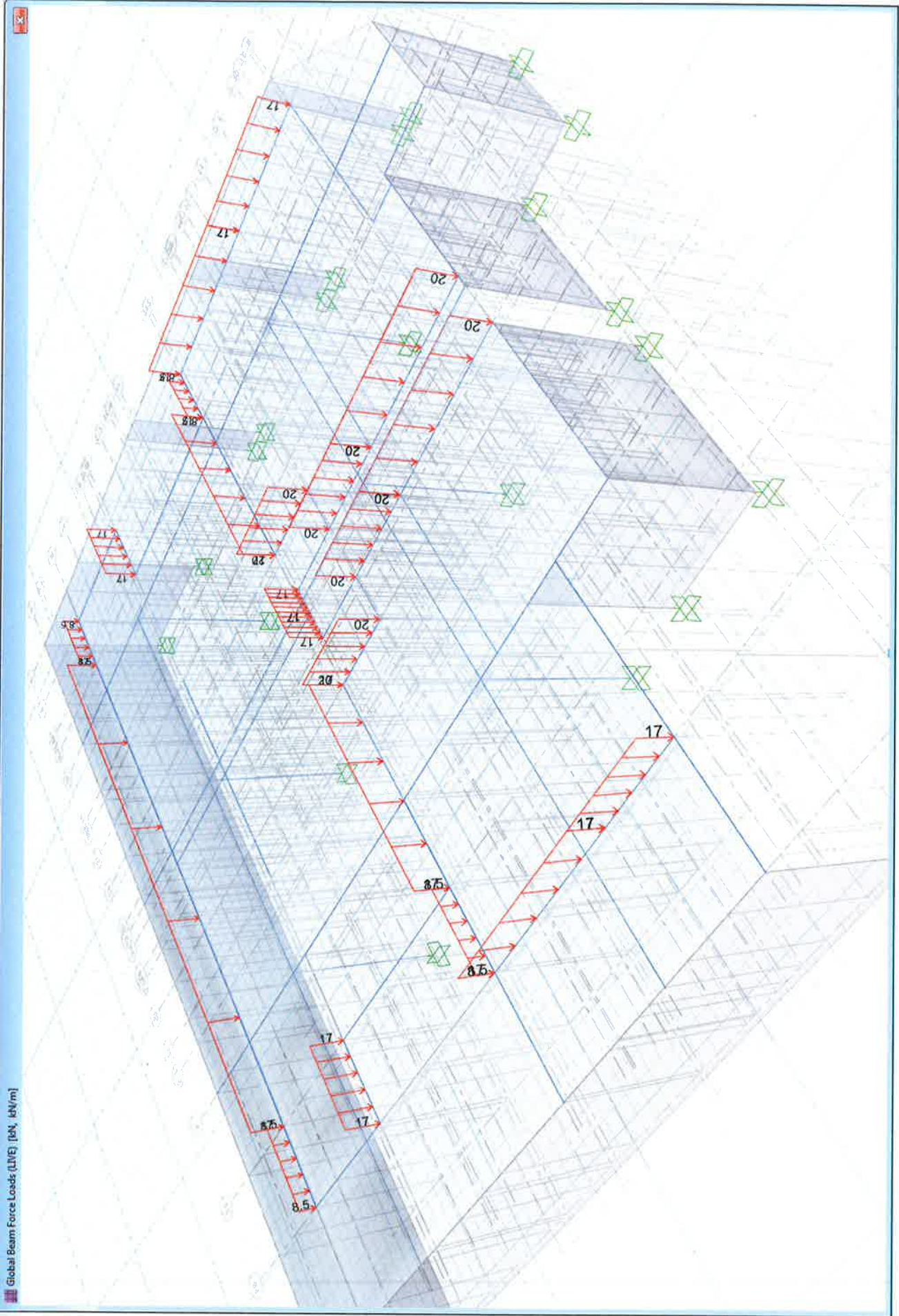


Live Load
Point Load

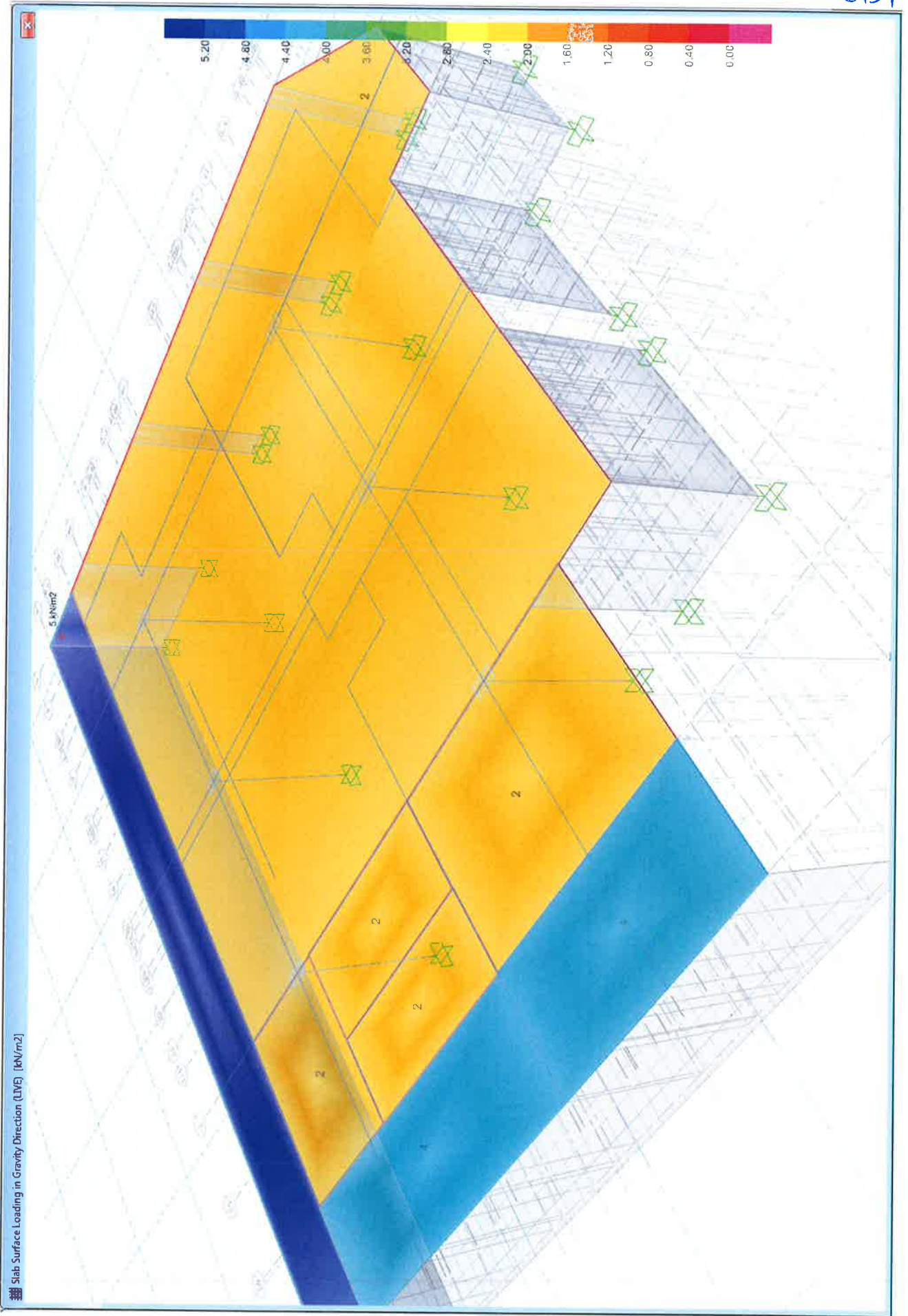


Live Load
Line Load

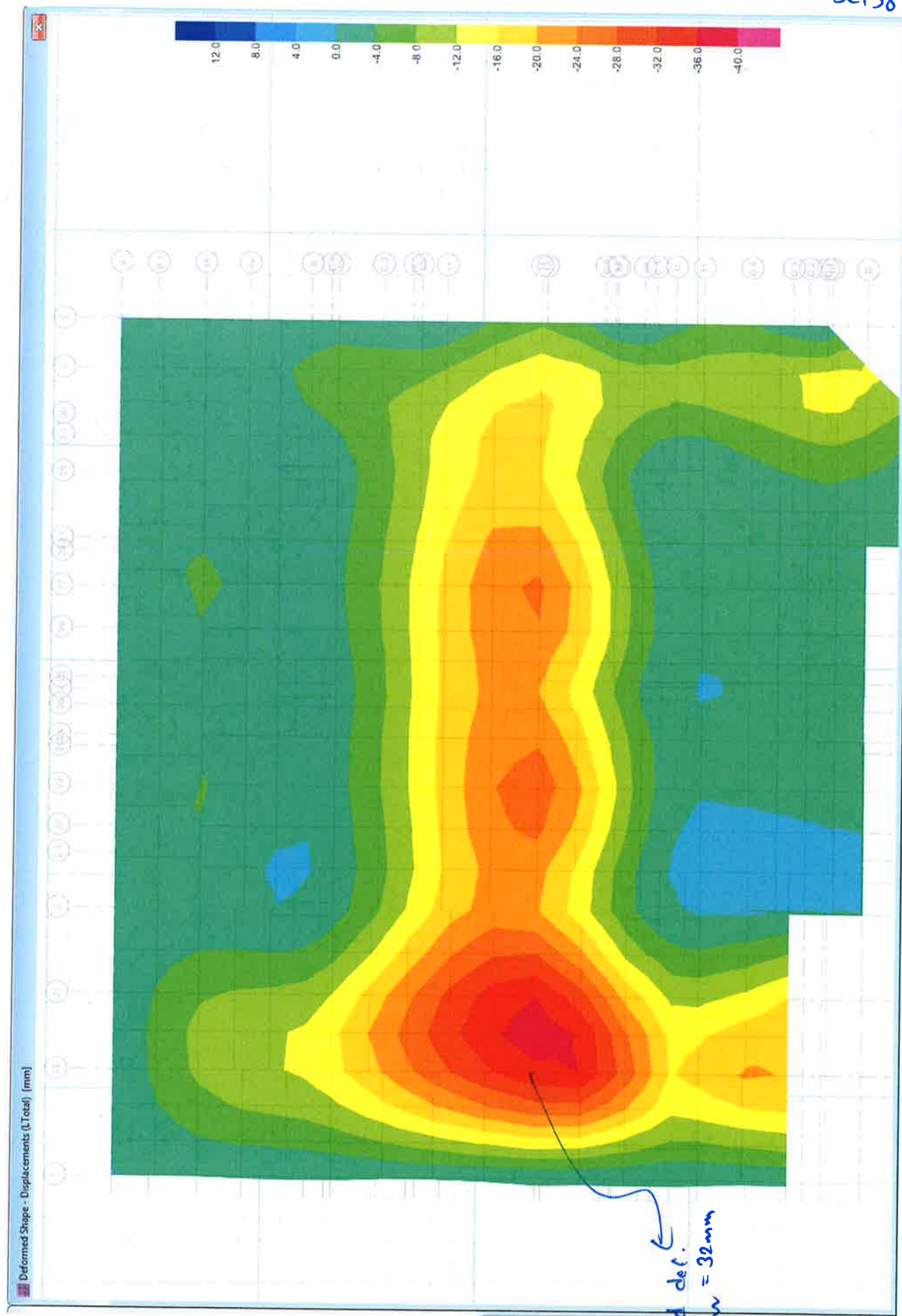
Global Beam Force Loads (LIVE) [kN, kN/m]



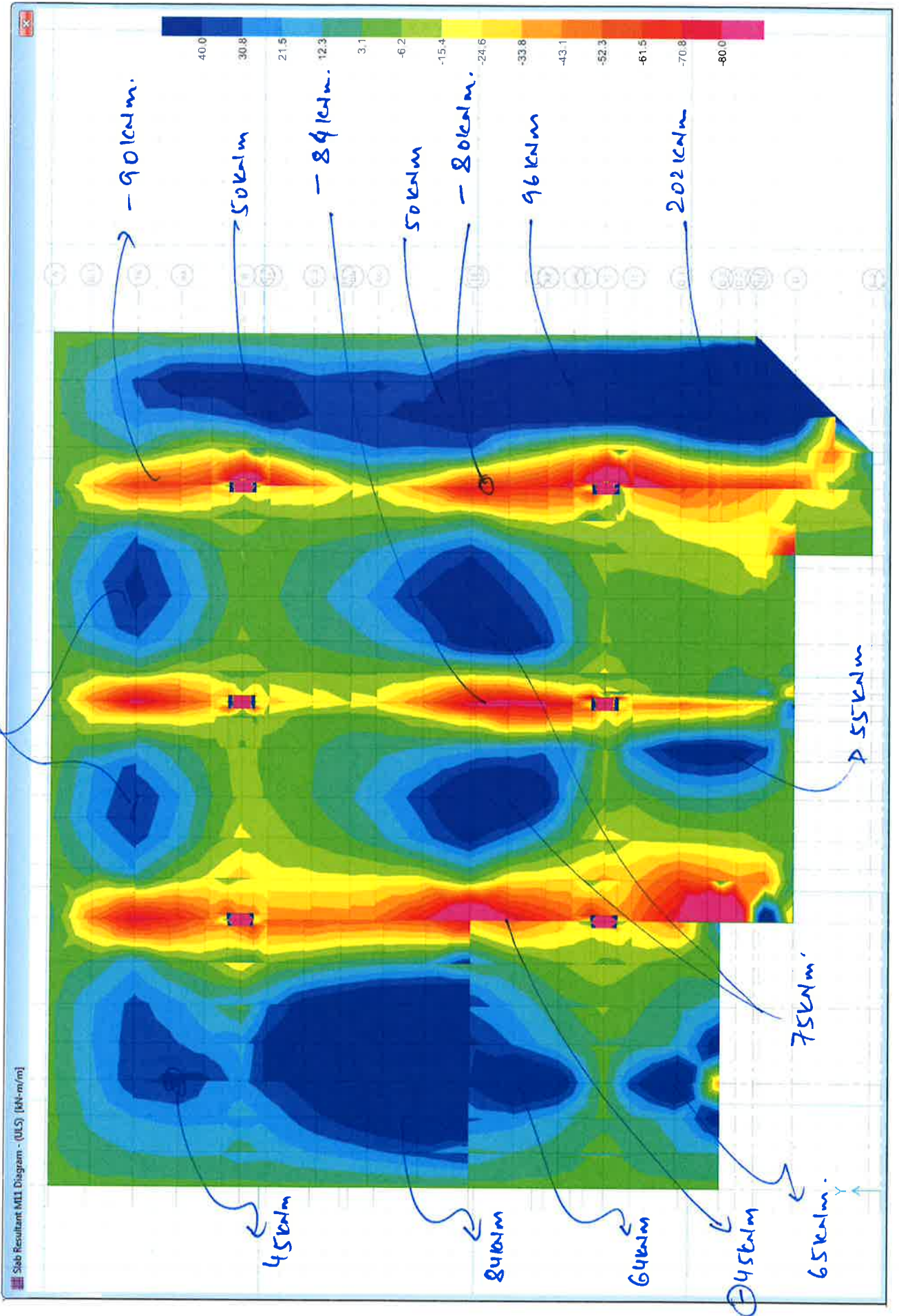
Surface Load



Expected long term deflection



Ultimate moment
M11 direction (Horizontal of plan)



Slab thickness

275 Thick \Rightarrow N12 @ 200 $\frac{c}{c}$ each way top (550 mm²)
N16 @ 200 $\frac{c}{c}$ each way bottom (1000 mm²)

200 Thick \Rightarrow N12 @ 200 $\frac{c}{c}$ each way top (550 mm²)
N16 @ 200 $\frac{c}{c}$ each way bottom (1000 mm²)

M11 Direction.

Additional Reinforcement

(a) , $M^* = 202 \text{ kNm}$ (pick) \rightarrow Conservative to design for this
o.o take avg of $\frac{202 + 96}{2} = +149 \text{ kNm}$.

$$A_{st} \text{ req} = \frac{149 \times 10^6}{0.8 \times 0.9 \times 500 (275 - 65)} \\ = 1970 \text{ mm}^2.$$

Additional Reinforcement Required = 1970 - 1000
= 970 mm² bottom

\therefore Adopt (a) as N16 @ 200 $\frac{c}{c}$ bottom additional bar

(b)

$M^* = +84 \text{ kNm}$ @ 275 Thick slab.

$$A_{st} \text{ req} = \frac{84 \times 10^6}{0.8 \times 0.9 \times 500 (275 - 65)} \\ = 1111 \text{ mm}^2.$$

Addition req - 1111 mm² adopt (b) ^{as} N16 @ 400 $\frac{c}{c}$ bottom

$M^* = +64 \text{ kNm}$ @ 200 Thick slab.

$$A_{st} \text{ req} = \frac{64 \times 10^6}{0.8 \times 0.9 \times 500 (200 - 65)} = 1316 \text{ mm}^2.$$

Addition req - 316 mm² \rightarrow Adopt N16 @ 400 $\frac{c}{c}$

c) $M^* = -90 \text{ kNm}$ @ 275 thick slab.

$$A_{st} \text{ req} = \frac{90 \times 10^6}{0.8 \times 0.9 \times 500 \times (275 - 65)}$$

$$= 1190 \text{ mm}^2$$

Additional reinforcement required = $1190 - 550$

$$= 640 \text{ mm}^2$$

Adopt N16 @ 200 c/c as additional reo at top.

$M^* = -45 \text{ kNm}$ @ 200 thick slab.

$$A_{st} \text{ req} = \frac{45 \times 10^6}{0.8 \times 0.9 \times 500 \times (200 - 65)}$$

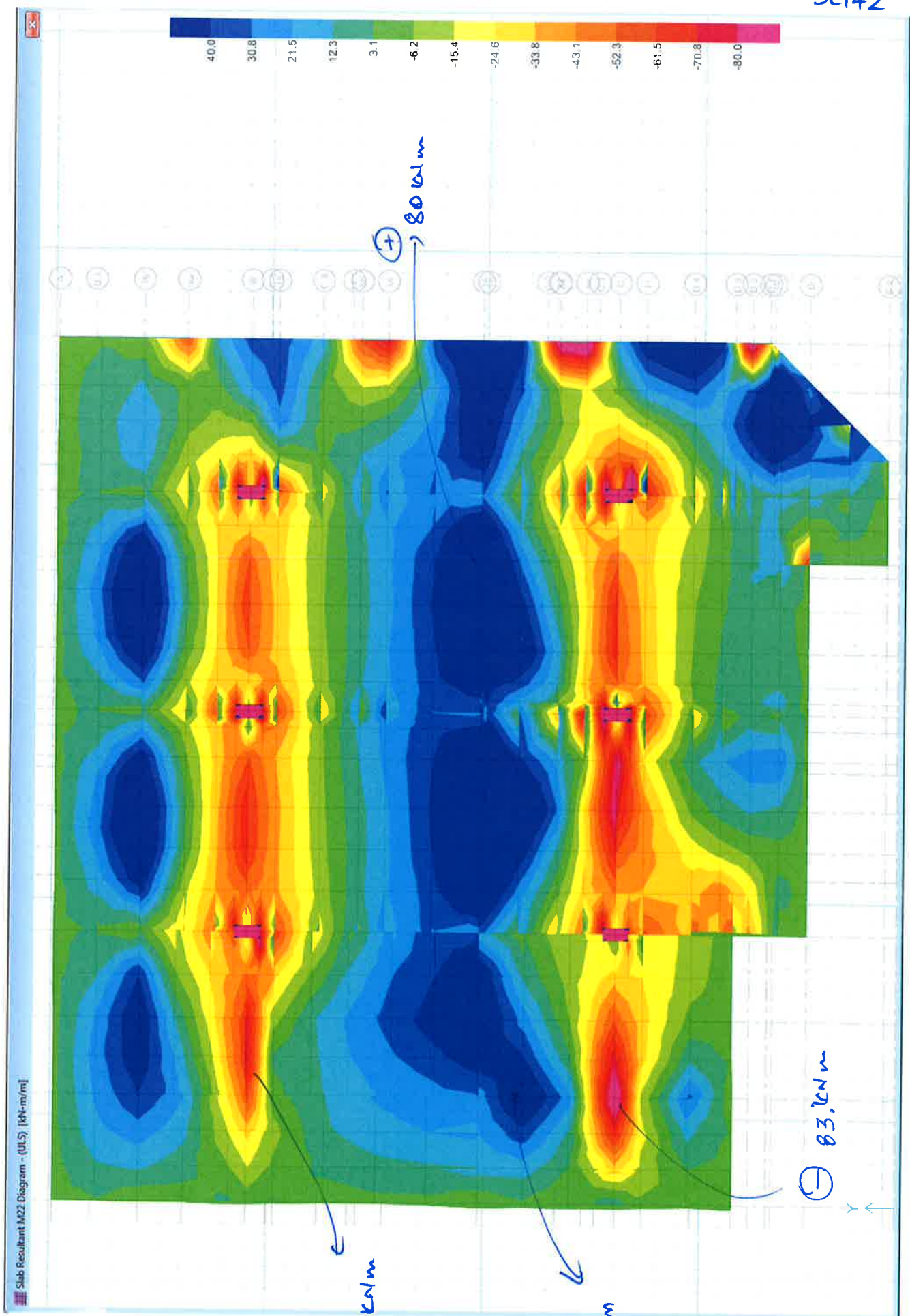
$$= 926 \text{ mm}^2$$

Addition reo as N16 @ 200 c/c top (ok)

d) $M^* = -84 \text{ kNm}$.

Adopt N16 @ 200 c/c top bar (additional)

Ultimate moment
M22 direction (Vertical of plan)



M22 Direction (Upper most and lowermost bar)

Maximum +ve moment $\oplus 80 \text{ kNm}$ @ 275 thick slab.

$$A_{st} \text{ req} = \frac{80 \times 10^6}{0.8 \times 0.9 \times 500 \times (275 - 50)} \\ = 988 \text{ mm}^2$$

Adopt N16 @ 200 ϕ bottom (LMB)

$M_{\text{max}} = \oplus 50 \text{ kNm}$ @ 200 thick slab.

$$A_{st} \text{ req} = \frac{50 \times 10^6}{0.8 \times 0.9 \times 500 \times (200 - 50)} \\ = 925 \text{ mm}^2$$

Adopted N16 @ 200 ϕ bottom (LMB)

-ve reinforcement-

(e) $M^x = \ominus 70 \text{ kNm}$ @ 275 thick slab

$$A_{st} \text{ req} = \frac{70 \times 10^6}{0.8 \times 0.9 \times 500 \times (275 - 50)} \\ = 864 \text{ mm}^2$$

$$\text{Additional reinforcement req} = 864 - 550 = 314 \text{ mm}^2$$

Adopt N16 @ 300 ϕ (UMB)

(f) $M^x = \ominus 83 \text{ kNm}$ @ 200 thick slab.

$$A_{st} \text{ req} = \frac{83 \times 10^6}{0.8 \times 0.9 \times 500 \times 150} = 1537 \text{ mm}^2$$

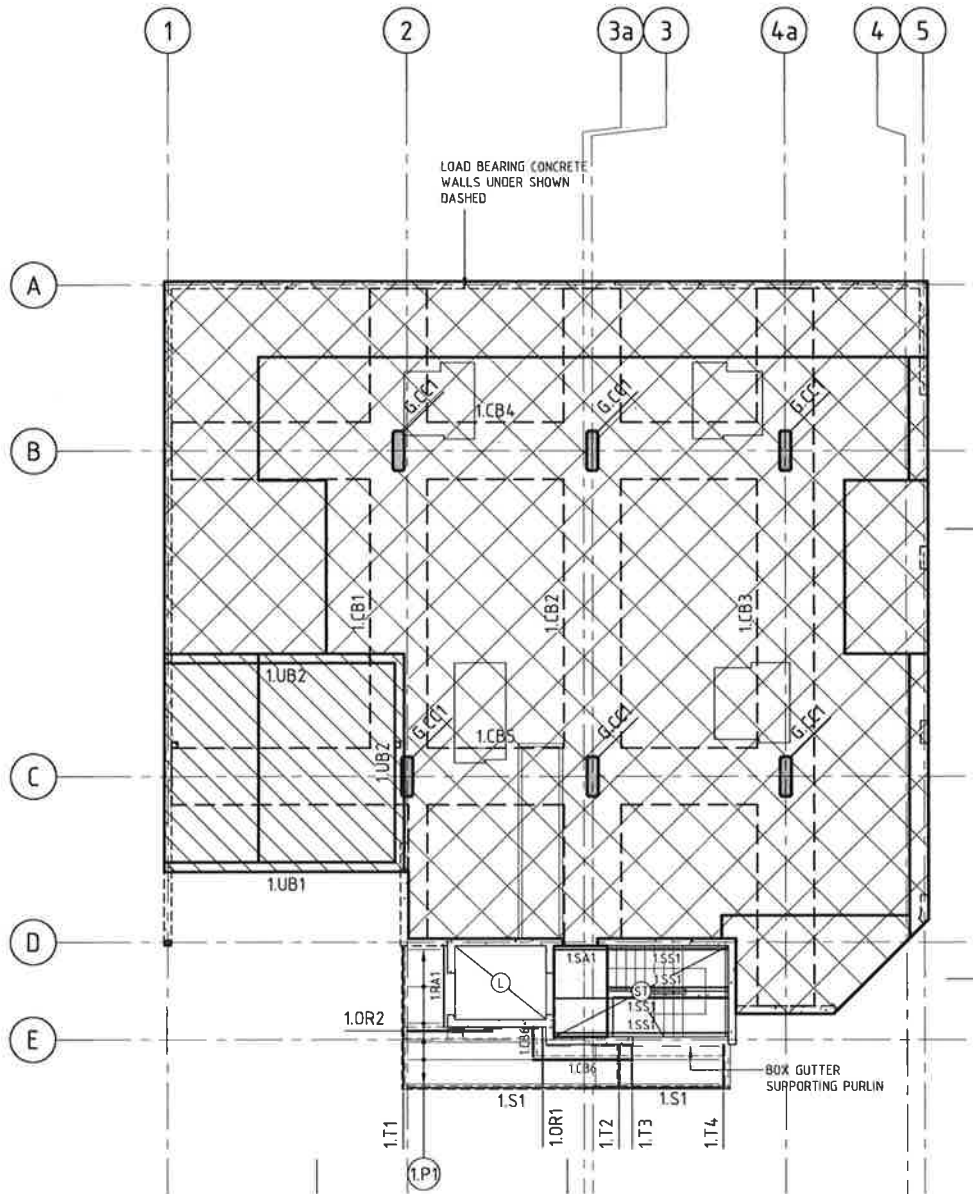
$$\text{Addition req req} = 1537 - 550 = 987 \text{ mm}^2$$

Adopt additional N16 @ 200 ϕ (UMB)

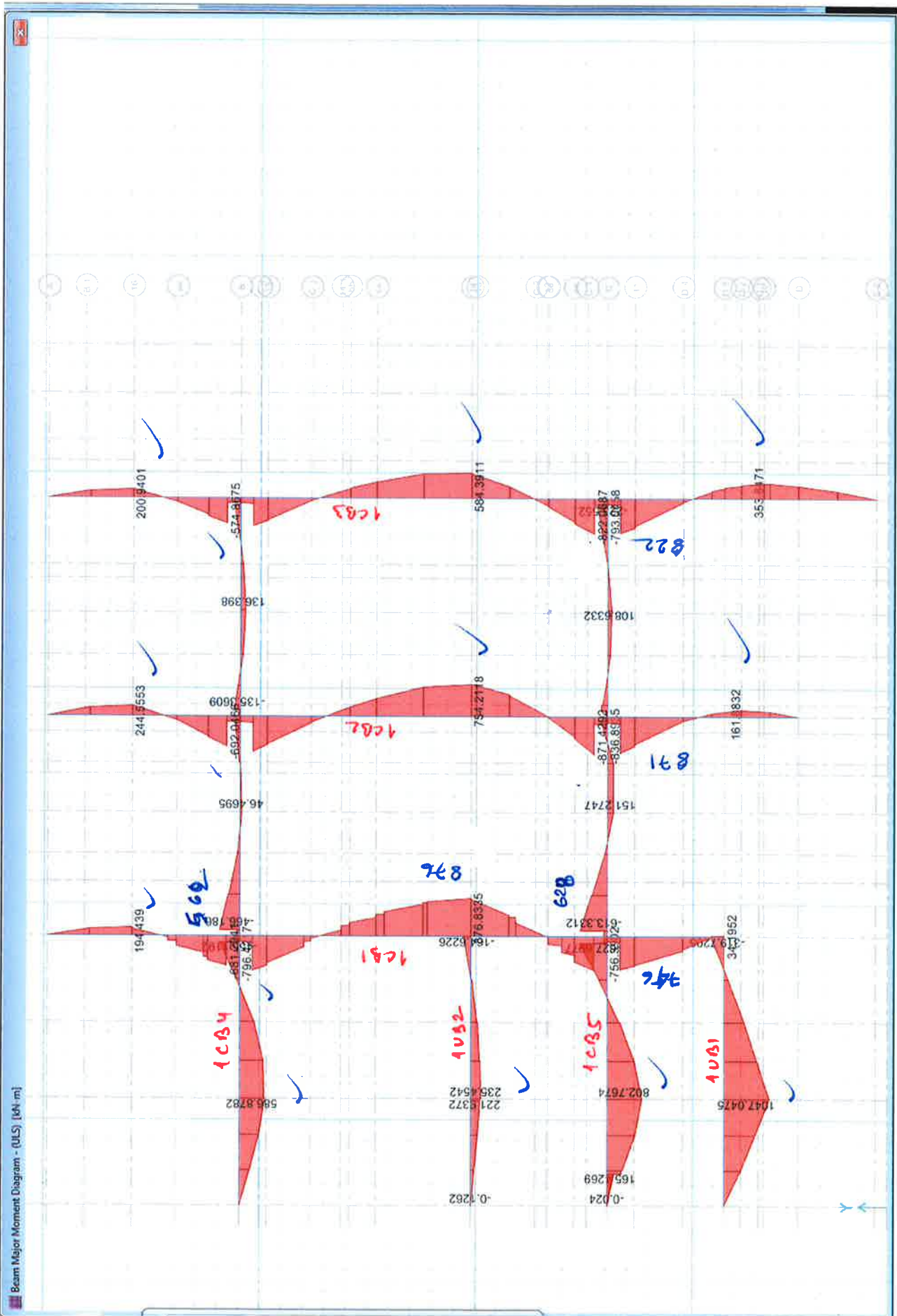
Checked :

Date :/..../.....

