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King & Dawson Ltd architects & engineers ltd

20th June 2019

Attention: Raymond Leslie Smith, Joan Bernadette Smith
Unit 9, 220 Thorndon Quay
Thorndon
Wellington 6011

Seismic Strengthening Design Stage Two: Draft Design

210-220 Thorndon Quay

1.0 Executive Summary

We (King & Dawson) have been engaged to complete a seismic strengthening design for the structure at the above address, to bring it above 67% of New Building Standard (%NBS) as defined by the *Seismic Assessment of Existing Buildings* (NZSEE et al, July 2017). Stage one of the design process involved confirming the current rating of the structure to the current standards - refer the attached document *Seismic Strengthening Design Stage One: DSA Review*. This found that the structure as it currently stands achieves 20-30% NBS. The draft design described here is sufficient to bring the structure up to 70-75% NBS. *Stage Three: Detailed Design* will finalize this proposal.

Image: Google Maps 2019

2.0 Background

2.1 Stage One: DSA Review

Refer attached document. Stage one involved assessing the expected failure mechanism of the structure and current NBS rating. This found that the structure currently achieves 20-30%NBS, governed by the plastic hinging of the front and rear perimeter frames under north/south seismic loading, allowing the columns to rotate past their curvature limit and losing gravity load bearing capacity.

2.2 Stage Two: Draft Design

Presented here is the draft design for a strengthening design that will bring the rating of the structure to 70-75%NBS. This will work by providing a secondary load path for north/south seismic loading that will be engaged as the frames begin to hinge. This in turn limits the interstorey drift of each level, preventing the columns from exceeding their curvature limits and maintaining their gravity resisting capacity.

2.3 Stage Three: Detailed Design

The final stage of the design process involves assessing the finer details of the design, such as connecting the new elements to the existing structure, and quantifying the level of required foundation work involved.

2.4 Life Safety Approach

Worth noting is that the approach taken by the Seismic Assessment of Existing Buildings in comparison to that of new building design. In new building design, a level of serviceability is expected following a major seismic event. This contrasts with assessments, where the focus is preserving human life, and varying degrees of structural damage is expected.

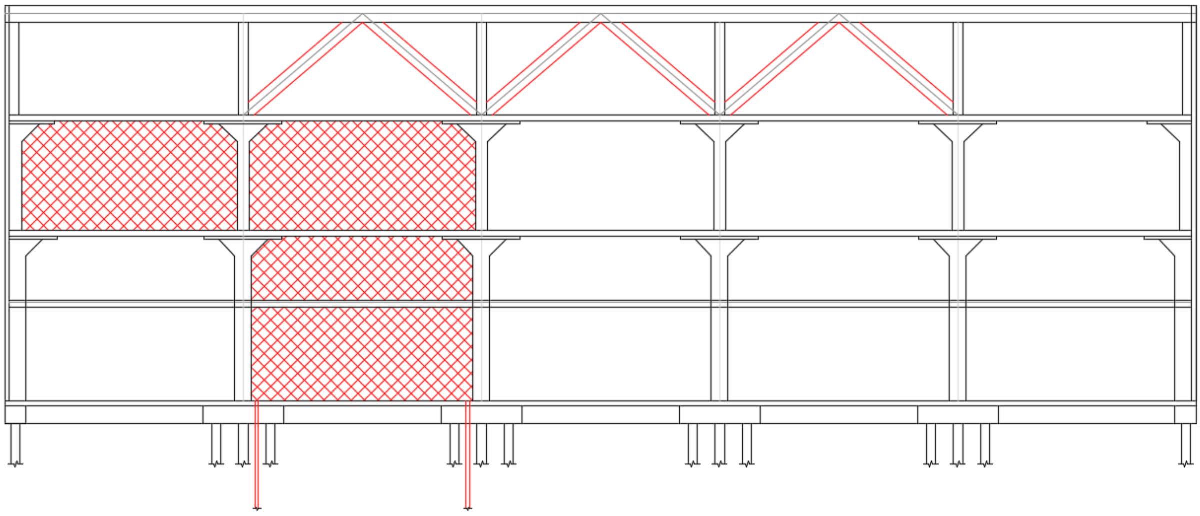
3.0 Strengthening Design Features

3.1 Shear Walls

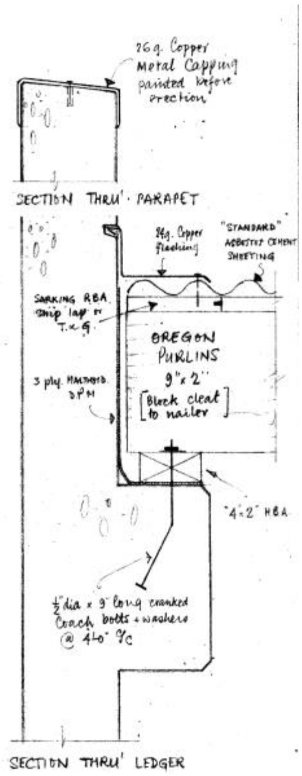
The primary element of the secondary load path is a series of four sprayed concrete shear walls, similar to that constructed in the 2010 strengthening. These will be located down the centreline of the building (Grid C).

3.2 Concentrically Braced Frames

At the apartment level, a series of three concentrically braced steel frames (CBF's) are proposed. These will incorporate existing concrete columns and the rolled steel joist (RSJ) that spans the length of the structure.



Above: Elevation of proposed works to Grid C
Below: Existing gable end to purlin connection



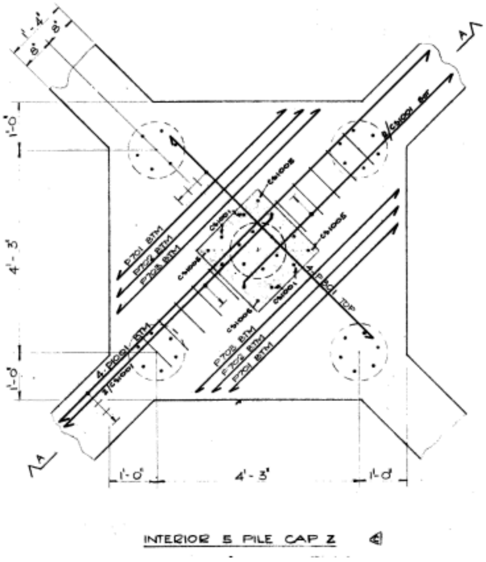
3.3 Gable End Connections

A secondary aspect of the proposed works involves strengthening the existing connection between the gable end walls and the roof structure. Stage One of this design found that the existing connection achieves 62%NBS. In order to bring this to above 67% additional fixings are required at regular spacings along the north and south walls.

4.0 Foundations

4.1 Uplift Demand