Concrete Beams.



SC145



Ultimate moment on concrete Beams

Upstand beam 1UB1

$$M_{max} = 1047 \text{ kNm}$$

900 deep

$$A_{st} \text{ req} = \frac{1047 \times 10^6}{360 \times 800} = 3635 \text{ mm}^2$$

Adopt 5N32 bars bottom.

And 3N32 bars top.

each face N12 @ 200 c/c

Lys N12 @ 200 c/c

Upstand beam 1UB2

Concrete beam @ edge of higher slab.

250 wide

570 deep

$$[+ftd \text{ depth} = 375 + 275 = 650]$$

$$M^* = 235 \text{ kNm}$$

$$A_{st} \text{ req} = \frac{235 \times 10^6}{360 \times 500} = 1305 \text{ mm}^2$$

Adopt 3N24 Top, 3N24 B.

w10 @ 200 c/c

1CB1 → 500-1500 wide 650 deep

Take $d = 500 \text{ mm}$

$$A_{st} \text{ req} = \frac{1}{360 \times 500} \times M^* = 5.56 \times 10^{-6}$$

778 → 4325 mm²

876 → 4870 mm²

Checked :

Date : .../.../...

Concrete beam 1CB1.

Width varies from 500 - 1500, depth 650

For -ve reo $d_{eff} = 650 - 30 - 100 = 520 \text{ mm}$

+ve reo $d_{eff} = 650 - 30 - 65 = 555 \text{ mm}$
 $\approx 550 \text{ mm.}$

Area of steel required. = $\frac{M^*}{0.8 \times 0.9 \times 500 \times d}$

for -ve = $5.34 \times 10^{-6} \cdot M^* = 5.34 \text{ M} \rightarrow M \text{ in kNm.}$

+ve = $5.05 \times 10^{-6} \cdot M^* = 5.05 \text{ M} \rightarrow M \text{ in kNm}$

Moment	Ast required	Adopt
- 746 kNm	3984 mm ²	Adopt 5N32 (T)
+ 876 kNm	4424 mm ²	Adopt 12N24 (B)
- 796 kNm	4250 mm ²	Adopt 11N24 (T)
+ 194 kNm	980 mm ²	Adopt 7N24 (B)

Adopt N12 @ 200 $\frac{1}{2}$ @ 500 wide &
 2 N12 @ 200 $\frac{1}{2}$ @ 1500 wide beam.

Concrete beam 1CB2

1500 wide 650 deep

Moment	Ast Req.	Adopt
+ 162 kNm	818 mm ²	7N24 (B)
- 871 kNm	4657 mm ²	12N24 (T)
+ 755 kNm	3813 mm ²	10N24 (B)
- 692 kNm	3695 mm ²	10N24 (T)
+ 245 kNm	1237 mm ²	7N24 (B)

Adopt 2/N12 @ 200 $\frac{1}{2}$ lig.

Checked :

Date : .../.../...

Concrete beam 1CB3.

1500 wide 650 deep.

Moment	Ast. Req.	Adopt
⊕ 354 kNm	1788 mm ²	7 N24 (B)
⊖ 822 kNm	4390 mm ²	12 N24 (T)
⊕ 585 kNm	2954 mm ²	9 N24 (B)
⊖ 574 kNm	3065 mm ²	9 N24 (T)
⊕ 200 kNm	1010 mm ²	7 N24 (B)

Adopt 2/N12 @ 200 c/c Lgs.

Concrete beam 1CB4. — 1500 wide 650 deep

Moment	Ast. Req.	Adopt
⊕ 586 kNm	2960 mm ²	10 N24 (B)
⊖ 562 kNm	3001 mm ²	10 N24 (T)
Rest. Not critical Use (7 N24 T & B)		

Adopt 2/N12 @ 200 c/c Lgs.

Concrete beam 1CB5 — 1500 wide 650 deep.

Moment	Ast. Req.	Adopt
⊕ 803 kNm	4055 mm ²	12 N24 (B)
⊖ 628 kNm	3354 mm ²	11 N24 (T)
Rest not critical Adopt 7 N24 T & B		

Adopt 2/N12 @ 200 c/c

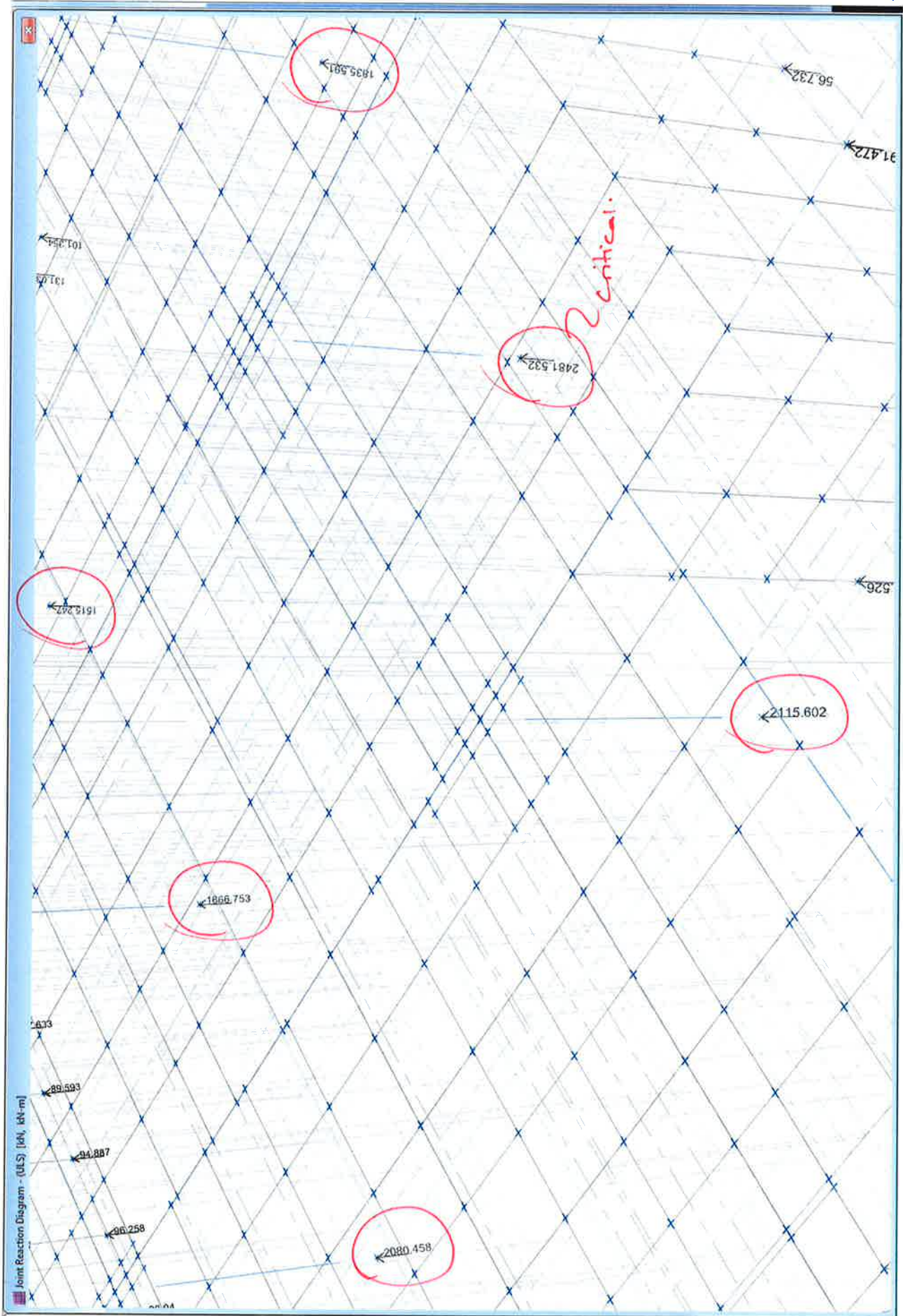
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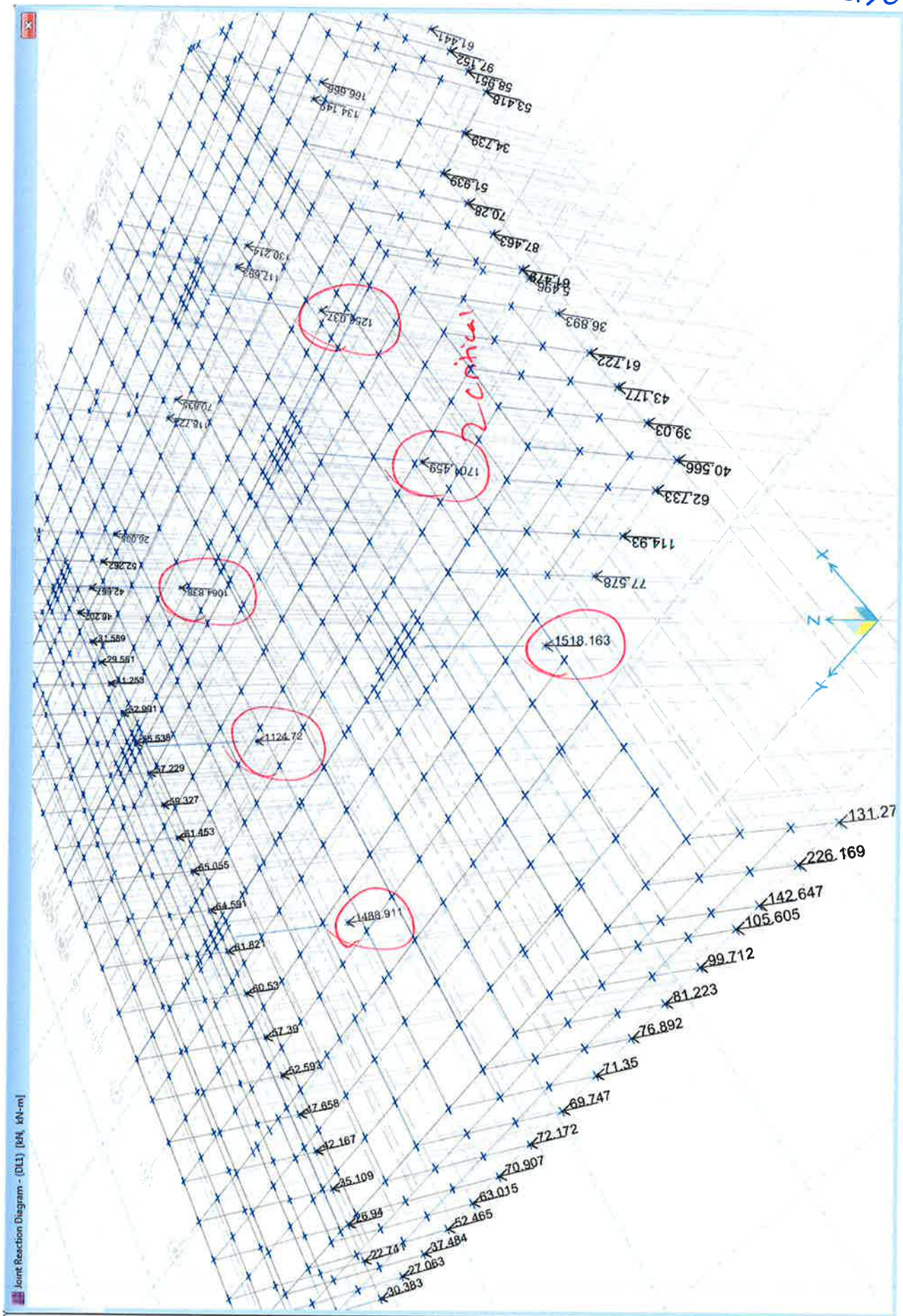
171016g

SC149

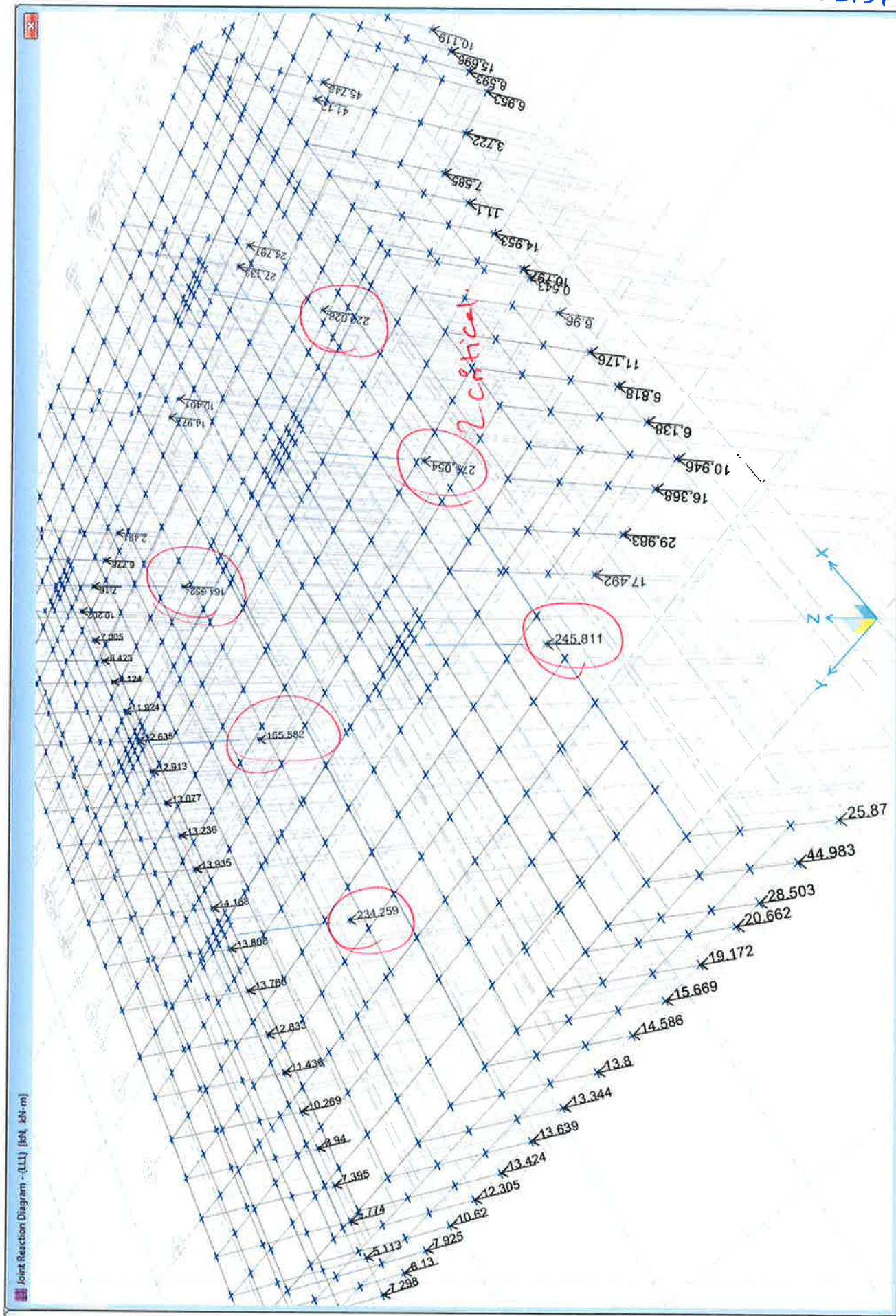
Ultimate Reaction on Columns



Total Dead load Reaction on Columns



Total Live load Reaction on Columns



Column design. (GCC1)

$$N^* = 2500 \text{ kN. (From SAFE)}$$

Design column from RAPT.

1050x300 Column with 10 N28 vertical bar

Refer to result from ~~SA~~ ^{OK} RAPT.

Column Under Fire (120/120/120)

$$P_{DL} = 1704 \text{ kN.}$$

$$N_f^* = P_{DL} + 0.4 P_u = 1814 \text{ kN}$$

$$P_u = 275 \text{ kN}$$

With 300x1050 column size

$f'_c = 50 \text{ MPa}$, Cover as 50mm. & 10 N28 vert. bar

$$\eta = \frac{N_f^*}{0.7 \left(\frac{A_c f'_c}{1.5} + \frac{A_s f_{sy}}{1.5} \right)}$$

$$= 0.196$$

$$\text{red ratio } (\rho) = 0.25$$

\therefore From table 5.6.4 A_s 3600

300 wide x 1050 long Concrete Column

with $F'_c = 50 \text{ MPa}$ and A_{eo} as 10 N28 vertical.

Pad footing PF1

Adopt

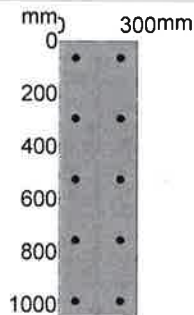
3400x3400x700 deep - F'_c 32 MPa

N20 @ 200 $\frac{1}{4}$ each way
bottom

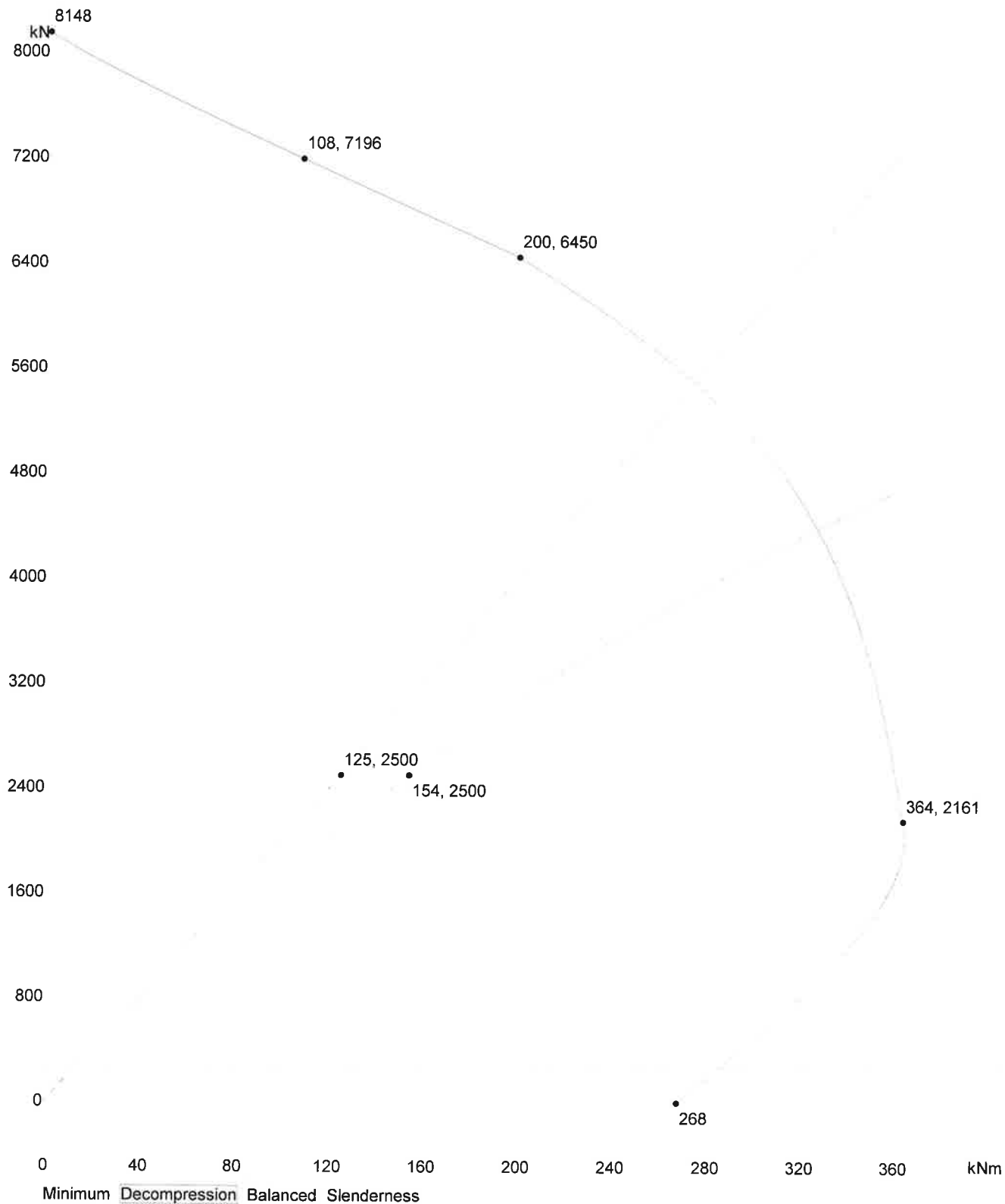
Checked :

Date : .../.../...

Rectangle 1050mm Deep x 300mm
Reinforcement Bar, 10 N28
Reinforcement Ratio - 1.96%
Australia - AS3600-2009
Australia - Australian Materials - 2009
Concrete Type - 40MPa
Composite Elements 90.00 degrees clockwise, - Left Face in
Compression
Length Unsupported = 2800mm
Effective Length Factor Braced = 1.00
Smaller End Moment = 0kNm
Larger End Moment = 125kNm
Minimum Moment = 38kNm
Buckling Load = 13278kN
Magnified Moment = 154kNm
Maximum Moment = 361kNm
Slenderness - Column OK



Column GCC1



PAD & PIER FOOTING DESIGN FOR BUILDING STRUCTURES

Pad footing PF1.

Type of Footing =	S	S=Square, R=Rectangular, B=Bored Pier
Side Dimension =	3300 mm	
Auto Size Calculation		
Specific Size Calculation		
Footing Depth =	450 mm	
Footing Concrete f'_c =	20 MPa	
Vertical Load =	2500.000 kN	Overturning Loads Permanent? N Y=Yes, N=No
Horizontal Load =	0.000 kN	
Applied Moment =	0.000 kNm	
Soil Type =	Stiff sandy clay, gravelly clay, sandy silt, compacted clay fill (Class II) ▼ (AS 4678-2002 Table D4)	
Cohesion =	5 kPa	Concrete Slab ? N Y=Yes, N=No
Friction Angle =	30 degrees	
Density =	18.0 kN/m³	
Bearing Capacity =	540 kPa	

RESISTANCE AGAINST UPLIFT

Footing weight =	117.61 kN
Interacting soil weight =	14.46 kN
Cohesion force on vertical faces =	14.85 kN
Contributing slab weight =	0.00 kN
0.9*Total load resisting uplift =	119.22 kN
Ratio 0.9*Resistance:Uplift =	N/A :OK

RESISTANCE TO OVERTURNING

Disturbing Moment at point 'A' on base =	0.00 kNm
Passive pressure strength at surface =	17.32 kPa
Passive pressure strength at base =	41.62 kPa
Depth, Z, to passive pressure switch =	268 mm
Passive pressure strength at depth Z =	31.79 kPa
Total passive soil force (Upper zone, R_1) =	25.85 kN
Total passive soil force (Lower zone, R_2) =	7.09 kN
M.R. due to passive soil forces R_1 & R_2 =	4.66 kNm
M.R. due to cohesion on vertical faces =	24.50 kNm
M.R. due to friction on vertical faces =	4.82 kNm
M.R. due to gravity & applied vert. loads =	4319.06 kNm
0.8*ΣM.R. (Due to all effects) =	3482.43 kNm
Ratio (0.8*ΣM.R. : O.T. about point 'A') =	N/A :OK

BEARING PRESSURE

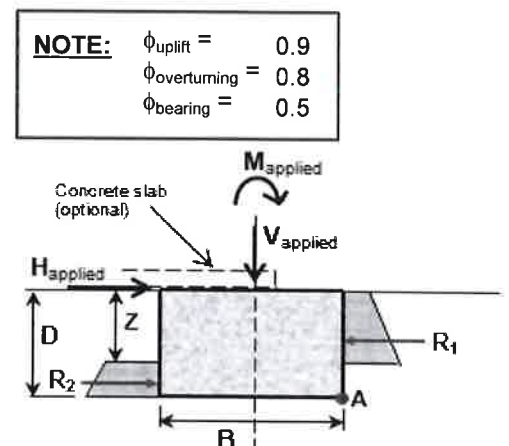
Moment at centreline of base of footing =	0.00 kNm
Net Vertical Force at base of footing, $F_{v,u}$	2641.13 kN
Effective eccentricity of vertical load, e_v =	0 mm
Width of bearing pressure block, W_{bp} =	3300 mm
Maximum soil bearing capacity =	2940.3 kN

> 2641.13 kN :OK

=> ADOPT 3300 mm sq. x 450 mm deep concrete pad.

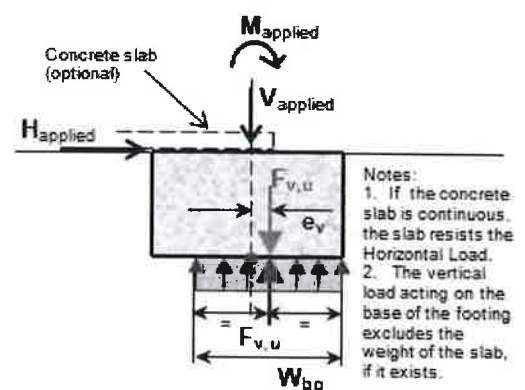
(NOTE: No concrete slab has been allowed for in the design.)

FOOTING REINFORCEMENT:



Forces Acting on Footing

Showing Soil Passive Pressure Distribution



Forces Acting on Footing

Showing Vertical Bearing Pressure Distribution

End of calculation

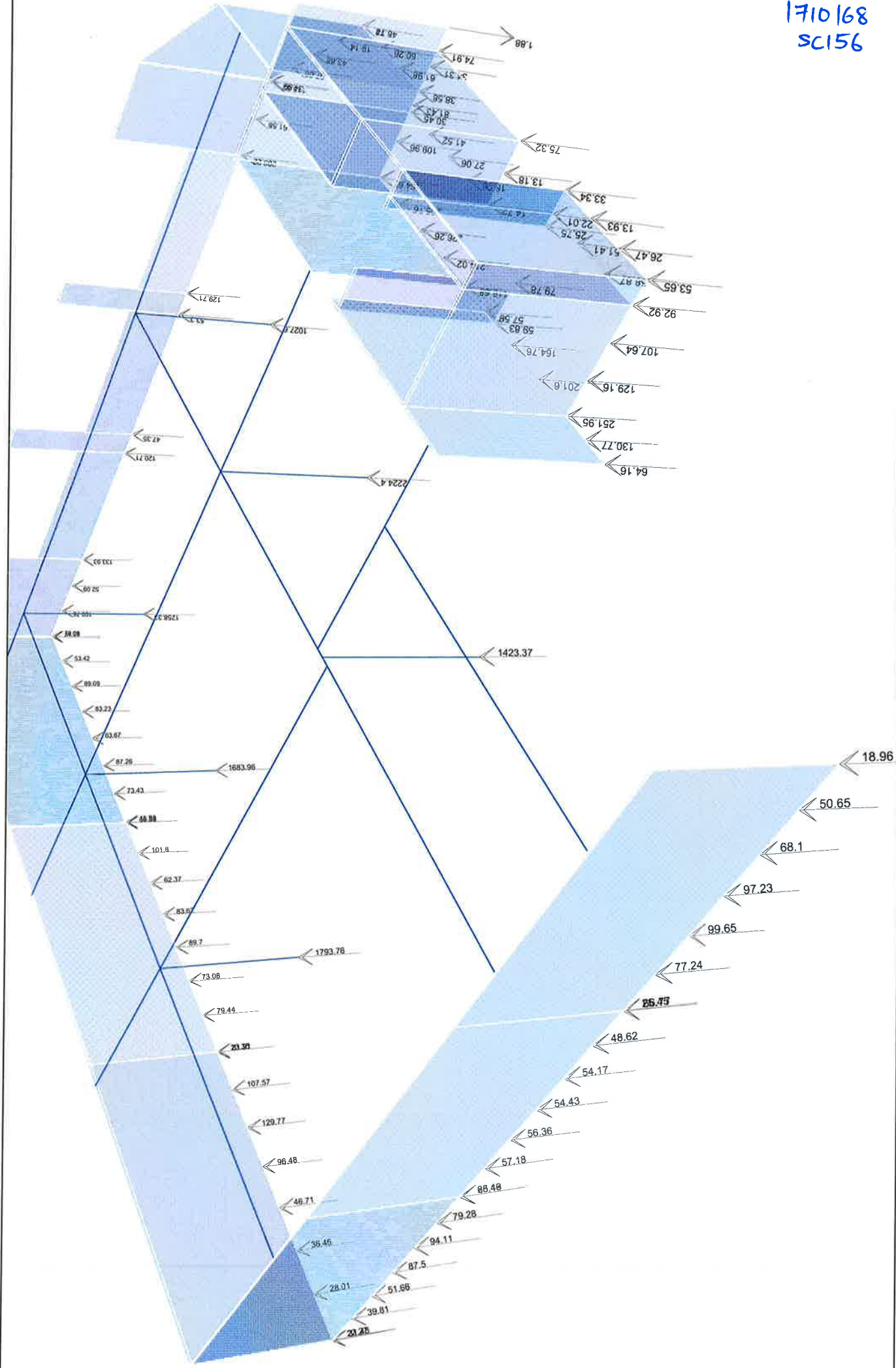
Req. Cals for pad footing PF-1.

PAD FOOTING	PF1		L(mm)		W (mm)	
	1050		300			
column size	DL	LL	KN	KN	KN	KN
	1704.00	275.00	1979.00	180	10.99	3.32
Vertical Load	1704.00		275.00		10.99	
	KN		KN		m2	
Effective depth	650		in mm			
	.7xa(sup)		105 mm			
Design bearing press	216.35		Kpa			
	935.68		KNm		275.20	
Total Max moment	1176.06		KNm		Moment per unit width	
	Area of steel req		1176.06			
Area of steel in %	0.0018		Max			
	Min area of steel req%		0.0015		0.0018	
Area of steel	1176.06		mm2		Adopt	
	Check Flexural Shear		662.03		N20 at 200	
Flexural Shear V*max	685.69		KN		Adopted area steel%	
	V*punch		2151.59		0.0024	
f _{cv}	1.92		Mpa			
	ØVuo		4638.11			
Check Punching Shear	685.69		KN			
	V*punch		2151.59			
f _{cv}	1.92		Mpa			
	ØVuo		4638.11			

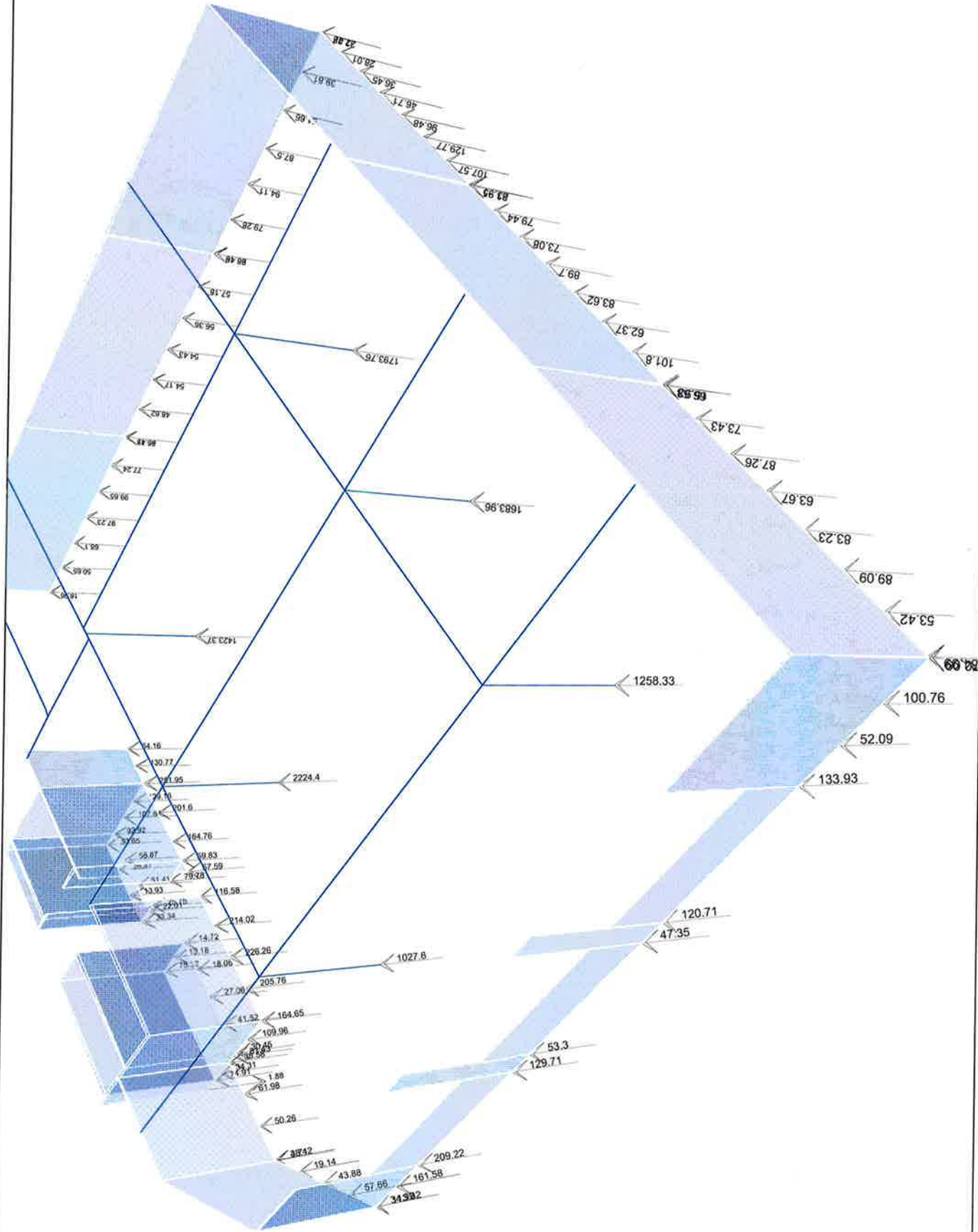
f_{ct} (Mpa) 32 f_{sy} (Mpa) 500 f_{ct}.f 3.39

1550 mm2

β1 0.8 β2 1 β3 1 f_{cv} (Mpa) 3.17 4



Result From ETAB3.



Critical line load. Footing M1 and M

Result from ETAB. - Along grid A.

length of wall = 20m.

$$\begin{aligned}\text{Ultimate Reaction} &= 54 + 89 + 84 + 64 + 88 + 74 + 67 \\ &\quad 102 + 63 + 84 + 90 + 73 + 80 + 84 + 108 \\ &\quad 130 + 97 + 47 + 37 + 28 + 33. \\ &= 520 + 684 + 372 \\ &= 1576.\end{aligned}$$

$$\therefore \text{Ultimate Line load} = \frac{1576}{20} = 78.8 \text{ kN/m}$$

Say 100 kN/m.

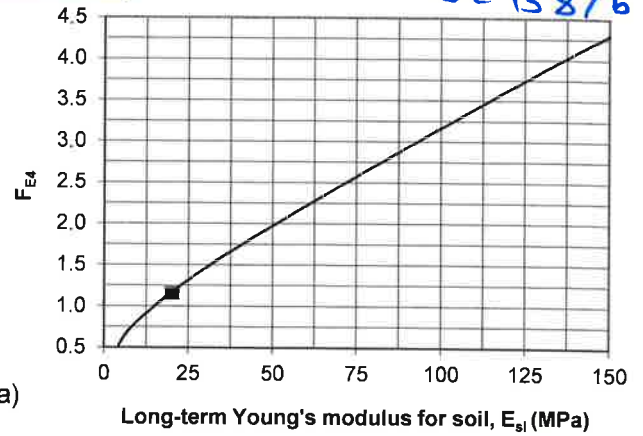
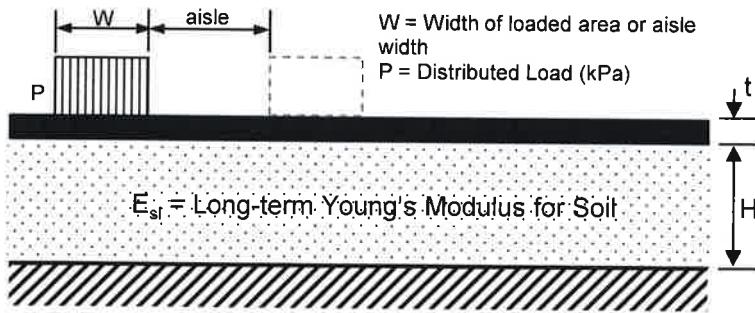
M1 - Adopt 600 wide footing and 700 deep for bearing to the ~~wall~~ footing supporting wall panels.
10N20, (5T, 5B) W10 @ 800 c/c

M - Adopt 400 wide footing 700 deep for tie footings.
8N20 - 4T, 4B
W10 @ 800 c/c.

DISTRIBUTED LOADING (CHART 1.4)

Slab on Ground
Car Park

1710168
SC 158/b



$$F_4 = F_{E4} \cdot F_{H4} \cdot F_{S4} \cdot f_{all} \cdot 1000/P$$

f_{all} = Design Tensile Strength (MPa)

$$f_{cf} = 0.7 (f_c)^{0.5}$$

$$f_{all} = k_1 \cdot k_2 \cdot f_{cf}$$

$$f_c = 32 \text{ MPa}$$

$$f_{cf} = 3.96 \text{ MPa}$$

$$k_1 = 0.8$$

$$k_2 = 0.75$$

$$f_{all} = 2.38 \text{ MPa}$$

Long-term Young's Modulus

$$E_{sl} = 20 \text{ MPa}$$

Width of Loaded Area or Aisle

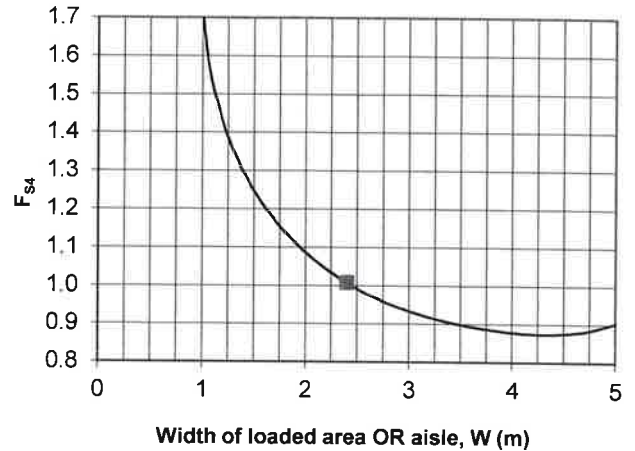
$$W = 2.4 \text{ m}$$

Depth of Soil Layer

$$H = 4 \text{ m}$$

Distributed Load

$$P = 5 \text{ kPa}$$



Factors

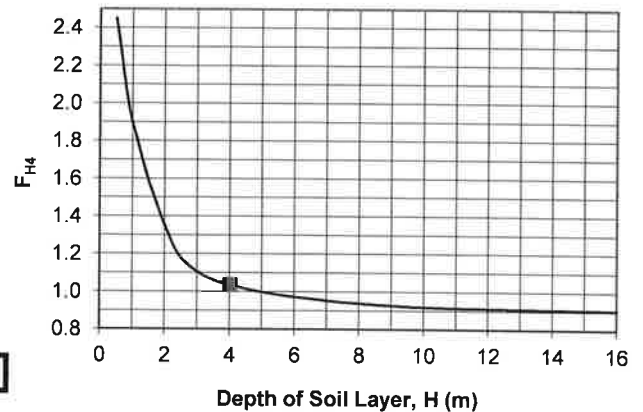
$$F_{E4} = 1.16$$

$$F_{S4} = 1.01$$

$$F_{H4} = 1.04$$

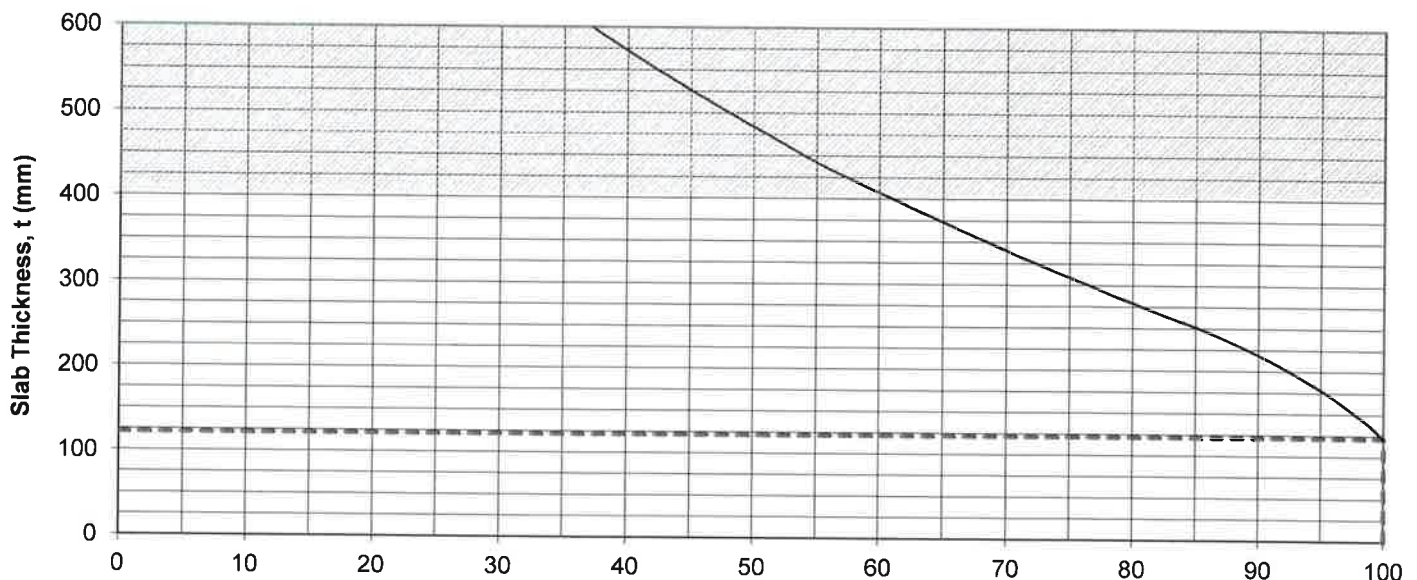
$$F_4 = 100.00 \text{ **}$$

F4 calculated as 578, which is off



Slab Thickness

$$t = 121 \text{ mm}$$



Note: Use computer analysis if the assessed thickness is in the shaded area ($t > 400 \text{ mm}$)

Outriggers DR 1.

we.

Line load. - loading width = $\frac{5.7}{2} \times 1.1$

$$dl = 0.4 \times 3.2 + 0.2 = 3.2 \text{ m.}$$

$$= 1.48 \text{ kN/m.}$$

$$L.L. = 0.25 \times 3.2 = 0.8 \text{ kN/m.}$$

$$w_{ult} = 1.48 \times 1.2 + 0.8 \times 1.5 = 3 \text{ kN/m.}$$

Point load $\rightarrow 0.6 \times 0.4 \times \frac{5.7}{2} = 0.7 \text{ kN.}$

$$P^* = 1.2 \times 0.7 = 0.84 \text{ kN.}$$

$$\therefore M^* = 3 \times \frac{1.5^2}{2} + 0.84 \times 1.5 = 4.64 \text{ kNm.}$$

$$I_{req} \text{ for } \frac{L}{250} \text{ of } dL_{is} = 0.78 \times 10^6$$

$$I_{req} \text{ for } \frac{L}{250} \text{ of } dL_p = 0.66 \times 10^6$$

$$I_{Total} = 1.44 \times 10^6 \text{ mm}^4.$$

Adopt 125 PFL

End G5 SHS. taking $0.6 \times 0.4 = 0.24 \text{ kN/m}$ line load. ~ 0.3

$$w_{ult} = 0.3 \times 1.2 = 0.36 \text{ kN/m.}$$

$$\text{Span} = 3.8 \text{ m.}$$

$$M_{max.} = 0.65 \text{ kNm.}$$

$$I_{req} \text{ for } \frac{L}{500} \text{ of } dL = 0.54 \times 10^6 \text{ mm}^4.$$

Adopt 65x65x4 SHS.

9F Point load from hanging $\rightarrow 1.8 \times 1.5 = 2.7 \text{ kN.}$

$$M_{max} = 2.7 \times \frac{3.8}{4} = 2.6 \text{ kNm.}$$

$$I_{req} = 649 \text{ kNm}^2$$

Checked :

Date :/...../.....

(OK)

Retaining wall.

Maintain top of Cast in situ wall at 33.3

Retaining height- $= 33.3 - 31.5$
 $= 1.8 \text{ m.}$

Adopt 200 thick wall to match with precast
Adjoining building load is not applied as it is
proposed to underpin the ex'g adjoining property.

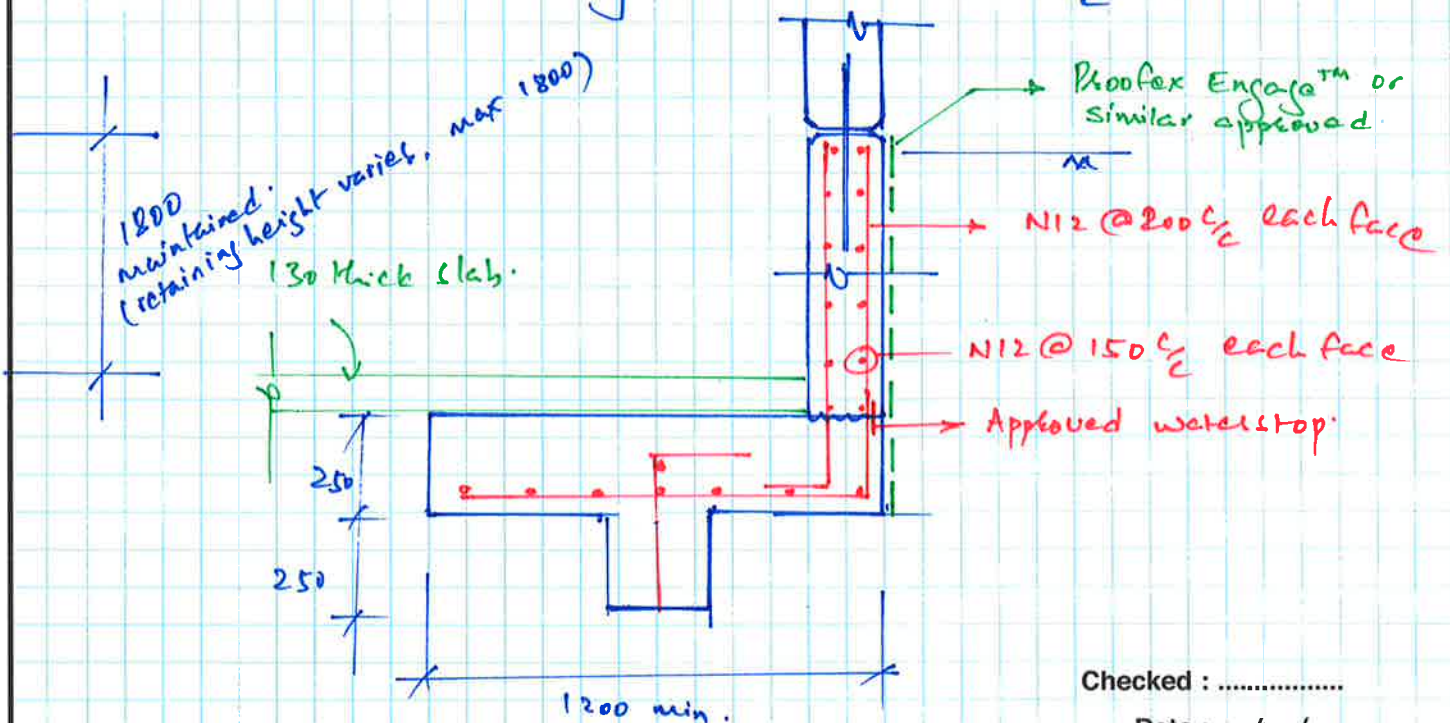
Live load = 5 kPa (Drive way and flat surface)

From Speed - Sheat.

N12 vertical @ 200 $\frac{L}{L}$ (OK)

N16 @ 300 $\frac{c}{c}$ each face horizontal - (ok) for strong degree of crack control

Alternately use - N12 @ 150 ^{Control} ₂ each face.



Checked :

Date :/...../.....

RETAINING WALL DESIGN

This design is in accordance with AS4678 — 2002 Earth-retaining Structures

WALL TYPE: REINFORCED CONCRETE

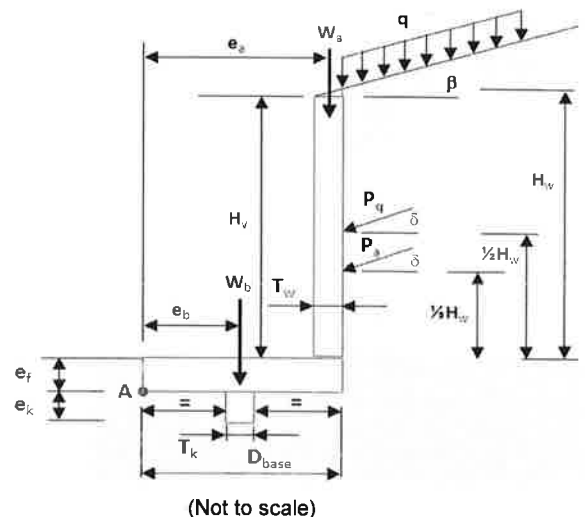
Design input parameters:

a) Geometry

H_v = Vertical height of wall =	1.80 m
T_w = Overall wall thickness =	0.200 m
D_{base} = Width of wall base (must be $\geq T_w$) =	1.20 m
β = Angle of backfill surface =	0.00 degrees
e_f = Embedment depth of footing =	0.25 m
T_k = Width of shear key =	0.25 m
e_k = Embedment depth of shear key =	0.25 m

b) Material Properties

δ = Friction angle between wall and soil =	18 degrees
γ_w = Density of wall material =	24.00 kN/m ³
Concrete compressive strength, f_c =	32 MPa
Reinforcement yield stress =	500 MPa
Vertical reinforcement bar diameter =	12 mm
Vertical reinforcement bar spacing =	200 mm



Retained soil type = Firm clay of medium to high plasticity, silty clay, sandy clay (AS 4678-2002 Table D4)

Cohesion, c_r :	5 kPa	Density, $\gamma_{s,r}$:	18.0 kN/m ³
Friction Angle, ϕ_r :	27 degrees		

Foundation soil type = Firm clay of medium to high plasticity, silty clay, sandy clay (AS 4678-2002 Table D4)

Cohesion, c_f :	5 kPa	Density, $\gamma_{s,f}$:	18.0 kN/m ³
Friction Angle, ϕ_f :	27 degrees	Bearing Strength, $q_{u,f}$:	360 kPa

c) Other

q_s = Surcharge on top of backfill surface = 5.00 kPa (Ref. AS 4678 — 2002 Table 1.1 for minimum value)

Design factors:

$k_1 = 1 / \sin(90^\circ) =$	1.00	$k_2 = \sin(90 - \phi_r) =$	0.89
$k_3 = \sin(90 + \delta) =$	0.95	$k_4 = \sin(\phi_r + \delta) =$	0.71
$k_5 = \sin(\phi_r - \beta) =$	0.45	$k_6 = \sin(90 - \beta) =$	1.00

Design height, $H_w = H_v + T_w \cdot \tan \beta =$ 1.80 m

NOTE:

Dead Load Factor =	1.2
Live Load Factor =	1.5
$\Phi_{\text{overturning}}$ =	0.8
Φ_{sliding} =	0.7
Φ_{bearing} =	0.33

Forces acting on the wall:

$K_a = (k_1 * k_2 / (k_3 + (k_4 * k_5 / k_6)^{0.5})))^2 =$	0.34
$P_a = 0.5 * K_a * \gamma_{s,r} * H_w^2 =$	10.05 kN/m
$P_q = K_a * q_s * H_w =$	3.10 kN/m

Check Overturning (Limit State Condition)

1. Overturning moments (about point A).

(a) Soil pressure

$$M_{a,A}^* = 1.2 * P_a * \cos(\delta) * (\frac{1}{2}H_w + e) = 9.75 \text{ kNm/m}$$

(b) Surcharge on top of backfill surface

$$M_{q,A}^* = 1.5 * P_q * \cos(\delta) * (\frac{1}{2}H_w + e) = 5.09 \text{ kNm/m}$$

$$\Rightarrow \text{Total Overturning Moment is } 14.84 \text{ kNm/m} \quad \leftarrow$$

2. Restoring moments (about point A).

$$M_{R,A} = 0.8 * [(W_a * e_a + W_b * e_b) + (P_a + P_q) * \sin(\delta) * D_{base}] = 15.60 \text{ kNm/m}$$

where

$$W_a \text{ (weight of the wall)} = \gamma_w * H_v * T_w = 8.64 \text{ kN/m}$$

$$e_a = D_{base} - T_w / 2 = 1.10 \text{ m}$$

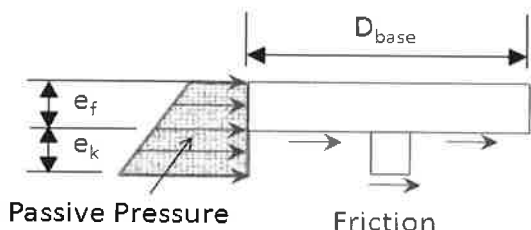
$$W_b \text{ (weight of footing, including shear key)} = 8.53 \text{ kN/m}$$

$$\therefore e_b = D_{base} / 2 = 0.60 \text{ m}$$

$$\Rightarrow \text{Total Restoring Moment is } 15.60 \text{ kNm/m} \quad \leftarrow$$

i.e. Restoring Moment > Overturning Moment \therefore Resistance to Overturning is OK

Check Sliding of Footing (Limit State Condition)



Resistance to horizontal sliding (Limit State Design)

a) Soil friction

$$N_f = W_a + W_b + (P_a + P_q) * \sin(\delta) = 21.24 \text{ kN/m}$$

$$u_f = \tan(\phi_f) = 0.51$$

$$\text{Friction resistance, } F_{\phi,f} = N_f * u_f = 10.82 \text{ kN/m}$$

b) Passive soil resistance

$$\text{Passive pressure strength at surface} = 16.32 \text{ kPa}$$

$$\text{Passive pressure strength at embedment depth, } (e_f + e_k) = 40.28 \text{ kPa}$$

$$\text{Passive soil resistance, } F_{p,f} = 14.15 \text{ kN/m}$$

Total Resistance, $F_{t,f}$

$$F_{t,f} = \Phi_{sliding} * (F_{\phi,f} + F_{p,f}) = 0.7 * (10.82 + 14.15) \text{ kN/m} = 17.48 \text{ kN/m} \quad \leftarrow$$

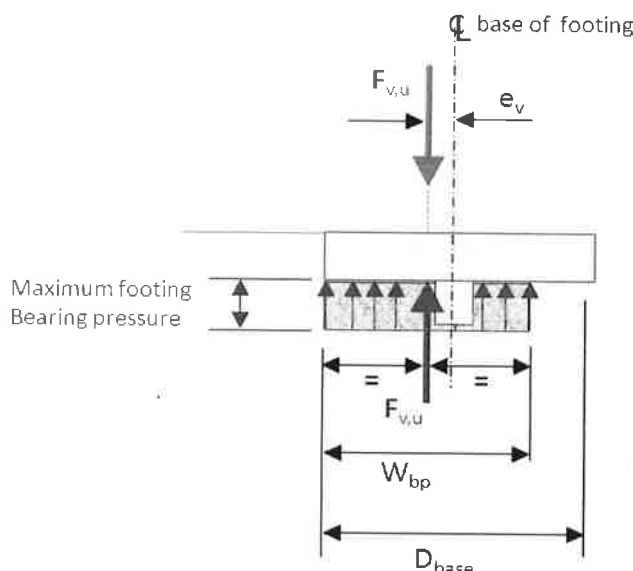
Total Sliding Force, $F_{s,f}$

$$F_{s,f} = (1.2 * P_a + 1.5 * P_q) * \cos(\delta) = 15.90 \text{ kN/m} \quad \leftarrow$$

i.e. Footing Sliding Resistance > Maximum Footing Sliding Force

\therefore Resistance to Sliding of Footing is OK

Check Soil Bearing Pressure (Limit State Condition)



Soil bearing pressure (Limit State Design)

Maximum Bearing Resistance, $\Phi \cdot q_{u,f}$

$$\Phi_{\text{bearing}} \cdot q_{u,f} = 0.33 \cdot 360 \text{ kPa} =$$

118.8 kPa ←

Maximum Bearing Pressure, $P_{m,f}$

Total Vertical Force acting on Footing =

$$F_{v,u} = 1.2 \cdot (W_a + W_b) + (1.2 \cdot P_a + 1.5 \cdot P_q) \cdot \sin(\delta) =$$

25.49 kN/m

Overturning Moment About Centre of Base:

$$M_{a,f} = 1.2 \cdot P_a \cdot \cos(\delta) \cdot [\frac{1}{2} H_w + e_f] =$$

9.75 kNm/m

$$M_{q,f} = 1.5 \cdot P_q \cdot \cos(\delta) \cdot [\frac{1}{2} H_w + e_f] =$$

5.09 kNm/m

Restoring Moment About Centre of Base:

$$M_{r,f} = 1.2 \cdot W_a \cdot (\frac{1}{2} D_{\text{base}} - e_a) =$$

-5.18 kNm/m

$$\text{Net Moment About Centre of Base} = M_{\text{net},f} = M_{a,f} + M_{q,f} + M_{r,f} =$$

9.66 kNm/m

$$\text{Width of wall footing} = D_{\text{base}} =$$

1.20 m

$$\text{Load Eccentricity About Centre of Base} = e_v = M_{\text{net},f} / F_{v,u} =$$

0.379 m

$$\text{Width of Bearing Pressure Block} = W_{bp} = 2 \cdot (\frac{1}{2} D_{\text{base}} - e_v) =$$

0.442 m

$$\text{The Maximum Soil Bearing Pressure, } P_{m,f} = F_{v,u} / W_{bp} =$$

58 kPa ←

i.e. Footing Bearing Resistance > Maximum Footing Bearing Pressure

∴ Soil Bearing Pressure Beneath Footing is OK

SUMMARY FOR REINFORCED CONCRETE RETAINING WALL FOOTING

WALL HEIGHT = 1.80 m

WALL WIDTH = 0.20 m

FOOTING = 1.20 m wide x 0.250 m deep. Grade N20 concrete.

Type additional information
into these three
lines of text

End of calculation

REINFORCED CONCRETE RETAINING WALL

This design is in accordance with AS 3600 — 2009 Concrete structures

With reference to p. ,

$$M^* = 1.2 * P_a * \cos(\delta) * \frac{1}{3}H_w + 1.5 * P_q * \cos(\delta) * \frac{1}{2}H_w = 10.87 \text{ kNm/m} \quad \leftarrow$$

Concrete wall thickness = 200 mm

Minimum cover to reinforcement = 50 mm

Effective depth to vertical reinforcement, d = 140 mm

Concrete 28 day compressive strength, f_c = 32 MPa

$$\text{Min. vertical reinforcement} = 0.20.D^2/d.f_{ct}.f_{sy} * 1000 \text{ mm}^2/\text{m} = 388 \text{ mm}^2/\text{m} \quad \text{Clause 8.1.4.1}$$

For min. horizontal reinforcement: Clause 11.6.2

Exposure classification / Degree of control over cracking: A1 / Strong

$$\Rightarrow \text{Min. horizontal reinforcement ratio, } p_{\min} = 0.0060$$

$$\text{Min. horizontal reinforcement} = p_{\min} * D * 1000 \text{ mm}^2/\text{m} = 1200 \text{ mm}^2/\text{m}$$

Vertical reinforcement

$$\text{Try N12 @ 200 c/c} \quad A_{st} = 550 \text{ mm}^2/\text{m} > \text{Min Vertical } A_{st} \therefore \text{OK}$$

$$\phi_b.M_u = 0.8*f_{sy}.A_{st}.d*(1 - 0.6*(A_{st}.f_{sy}/(b.d.f_c))) = 29.7 \text{ kNm/m} > M^* = 10.87 \text{ kNm/m} \therefore \text{OK}$$

Horizontal reinforcement

$$\text{Try N16 @ 150c/c} \quad A_{st} = 1333 \text{ mm}^2/\text{m} > \text{Min Horizontal } A_{st} \therefore \text{OK}$$

ADOPT 200mm thick reinforced concrete wall.

Concrete Grade N32.

N12 @ 200 c/c vertical reinforcement placed at 140mm from the front face of the wall.

N16 @ 150c/c horizontal reinforcement.

Type any additional recommendation in this line

End of calculation

Alt. Use N12 @ 150 c/c each face
ie $A_{st} = 1466 \text{ mm}^2$.

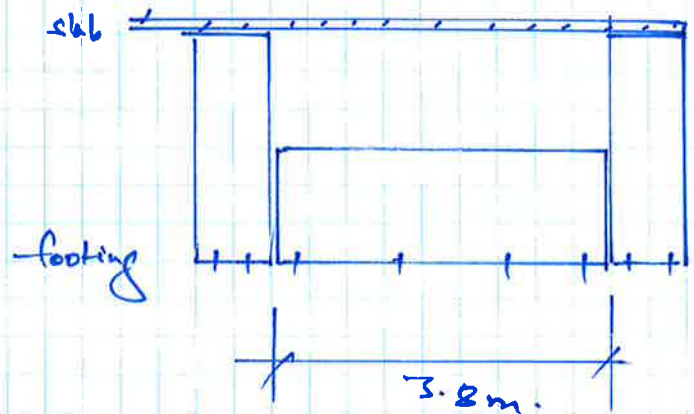
Precast panel retaining wall.

$$\text{Span} = 3.8 \text{ m}$$

Max. retaining height-

$$= 32.9 - 31.5$$

$$= 1.4 \text{ m} \sim 1.5 \text{ m.}$$



@ 1.5 m

$$K_o \gamma H = 0.6 \times 18 \times 1.5 \\ = 16.2 \text{ kPa.}$$

$$K_o q = 0.6 \times 5 = 3 \text{ kPa.}$$

@ 1.0 m.

$$K_o \gamma H = 0.6 \times 18 \times 1 \\ = 10.8 \text{ kPa.}$$

average at lower

$$K_o \gamma H = \frac{16.2 + 10.8}{2} \\ = 13.5 \text{ kPa.}$$

$$K_o q = 3 \text{ kPa}$$



$$\text{Ultimate} = 13.5 \times 1.25 + 3 \times 1.5 = 21.38 \text{ kPa.}$$

Checking wall horizontally only (supported from two sides)

$$M^* = 21.38 \times 1 \times \frac{3.8^2}{8} = 38.6 \text{ kNm.}$$

$$A_{st} \text{ req} = \frac{38.6 \times 10^6}{0.8 \times 0.9 \times 500 \times 150} = 714 \text{ mm}^2.$$

\therefore Adopt N12 @ 200 c/c int. face + SL82 each face

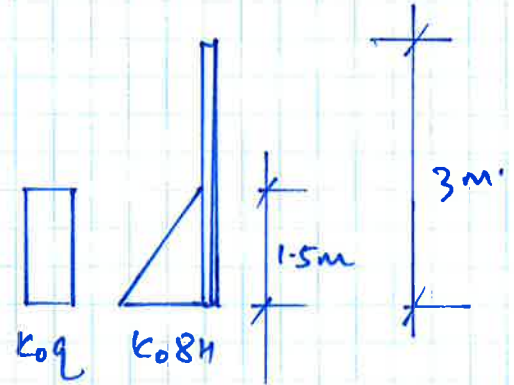
Checked :

Date :/..../....

full height panel. 600 wide.

$$k_{oq} = 3 \text{ kPa.}$$

$$k_{o8H} = 0.6 \times 18 + 1.5 \\ = 16.2 \text{ kPa.}$$



Reaction from loading width of
 $3.6 + 0.6 = 4.2 \text{ m.}$

$$k_{oq} = 4.2 \times 3 = 12.6 \text{ kPa.}$$

$$k_{o8H} = 4.2 \times 16.2 = 68.04.$$

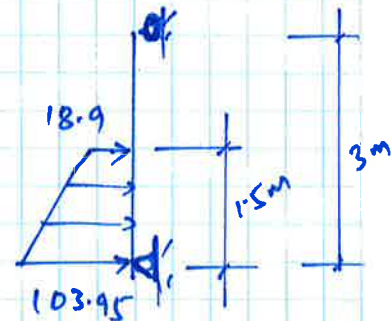
Ultimate

$$Q^* = 12.6 \times 1.5 = 18.9$$

$$p^* = 68.04 \times 1.25 = 85.05$$

Analysis from microstran

$$M_{max} = 31.9 \text{ kNm}$$



$$A_{st} \text{ req} = \frac{31.9 \times 10^6}{0.8 \times 0.9 \times 500 \times 150} \\ = 590 \text{ mm}^2$$

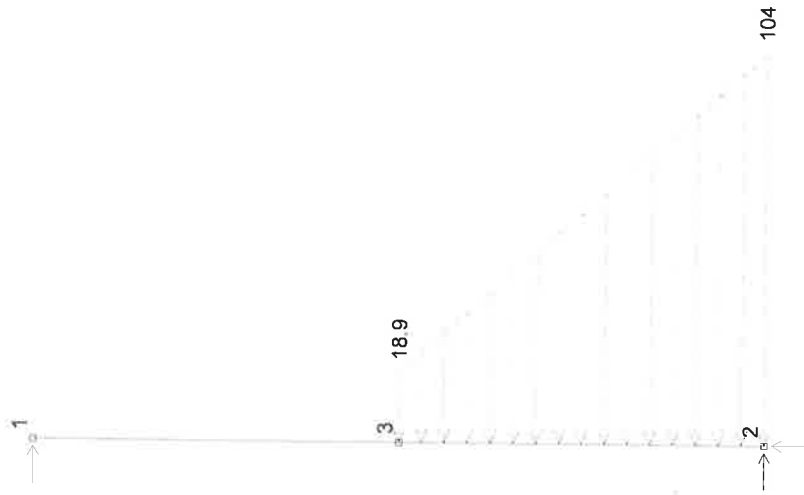
\therefore Adopt 8N16 - 4 each face full ht + 5L82 each face.

Load Cases:
1 P ult

Y
Z
X

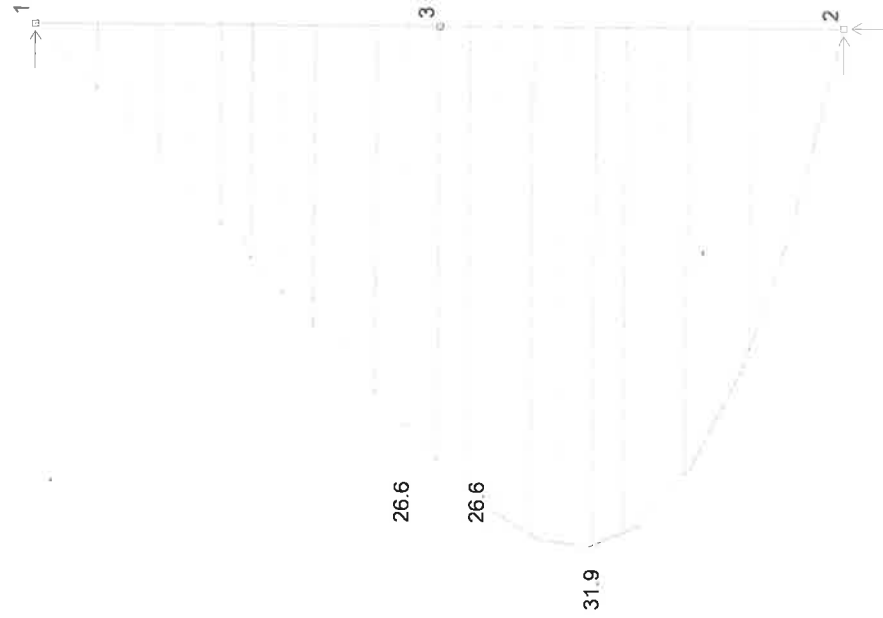
theta: 270 phi: 0

Microstran [V8.11r]



79135
8910171

Load Cases:
1 P ult



Y
Z
X

theta: 270 phi: 0

Bending Moment, Mz

Microstran [V8.11r]

C:\Users\ramesh\Desktop\Ramesh\Retaining wall

1710168
SC168

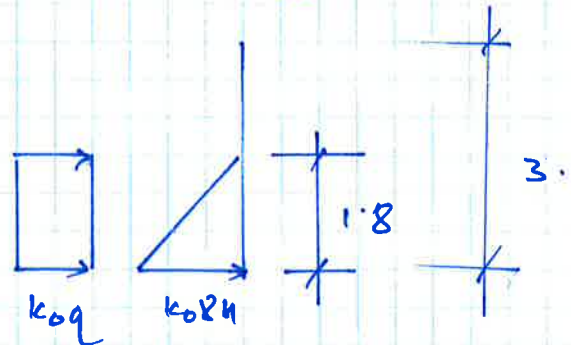
In general for full ht. panels. - Retaining wall,

$$k_{oq} = 3 \text{ kPa.}$$

$$k_{o8H} = 0.64 \times 18 \times 1.8$$

$$= 19.44$$

$$\approx 19.5 \text{ kPa}$$



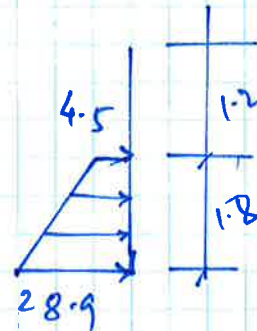
Ultimate \Rightarrow

$$q^* = 3 \times 9.5 = 4.5 \text{ kPa.}$$

$$p^* = 19.5 \times 1.25 = 24.4 \text{ kPa}$$

From microstran

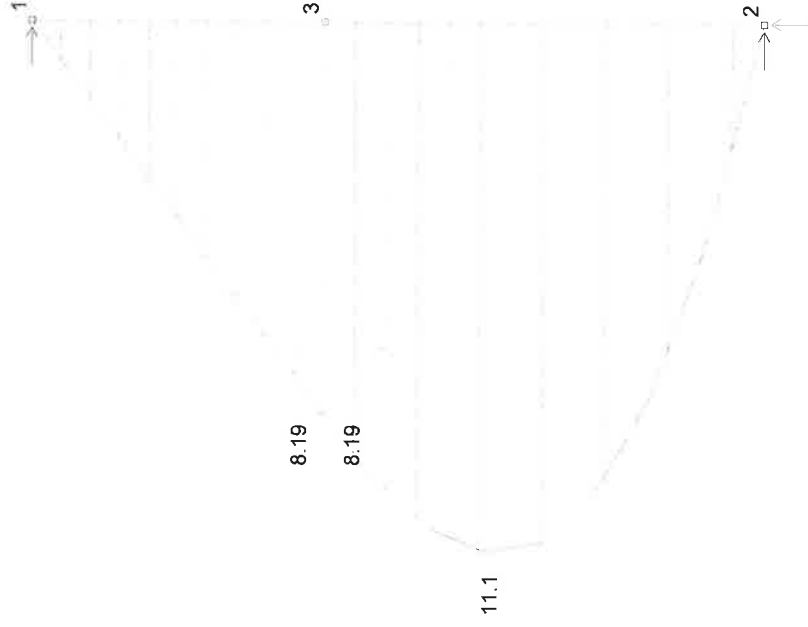
$$M_{max} = 11.1 \text{ kNm.}$$



$$A_{st \text{ req.}} = \frac{11.1 \times 10^6}{0.64 \times 0.9 \times 500 \times 150} = 257 \text{ mm}^2$$

Adopt SL82 each face + N12 @ 400 Σ vertical
(Additional full height bar)

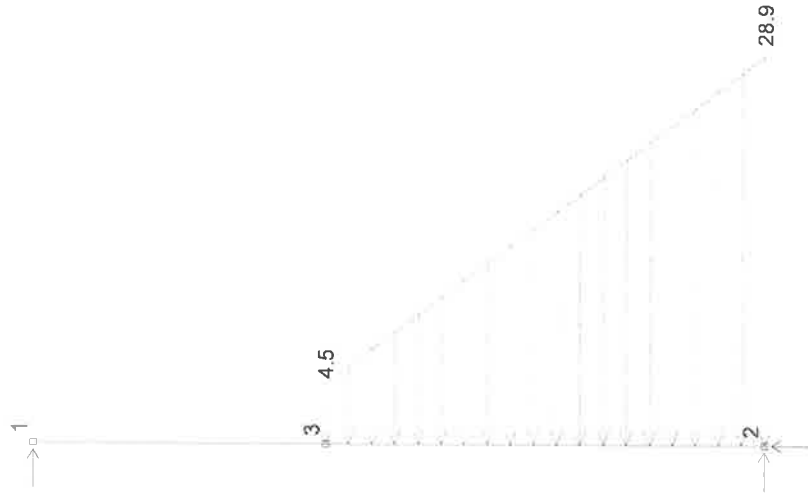
Load Cases:
1 P ult



Y
Z
X
theta: 270 phi: 0

Bending Moment, Mz

Load Cases:
1 P ult



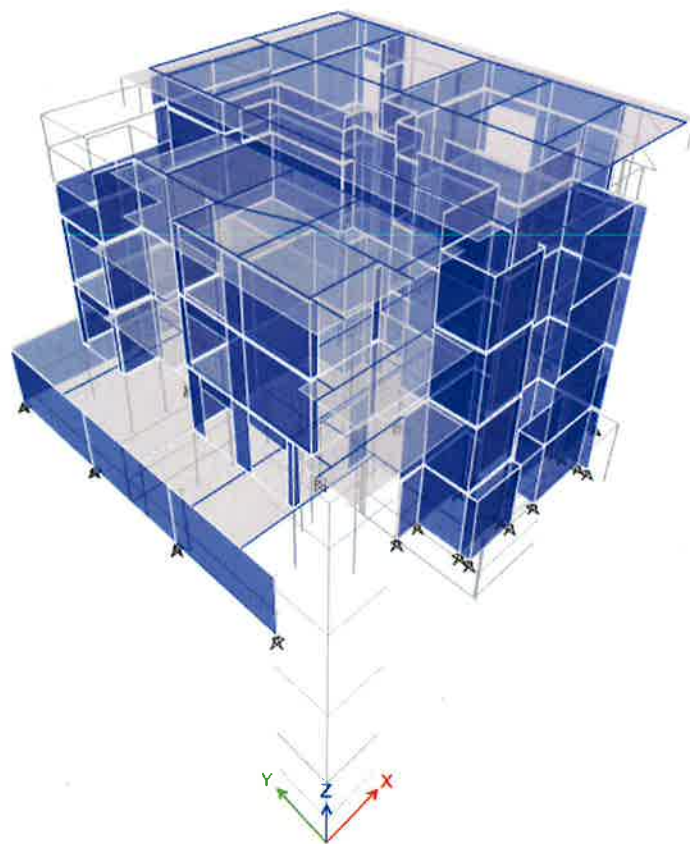
Y
Z
X

theta: 270 phi: 0

Microstran [V8.11r]

Building Stability Check.

ETABS[®] version 17
Integrated Building Design Software



Stability Report

Proposed Residential Development

Model File: Model 01, Revision 0

8/10/2018

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1 Loads

This chapter provides loading information as applied to the model.

1.1 Auto Seismic Loading

Table 1.1 - Auto Seismic - AS 1170:2007 (Part 1 of 2)

Load Pattern	Type	Direction	Eccentricity %	Ecc. Overridden	Period Method	kt	Top Story	Bottom Story	Site Class
EQ X	Seismic	X		No	Program Calculated	0.06	Lower Roof	Base	D
EQ X	Seismic	X + Ecc. Y	10	No	Program Calculated	0.06	Lower Roof	Base	D
EQ X	Seismic	X - Ecc. Y	10	No	Program Calculated	0.06	Lower Roof	Base	D
EQ Y	Seismic	Y		No	Program Calculated	0.06	Lower Roof	Base	D
EQ Y	Seismic	Y + Ecc. X	10	No	Program Calculated	0.06	Lower Roof	Base	D
EQ Y	Seismic	Y - Ecc. X	10	No	Program Calculated	0.06	Lower Roof	Base	D

Table 1.1 - Auto Seismic - AS 1170:2007 (Part 2 of 2)

kp	Z	Sp	μ	Period Used sec	Coeff Used	Weight Used kN	Base Shear kN
1	0.1	0.67	3	0.144	0.082187	11847.6024	973.715
1	0.1	0.67	3	0.144	0.082187	11847.6024	973.715
1	0.1	0.67	3	0.144	0.082187	11847.6024	973.715
1	0.1	0.67	3	0.148	0.082187	11847.6024	973.715
1	0.1	0.67	3	0.148	0.082187	11847.6024	973.715
1	0.1	0.67	3	0.148	0.082187	11847.6024	973.715

AS 1170 2007 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQ X according to AS 1170 2007, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 10% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, k_t [AS 6.2.3]

$$k_t = 0.06m$$

Structure Height Above Base, h_n

$$h_n = 12.575 \text{ m}$$

Factors and Coefficients

Probability Factor, k_p [AS Table 3.1]

$$k_p = 1$$

Hazard Factor, Z [AS Table 3.2]

$$Z = 0.1$$

Structural Performance Factor, S_p [AS Table 6.5(A)]

$$S_p = 0.67$$

Structural Ductility Factor, μ [AS Table 6.5(A)]

$$\mu = 3$$

Site Sub-soil Class [AS 4.1.1] = De - Deep or Soft Soil

Equivalent Lateral Forces

Seismic Design Action Coefficient, $C_{d(T)}$ [AS 6.2.1]

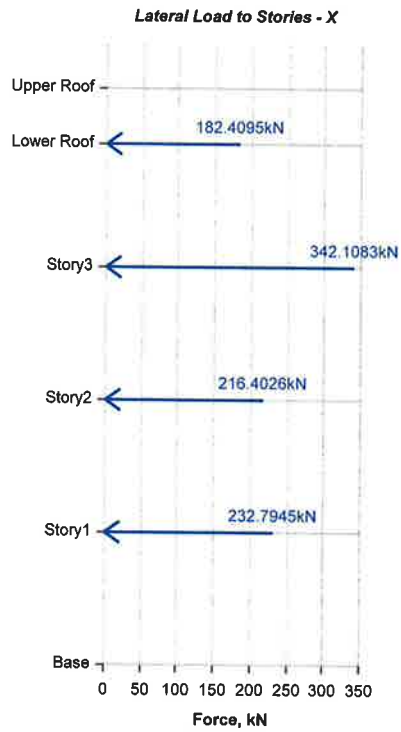
$$C_d(T_i) = \frac{k_p Z C_h(T_i) S_p}{\mu}$$

Calculated Base Shear

Direction	Period Used (sec)	$C_{d(T)}$	W (kN)	V (kN)
X	0.144	0.082187	11847.6024	973.715
X + Ecc. Y	0.144	0.082187	11847.6024	973.715
X - Ecc. Y	0.144	0.082187	11847.6024	973.715

Applied Story Forces

Loads



Story	Elevation m	X-Dir kN	Y-Dir kN
Upper Roof	13.925	0	0
Lower Roof	12.575	182.4095	0
Story3	9.615	342.1083	0
Story2	6.405	216.4026	0
Story1	3.195	232.7945	0
Base	0	0	0

AS 1170 2007 Auto Seismic Load Calculation

This calculation presents the automatically generated lateral seismic loads for load pattern EQ Y according to AS 1170 2007, as calculated by ETABS.

Direction and Eccentricity

Direction = Multiple

Eccentricity Ratio = 10% for all diaphragms

Structural Period

Period Calculation Method = Program Calculated

Coefficient, k_1 [AS 6.2.3]

$$k_1 = 0.06m$$

Structure Height Above Base, h_n

$$h_n = 12.575 \text{ m}$$

Factors and Coefficients

Probability Factor, k_p [AS Table 3.1]

$$k_p = 1$$

Hazard Factor, Z [AS Table 3.2]

$$Z = 0.1$$

Structural Performance Factor, S_p [AS Table 6.5(A)]

$$S_p = 0.67$$

Structural Ductility Factor, μ [AS Table 6.5(A)]

$$\mu = 3$$

Site Sub-soil Class [AS 4.1.1] = De - Deep or Soft Soil

Equivalent Lateral Forces

Seismic Design Action Coefficient, $C_{d(T)}$ [AS 6.2.1]

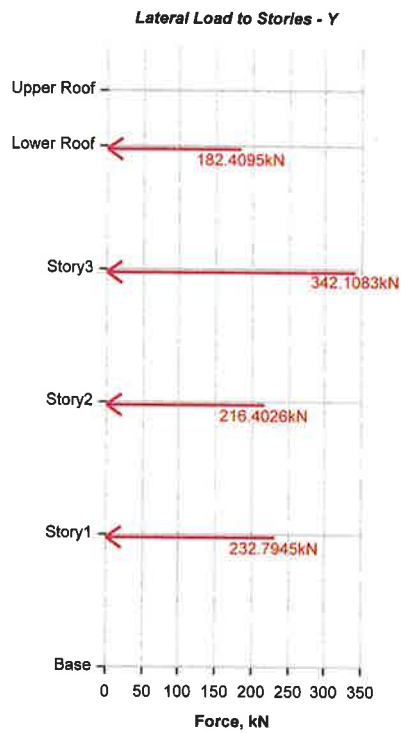
$$C_d(T_1) = \frac{k_p Z C_h(T_1) S_p}{\mu}$$

Calculated Base Shear

Direction	Period Used (sec)	$C_{d(T)}$	W (kN)	V (kN)
Y	0.148	0.082187	11847.6024	973.715
Y + Ecc. X	0.148	0.082187	11847.6024	973.715
Y - Ecc. X	0.148	0.082187	11847.6024	973.715

Applied Story Forces

Loads



Story	Elevation m	X-Dir kN	Y-Dir kN
Upper Roof	13.925	0	0
Lower Roof	12.575	0	182.4095
Story3	9.615	0	342.1083
Story2	6.405	0	216.4026
Story1	3.195	0	232.7945
Base	0	0	0

Loads

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2 Analysis Results

This chapter provides analysis results.

2.1 Story Results

Table 2.1 - Story Max/Avg Displacements

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Upper Roof	Dead	X	0.749	0.391	1.916
Upper Roof	Dead	Y	0.733	0.444	1.652
Lower Roof	Dead	X	0.655	0.358	1.829
Lower Roof	Dead	Y	0.204	0.046	4.213
Story3	Dead	X	0.451	0.261	1.729
Story3	Dead	Y	0.44	0.212	2.077
Story2	Dead	X	0.239	0.146	1.638
Story2	Dead	Y	0.220	0.13	1.756
Upper Roof	Live	X	0.148	0.081	1.836
Upper Roof	Live	Y	0.115	0.051	1.692
Lower Roof	Live	X	0.146	0.081	1.798
Story3	Live	X	0.089	0.053	1.667
Story3	Live	Y	0.073	0.03	2.408
Story2	Live	X	0.046	0.020	1.572
Story2	Live	Y	0.04	0.022	1.857
Upper Roof	SDead	X	0.05	0.027	1.882
Upper Roof	SDead	Y	0.046	0.027	1.697
Lower Roof	SDead	X	0.042	0.023	1.842
Lower Roof	SDead	Y	0.015	0.005	2.989
Story3	SDead	X	0.029	0.017	1.668
Story3	SDead	Y	0.029	0.014	1.994
Story2	SDead	X	0.015	0.009	1.577
Story2	SDead	Y	0.015	0.009	1.674
Upper Roof	Wind 1	X	0.211	0.205	1.032
Upper Roof	Wind 1	Y	0.036	0.032	1.142
Lower Roof	Wind 1	X	0.215	0.21	1.026
Lower Roof	Wind 1	Y	0.028	0.024	1.168
Story3	Wind 1	X	0.146	0.139	1.039
Story3	Wind 1	Y	0.022	0.016	1.383
Story2	Wind 1	X	0.085	0.08	1.062
Story2	Wind 1	Y	0.014	0.011	1.321
Upper Roof	Wind 2	Y	0.246	0.169	1.294
Lower Roof	Wind 2	X	0.068	0.034	2.028
Lower Roof	Wind 2	Y	0.166	0.138	1.136
Story3	Wind 2	Y	0.175	0.125	1.393
Story2	Wind 2	Y	0.107	0.076	1.435
Upper Roof	Wind 3	X	0.31	0.217	1.428
Lower Roof	Wind 3	X	0.258	0.19	1.359
Lower Roof	Wind 3	Y	0.117	0.083	1.421
Story3	Wind 3	X	0.205	0.146	1.411
Story2	Wind 3	X	0.121	0.086	1.405
Upper Roof	Wind 4	X	0.116	0.029	4.013
Upper Roof	Wind 4	Y	0.202	0.136	1.483
Lower Roof	Wind 4	Y	0.274	0.239	1.143
Story3	Wind 4	X	0.08	0.02	3.987

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	Wind 4	Y	0.197	0.125	1.574
Story2	Wind 4	X	0.047	0.014	3.252
Story2	Wind 4	Y	0.12	0.078	1.531
Upper Roof	EQ X 1	X	0.876	0.814	1.076
Upper Roof	EQ X 1	Y	0.144	0.093	1.549
Lower Roof	EQ X 1	X	0.733	0.713	1.028
Lower Roof	EQ X 1	Y	0.138	0.125	1.105
Story3	EQ X 1	X	0.622	0.572	1.087
Story2	EQ X 1	X	0.376	0.344	1.092
Upper Roof	EQ X 2	X	0.981	0.828	1.185
Lower Roof	EQ X 2	X	0.776	0.7	1.11
Lower Roof	EQ X 2	Y	0.225	0.185	1.218
Story3	EQ X 2	X	0.689	0.58	1.188
Story2	EQ X 2	X	0.416	0.35	1.189
Upper Roof	EQ X 3	X	0.831	0.801	1.038
Upper Roof	EQ X 3	Y	0.146	0.12	1.21
Lower Roof	EQ X 3	X	0.764	0.727	1.051
Story3	EQ X 3	X	0.59	0.562	1.051
Story3	EQ X 3	Y	0.102	0.064	1.599
Story2	EQ X 3	X	0.345	0.327	1.056
Story2	EQ X 3	Y	0.067	0.042	1.612
Upper Roof	EQ Y 1	Y	0.86	0.808	1.064
Lower Roof	EQ Y 1	X	0.115	0.111	1.043
Lower Roof	EQ Y 1	Y	0.765	0.762	1.004
Story3	EQ Y 1	Y	0.631	0.6	1.052
Story2	EQ Y 1	X	0.061	0.038	1.588
Story2	EQ Y 1	Y	0.399	0.375	1.064
Upper Roof	EQ Y 2	Y	0.963	0.842	1.144
Lower Roof	EQ Y 2	X	0.176	0.124	1.417
Lower Roof	EQ Y 2	Y	0.733	0.705	1.039
Story3	EQ Y 2	Y	0.703	0.604	1.164
Story2	EQ Y 2	Y	0.439	0.377	1.165
Upper Roof	EQ Y 3	X	0.127	0.091	1.398
Upper Roof	EQ Y 3	Y	0.802	0.778	1.03
Lower Roof	EQ Y 3	X	0.158	0.097	1.632
Lower Roof	EQ Y 3	Y	0.853	0.82	1.04
Story3	EQ Y 3	X	0.11	0.066	1.668
Story3	EQ Y 3	Y	0.649	0.597	1.087
Story2	EQ Y 3	X	0.061	0.044	1.379
Story2	EQ Y 3	Y	0.39	0.367	1.063
Upper Roof	Serv 01 (G)	X	0.799	0.418	1.913
Upper Roof	Serv 01 (G)	Y	0.779	0.471	1.654
Lower Roof	Serv 01 (G)	X	0.697	0.381	1.83
Lower Roof	Serv 01 (G)	Y	0.219	0.053	4.096
Story3	Serv 01 (G)	X	0.48	0.278	1.725
Story3	Serv 01 (G)	Y	0.469	0.226	2.071
Story2	Serv 01 (G)	X	0.254	0.155	1.634
Story2	Serv 01 (G)	Y	0.244	0.139	1.76
Upper Roof	Serv 02 (G+0.7Q)	X	0.903	0.474	1.904

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Upper Roof	Serv 02 (G+0.7Q)	Y	0.86	0.514	1.674
Lower Roof	Serv 02 (G+0.7Q)	X	0.799	0.438	1.826
Lower Roof	Serv 02 (G+0.7Q)	Y	0.24	0.051	4.733
Story3	Serv 02 (G+0.7Q)	X	0.542	0.316	1.718
Story3	Serv 02 (G+0.7Q)	Y	0.52	0.248	2.1
Story2	Serv 02 (G+0.7Q)	X	0.285	0.175	1.627
Story2	Serv 02 (G+0.7Q)	Y	0.272	0.154	1.77
Upper Roof	Serv 03 (G+Wx) Max	X	0.666	0.28	2.375
Upper Roof	Serv 03 (G+Wx) Max	Y	0.941	0.597	1.577
Lower Roof	Serv 03 (G+Wx) Max	X	0.56	0.244	2.3
Lower Roof	Serv 03 (G+Wx) Max	Y	0.324	0.146	2.218
Story3	Serv 03 (G+Wx) Max	X	0.389	0.185	2.101
Story3	Serv 03 (G+Wx) Max	Y	0.582	0.309	1.884
Story2	Serv 03 (G+Wx) Max	X	0.2	0.102	1.963
Story2	Serv 03 (G+Wx) Max	Y	0.313	0.188	1.664
Upper Roof	Serv 03 (G+Wx) Min	X	1.006	0.563	1.788
Upper Roof	Serv 03 (G+Wx) Min	Y	0.732	0.381	1.924
Lower Roof	Serv 03 (G+Wx) Min	X	0.87	0.508	1.712
Lower Roof	Serv 03 (G+Wx) Min	Y	0.296	0.106	2.779
Story3	Serv 03 (G+Wx) Min	X	0.617	0.376	1.643
Story3	Serv 03 (G+Wx) Min	Y	0.433	0.145	2.985
Story2	Serv 03 (G+Wx) Min	X	0.334	0.213	1.571
Story2	Serv 03 (G+Wx) Min	Y	0.219	0.088	2.501
Upper Roof	Serv 04 (G-Wx) Max	X	0.592	0.273	2.173
Upper Roof	Serv 04 (G-Wx) Max	Y	0.826	0.562	1.471
Lower Roof	Serv 04 (G-Wx) Max	X	0.524	0.254	2.066
Lower Roof	Serv 04 (G-Wx) Max	Y	0.355	0.213	1.665
Story3	Serv 04 (G-Wx) Max	X	0.343	0.181	1.897
Story3	Serv 04 (G-Wx) Max	Y	0.505	0.308	1.641
Story2	Serv 04 (G-Wx) Max	X	0.174	0.098	1.772
Story2	Serv 04 (G-Wx) Max	Y	0.27	0.19	1.418
Upper Roof	Serv 04 (G-Wx) Min	X	0.933	0.554	1.682
Upper Roof	Serv 04 (G-Wx) Min	Y	0.617	0.345	1.788
Lower Roof	Serv 04 (G-Wx) Min	X	0.833	0.518	1.609
Story3	Serv 04 (G-Wx) Min	X	0.571	0.371	1.538
Story3	Serv 04 (G-Wx) Min	Y	0.356	0.144	2.475
Story2	Serv 04 (G-Wx) Min	X	0.308	0.209	1.474
Story2	Serv 04 (G-Wx) Min	Y	0.176	0.09	1.961
Upper Roof	Serv 05 (G+Wy)	X	0.799	0.418	1.913
Upper Roof	Serv 05 (G+Wy)	Y	0.779	0.471	1.654
Lower Roof	Serv 05 (G+Wy)	X	0.697	0.381	1.83
Lower Roof	Serv 05 (G+Wy)	Y	0.219	0.053	4.096
Story3	Serv 05 (G+Wy)	X	0.48	0.278	1.725
Story3	Serv 05 (G+Wy)	Y	0.469	0.226	2.071
Story2	Serv 05 (G+Wy)	X	0.254	0.155	1.634
Story2	Serv 05 (G+Wy)	Y	0.244	0.139	1.76
Upper Roof	Serv 06 (G-Wy)	X	0.799	0.418	1.913
Upper Roof	Serv 06 (G-Wy)	Y	0.779	0.471	1.654
Lower Roof	Serv 06 (G-Wy)	X	0.697	0.381	1.83

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Serv 06 (G-Wy)	Y	0.219	0.053	4.096
Story3	Serv 06 (G-Wy)	X	0.48	0.278	1.725
Story3	Serv 06 (G-Wy)	Y	0.469	0.226	2.071
Story2	Serv 06 (G-Wy)	X	0.254	0.155	1.634
Story2	Serv 06 (G-Wy)	Y	0.244	0.139	1.76
Upper Roof	Serv 07 (G+0.7Wx+0.3Wy) Max	X	0.705	0.321	2.196
Upper Roof	Serv 07 (G+0.7Wx+0.3Wy) Max	Y	0.893	0.559	1.596
Lower Roof	Serv 07 (G+0.7Wx+0.3Wy) Max	X	0.601	0.284	2.112
Lower Roof	Serv 07 (G+0.7Wx+0.3Wy) Max	Y	0.293	0.118	2.471
Story3	Serv 07 (G+0.7Wx+0.3Wy) Max	X	0.416	0.213	1.954
Story3	Serv 07 (G+0.7Wx+0.3Wy) Max	Y	0.548	0.284	1.928
Story2	Serv 07 (G+0.7Wx+0.3Wy) Max	X	0.216	0.118	1.833
Story2	Serv 07 (G+0.7Wx+0.3Wy) Max	Y	0.292	0.173	1.687
Upper Roof	Serv 07 (G+0.7Wx+0.3Wy) Min	X	0.944	0.519	1.818
Upper Roof	Serv 07 (G+0.7Wx+0.3Wy) Min	Y	0.746	0.408	1.831
Lower Roof	Serv 07 (G+0.7Wx+0.3Wy) Min	X	0.818	0.47	1.741
Lower Roof	Serv 07 (G+0.7Wx+0.3Wy) Min	Y	0.241	0.059	4.105
Story3	Serv 07 (G+0.7Wx+0.3Wy) Min	X	0.576	0.347	1.662
Story3	Serv 07 (G+0.7Wx+0.3Wy) Min	Y	0.444	0.169	2.621
Story2	Serv 07 (G+0.7Wx+0.3Wy) Min	X	0.31	0.196	1.586
Story2	Serv 07 (G+0.7Wx+0.3Wy) Min	Y	0.227	0.103	2.203
Upper Roof	Serv 08 (G+0.5Wx+0.5Wy) Max	X	0.731	0.348	2.102
Upper Roof	Serv 08 (G+0.5Wx+0.5Wy) Max	Y	0.861	0.535	1.611
Lower Roof	Serv 08 (G+0.5Wx+0.5Wy) Max	X	0.627	0.311	2.017
Lower Roof	Serv 08 (G+0.5Wx+0.5Wy) Max	Y	0.272	0.1	2.71
Story3	Serv 08 (G+0.5Wx+0.5Wy) Max	X	0.434	0.231	1.878
Story3	Serv 08 (G+0.5Wx+0.5Wy) Max	Y	0.526	0.268	1.962
Story2	Serv 08 (G+0.5Wx+0.5Wy) Max	X	0.227	0.128	1.767
Story2	Serv 08 (G+0.5Wx+0.5Wy) Max	Y	0.279	0.164	1.704
Upper Roof	Serv 08 (G+0.5Wx+0.5Wy) Min	X	0.904	0.491	1.841
Upper Roof	Serv 08 (G+0.5Wx+0.5Wy) Min	Y	0.755	0.425	1.777
Lower Roof	Serv 08 (G+0.5Wx+0.5Wy) Min	X	0.784	0.445	1.762
Story3	Serv 08 (G+0.5Wx+0.5Wy) Min	X	0.55	0.328	1.677
Story3	Serv 08 (G+0.5Wx+0.5Wy) Min	Y	0.451	0.185	2.435
Story2	Serv 08 (G+0.5Wx+0.5Wy) Min	X	0.295	0.184	1.597
Story2	Serv 08 (G+0.5Wx+0.5Wy) Min	Y	0.232	0.113	2.052
Upper Roof	Serv 09 (G+0.3Wx+0.7Wy) Max	X	0.759	0.377	2.016
Upper Roof	Serv 09 (G+0.3Wx+0.7Wy) Max	Y	0.828	0.509	1.627
Lower Roof	Serv 09 (G+0.3Wx+0.7Wy) Max	X	0.656	0.34	1.931
Lower Roof	Serv 09 (G+0.3Wx+0.7Wy) Max	Y	0.25	0.081	3.087
Story3	Serv 09 (G+0.3Wx+0.7Wy) Max	X	0.453	0.25	1.808
Story3	Serv 09 (G+0.3Wx+0.7Wy) Max	Y	0.503	0.251	2.002
Story2	Serv 09 (G+0.3Wx+0.7Wy) Max	X	0.238	0.139	1.706
Story2	Serv 09 (G+0.3Wx+0.7Wy) Max	Y	0.265	0.153	1.725
Upper Roof	Serv 09 (G+0.3Wx+0.7Wy) Min	X	0.861	0.461	1.868
Upper Roof	Serv 09 (G+0.3Wx+0.7Wy) Min	Y	0.765	0.444	1.723
Lower Roof	Serv 09 (G+0.3Wx+0.7Wy) Min	X	0.748	0.419	1.787
Story3	Serv 09 (G+0.3Wx+0.7Wy) Min	X	0.521	0.307	1.695
Story3	Serv 09 (G+0.3Wx+0.7Wy) Min	Y	0.458	0.202	2.267

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Serv 09 (G+0.3Wx+0.7Wy) Min	X	0.278	0.172	1.611
Story2	Serv 09 (G+0.3Wx+0.7Wy) Min	Y	0.237	0.124	1.917
Upper Roof	Serv 10 (G-0.7Wx-0.3Wy) Max	X	0.654	0.316	2.07
Upper Roof	Serv 10 (G-0.7Wx-0.3Wy) Max	Y	0.812	0.535	1.519
Lower Roof	Serv 10 (G-0.7Wx-0.3Wy) Max	X	0.576	0.292	1.974
Lower Roof	Serv 10 (G-0.7Wx-0.3Wy) Max	Y	0.315	0.166	1.899
Story3	Serv 10 (G-0.7Wx-0.3Wy) Max	X	0.384	0.21	1.829
Story3	Serv 10 (G-0.7Wx-0.3Wy) Max	Y	0.494	0.283	1.743
Story2	Serv 10 (G-0.7Wx-0.3Wy) Max	X	0.198	0.115	1.717
Story2	Serv 10 (G-0.7Wx-0.3Wy) Max	Y	0.262	0.175	1.499
Upper Roof	Serv 10 (G-0.7Wx-0.3Wy) Min	X	0.893	0.514	1.738
Upper Roof	Serv 10 (G-0.7Wx-0.3Wy) Min	Y	0.666	0.383	1.739
Lower Roof	Serv 10 (G-0.7Wx-0.3Wy) Min	X	0.793	0.477	1.662
Story3	Serv 10 (G-0.7Wx-0.3Wy) Min	X	0.544	0.343	1.583
Story3	Serv 10 (G-0.7Wx-0.3Wy) Min	Y	0.39	0.168	2.313
Story2	Serv 10 (G-0.7Wx-0.3Wy) Min	X	0.292	0.193	1.512
Story2	Serv 10 (G-0.7Wx-0.3Wy) Min	Y	0.196	0.104	1.881
Upper Roof	Serv 11 (G-0.5Wx-0.5Wy) Max	X	0.694	0.344	2.017
Upper Roof	Serv 11 (G-0.5Wx-0.5Wy) Max	Y	0.803	0.517	1.553
Lower Roof	Serv 11 (G-0.5Wx-0.5Wy) Max	X	0.609	0.316	1.926
Lower Roof	Serv 11 (G-0.5Wx-0.5Wy) Max	Y	0.288	0.135	2.141
Story3	Serv 11 (G-0.5Wx-0.5Wy) Max	X	0.41	0.229	1.794
Story3	Serv 11 (G-0.5Wx-0.5Wy) Max	Y	0.487	0.268	1.82
Story2	Serv 11 (G-0.5Wx-0.5Wy) Max	X	0.213	0.126	1.689
Story2	Serv 11 (G-0.5Wx-0.5Wy) Max	Y	0.257	0.165	1.56
Upper Roof	Serv 11 (G-0.5Wx-0.5Wy) Min	X	0.867	0.487	1.78
Upper Roof	Serv 11 (G-0.5Wx-0.5Wy) Min	Y	0.697	0.407	1.712
Lower Roof	Serv 11 (G-0.5Wx-0.5Wy) Min	X	0.766	0.45	1.701
Story3	Serv 11 (G-0.5Wx-0.5Wy) Min	X	0.526	0.325	1.617
Story3	Serv 11 (G-0.5Wx-0.5Wy) Min	Y	0.412	0.184	2.231
Story2	Serv 11 (G-0.5Wx-0.5Wy) Min	X	0.281	0.182	1.541
Story2	Serv 11 (G-0.5Wx-0.5Wy) Min	Y	0.21	0.114	1.84
Upper Roof	Serv 12 (G-0.3Wx-0.7Wy) Max	X	0.737	0.374	1.97
Upper Roof	Serv 12 (G-0.3Wx-0.7Wy) Max	Y	0.793	0.498	1.593
Lower Roof	Serv 12 (G-0.3Wx-0.7Wy) Max	X	0.645	0.343	1.882
Lower Roof	Serv 12 (G-0.3Wx-0.7Wy) Max	Y	0.26	0.101	2.566
Story3	Serv 12 (G-0.3Wx-0.7Wy) Max	X	0.439	0.249	1.762
Story3	Serv 12 (G-0.3Wx-0.7Wy) Max	Y	0.48	0.251	1.914
Story2	Serv 12 (G-0.3Wx-0.7Wy) Max	X	0.23	0.138	1.663
Story2	Serv 12 (G-0.3Wx-0.7Wy) Max	Y	0.252	0.154	1.634
Upper Roof	Serv 12 (G-0.3Wx-0.7Wy) Min	X	0.839	0.458	1.83
Upper Roof	Serv 12 (G-0.3Wx-0.7Wy) Min	Y	0.731	0.434	1.686
Lower Roof	Serv 12 (G-0.3Wx-0.7Wy) Min	X	0.738	0.422	1.749
Story3	Serv 12 (G-0.3Wx-0.7Wy) Min	X	0.507	0.306	1.657
Story3	Serv 12 (G-0.3Wx-0.7Wy) Min	Y	0.435	0.202	2.157
Story2	Serv 12 (G-0.3Wx-0.7Wy) Min	X	0.27	0.171	1.576
Story2	Serv 12 (G-0.3Wx-0.7Wy) Min	Y	0.224	0.124	1.803
Upper Roof	Serv 13 (G+0.7Wx-0.3Wy) Max	X	0.705	0.321	2.196
Upper Roof	Serv 13 (G+0.7Wx-0.3Wy) Max	Y	0.893	0.559	1.596

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Serv 13 (G+0.7Wx-0.3Wy) Max	X	0.601	0.284	2.112
Lower Roof	Serv 13 (G+0.7Wx-0.3Wy) Max	Y	0.293	0.118	2.471
Story3	Serv 13 (G+0.7Wx-0.3Wy) Max	X	0.416	0.213	1.954
Story3	Serv 13 (G+0.7Wx-0.3Wy) Max	Y	0.548	0.284	1.928
Story2	Serv 13 (G+0.7Wx-0.3Wy) Max	X	0.216	0.118	1.833
Story2	Serv 13 (G+0.7Wx-0.3Wy) Max	Y	0.292	0.173	1.687
Upper Roof	Serv 13 (G+0.7Wx-0.3Wy) Min	X	0.944	0.519	1.818
Upper Roof	Serv 13 (G+0.7Wx-0.3Wy) Min	Y	0.746	0.408	1.831
Lower Roof	Serv 13 (G+0.7Wx-0.3Wy) Min	X	0.818	0.47	1.741
Lower Roof	Serv 13 (G+0.7Wx-0.3Wy) Min	Y	0.241	0.059	4.105
Story3	Serv 13 (G+0.7Wx-0.3Wy) Min	X	0.576	0.347	1.662
Story3	Serv 13 (G+0.7Wx-0.3Wy) Min	Y	0.444	0.169	2.621
Story2	Serv 13 (G+0.7Wx-0.3Wy) Min	X	0.31	0.196	1.586
Story2	Serv 13 (G+0.7Wx-0.3Wy) Min	Y	0.227	0.103	2.203
Upper Roof	Serv 14 (G+0.5Wx-0.5Wy) Max	X	0.731	0.348	2.102
Upper Roof	Serv 14 (G+0.5Wx-0.5Wy) Max	Y	0.861	0.535	1.611
Lower Roof	Serv 14 (G+0.5Wx-0.5Wy) Max	X	0.627	0.311	2.017
Lower Roof	Serv 14 (G+0.5Wx-0.5Wy) Max	Y	0.272	0.1	2.71
Story3	Serv 14 (G+0.5Wx-0.5Wy) Max	X	0.434	0.231	1.878
Story3	Serv 14 (G+0.5Wx-0.5Wy) Max	Y	0.526	0.268	1.962
Story2	Serv 14 (G+0.5Wx-0.5Wy) Max	X	0.227	0.128	1.767
Story2	Serv 14 (G+0.5Wx-0.5Wy) Max	Y	0.279	0.164	1.704
Upper Roof	Serv 14 (G+0.5Wx-0.5Wy) Min	X	0.904	0.491	1.841
Upper Roof	Serv 14 (G+0.5Wx-0.5Wy) Min	Y	0.755	0.425	1.777
Lower Roof	Serv 14 (G+0.5Wx-0.5Wy) Min	X	0.784	0.445	1.762
Story3	Serv 14 (G+0.5Wx-0.5Wy) Min	X	0.55	0.328	1.677
Story3	Serv 14 (G+0.5Wx-0.5Wy) Min	Y	0.451	0.185	2.435
Story2	Serv 14 (G+0.5Wx-0.5Wy) Min	X	0.295	0.184	1.597
Story2	Serv 14 (G+0.5Wx-0.5Wy) Min	Y	0.232	0.113	2.052
Upper Roof	Serv 15 (G+0.3Wx-0.7Wy) Max	X	0.759	0.377	2.016
Upper Roof	Serv 15 (G+0.3Wx-0.7Wy) Max	Y	0.828	0.509	1.627
Lower Roof	Serv 15 (G+0.3Wx-0.7Wy) Max	X	0.656	0.34	1.931
Lower Roof	Serv 15 (G+0.3Wx-0.7Wy) Max	Y	0.25	0.081	3.087
Story3	Serv 15 (G+0.3Wx-0.7Wy) Max	X	0.453	0.25	1.808
Story3	Serv 15 (G+0.3Wx-0.7Wy) Max	Y	0.503	0.251	2.002
Story2	Serv 15 (G+0.3Wx-0.7Wy) Max	X	0.238	0.139	1.706
Story2	Serv 15 (G+0.3Wx-0.7Wy) Max	Y	0.265	0.153	1.725
Upper Roof	Serv 15 (G+0.3Wx-0.7Wy) Min	X	0.861	0.461	1.868
Upper Roof	Serv 15 (G+0.3Wx-0.7Wy) Min	Y	0.765	0.444	1.723
Lower Roof	Serv 15 (G+0.3Wx-0.7Wy) Min	X	0.748	0.419	1.787
Story3	Serv 15 (G+0.3Wx-0.7Wy) Min	X	0.521	0.307	1.695
Story3	Serv 15 (G+0.3Wx-0.7Wy) Min	Y	0.458	0.202	2.267
Story2	Serv 15 (G+0.3Wx-0.7Wy) Min	X	0.278	0.172	1.611
Story2	Serv 15 (G+0.3Wx-0.7Wy) Min	Y	0.237	0.124	1.917
Upper Roof	Serv 16 (G-0.7Wx+0.3Wy) Max	X	0.654	0.316	2.07
Upper Roof	Serv 16 (G-0.7Wx+0.3Wy) Max	Y	0.812	0.535	1.519
Lower Roof	Serv 16 (G-0.7Wx+0.3Wy) Max	X	0.576	0.292	1.974
Lower Roof	Serv 16 (G-0.7Wx+0.3Wy) Max	Y	0.315	0.166	1.899
Story3	Serv 16 (G-0.7Wx+0.3Wy) Max	X	0.384	0.21	1.829

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	Serv 16 (G-0.7Wx+0.3Wy) Max	Y	0.494	0.283	1.743
Story2	Serv 16 (G-0.7Wx+0.3Wy) Max	X	0.198	0.115	1.717
Story2	Serv 16 (G-0.7Wx+0.3Wy) Max	Y	0.262	0.175	1.499
Upper Roof	Serv 16 (G-0.7Wx+0.3Wy) Min	X	0.893	0.514	1.738
Upper Roof	Serv 16 (G-0.7Wx+0.3Wy) Min	Y	0.666	0.383	1.739
Lower Roof	Serv 16 (G-0.7Wx+0.3Wy) Min	X	0.793	0.477	1.662
Story3	Serv 16 (G-0.7Wx+0.3Wy) Min	X	0.544	0.343	1.583
Story3	Serv 16 (G-0.7Wx+0.3Wy) Min	Y	0.39	0.168	2.313
Story2	Serv 16 (G-0.7Wx+0.3Wy) Min	X	0.292	0.193	1.512
Story2	Serv 16 (G-0.7Wx+0.3Wy) Min	Y	0.196	0.104	1.881
Upper Roof	Serv 17 (G-0.5Wx+0.5Wy) Max	X	0.694	0.344	2.017
Upper Roof	Serv 17 (G-0.5Wx+0.5Wy) Max	Y	0.803	0.517	1.553
Lower Roof	Serv 17 (G-0.5Wx+0.5Wy) Max	X	0.609	0.316	1.926
Lower Roof	Serv 17 (G-0.5Wx+0.5Wy) Max	Y	0.288	0.135	2.141
Story3	Serv 17 (G-0.5Wx+0.5Wy) Max	X	0.41	0.229	1.794
Story3	Serv 17 (G-0.5Wx+0.5Wy) Max	Y	0.487	0.268	1.82
Story2	Serv 17 (G-0.5Wx+0.5Wy) Max	X	0.213	0.126	1.689
Story2	Serv 17 (G-0.5Wx+0.5Wy) Max	Y	0.257	0.165	1.56
Upper Roof	Serv 17 (G-0.5Wx+0.5Wy) Min	X	0.867	0.487	1.78
Upper Roof	Serv 17 (G-0.5Wx+0.5Wy) Min	Y	0.697	0.407	1.712
Lower Roof	Serv 17 (G-0.5Wx+0.5Wy) Min	X	0.766	0.45	1.701
Story3	Serv 17 (G-0.5Wx+0.5Wy) Min	X	0.526	0.325	1.617
Story3	Serv 17 (G-0.5Wx+0.5Wy) Min	Y	0.412	0.184	2.231
Story2	Serv 17 (G-0.5Wx+0.5Wy) Min	X	0.281	0.182	1.541
Story2	Serv 17 (G-0.5Wx+0.5Wy) Min	Y	0.21	0.114	1.84
Upper Roof	Serv 18 (G-0.3Wx+0.7Wy) Max	X	0.737	0.374	1.97
Upper Roof	Serv 18 (G-0.3Wx+0.7Wy) Max	Y	0.793	0.498	1.593
Lower Roof	Serv 18 (G-0.3Wx+0.7Wy) Max	X	0.645	0.343	1.882
Lower Roof	Serv 18 (G-0.3Wx+0.7Wy) Max	Y	0.26	0.101	2.566
Story3	Serv 18 (G-0.3Wx+0.7Wy) Max	X	0.439	0.249	1.762
Story3	Serv 18 (G-0.3Wx+0.7Wy) Max	Y	0.48	0.251	1.914
Story2	Serv 18 (G-0.3Wx+0.7Wy) Max	X	0.23	0.138	1.663
Story2	Serv 18 (G-0.3Wx+0.7Wy) Max	Y	0.252	0.154	1.634
Upper Roof	Serv 18 (G-0.3Wx+0.7Wy) Min	X	0.839	0.458	1.83
Upper Roof	Serv 18 (G-0.3Wx+0.7Wy) Min	Y	0.731	0.434	1.686
Lower Roof	Serv 18 (G-0.3Wx+0.7Wy) Min	X	0.738	0.422	1.749
Story3	Serv 18 (G-0.3Wx+0.7Wy) Min	X	0.507	0.306	1.657
Story3	Serv 18 (G-0.3Wx+0.7Wy) Min	Y	0.435	0.202	2.157
Story2	Serv 18 (G-0.3Wx+0.7Wy) Min	X	0.27	0.171	1.576
Story2	Serv 18 (G-0.3Wx+0.7Wy) Min	Y	0.224	0.124	1.803
Upper Roof	Ult 01 (1.35G)	X	1.079	0.564	1.913
Upper Roof	Ult 01 (1.35G)	Y	1.052	0.636	1.654
Lower Roof	Ult 01 (1.35G)	X	0.941	0.514	1.83
Lower Roof	Ult 01 (1.35G)	Y	0.296	0.072	4.096
Story3	Ult 01 (1.35G)	X	0.648	0.376	1.725
Story3	Ult 01 (1.35G)	Y	0.633	0.306	2.071
Story2	Ult 01 (1.35G)	X	0.343	0.21	1.634
Story2	Ult 01 (1.35G)	Y	0.33	0.187	1.76
Upper Roof	Ult 02 (1.2G+1.5Q)	X	1.181	0.622	1.898

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Upper Roof	Ult 02 (1.2G+1.5Q)	Y	1.108	0.657	1.687
Lower Roof	Ult 02 (1.2G+1.5Q)	X	1.055	0.579	1.823
Lower Roof	Ult 02 (1.2G+1.5Q)	Y	0.308	0.058	5.283
Story3	Ult 02 (1.2G+1.5Q)	X	0.71	0.414	1.714
Story3	Ult 02 (1.2G+1.5Q)	Y	0.673	0.317	2.12
Story2	Ult 02 (1.2G+1.5Q)	X	0.372	0.229	1.623
Story2	Ult 02 (1.2G+1.5Q)	Y	0.353	0.199	1.776
Upper Roof	Ult 03 (1.2G+Wx+0.4Q) Max	X	0.819	0.328	2.494
Upper Roof	Ult 03 (1.2G+Wx+0.4Q) Max	Y	1.223	0.777	1.573
Lower Roof	Ult 03 (1.2G+Wx+0.4Q) Max	X	0.69	0.285	2.427
Lower Roof	Ult 03 (1.2G+Wx+0.4Q) Max	Y	0.431	0.201	2.149
Story3	Ult 03 (1.2G+Wx+0.4Q) Max	X	0.476	0.217	2.199
Story3	Ult 03 (1.2G+Wx+0.4Q) Max	Y	0.761	0.407	1.869
Story2	Ult 03 (1.2G+Wx+0.4Q) Max	X	0.243	0.118	2.051
Story2	Ult 03 (1.2G+Wx+0.4Q) Max	Y	0.411	0.248	1.655
Upper Roof	Ult 03 (1.2G+Wx+0.4Q) Min	X	1.327	0.75	1.77
Upper Roof	Ult 03 (1.2G+Wx+0.4Q) Min	Y	0.911	0.454	2.004
Lower Roof	Ult 03 (1.2G+Wx+0.4Q) Min	X	1.152	0.679	1.697
Lower Roof	Ult 03 (1.2G+Wx+0.4Q) Min	Y	0.424	0.176	2.407
Story3	Ult 03 (1.2G+Wx+0.4Q) Min	X	0.817	0.501	1.63
Story3	Ult 03 (1.2G+Wx+0.4Q) Min	Y	0.538	0.162	3.314
Story2	Ult 03 (1.2G+Wx+0.4Q) Min	X	0.442	0.283	1.561
Story2	Ult 03 (1.2G+Wx+0.4Q) Min	Y	0.272	0.099	2.75
Upper Roof	Ult 04 (1.2G-Wx+0.4Q) Max	X	0.709	0.317	2.239
Upper Roof	Ult 04 (1.2G-Wx+0.4Q) Max	Y	1.052	0.725	1.451
Lower Roof	Ult 04 (1.2G-Wx+0.4Q) Max	X	0.637	0.3	2.125
Lower Roof	Ult 04 (1.2G-Wx+0.4Q) Max	Y	0.478	0.301	1.588
Story3	Ult 04 (1.2G-Wx+0.4Q) Max	X	0.407	0.21	1.94
Story3	Ult 04 (1.2G-Wx+0.4Q) Max	Y	0.646	0.405	1.593
Story2	Ult 04 (1.2G-Wx+0.4Q) Max	X	0.203	0.112	1.808
Story2	Ult 04 (1.2G-Wx+0.4Q) Max	Y	0.347	0.252	1.378
Upper Roof	Ult 04 (1.2G-Wx+0.4Q) Min	X	1.217	0.738	1.651
Upper Roof	Ult 04 (1.2G-Wx+0.4Q) Min	Y	0.74	0.402	1.84
Lower Roof	Ult 04 (1.2G-Wx+0.4Q) Min	X	1.099	0.694	1.582
Lower Roof	Ult 04 (1.2G-Wx+0.4Q) Min	Y	0.27	0.076	3.568
Story3	Ult 04 (1.2G-Wx+0.4Q) Min	X	0.747	0.494	1.512
Story3	Ult 04 (1.2G-Wx+0.4Q) Min	Y	0.423	0.161	2.636
Story2	Ult 04 (1.2G-Wx+0.4Q) Min	X	0.403	0.277	1.451
Story2	Ult 04 (1.2G-Wx+0.4Q) Min	Y	0.207	0.102	2.032
Upper Roof	Ult 05 (1.2G+Wy+0.4Q)	X	1.018	0.533	1.909
Upper Roof	Ult 05 (1.2G+Wy+0.4Q)	Y	0.981	0.59	1.664
Lower Roof	Ult 05 (1.2G+Wy+0.4Q)	X	0.894	0.489	1.828
Lower Roof	Ult 05 (1.2G+Wy+0.4Q)	Y	0.275	0.063	4.391
Story3	Ult 05 (1.2G+Wy+0.4Q)	X	0.612	0.355	1.722
Story3	Ult 05 (1.2G+Wy+0.4Q)	Y	0.592	0.284	2.086
Story2	Ult 05 (1.2G+Wy+0.4Q)	X	0.323	0.198	1.631
Story2	Ult 05 (1.2G+Wy+0.4Q)	Y	0.309	0.175	1.765
Upper Roof	Ult 06 (1.2G-Wy+0.4Q)	X	1.018	0.533	1.909
Upper Roof	Ult 06 (1.2G-Wy+0.4Q)	Y	0.981	0.59	1.664

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 06 (1.2G-Wy+0.4Q)	X	0.894	0.489	1.828
Lower Roof	Ult 06 (1.2G-Wy+0.4Q)	Y	0.275	0.063	4.391
Story3	Ult 06 (1.2G-Wy+0.4Q)	X	0.612	0.355	1.722
Story3	Ult 06 (1.2G-Wy+0.4Q)	Y	0.592	0.284	2.086
Story2	Ult 06 (1.2G-Wy+0.4Q)	X	0.323	0.198	1.631
Story2	Ult 06 (1.2G-Wy+0.4Q)	Y	0.309	0.175	1.765
Upper Roof	Ult 07 (0.9G+Wx) Max	X	0.52	0.171	3.045
Upper Roof	Ult 07 (0.9G+Wx) Max	Y	0.943	0.612	1.542
Lower Roof	Ult 07 (0.9G+Wx) Max	X	0.423	0.138	3.069
Lower Roof	Ult 07 (0.9G+Wx) Max	Y	0.354	0.186	1.898
Story3	Ult 07 (0.9G+Wx) Max	X	0.297	0.112	2.655
Story3	Ult 07 (0.9G+Wx) Max	Y	0.591	0.327	1.807
Story2	Ult 07 (0.9G+Wx) Max	X	0.149	0.06	2.463
Story2	Ult 07 (0.9G+Wx) Max	Y	0.322	0.198	1.625
Upper Roof	Ult 07 (0.9G+Wx) Min	X	1.028	0.592	1.736
Upper Roof	Ult 07 (0.9G+Wx) Min	Y	0.631	0.289	2.185
Lower Roof	Ult 07 (0.9G+Wx) Min	X	0.885	0.532	1.662
Lower Roof	Ult 07 (0.9G+Wx) Min	Y	0.375	0.191	1.968
Story3	Ult 07 (0.9G+Wx) Min	X	0.637	0.396	1.608
Story3	Ult 07 (0.9G+Wx) Min	Y	0.368	0.082	4.473
Story2	Ult 07 (0.9G+Wx) Min	X	0.348	0.225	1.545
Story2	Ult 07 (0.9G+Wx) Min	Y	0.182	0.049	3.758
Upper Roof	Ult 08 (0.9G-Wx) Max	X	0.41	0.159	2.577
Upper Roof	Ult 08 (0.9G-Wx) Max	Y	0.772	0.559	1.38
Lower Roof	Ult 08 (0.9G-Wx) Max	X	0.369	0.153	2.414
Lower Roof	Ult 08 (0.9G-Wx) Max	Y	0.4	0.287	1.396
Story3	Ult 08 (0.9G-Wx) Max	X	0.227	0.105	2.167
Story3	Ult 08 (0.9G-Wx) Max	Y	0.476	0.325	1.463
Story2	Ult 08 (0.9G-Wx) Max	X	0.109	0.054	2.006
Story2	Ult 08 (0.9G-Wx) Max	Y	0.257	0.201	1.278
Upper Roof	Ult 08 (0.9G-Wx) Min	X	0.918	0.58	1.584
Upper Roof	Ult 08 (0.9G-Wx) Min	Y	0.46	0.236	1.945
Lower Roof	Ult 08 (0.9G-Wx) Min	X	0.831	0.548	1.518
Lower Roof	Ult 08 (0.9G-Wx) Min	Y	0.221	0.09	2.452
Story3	Ult 08 (0.9G-Wx) Min	X	0.567	0.389	1.458
Story3	Ult 08 (0.9G-Wx) Min	Y	0.253	0.08	3.148
Story2	Ult 08 (0.9G-Wx) Min	X	0.309	0.219	1.406
Story2	Ult 08 (0.9G-Wx) Min	Y	0.118	0.052	2.28
Upper Roof	Ult 09 (0.9G+Wy)	X	0.719	0.376	1.913
Upper Roof	Ult 09 (0.9G+Wy)	Y	0.701	0.424	1.654
Lower Roof	Ult 09 (0.9G+Wy)	X	0.627	0.343	1.83
Lower Roof	Ult 09 (0.9G+Wy)	Y	0.197	0.048	4.096
Story3	Ult 09 (0.9G+Wy)	X	0.432	0.25	1.725
Story3	Ult 09 (0.9G+Wy)	Y	0.422	0.204	2.071
Story2	Ult 09 (0.9G+Wy)	X	0.229	0.14	1.634
Story2	Ult 09 (0.9G+Wy)	Y	0.22	0.125	1.76
Upper Roof	Ult 10 (0.9G-Wy)	X	0.719	0.376	1.913
Upper Roof	Ult 10 (0.9G-Wy)	Y	0.701	0.424	1.654
Lower Roof	Ult 10 (0.9G-Wy)	X	0.627	0.343	1.83

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 10 (0.9G-Wy)	Y	0.197	0.048	4.096
Story3	Ult 10 (0.9G-Wy)	X	0.432	0.25	1.725
Story3	Ult 10 (0.9G-Wy)	Y	0.422	0.204	2.071
Story2	Ult 10 (0.9G-Wy)	X	0.229	0.14	1.634
Story2	Ult 10 (0.9G-Wy)	Y	0.22	0.125	1.76
Upper Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	X	0.58	0.232	2.495
Upper Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	Y	0.871	0.555	1.568
Lower Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	X	0.484	0.199	2.43
Lower Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	Y	0.307	0.145	2.117
Story3	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	X	0.337	0.153	2.2
Story3	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	Y	0.54	0.29	1.862
Story2	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	X	0.172	0.084	2.05
Story2	Ult 11 (0.9G+0.7Wx+0.3Wy) Max	Y	0.291	0.176	1.653
Upper Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	X	0.935	0.527	1.774
Upper Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	Y	0.652	0.329	1.98
Lower Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	X	0.808	0.476	1.698
Lower Roof	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	Y	0.293	0.119	2.461
Story3	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	X	0.575	0.352	1.633
Story3	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	Y	0.385	0.119	3.237
Story2	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	X	0.312	0.2	1.563
Story2	Ult 11 (0.9G+0.7Wx+0.3Wy) Min	Y	0.194	0.071	2.71
Upper Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	X	0.619	0.273	2.267
Upper Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	Y	0.822	0.518	1.588
Lower Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	X	0.525	0.24	2.185
Lower Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	Y	0.275	0.117	2.349
Story3	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	X	0.364	0.181	2.012
Story3	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	Y	0.506	0.265	1.908
Story2	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	X	0.189	0.1	1.884
Story2	Ult 12 (0.9G+0.5Wx+0.5Wy) Max	Y	0.271	0.162	1.677
Upper Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	X	0.874	0.484	1.805
Upper Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	Y	0.666	0.356	1.869
Lower Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	X	0.756	0.438	1.728
Lower Roof	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	Y	0.238	0.071	3.342
Story3	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	X	0.534	0.323	1.653
Story3	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	Y	0.395	0.143	2.763
Story2	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	X	0.288	0.183	1.579
Story2	Ult 12 (0.9G+0.5Wx+0.5Wy) Min	Y	0.201	0.087	2.319
Upper Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	X	0.659	0.314	2.097
Upper Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	Y	0.774	0.48	1.611
Lower Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	X	0.566	0.281	2.012
Lower Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	Y	0.244	0.09	2.725
Story3	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	X	0.391	0.209	1.874
Story3	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	Y	0.473	0.241	1.964
Story2	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	X	0.205	0.116	1.764
Story2	Ult 13 (0.9G+0.3Wx+0.7Wy) Max	Y	0.251	0.147	1.705
Upper Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	X	0.812	0.441	1.842
Upper Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	Y	0.68	0.383	1.774
Lower Roof	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	X	0.704	0.4	1.763
Story3	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	X	0.493	0.294	1.678

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	Y	0.406	0.167	2.426
Story2	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	X	0.264	0.165	1.598
Story2	Ult 13 (0.9G+0.3Wx+0.7Wy) Min	Y	0.209	0.102	2.045
Upper Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	X	0.503	0.224	2.243
Upper Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	Y	0.751	0.519	1.447
Lower Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	X	0.447	0.21	2.128
Lower Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	Y	0.339	0.215	1.578
Story3	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	X	0.288	0.148	1.943
Story3	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	Y	0.46	0.289	1.592
Story2	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	X	0.145	0.08	1.811
Story2	Ult 14 (0.9G-0.7Wx-0.3Wy) Max	Y	0.246	0.178	1.38
Upper Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	X	0.859	0.519	1.655
Upper Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	Y	0.532	0.293	1.819
Lower Roof	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	X	0.77	0.486	1.584
Story3	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	X	0.527	0.347	1.516
Story3	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	Y	0.304	0.117	2.588
Story2	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	X	0.285	0.196	1.455
Story2	Ult 14 (0.9G-0.7Wx-0.3Wy) Min	Y	0.149	0.074	2.015
Upper Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	X	0.565	0.268	2.11
Upper Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	Y	0.736	0.492	1.498
Lower Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	X	0.498	0.248	2.01
Lower Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	Y	0.299	0.167	1.785
Story3	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	X	0.329	0.178	1.855
Story3	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	Y	0.449	0.264	1.697
Story2	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	X	0.169	0.097	1.738
Story2	Ult 15 (0.9G-0.5Wx-0.5Wy) Max	Y	0.239	0.163	1.463
Upper Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	X	0.819	0.478	1.713
Upper Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	Y	0.58	0.33	1.758
Lower Roof	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	X	0.729	0.445	1.638
Story3	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	X	0.5	0.32	1.563
Story3	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	Y	0.338	0.142	2.376
Story2	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	X	0.269	0.18	1.495
Story2	Ult 15 (0.9G-0.5Wx-0.5Wy) Min	Y	0.169	0.088	1.912
Upper Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	X	0.626	0.311	2.015
Upper Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	Y	0.722	0.465	1.555
Lower Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	X	0.55	0.286	1.924
Lower Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	Y	0.258	0.12	2.156
Story3	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	X	0.37	0.207	1.792
Story3	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	Y	0.438	0.24	1.824
Story2	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	X	0.193	0.114	1.687
Story2	Ult 16 (0.9G-0.3Wx-0.7Wy) Max	Y	0.231	0.148	1.563
Upper Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	X	0.779	0.437	1.782
Upper Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	Y	0.629	0.368	1.71
Lower Roof	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	X	0.688	0.404	1.703
Story3	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	X	0.473	0.292	1.618
Story3	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	Y	0.371	0.167	2.227
Story2	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	X	0.253	0.164	1.543
Story2	Ult 16 (0.9G-0.3Wx-0.7Wy) Min	Y	0.189	0.103	1.838
Upper Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	X	0.58	0.232	2.495

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Upper Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	Y	0.871	0.555	1.568
Lower Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	X	0.484	0.199	2.43
Lower Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	Y	0.307	0.145	2.117
Story3	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	X	0.337	0.153	2.2
Story3	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	Y	0.54	0.29	1.862
Story2	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	X	0.172	0.084	2.05
Story2	Ult 17 (0.9G+0.7Wx-0.3Wy) Max	Y	0.291	0.176	1.653
Upper Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	X	0.935	0.527	1.774
Upper Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	Y	0.652	0.329	1.98
Lower Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	X	0.808	0.476	1.698
Lower Roof	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	Y	0.293	0.119	2.461
Story3	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	X	0.575	0.352	1.633
Story3	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	Y	0.385	0.119	3.237
Story2	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	X	0.312	0.2	1.563
Story2	Ult 17 (0.9G+0.7Wx-0.3Wy) Min	Y	0.194	0.071	2.71
Upper Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	X	0.619	0.273	2.267
Upper Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	Y	0.822	0.518	1.588
Lower Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	X	0.525	0.24	2.185
Lower Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	Y	0.275	0.117	2.349
Story3	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	X	0.364	0.181	2.012
Story3	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	Y	0.506	0.265	1.908
Story2	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	X	0.189	0.1	1.884
Story2	Ult 18 (0.9G+0.5Wx-0.5Wy) Max	Y	0.271	0.162	1.677
Upper Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	X	0.874	0.484	1.805
Upper Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	Y	0.666	0.356	1.869
Lower Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	X	0.756	0.438	1.728
Lower Roof	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	Y	0.238	0.071	3.342
Story3	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	X	0.534	0.323	1.653
Story3	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	Y	0.395	0.143	2.763
Story2	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	X	0.288	0.183	1.579
Story2	Ult 18 (0.9G+0.5Wx-0.5Wy) Min	Y	0.201	0.087	2.319
Upper Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	X	0.659	0.314	2.097
Upper Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	Y	0.774	0.48	1.611
Lower Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	X	0.566	0.281	2.012
Lower Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	Y	0.244	0.09	2.725
Story3	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	X	0.391	0.209	1.874
Story3	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	Y	0.473	0.241	1.964
Story2	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	X	0.205	0.116	1.764
Story2	Ult 19 (0.9G+0.3Wx-0.7Wy) Max	Y	0.251	0.147	1.705
Upper Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	X	0.812	0.441	1.842
Upper Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	Y	0.68	0.383	1.774
Lower Roof	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	X	0.704	0.4	1.763
Story3	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	X	0.493	0.294	1.678
Story3	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	Y	0.406	0.167	2.426
Story2	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	X	0.264	0.165	1.598
Story2	Ult 19 (0.9G+0.3Wx-0.7Wy) Min	Y	0.209	0.102	2.045
Upper Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	X	0.503	0.224	2.243
Upper Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	Y	0.751	0.519	1.447
Lower Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	X	0.447	0.21	2.128

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	Y	0.339	0.215	1.578
Story3	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	X	0.288	0.148	1.943
Story3	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	Y	0.46	0.289	1.592
Story2	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	X	0.145	0.08	1.811
Story2	Ult 20 (0.9G-0.7Wx+0.3Wy) Max	Y	0.246	0.178	1.38
Upper Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	X	0.859	0.519	1.655
Upper Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	Y	0.532	0.293	1.819
Lower Roof	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	X	0.77	0.486	1.584
Story3	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	X	0.527	0.347	1.516
Story3	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	Y	0.304	0.117	2.588
Story2	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	X	0.285	0.196	1.455
Story2	Ult 20 (0.9G-0.7Wx+0.3Wy) Min	Y	0.149	0.074	2.015
Upper Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	X	0.565	0.268	2.11
Upper Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	Y	0.736	0.492	1.498
Lower Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	X	0.498	0.248	2.01
Lower Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	Y	0.299	0.167	1.785
Story3	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	X	0.329	0.178	1.855
Story3	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	Y	0.449	0.264	1.697
Story2	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	X	0.169	0.097	1.738
Story2	Ult 21 (0.9G-0.5Wx+0.5Wy) Max	Y	0.239	0.163	1.463
Upper Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	X	0.819	0.478	1.713
Upper Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	Y	0.58	0.33	1.758
Lower Roof	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	X	0.729	0.445	1.638
Story3	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	X	0.5	0.32	1.563
Story3	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	Y	0.338	0.142	2.376
Story2	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	X	0.269	0.18	1.495
Story2	Ult 21 (0.9G-0.5Wx+0.5Wy) Min	Y	0.169	0.088	1.912
Upper Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	X	0.626	0.311	2.015
Upper Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	Y	0.722	0.465	1.555
Lower Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	X	0.55	0.286	1.924
Lower Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	Y	0.258	0.12	2.156
Story3	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	X	0.37	0.207	1.792
Story3	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	Y	0.438	0.24	1.824
Story2	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	X	0.193	0.114	1.687
Story2	Ult 22 (0.9G-0.3Wx+0.7Wy) Max	Y	0.231	0.148	1.563
Upper Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	X	0.779	0.437	1.782
Upper Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	Y	0.629	0.368	1.71
Lower Roof	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	X	0.688	0.404	1.703
Story3	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	X	0.473	0.292	1.618
Story3	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	Y	0.371	0.167	2.227
Story2	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	X	0.253	0.164	1.543
Story2	Ult 22 (0.9G-0.3Wx+0.7Wy) Min	Y	0.189	0.103	1.838
Upper Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	X	0.879	0.39	2.254
Upper Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	Y	1.15	0.721	1.595
Lower Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	X	0.752	0.346	2.173
Lower Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	Y	0.385	0.159	2.413
Story3	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	X	0.517	0.258	2.002
Story3	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	Y	0.71	0.37	1.919
Story2	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	X	0.267	0.142	1.875

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Max	Y	0.381	0.226	1.681
Upper Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	X	1.234	0.685	1.802
Upper Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	Y	0.932	0.495	1.883
Lower Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	X	1.075	0.622	1.728
Lower Roof	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	Y	0.342	0.104	3.27
Story3	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	X	0.755	0.457	1.651
Story3	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	Y	0.555	0.199	2.788
Story2	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	X	0.406	0.258	1.577
Story2	Ult 23 (1.2G+0.7Wx+0.3Wy+0.4Q) Min	Y	0.283	0.122	2.325
Upper Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	X	0.918	0.431	2.131
Upper Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	Y	1.102	0.684	1.612
Lower Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	X	0.792	0.387	2.048
Lower Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	Y	0.353	0.132	2.682
Story3	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	X	0.544	0.286	1.902
Story3	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	Y	0.676	0.345	1.958
Story2	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	X	0.283	0.158	1.788
Story2	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Max	Y	0.36	0.212	1.7
Upper Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	X	1.173	0.642	1.828
Upper Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	Y	0.946	0.522	1.812
Lower Roof	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	X	1.023	0.584	1.752
Story3	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	X	0.714	0.428	1.668
Story3	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	Y	0.565	0.223	2.533
Story2	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	X	0.382	0.241	1.589
Story2	Ult 24 (1.2G+0.5Wx+0.5Wy+0.4Q) Min	Y	0.29	0.137	2.12
Upper Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	X	0.958	0.472	2.031
Upper Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	Y	1.054	0.646	1.631
Lower Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	X	0.833	0.428	1.947
Lower Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	Y	0.322	0.104	3.093
Story3	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	X	0.571	0.314	1.821
Story3	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	Y	0.643	0.321	2.003
Story2	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	X	0.299	0.174	1.716
Story2	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Max	Y	0.34	0.197	1.723
Upper Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	X	1.111	0.598	1.857
Upper Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	Y	0.96	0.549	1.749
Lower Roof	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	X	0.972	0.546	1.779
Story3	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	X	0.673	0.399	1.687
Story3	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	Y	0.576	0.247	2.328
Story2	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	X	0.359	0.224	1.604
Story2	Ult 25 (1.2G+0.3Wx+0.7Wy+0.4Q) Min	Y	0.298	0.152	1.957
Upper Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	X	0.802	0.382	2.1
Upper Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	Y	1.03	0.684	1.506
Lower Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	X	0.714	0.357	2.003
Lower Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	Y	0.417	0.23	1.817
Story3	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	X	0.468	0.253	1.848
Story3	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	Y	0.63	0.369	1.707
Story2	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	X	0.239	0.138	1.731
Story2	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Max	Y	0.335	0.229	1.467
Upper Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	X	1.158	0.676	1.712
Upper Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	Y	0.812	0.458	1.772

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	X	1.037	0.633	1.639
Story3	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	X	0.706	0.452	1.562
Story3	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	Y	0.474	0.198	2.399
Story2	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	X	0.379	0.254	1.493
Story2	Ult 26 (1.2G-0.7Wx-0.3Wy+0.4Q) Min	Y	0.238	0.124	1.919
Upper Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	X	0.864	0.425	2.031
Upper Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	Y	1.016	0.657	1.546
Lower Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	X	0.766	0.395	1.941
Lower Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	Y	0.377	0.182	2.07
Story3	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	X	0.509	0.282	1.803
Story3	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	Y	0.619	0.345	1.796
Story2	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	X	0.263	0.155	1.695
Story2	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Max	Y	0.328	0.213	1.537
Upper Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	X	1.118	0.635	1.759
Upper Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	Y	0.86	0.496	1.735
Lower Roof	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	X	0.997	0.592	1.684
Story3	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	X	0.679	0.425	1.6
Story3	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	Y	0.508	0.222	2.285
Story2	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	X	0.363	0.238	1.526
Story2	Ult 27 (1.2G-0.5Wx-0.5Wy+0.4Q) Min	Y	0.258	0.139	1.863
Upper Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	X	0.925	0.469	1.975
Upper Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	Y	1.002	0.63	1.59
Lower Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	X	0.817	0.432	1.89
Lower Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	Y	0.336	0.134	2.503
Story3	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	X	0.55	0.312	1.766
Story3	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	Y	0.608	0.32	1.899
Story2	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	X	0.287	0.172	1.665
Story2	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Max	Y	0.32	0.198	1.617
Upper Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	X	1.078	0.595	1.813
Upper Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	Y	0.909	0.533	1.704
Lower Roof	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	X	0.956	0.551	1.735
Story3	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	X	0.652	0.397	1.643
Story3	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	Y	0.541	0.247	2.193
Story2	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	X	0.347	0.222	1.563
Story2	Ult 28 (1.2G-0.3Wx-0.7Wy+0.4Q) Min	Y	0.279	0.153	1.818
Upper Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	X	0.879	0.39	2.254
Upper Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	Y	1.15	0.721	1.595
Lower Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	X	0.752	0.346	2.173
Lower Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	Y	0.385	0.159	2.413
Story3	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	X	0.517	0.258	2.002
Story3	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	Y	0.71	0.37	1.919
Story2	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	X	0.267	0.142	1.875
Story2	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Max	Y	0.381	0.226	1.681
Upper Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	X	1.234	0.685	1.802
Upper Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	Y	0.932	0.495	1.883
Lower Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	X	1.075	0.622	1.728
Lower Roof	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	Y	0.342	0.104	3.27
Story3	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	X	0.755	0.457	1.651
Story3	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	Y	0.555	0.199	2.788

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	X	0.406	0.258	1.577
Story2	Ult 29 (1.2G+0.7Wx-0.3Wy+0.4Q) Min	Y	0.283	0.122	2.325
Upper Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	X	0.918	0.431	2.131
Upper Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	Y	1.102	0.684	1.612
Lower Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	X	0.792	0.387	2.048
Lower Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	Y	0.353	0.132	2.682
Story3	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	X	0.544	0.286	1.902
Story3	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	Y	0.676	0.345	1.958
Story2	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	X	0.283	0.158	1.788
Story2	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Max	Y	0.36	0.212	1.7
Upper Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	X	1.173	0.642	1.828
Upper Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	Y	0.946	0.522	1.812
Lower Roof	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	X	1.023	0.584	1.752
Story3	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	X	0.714	0.428	1.668
Story3	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	Y	0.565	0.223	2.533
Story2	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	X	0.382	0.241	1.589
Story2	Ult 30 (1.2G+0.5Wx-0.5Wy+0.4Q) Min	Y	0.29	0.137	2.12
Upper Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	X	0.958	0.472	2.031
Upper Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	Y	1.054	0.646	1.631
Lower Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	X	0.833	0.428	1.947
Lower Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	Y	0.322	0.104	3.093
Story3	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	X	0.571	0.314	1.821
Story3	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	Y	0.643	0.321	2.003
Story2	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	X	0.299	0.174	1.716
Story2	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Max	Y	0.34	0.197	1.723
Upper Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	X	1.111	0.598	1.857
Upper Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	Y	0.96	0.549	1.749
Lower Roof	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	X	0.972	0.546	1.779
Story3	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	X	0.673	0.399	1.687
Story3	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	Y	0.576	0.247	2.328
Story2	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	X	0.359	0.224	1.604
Story2	Ult 31 (1.2G+0.3Wx-0.7Wy+0.4Q) Min	Y	0.298	0.152	1.957
Upper Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	X	0.802	0.382	2.1
Upper Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	Y	1.03	0.684	1.506
Lower Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	X	0.714	0.357	2.003
Lower Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	Y	0.417	0.23	1.817
Story3	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	X	0.468	0.253	1.848
Story3	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	Y	0.63	0.369	1.707
Story2	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	X	0.239	0.138	1.731
Story2	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Max	Y	0.335	0.229	1.467
Upper Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	X	1.158	0.676	1.712
Upper Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	Y	0.812	0.458	1.772
Lower Roof	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	X	1.037	0.633	1.639
Story3	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	X	0.706	0.452	1.562
Story3	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	Y	0.474	0.198	2.399
Story2	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	X	0.379	0.254	1.493
Story2	Ult 32 (1.2G-0.7Wx+0.3Wy+0.4Q) Min	Y	0.238	0.124	1.919
Upper Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	X	0.864	0.425	2.031
Upper Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	Y	1.016	0.657	1.546

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	X	0.766	0.395	1.941
Lower Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	Y	0.377	0.182	2.07
Story3	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	X	0.509	0.282	1.803
Story3	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	Y	0.619	0.345	1.796
Story2	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	X	0.263	0.155	1.695
Story2	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Max	Y	0.328	0.213	1.537
Upper Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	X	1.118	0.635	1.759
Upper Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	Y	0.86	0.496	1.735
Lower Roof	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	X	0.997	0.592	1.684
Story3	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	X	0.679	0.425	1.6
Story3	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	Y	0.508	0.222	2.285
Story2	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	X	0.363	0.238	1.526
Story2	Ult 33 (1.2G-0.5Wx+0.5Wy+0.4Q) Min	Y	0.258	0.139	1.863
Upper Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	X	0.925	0.469	1.975
Upper Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	Y	1.002	0.63	1.59
Lower Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	X	0.817	0.432	1.89
Lower Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	Y	0.336	0.134	2.503
Story3	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	X	0.55	0.312	1.766
Story3	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	Y	0.608	0.32	1.899
Story2	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	X	0.287	0.172	1.665
Story2	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Max	Y	0.32	0.198	1.617
Upper Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	X	1.078	0.595	1.813
Upper Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	Y	0.909	0.533	1.704
Lower Roof	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	X	0.956	0.551	1.735
Story3	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	X	0.652	0.397	1.643
Story3	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	Y	0.541	0.247	2.193
Story2	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	X	0.347	0.222	1.563
Story2	Ult 34 (1.2G-0.3Wx+0.7Wy+0.4Q) Min	Y	0.279	0.153	1.818
Upper Roof	Ult 35 (G+EQX+0.3Q) Max	X	0.79	0.464	1.704
Upper Roof	Ult 35 (G+EQX+0.3Q) Max	Y	0.959	0.652	1.471
Lower Roof	Ult 35 (G+EQX+0.3Q) Max	X	0.694	0.365	1.902
Lower Roof	Ult 35 (G+EQX+0.3Q) Max	Y	0.374	0.236	1.58
Story3	Ult 35 (G+EQX+0.3Q) Max	X	0.497	0.331	1.504
Story3	Ult 35 (G+EQX+0.3Q) Max	Y	0.593	0.371	1.597
Story2	Ult 35 (G+EQX+0.3Q) Max	X	0.281	0.198	1.418
Story2	Ult 35 (G+EQX+0.3Q) Max	Y	0.323	0.227	1.424
Upper Roof	Ult 35 (G+EQX+0.3Q) Min	X	0.635	0.282	2.255
Upper Roof	Ult 35 (G+EQX+0.3Q) Min	Y	0.759	0.516	1.47
Lower Roof	Ult 35 (G+EQX+0.3Q) Min	X	0.554	0.251	2.203
Lower Roof	Ult 35 (G+EQX+0.3Q) Min	Y	0.321	0.123	2.599
Story3	Ult 35 (G+EQX+0.3Q) Min	X	0.394	0.213	1.853
Story3	Ult 35 (G+EQX+0.3Q) Min	Y	0.453	0.236	1.917
Story2	Ult 35 (G+EQX+0.3Q) Min	X	0.224	0.135	1.655
Story2	Ult 35 (G+EQX+0.3Q) Min	Y	0.245	0.151	1.624
Upper Roof	Ult 36 (G-EQX+0.3Q) Max	X	1.615	1.165	1.386
Upper Roof	Ult 36 (G-EQX+0.3Q) Max	Y	0.869	0.462	1.879
Lower Roof	Ult 36 (G-EQX+0.3Q) Max	X	1.43	1.061	1.347
Story3	Ult 36 (G-EQX+0.3Q) Max	X	1.045	0.801	1.304
Story3	Ult 36 (G-EQX+0.3Q) Max	Y	0.537	0.239	2.249

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Ult 36 (G-EQX+0.3Q) Max	X	0.595	0.47	1.266
Story2	Ult 36 (G-EQX+0.3Q) Max	Y	0.289	0.151	1.921
Upper Roof	Ult 36 (G-EQX+0.3Q) Min	X	1.824	1.347	1.354
Upper Roof	Ult 36 (G-EQX+0.3Q) Min	Y	0.669	0.327	2.046
Lower Roof	Ult 36 (G-EQX+0.3Q) Min	X	1.517	1.175	1.291
Lower Roof	Ult 36 (G-EQX+0.3Q) Min	Y	0.346	0.132	2.627
Story3	Ult 36 (G-EQX+0.3Q) Min	X	1.181	0.921	1.282
Story3	Ult 36 (G-EQX+0.3Q) Min	Y	0.398	0.104	3.821
Story2	Ult 36 (G-EQX+0.3Q) Min	X	0.674	0.538	1.252
Story2	Ult 36 (G-EQX+0.3Q) Min	Y	0.212	0.075	2.835
Upper Roof	Ult 37 (G+EQY+0.3Q) Max	X	0.717	0.27	2.66
Upper Roof	Ult 37 (G+EQY+0.3Q) Max	Y	1.771	1.364	1.298
Lower Roof	Ult 37 (G+EQY+0.3Q) Max	X	0.582	0.238	2.448
Lower Roof	Ult 37 (G+EQY+0.3Q) Max	Y	1.016	0.872	1.165
Story3	Ult 37 (G+EQY+0.3Q) Max	X	0.415	0.183	2.27
Story3	Ult 37 (G+EQY+0.3Q) Max	Y	1.194	0.905	1.319
Story2	Ult 37 (G+EQY+0.3Q) Max	X	0.211	0.091	2.316
Story2	Ult 37 (G+EQY+0.3Q) Max	Y	0.696	0.56	1.242
Upper Roof	Ult 37 (G+EQY+0.3Q) Min	X	0.93	0.454	2.048
Upper Roof	Ult 37 (G+EQY+0.3Q) Min	Y	1.568	1.227	1.278
Lower Roof	Ult 37 (G+EQY+0.3Q) Min	X	0.668	0.351	1.903
Lower Roof	Ult 37 (G+EQY+0.3Q) Min	Y	0.961	0.758	1.268
Story3	Ult 37 (G+EQY+0.3Q) Min	X	0.553	0.304	1.821
Story3	Ult 37 (G+EQY+0.3Q) Min	Y	1.051	0.768	1.368
Story2	Ult 37 (G+EQY+0.3Q) Min	X	0.289	0.159	1.814
Story2	Ult 37 (G+EQY+0.3Q) Min	Y	0.616	0.482	1.276
Upper Roof	Ult 38 (G-EQY+0.3Q) Max	X	0.758	0.427	1.777
Upper Roof	Ult 38 (G-EQY+0.3Q) Max	Y	0.556	0.248	2.239
Lower Roof	Ult 38 (G-EQY+0.3Q) Max	X	0.813	0.459	1.771
Lower Roof	Ult 38 (G-EQY+0.3Q) Max	Y	0.801	0.653	1.227
Story3	Ult 38 (G-EQY+0.3Q) Max	X	0.46	0.282	1.632
Story3	Ult 38 (G-EQY+0.3Q) Max	Y	0.525	0.297	1.769
Story2	Ult 38 (G-EQY+0.3Q) Max	X	0.245	0.168	1.458
Story2	Ult 38 (G-EQY+0.3Q) Max	Y	0.28	0.187	1.499
Upper Roof	Ult 38 (G-EQY+0.3Q) Min	X	0.971	0.611	1.589
Upper Roof	Ult 38 (G-EQY+0.3Q) Min	Y	0.627	0.386	1.628
Lower Roof	Ult 38 (G-EQY+0.3Q) Min	X	0.899	0.572	1.57
Lower Roof	Ult 38 (G-EQY+0.3Q) Min	Y	0.976	0.767	1.271
Story3	Ult 38 (G-EQY+0.3Q) Min	X	0.599	0.403	1.485
Story3	Ult 38 (G-EQY+0.3Q) Min	Y	0.657	0.433	1.515
Story2	Ult 38 (G-EQY+0.3Q) Min	X	0.326	0.237	1.373
Story2	Ult 38 (G-EQY+0.3Q) Min	Y	0.356	0.265	1.346
Upper Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	X	0.855	0.515	1.66
Upper Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	Y	1.247	0.915	1.363
Lower Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	X	0.747	0.415	1.799
Lower Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	Y	0.61	0.482	1.264
Story3	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	X	0.535	0.362	1.476
Story3	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	Y	0.804	0.572	1.405
Story2	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	X	0.308	0.219	1.405

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Max	Y	0.455	0.349	1.303
Upper Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	X	0.653	0.278	2.352
Upper Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	Y	0.985	0.738	1.336
Lower Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	X	0.564	0.268	2.109
Lower Roof	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	Y	0.541	0.335	1.614
Story3	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	X	0.401	0.209	1.917
Story3	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	Y	0.621	0.396	1.568
Story2	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	X	0.233	0.137	1.702
Story2	Ult 39 (G+1.0EQX+0.3EQY+0.3Q) Min	Y	0.352	0.252	1.4
Upper Roof	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	Y	1.365	1.008	1.354
Lower Roof	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	X	0.4	0.064	6.292
Lower Roof	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	Y	0.695	0.554	1.253
Story3	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	X	0.271	0.074	3.676
Story3	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	Y	0.893	0.638	1.4
Story2	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	X	0.155	0.054	2.895
Story2	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Max	Y	0.51	0.394	1.295
Upper Roof	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Min	Y	1.164	0.872	1.335
Lower Roof	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Min	X	0.359	0.05	7.21
Lower Roof	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Min	Y	0.641	0.441	1.455
Story3	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Min	Y	0.752	0.502	1.497
Story2	Ult 40 (G+0.5EQX+0.5EQY+0.3Q) Min	Y	0.43	0.316	1.359
Upper Roof	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Max	Y	1.815	1.413	1.284
Lower Roof	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Max	Y	1.059	0.927	1.142
Story3	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Max	Y	1.224	0.946	1.294
Story2	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Max	Y	0.716	0.585	1.224
Upper Roof	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	X	0.698	0.237	2.948
Upper Roof	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	Y	1.552	1.235	1.257
Lower Roof	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	X	0.461	0.154	2.993
Lower Roof	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	Y	0.989	0.779	1.269
Story3	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	X	0.392	0.153	2.569
Story3	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	Y	1.04	0.768	1.353
Story2	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	X	0.191	0.068	2.822
Story2	Ult 41 (G+0.3EQX+1.0EQY+0.3Q) Min	Y	0.612	0.484	1.265
Upper Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	X	1.59	1.161	1.369
Upper Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	Y	0.642	0.241	2.665
Lower Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	X	1.452	1.078	1.347
Lower Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	Y	0.376	0.23	1.633
Story3	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	X	1.037	0.801	1.295
Story3	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	Y	0.374	0.081	4.588
Story2	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	X	0.589	0.471	1.249
Story2	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Max	Y	0.186	0.052	3.591
Upper Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Min	X	1.862	1.398	1.332
Lower Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Min	X	1.565	1.225	1.277
Lower Roof	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Min	Y	0.602	0.378	1.593
Story3	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Min	X	1.209	0.954	1.267
Story2	Ult 42 (G-1.0EQX-0.3EQY+0.3Q) Min	X	0.692	0.561	1.235
Upper Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Max	X	1.187	0.796	1.491
Upper Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Max	Y	0.464	0.107	4.336
Lower Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Max	X	1.122	0.76	1.475

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Max	Y	0.487	0.336	1.45
Story3	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Max	X	0.752	0.542	1.389
Story2	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Max	X	0.42	0.319	1.317
Upper Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	X	1.397	0.979	1.427
Lower Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	X	1.208	0.874	1.382
Lower Roof	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	Y	0.661	0.45	1.47
Story3	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	X	0.89	0.662	1.344
Story3	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	Y	0.423	0.165	2.57
Story2	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	X	0.5	0.388	1.288
Story2	Ult 43 (G-0.5EQX-0.5EQY+0.3Q) Min	Y	0.209	0.095	2.203
Upper Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	X	0.99	0.644	1.537
Upper Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	Y	0.588	0.256	2.296
Lower Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	X	1.02	0.656	1.554
Lower Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	Y	0.816	0.674	1.21
Story3	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	X	0.626	0.436	1.435
Story3	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	Y	0.537	0.293	1.829
Story2	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	X	0.344	0.26	1.321
Story2	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Max	Y	0.291	0.187	1.56
Upper Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	X	1.265	0.883	1.433
Upper Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	Y	0.681	0.434	1.569
Lower Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	X	1.132	0.803	1.409
Lower Roof	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	Y	1.042	0.823	1.267
Story3	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	X	0.801	0.591	1.355
Story3	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	Y	0.708	0.471	1.504
Story2	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	X	0.448	0.349	1.283
Story2	Ult 44 (G-0.3EQX-1.0EQY+0.3Q) Min	Y	0.385	0.285	1.352
Upper Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	X	0.774	0.468	1.652
Upper Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	Y	0.733	0.431	1.701
Lower Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	X	0.683	0.349	1.959
Story3	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	X	0.491	0.334	1.471
Story3	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	Y	0.425	0.212	2.008
Story2	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	X	0.272	0.197	1.384
Story2	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Max	Y	0.216	0.118	1.83
Upper Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	X	0.572	0.231	2.475
Upper Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	Y	0.472	0.254	1.858
Lower Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	X	0.501	0.201	2.49
Lower Roof	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	Y	0.33	0.122	2.692
Story3	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	X	0.357	0.18	1.979
Story3	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	Y	0.242	0.035	6.867
Story2	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	X	0.197	0.114	1.725
Story2	Ult 45 (G+1.0EQX-0.3EQY+0.3Q) Min	Y	0.113	0.026	4.318
Upper Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	Y	0.509	0.202	2.522
Lower Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	X	0.389	0.047	8.251
Lower Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	Y	0.351	0.208	1.685
Story3	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	X	0.197	0.025	7.768
Story3	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	Y	0.262	0.037	7.041
Story2	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	X	0.095	0.016	6.064
Story2	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Max	Y	0.11	0.018	6.227
Upper Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	X	0.521	0.165	3.167

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Upper Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	Y	0.308	0.065	4.7
Lower Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	X	0.475	0.161	2.958
Lower Roof	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	Y	0.525	0.322	1.63
Story3	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	X	0.284	0.095	2.98
Story3	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	Y	0.319	0.099	3.216
Story2	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	X	0.133	0.048	2.77
Story2	Ult 46 (G+0.5EQX-0.5EQY+0.3Q) Min	Y	0.15	0.059	2.518
Upper Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	X	0.464	0.155	2.993
Upper Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	Y	0.502	0.199	2.517
Lower Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	X	0.58	0.228	2.542
Lower Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	Y	0.735	0.598	1.229
Story3	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	X	0.258	0.094	2.747
Story3	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	Y	0.474	0.256	1.85
Story2	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	X	0.129	0.059	2.189
Story2	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Max	Y	0.268	0.175	1.531
Upper Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	X	0.739	0.394	1.877
Upper Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	Y	0.595	0.377	1.576
Lower Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	X	0.692	0.376	1.843
Lower Roof	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	Y	0.961	0.746	1.288
Story3	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	X	0.437	0.251	1.741
Story3	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	Y	0.645	0.434	1.486
Story2	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	X	0.227	0.144	1.574
Story2	Ult 47 (G+0.3EQX-1.0EQY+0.3Q) Min	Y	0.349	0.267	1.308
Upper Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	X	1.577	1.114	1.416
Upper Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	Y	1.156	0.725	1.595
Lower Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	X	1.383	1.011	1.367
Lower Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	Y	0.372	0.227	1.636
Story3	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	X	1.017	0.769	1.323
Story3	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	Y	0.743	0.437	1.699
Story2	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	X	0.578	0.448	1.29
Story2	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Max	Y	0.416	0.273	1.527
Upper Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	X	1.85	1.351	1.369
Upper Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	Y	0.895	0.548	1.633
Lower Roof	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	X	1.495	1.159	1.29
Story3	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	X	1.195	0.925	1.293
Story3	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	Y	0.561	0.262	2.146
Story2	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	X	0.681	0.537	1.268
Story2	Ult 48 (G-1.0EQX+0.3EQY+0.3Q) Min	Y	0.315	0.173	1.815
Upper Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	X	1.166	0.718	1.623
Upper Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	Y	1.32	0.913	1.445
Lower Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	X	1.006	0.65	1.549
Lower Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	Y	0.575	0.427	1.349
Story3	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	X	0.73	0.493	1.48
Story3	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	Y	0.861	0.57	1.511
Story2	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	X	0.402	0.28	1.436
Story2	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Max	Y	0.482	0.35	1.377
Upper Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	X	1.376	0.901	1.527
Upper Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	Y	1.118	0.777	1.44
Lower Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	X	1.093	0.763	1.432

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	Y	0.522	0.313	1.668
Story3	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	X	0.867	0.613	1.414
Story3	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	Y	0.72	0.434	1.659
Story2	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	X	0.482	0.349	1.381
Story2	Ult 49 (G-0.5EQX+0.5EQY+0.3Q) Min	Y	0.403	0.273	1.475
Upper Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	X	0.948	0.487	1.949
Upper Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	Y	1.788	1.356	1.318
Lower Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	X	0.789	0.435	1.815
Lower Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	Y	0.988	0.851	1.161
Story3	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	X	0.576	0.334	1.725
Story3	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	Y	1.205	0.905	1.332
Story2	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	X	0.307	0.182	1.69
Story2	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Max	Y	0.699	0.559	1.252
Upper Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	X	1.224	0.726	1.686
Upper Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	Y	1.525	1.178	1.294
Lower Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	X	0.901	0.582	1.548
Lower Roof	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	Y	0.917	0.702	1.306
Story3	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	X	0.756	0.491	1.539
Story3	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	Y	1.02	0.727	1.403
Story2	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	X	0.411	0.272	1.513
Story2	Ult 50 (G-0.3EQX+1.0EQY+0.3Q) Min	Y	0.595	0.458	1.301
Upper Roof	ENVELOPE Max	X	0.855	0.515	1.66
Upper Roof	ENVELOPE Max	Y	1.815	1.413	1.284
Lower Roof	ENVELOPE Max	X	0.747	0.415	1.799
Lower Roof	ENVELOPE Max	Y	1.059	0.927	1.142
Story3	ENVELOPE Max	X	0.535	0.362	1.476
Story3	ENVELOPE Max	Y	1.224	0.946	1.294
Story2	ENVELOPE Max	X	0.308	0.219	1.405
Story2	ENVELOPE Max	Y	0.716	0.585	1.224
Upper Roof	ENVELOPE Min	X	1.862	1.398	1.332
Upper Roof	ENVELOPE Min	Y	0.681	0.434	1.569
Lower Roof	ENVELOPE Min	X	1.565	1.225	1.277
Lower Roof	ENVELOPE Min	Y	1.042	0.823	1.267
Story3	ENVELOPE Min	X	1.209	0.954	1.267
Story3	ENVELOPE Min	Y	0.708	0.471	1.504
Story2	ENVELOPE Min	X	0.692	0.561	1.235
Story2	ENVELOPE Min	Y	0.385	0.285	1.352
Upper Roof	DStIS1	X	1.079	0.564	1.913
Upper Roof	DStIS1	Y	1.052	0.636	1.654
Lower Roof	DStIS1	X	0.941	0.514	1.83
Lower Roof	DStIS1	Y	0.296	0.072	4.096
Story3	DStIS1	X	0.648	0.376	1.725
Story3	DStIS1	Y	0.633	0.306	2.071
Story2	DStIS1	X	0.343	0.21	1.634
Story2	DStIS1	Y	0.33	0.187	1.76
Upper Roof	DStIS2	X	1.181	0.622	1.898
Upper Roof	DStIS2	Y	1.108	0.657	1.687
Lower Roof	DStIS2	X	1.055	0.579	1.823
Lower Roof	DStIS2	Y	0.308	0.058	5.283

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	DStIS2	X	0.71	0.414	1.714
Story3	DStIS2	Y	0.673	0.317	2.12
Story2	DStIS2	X	0.372	0.229	1.623
Story2	DStIS2	Y	0.353	0.199	1.776
Upper Roof	DStIS3 Max	X	0.819	0.328	2.494
Upper Roof	DStIS3 Max	Y	1.223	0.777	1.573
Lower Roof	DStIS3 Max	X	0.69	0.285	2.427
Lower Roof	DStIS3 Max	Y	0.431	0.201	2.149
Story3	DStIS3 Max	X	0.476	0.217	2.199
Story3	DStIS3 Max	Y	0.761	0.407	1.869
Story2	DStIS3 Max	X	0.243	0.118	2.051
Story2	DStIS3 Max	Y	0.411	0.248	1.655
Upper Roof	DStIS3 Min	X	1.327	0.75	1.77
Upper Roof	DStIS3 Min	Y	0.911	0.454	2.004
Lower Roof	DStIS3 Min	X	1.152	0.679	1.697
Lower Roof	DStIS3 Min	Y	0.424	0.176	2.407
Story3	DStIS3 Min	X	0.817	0.501	1.63
Story3	DStIS3 Min	Y	0.538	0.162	3.314
Story2	DStIS3 Min	X	0.442	0.283	1.561
Story2	DStIS3 Min	Y	0.272	0.099	2.75
Upper Roof	DStIS4 Max	X	0.709	0.317	2.239
Upper Roof	DStIS4 Max	Y	1.052	0.725	1.451
Lower Roof	DStIS4 Max	X	0.637	0.3	2.125
Lower Roof	DStIS4 Max	Y	0.478	0.301	1.588
Story3	DStIS4 Max	X	0.407	0.21	1.94
Story3	DStIS4 Max	Y	0.646	0.405	1.593
Story2	DStIS4 Max	X	0.203	0.112	1.808
Story2	DStIS4 Max	Y	0.347	0.252	1.378
Upper Roof	DStIS4 Min	X	1.217	0.738	1.651
Upper Roof	DStIS4 Min	Y	0.74	0.402	1.84
Lower Roof	DStIS4 Min	X	1.099	0.694	1.582
Lower Roof	DStIS4 Min	Y	0.27	0.076	3.568
Story3	DStIS4 Min	X	0.747	0.494	1.512
Story3	DStIS4 Min	Y	0.423	0.161	2.636
Story2	DStIS4 Min	X	0.403	0.277	1.451
Story2	DStIS4 Min	Y	0.207	0.102	2.032
Upper Roof	DStIS5 Max	X	0.76	0.296	2.566
Upper Roof	DStIS5 Max	Y	1.177	0.753	1.563
Lower Roof	DStIS5 Max	X	0.632	0.252	2.508
Lower Roof	DStIS5 Max	Y	0.419	0.202	2.072
Story3	DStIS5 Max	X	0.441	0.195	2.258
Story3	DStIS5 Max	Y	0.731	0.395	1.852
Story2	DStIS5 Max	X	0.225	0.107	2.102
Story2	DStIS5 Max	Y	0.395	0.24	1.648
Upper Roof	DStIS5 Min	X	1.268	0.717	1.767
Upper Roof	DStIS5 Min	Y	0.865	0.43	2.011
Lower Roof	DStIS5 Min	X	1.094	0.647	1.692
Lower Roof	DStIS5 Min	Y	0.409	0.174	2.342
Story3	DStIS5 Min	X	0.781	0.48	1.629

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story3	DStIS5 Min	Y	0.509	0.15	3.388
Story2	DStIS5 Min	X	0.424	0.272	1.55
Story2	DStIS5 Min	Y	0.256	0.09	2.835
Upper Roof	DStIS6 Max	X	0.85	0.288	2.984
Upper Roof	DStIS6 Max	Y	1.005	0.7	1.435
Lower Roof	DStIS6 Max	X	0.578	0.267	2.165
Lower Roof	DStIS6 Max	Y	0.466	0.303	1.54
Story3	DStIS6 Max	X	0.371	0.128	1.971
Story3	DStIS6 Max	Y	0.616	0.393	1.568
Story2	DStIS6 Max	X	0.155	0.101	1.534
Story2	DStIS6 Max	Y	0.331	0.243	1.361
Upper Roof	DStIS6 Min	X	1.158	0.705	1.642
Upper Roof	DStIS6 Min	Y	0.693	0.378	1.836
Lower Roof	DStIS6 Min	X	1.04	0.662	1.572
Lower Roof	DStIS6 Min	Y	0.254	0.074	3.438
Story3	DStIS6 Min	X	0.711	0.473	1.505
Story3	DStIS6 Min	Y	0.394	0.148	2.655
Story2	DStIS6 Min	X	0.355	0.288	1.448
Story2	DStIS6 Min	Y	0.191	0.093	2.048
Upper Roof	DStIS7 Max	X	0.52	0.171	3.045
Upper Roof	DStIS7 Max	Y	0.943	0.612	1.542
Lower Roof	DStIS7 Max	X	0.428	0.138	3.069
Lower Roof	DStIS7 Max	Y	0.354	0.186	1.898
Story3	DStIS7 Max	X	0.297	0.112	2.655
Story3	DStIS7 Max	Y	0.591	0.327	1.807
Story2	DStIS7 Max	X	0.148	0.06	2.433
Story2	DStIS7 Max	Y	0.322	0.198	1.625
Upper Roof	DStIS7 Min	X	1.025	0.592	1.738
Upper Roof	DStIS7 Min	Y	0.631	0.289	2.185
Lower Roof	DStIS7 Min	X	0.935	0.582	1.602
Lower Roof	DStIS7 Min	Y	0.375	0.191	1.968
Story3	DStIS7 Min	X	0.637	0.396	1.608
Story3	DStIS7 Min	Y	0.368	0.082	4.473
Story2	DStIS7 Min	X	0.348	0.225	1.545
Story2	DStIS7 Min	Y	0.182	0.049	3.758
Upper Roof	DStIS8 Max	X	0.41	0.159	2.577
Upper Roof	DStIS8 Max	Y	0.772	0.559	1.38
Lower Roof	DStIS8 Max	X	0.689	0.183	2.414
Lower Roof	DStIS8 Max	Y	0.4	0.287	1.396
Story3	DStIS8 Max	X	0.227	0.105	2.167
Story3	DStIS8 Max	Y	0.476	0.325	1.463
Story2	DStIS8 Max	X	0.199	0.054	2.003
Story2	DStIS8 Max	Y	0.257	0.201	1.278
Upper Roof	DStIS8 Min	X	0.918	0.58	1.584
Upper Roof	DStIS8 Min	Y	0.46	0.236	1.945
Lower Roof	DStIS8 Min	X	0.801	0.548	1.519
Lower Roof	DStIS8 Min	Y	0.221	0.09	2.452
Story3	DStIS8 Min	X	0.537	0.389	1.458
Story3	DStIS8 Min	Y	0.253	0.08	3.148

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Story2	DStIS8 Min	X	0.309	0.219	1.406
Story2	DStIS8 Min	Y	0.118	0.052	2.28
Upper Roof	DStIS9 Max	X	0.79	0.464	1.704
Upper Roof	DStIS9 Max	Y	0.959	0.652	1.471
Lower Roof	DStIS9 Max	X	0.694	0.365	1.902
Lower Roof	DStIS9 Max	Y	0.374	0.236	1.58
Story3	DStIS9 Max	X	0.497	0.331	1.504
Story3	DStIS9 Max	Y	0.593	0.371	1.597
Story2	DStIS9 Max	X	0.281	0.198	1.418
Story2	DStIS9 Max	Y	0.323	0.227	1.424
Upper Roof	DStIS9 Min	X	0.635	0.282	2.255
Upper Roof	DStIS9 Min	Y	0.759	0.516	1.47
Lower Roof	DStIS9 Min	X	0.554	0.251	2.203
Lower Roof	DStIS9 Min	Y	0.321	0.123	2.599
Story3	DStIS9 Min	X	0.394	0.213	1.853
Story3	DStIS9 Min	Y	0.453	0.236	1.917
Story2	DStIS9 Min	X	0.224	0.135	1.655
Story2	DStIS9 Min	Y	0.245	0.151	1.624
Upper Roof	DStIS10 Max	X	1.615	1.165	1.386
Upper Roof	DStIS10 Max	Y	0.869	0.462	1.879
Lower Roof	DStIS10 Max	X	1.43	1.061	1.347
Story3	DStIS10 Max	X	1.045	0.801	1.304
Story3	DStIS10 Max	Y	0.537	0.239	2.249
Story2	DStIS10 Max	X	0.595	0.47	1.266
Story2	DStIS10 Max	Y	0.289	0.151	1.921
Upper Roof	DStIS10 Min	X	1.824	1.347	1.354
Upper Roof	DStIS10 Min	Y	0.669	0.327	2.046
Lower Roof	DStIS10 Min	X	1.517	1.175	1.291
Lower Roof	DStIS10 Min	Y	0.346	0.132	2.627
Story3	DStIS10 Min	X	1.181	0.921	1.282
Story3	DStIS10 Min	Y	0.398	0.104	3.821
Story2	DStIS10 Min	X	0.674	0.538	1.252
Story2	DStIS10 Min	Y	0.212	0.075	2.835
Upper Roof	DStIS11 Max	X	0.717	0.27	2.66
Upper Roof	DStIS11 Max	Y	1.771	1.364	1.298
Lower Roof	DStIS11 Max	X	0.582	0.238	2.448
Lower Roof	DStIS11 Max	Y	1.016	0.872	1.165
Story3	DStIS11 Max	X	0.415	0.183	2.27
Story3	DStIS11 Max	Y	1.194	0.905	1.319
Story2	DStIS11 Max	X	0.211	0.091	2.316
Story2	DStIS11 Max	Y	0.696	0.56	1.242
Upper Roof	DStIS11 Min	X	0.93	0.454	2.048
Upper Roof	DStIS11 Min	Y	1.568	1.227	1.278
Lower Roof	DStIS11 Min	X	0.668	0.351	1.903
Lower Roof	DStIS11 Min	Y	0.961	0.758	1.268
Story3	DStIS11 Min	X	0.553	0.304	1.821
Story3	DStIS11 Min	Y	1.051	0.768	1.368
Story2	DStIS11 Min	X	0.289	0.159	1.814
Story2	DStIS11 Min	Y	0.616	0.482	1.276

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Upper Roof	DStIS12 Max	X	0.758	0.427	1.777
Upper Roof	DStIS12 Max	Y	0.556	0.248	2.239
Lower Roof	DStIS12 Max	X	0.813	0.459	1.771
Lower Roof	DStIS12 Max	Y	0.801	0.653	1.227
Story3	DStIS12 Max	X	0.46	0.282	1.632
Story3	DStIS12 Max	Y	0.525	0.297	1.769
Story2	DStIS12 Max	X	0.245	0.168	1.458
Story2	DStIS12 Max	Y	0.28	0.187	1.499
Upper Roof	DStIS12 Min	X	0.971	0.611	1.589
Upper Roof	DStIS12 Min	Y	0.627	0.386	1.628
Lower Roof	DStIS12 Min	X	0.899	0.572	1.57
Lower Roof	DStIS12 Min	Y	0.976	0.767	1.271
Story3	DStIS12 Min	X	0.599	0.403	1.485
Story3	DStIS12 Min	Y	0.657	0.433	1.515
Story2	DStIS12 Min	X	0.326	0.237	1.373
Story2	DStIS12 Min	Y	0.356	0.265	1.346
Upper Roof	DStIS13 Max	X	0.794	0.488	1.628
Upper Roof	DStIS13 Max	Y	0.925	0.634	1.459
Lower Roof	DStIS13 Max	X	0.699	0.389	1.795
Lower Roof	DStIS13 Max	Y	0.365	0.238	1.534
Story3	DStIS13 Max	X	0.503	0.346	1.451
Story3	DStIS13 Max	Y	0.571	0.362	1.576
Story2	DStIS13 Max	X	0.285	0.207	1.378
Story2	DStIS13 Max	Y	0.311	0.219	1.422
Upper Roof	DStIS13 Min	X	0.639	0.306	2.09
Upper Roof	DStIS13 Min	Y	0.725	0.498	1.454
Lower Roof	DStIS13 Min	X	0.558	0.276	2.026
Lower Roof	DStIS13 Min	Y	0.312	0.125	2.502
Story3	DStIS13 Min	X	0.4	0.229	1.746
Story3	DStIS13 Min	Y	0.431	0.227	1.897
Story2	DStIS13 Min	X	0.227	0.144	1.583
Story2	DStIS13 Min	Y	0.233	0.144	1.614
Upper Roof	DStIS14 Max	X	1.571	1.141	1.377
Upper Roof	DStIS14 Max	Y	0.834	0.444	1.879
Lower Roof	DStIS14 Max	X	1.386	1.037	1.337
Story3	DStIS14 Max	X	1.018	0.785	1.296
Story3	DStIS14 Max	Y	0.516	0.23	2.242
Story2	DStIS14 Max	X	0.582	0.461	1.261
Story2	DStIS14 Max	Y	0.278	0.145	1.924
Upper Roof	DStIS14 Min	X	1.78	1.323	1.345
Upper Roof	DStIS14 Min	Y	0.634	0.308	2.056
Lower Roof	DStIS14 Min	X	1.473	1.151	1.28
Lower Roof	DStIS14 Min	Y	0.335	0.131	2.563
Story3	DStIS14 Min	X	1.154	0.905	1.276
Story3	DStIS14 Min	Y	0.377	0.095	3.948
Story2	DStIS14 Min	X	0.661	0.53	1.247
Story2	DStIS14 Min	Y	0.2	0.068	2.924
Upper Roof	DStIS15 Max	X	0.673	0.245	2.743
Upper Roof	DStIS15 Max	Y	1.737	1.346	1.29

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	DStIS15 Max	X	0.538	0.213	2.522
Lower Roof	DStIS15 Max	Y	1.007	0.873	1.153
Story3	DStIS15 Max	X	0.388	0.166	2.33
Story3	DStIS15 Max	Y	1.172	0.896	1.307
Story2	DStIS15 Max	X	0.198	0.083	2.393
Story2	DStIS15 Max	Y	0.684	0.554	1.235
Upper Roof	DStIS15 Min	X	0.885	0.43	2.06
Upper Roof	DStIS15 Min	Y	1.534	1.209	1.269
Lower Roof	DStIS15 Min	X	0.624	0.327	1.911
Lower Roof	DStIS15 Min	Y	0.952	0.759	1.254
Story3	DStIS15 Min	X	0.527	0.288	1.83
Story3	DStIS15 Min	Y	1.029	0.759	1.356
Story2	DStIS15 Min	X	0.276	0.151	1.828
Story2	DStIS15 Min	Y	0.604	0.476	1.268
Upper Roof	DStIS16 Max	X	0.714	0.403	1.774
Upper Roof	DStIS16 Max	Y	0.558	0.266	2.093
Lower Roof	DStIS16 Max	X	0.769	0.435	1.769
Lower Roof	DStIS16 Max	Y	0.79	0.652	1.212
Story3	DStIS16 Max	X	0.433	0.266	1.63
Story3	DStIS16 Max	Y	0.521	0.305	1.706
Story2	DStIS16 Max	X	0.232	0.16	1.452
Story2	DStIS16 Max	Y	0.284	0.195	1.46
Upper Roof	DStIS16 Min	X	0.926	0.587	1.578
Upper Roof	DStIS16 Min	Y	0.629	0.404	1.559
Lower Roof	DStIS16 Min	X	0.855	0.548	1.56
Lower Roof	DStIS16 Min	Y	0.964	0.766	1.258
Story3	DStIS16 Min	X	0.572	0.387	1.477
Story3	DStIS16 Min	Y	0.653	0.442	1.477
Story2	DStIS16 Min	X	0.312	0.228	1.367
Story2	DStIS16 Min	Y	0.357	0.271	1.319
Upper Roof	DStID1	X	0.799	0.418	1.913
Upper Roof	DStID1	Y	0.779	0.471	1.654
Lower Roof	DStID1	X	0.697	0.381	1.83
Lower Roof	DStID1	Y	0.219	0.053	4.096
Story3	DStID1	X	0.48	0.278	1.725
Story3	DStID1	Y	0.469	0.226	2.071
Story2	DStID1	X	0.254	0.155	1.634
Story2	DStID1	Y	0.244	0.139	1.76
Upper Roof	DStID2	X	0.947	0.498	1.901
Upper Roof	DStID2	Y	0.895	0.532	1.681
Lower Roof	DStID2	X	0.843	0.462	1.824
Lower Roof	DStID2	Y	0.249	0.05	5.027
Story3	DStID2	X	0.569	0.332	1.716
Story3	DStID2	Y	0.542	0.257	2.111
Story2	DStID2	X	0.299	0.184	1.625
Story2	DStID2	Y	0.284	0.16	1.773
Upper Roof	DCmpD1	X	0.799	0.418	1.913
Upper Roof	DCmpD1	Y	0.779	0.471	1.654
Lower Roof	DCmpD1	X	0.697	0.381	1.83

Table 2.1 - Story Max/Avg Displacements (continued)

Story	Load Case/Combo	Direction	Maximum mm	Average mm	Ratio
Lower Roof	DCmpD1	Y	0.219	0.053	4.096
Story3	DCmpD1	X	0.48	0.278	1.725
Story3	DCmpD1	Y	0.469	0.226	2.071
Story2	DCmpD1	X	0.264	0.165	1.634
Story2	DCmpD1	Y	0.244	0.139	1.76
Upper Roof	DCmpD2	X	0.847	0.498	1.901
Upper Roof	DCmpD2	Y	0.895	0.532	1.681
Lower Roof	DCmpD2	X	0.843	0.462	1.824
Lower Roof	DCmpD2	Y	0.249	0.05	5.027
Story3	DCmpD2	X	0.569	0.362	1.718
Story3	DCmpD2	Y	0.542	0.257	2.111
Story2	DCmpD2	X	0.299	0.184	1.625
Story2	DCmpD2	Y	0.284	0.16	1.773
Upper Roof	DCmpS1	X	1.078	0.584	1.843
Upper Roof	DCmpS1	Y	1.052	0.636	1.654
Lower Roof	DCmpS1	X	0.941	0.514	1.83
Lower Roof	DCmpS1	Y	0.296	0.072	4.096
Story3	DCmpS1	X	0.648	0.376	1.725
Story3	DCmpS1	Y	0.633	0.306	2.071
Story2	DCmpS1	X	0.343	0.21	1.634
Story2	DCmpS1	Y	0.33	0.187	1.76
Upper Roof	DCmpS2	X	1.301	0.685	1.9
Upper Roof	DCmpS2	Y	1.225	0.727	1.684
Lower Roof	DCmpS2	X	1.159	0.636	1.824
Lower Roof	DCmpS2	Y	0.341	0.066	5.139
Story3	DCmpS2	X	0.782	0.463	1.715
Story3	DCmpS2	Y	0.743	0.351	2.115
Story2	DCmpS2	X	0.41	0.253	1.624
Story2	DCmpS2	Y	0.39	0.22	1.774
Upper Roof	DCmpC1	X	1.123	0.586	1.916
Upper Roof	DCmpC1	Y	1.1	0.666	1.652
Lower Roof	DCmpC1	X	0.936	0.537	1.829
Lower Roof	DCmpC1	Y	0.306	0.073	4.213
Story3	DCmpC1	X	0.677	0.391	1.729
Story3	DCmpC1	Y	0.66	0.318	2.077
Story2	DCmpC1	X	0.368	0.219	1.638
Story2	DCmpC1	Y	0.344	0.195	1.766

2.2 Modal Results

Table 2.2 - Modal Periods and Frequencies

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad ² /sec ²
Modal	1	0.148	6.741	42.3547	1793.9178
Modal	2	0.144	6.938	43.7831	1916.9586
Modal	3	0.127	7.875	49.4784	2448.1126
Modal	4	0.093	10.193	64.0736	4105.4293
Modal	5	0.09	11.133	69.9528	4893.3966
Modal	6	0.068	17.193	108.0257	11669.5461

Table 2.2 - Modal Periods and Frequencies (continued)

Case	Mode	Period sec	Frequency cyc/sec	Circular Frequency rad/sec	Eigenvalue rad ² /sec ²
Modal	7	0.058	17.279	108.5656	11786.492
Modal	8	0.056	17.766	111.6291	12461.0468
Modal	9	0.053	18.814	118.2115	13973.9588
Modal	10	0.051	19.838	123.2516	15225.478
Modal	11	0.049	20.497	128.7849	16585.5557
Modal	12	0.046	21.895	137.5692	18925.0949
Modal	13	0.043	23.507	147.6958	21814.0406
Modal	14	0.039	25.791	161.9858	26239.4064
Modal	15	0.033	30.07	188.9326	35695.5214
Modal	16	0.029	33.905	213.0336	45383.3004
Modal	17	0.022	45.872	288.2202	83070.8591
Modal	18	0.019	51.314	322.4165	103952.4104
Modal	19	0.011	92.028	578.2318	334352.0482
Modal	20	0.011	93.237	585.8229	343168.4248

Table 2.3 - Modal Participating Mass Ratios (Part 1 of 2)

Case	Mode	Period sec	UX	UY	UZ	Sum UX	Sum UY	Sum UZ
Modal	1	0.148	0.1722	0.4335	0	0.1722	0.4335	0
Modal	2	0.144	0.209	0.0424	0	0.3752	0.4759	0
Modal	3	0.127	0.1899	0.1463	0	0.5651	0.6222	0
Modal	4	0.096	0.006	0.015	0	0.5711	0.6402	0
Modal	5	0.09	0.0007	0.0029	0	0.5718	0.6431	0
Modal	6	0.058	0.0312	0.0098	0	0.573	0.6524	0
Modal	7	0.058	0.0056	0.104	0	0.5786	0.7564	0
Modal	8	0.056	0.0055	0.0454	0	0.5841	0.8017	0
Modal	9	0.053	0.0104	0.0578	0	0.5945	0.8595	0
Modal	10	0.051	0.0232	0.0228	0	0.6176	0.8823	0
Modal	11	0.049	0.0612	0.0256	0	0.6789	0.9079	0
Modal	12	0.046	0.1784	0.0108	0	0.8573	0.9186	0
Modal	13	0.043	0.0411	0.0053	0	0.8984	0.9239	0
Modal	14	0.039	0.0094	0.0035	0	0.9079	0.9274	0
Modal	15	0.033	0.0283	0.0026	0	0.9362	0.9301	0
Modal	16	0.029	0.0015	0.0177	0	0.9376	0.9478	0
Modal	17	0.022	0.0072	0.0045	0	0.9448	0.9522	0
Modal	18	0.019	0.0078	0.0122	0	0.9526	0.9644	0
Modal	19	0.011	0.0009	0.0228	0	0.9536	0.9872	0
Modal	20	0.011	0.0335	0.0016	0	0.987	0.9886	0

Table 2.3 - Modal Participating Mass Ratios (Part 2 of 2)

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	1	0.2669	0.1278	0.0269	0.2669	0.1278	0.0269
Modal	2	0.03	0.1344	0.2305	0.2969	0.3122	0.2574
Modal	3	0.0846	0.1425	0.1668	0.3815	0.4548	0.4242
Modal	4	0.0057	0.0067	0.0148	0.3872	0.4605	0.4391
Modal	5	0.0015	0.0007	0.0021	0.3887	0.4612	0.4412
Modal	6	0.0099	0.0005	0.0007	0.3986	0.4618	0.4419
Modal	7	0.1642	0.0066	0.0025	0.5628	0.4682	0.4444

Table 2.3 - Modal Participating Mass Ratios (Part 2 of 2, continued)

Case	Mode	RX	RY	RZ	Sum RX	Sum RY	Sum RZ
Modal	8	0.069	0.0033	0.0044	0.6317	0.4715	0.4488
Modal	9	0.1247	0.0156	0.0025	0.7565	0.4871	0.4513
Modal	10	0.0539	0.0272	3.048E-05	0.8104	0.5143	0.4514
Modal	11	0.0424	0.061	0.0018	0.8528	0.5752	0.4531
Modal	12	0.0129	0.2367	0.002	0.8658	0.8119	0.4551
Modal	13	0.0114	0.0479	0.0004	0.8771	0.8598	0.4555
Modal	14	0.0076	0.0085	0.0206	0.8847	0.8683	0.4761
Modal	15	0.0053	0.0339	0.0117	0.89	0.9022	0.4878
Modal	16	0.0248	0.0024	0.0466	0.9148	0.9046	0.5344
Modal	17	0.0045	0.0043	0.0075	0.9193	0.9088	0.5419
Modal	18	0.0086	0.004	0.075	0.9279	0.9129	0.6169
Modal	19	0.0275	0.001	0.0053	0.9554	0.9139	0.6222
Modal	20	0.0019	0.0405	0.0048	0.9573	0.9543	0.6271

Table 2.4 - Modal Participation Factors

Case	Mode	Period sec	UX kN-m	UY kN-m	UZ kN-m	RX kN-m	RY kN-m	RZ kN-m	Modal Mass kN-m-s ²	Modal Stiffness kN-m
Modal	1	0.148	1.5E-05	2.3E-05	0	-0.061643	0.042658	0.045143	1E-06	0.00179
Modal	2	0.144	-1.6E-05	7E-06	0	-0.02065	-0.051239	0.132163	1E-06	0.00192
Modal	3	0.127	-1.5E-05	1.4E-05	0	-0.034708	-0.045048	-0.112424	1E-06	0.00245
Modal	4	0.098	3E-06	-5E-06	0	0.009032	0.008982	0.033657	1E-06	0.00411
Modal	5	0.09	1E-06	2E-06	0	-0.004595	0.003155	0.012597	1E-06	0.00489
Modal	6	0.058	-1E-06	3E-06	0	0.011871	0.002599	-0.007147	1E-06	0.01167
Modal	7	0.058	-3E-06	-1.1E-05	0	-0.048341	0.009692	-0.013768	1E-06	0.01179
Modal	8	0.056	-3E-06	-8E-06	0	-0.031338	0.006818	-0.018318	1E-06	0.01246
Modal	9	0.053	-4E-06	-9E-06	0	-0.042136	0.014887	-0.013787	1E-06	0.01397
Modal	10	0.051	5E-06	5E-06	0	0.027708	-0.019685	0.00152	1E-06	0.01523
Modal	11	0.049	-9E-06	6E-06	0	0.024583	0.029458	-0.011545	1E-06	0.01659
Modal	12	0.046	1.5E-05	-4E-06	0	-0.013575	-0.058047	-0.012215	1E-06	0.01893
Modal	13	0.043	7E-06	-3E-06	0	-0.012718	-0.026104	-0.005531	1E-06	0.02181
Modal	14	0.039	-3E-06	-2E-06	0	-0.010395	0.010989	0.039553	1E-06	0.02624
Modal	15	0.033	6E-06	-2E-06	0	-0.00866	-0.021977	-0.029759	1E-06	0.0357
Modal	16	0.029	-1E-06	-5E-06	0	-0.018783	0.005829	0.059418	1E-06	0.04538
Modal	17	0.022	-3E-06	2E-06	0	0.007994	0.007785	-0.02389	1E-06	0.08307
Modal	18	0.019	3E-06	4E-06	0	0.01108	-0.007569	-0.075382	1E-06	0.10395
Modal	19	0.011	1E-06	-5E-06	0	-0.019785	-0.003768	0.020051	1E-06	0.33435
Modal	20	0.011	6E-06	1E-06	0	0.005156	-0.024006	0.019097	1E-06	0.34319

Table 2.5 - Modal Load Participation Ratios

Case	Item Type	Item	Static %	Dynamic %
Modal	Acceleration	UX	100	98.7
Modal	Acceleration	UY	100	98.88
Modal	Acceleration	UZ	0	0

Table 2.6 - Modal Direction Factors

Case	Mode	Period sec	UX	UY	UZ	RZ
Modal	1	0.148	0.295	0.693	0	0.012
Modal	2	0.144	0.434	0.091	0	0.475
Modal	3	0.127	0.478	0.348	0	0.175
Modal	4	0.098	0.007	0.014	0	0.979
Modal	5	0.09	0.002	0.007	0	0.991
Modal	6	0.058	0.004	0.063	0	0.934
Modal	7	0.058	0.036	0.882	0	0.082
Modal	8	0.056	0.023	0.257	0	0.72
Modal	9	0.053	0.104	0.689	0	0.207
Modal	10	0.051	0.288	0.408	0	0.304
Modal	11	0.049	0.347	0.147	0	0.506
Modal	12	0.046	0.756	0.047	0	0.197
Modal	13	0.043	0.598	0.149	0	0.253
Modal	14	0.039	0.109	0.147	0	0.744
Modal	15	0.033	0.671	0.086	0	0.243
Modal	16	0.029	0.08	0.818	0	0.102
Modal	17	0.022	0.395	0.173	0	0.432
Modal	18	0.019	0.071	0.114	0	0.815
Modal	19	0.011	0.052	0.327	0	0.621
Modal	20	0.011	0.742	0.074	0	0.183

Roof bracing — R_{S1} and R_{BR1} .
 R_{S2} & R_{BR1} .

Lateral Load from

$$EQ = 183 \text{ kN. (from ETAS)}$$

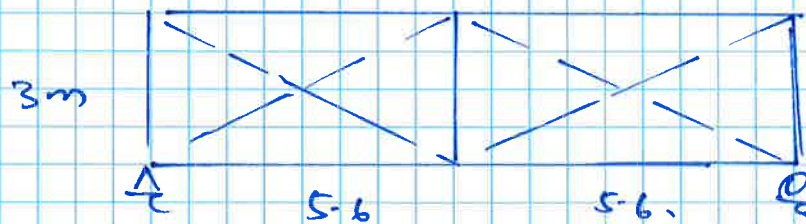
$$\text{Wind} = 1.13 * 19 * 4.8 / 2 = 51.5 \text{ kN.}$$

EQ load is critical.

∴ Designing roof bracing for EQ load.

Higher roof:

take half load
as the load
share by
lower roof
bracing.



$$\text{Half load} = \frac{183}{2} = 91.5 \text{ kN.}$$

$$\text{Line load} = 8.2 \text{ kN/m}$$

$$M_{\max} = \frac{8.2 * 11.2^2}{8} = 128.6 \text{ kN.}$$

$$\text{Lever arm} = 3 \text{ m.}$$

$$\therefore \text{Compression on strut} = \frac{128.6}{3} = 42.8 \text{ kN} \approx 50 \text{ kN.}$$

$$\text{Leff} = 5.6 \text{ m}$$

Adopt $R_{S1} = 100 \times 100 \times 4 \text{ SHS}$

R_{BR1} — Adopt $65 \times 5 \text{ EA.}$

$$\phi N_c = 102 \text{ kN} > \text{reqd. (OK)}$$

(OK)

R_{S2}
Lower roof

$$\text{Leff for strut} = 4.6 \text{ m.}$$

$$\text{Adopt } 89 \times 35 \text{ SHS; } \phi N_c = 90.1 \text{ kN} > \text{reqd. (OK)}$$

Checked :

Date : .../.../...

Tie beams

$$\text{Wind Load} = 1.13 \times \frac{4.8}{2} = 2.71 \text{ kN/m}$$

$$W_{\text{ult}} = 2.71 \text{ kN/m}$$

$$W_{\text{sew}} = 0.77 \times \frac{4.8}{2} = 1.85 \text{ kN/m}$$

RTB1.

Span 5600 mm.

$$M_{\text{max}} = 2.71 \times \frac{5.6^2}{8} = 10.6 \text{ kNm}$$

$$I_{\text{req}} \text{ for } \frac{1}{250} \text{ of } W_{\text{sew}} = 5.29 \times 10^6$$

Adopt 150 PFC.

Comp. on Tie beam due to EQ Load = 50 kN.

Comp. Capacity of RTB1 for $L_{\text{eff}} = 5.6 \text{ m} > \text{req. (OK)}$

Adopt 150 PFC on Flat; provide anchor @ 900 ϕ to precast panel.

RTB2

Max span = 4.6 m.

$$\therefore M_{\text{max}} = 2.71 \times \frac{4.6^2}{8} = 7.2 \text{ kNm}$$

$$I_{\text{req}} \text{ for } \frac{1}{250} \text{ of } W_{\text{sew}} = 2.93 \times 10^6$$

Adopt 125 PFC on flat

From ETAB.

Total lateral force on 3rd level.

$$= 183 + 343 = 526 \text{ kN.}$$

Precast panels at all 4-sides

o/c (OK)

Total lateral force on 2nd level

$$= 183 + 343 + 217$$

$$= 743 \text{ kN.}$$

Precast panels at all 4-side + lift and stair shaft

o/c (OK)

Total lateral force on 1st level.

$$= 183 + 343 + 217 + 233$$

$$= 976 \text{ kN.}$$

Precast panel - (200 thick as retaining)
at all side + lift & stair shaft.

o/c (OK)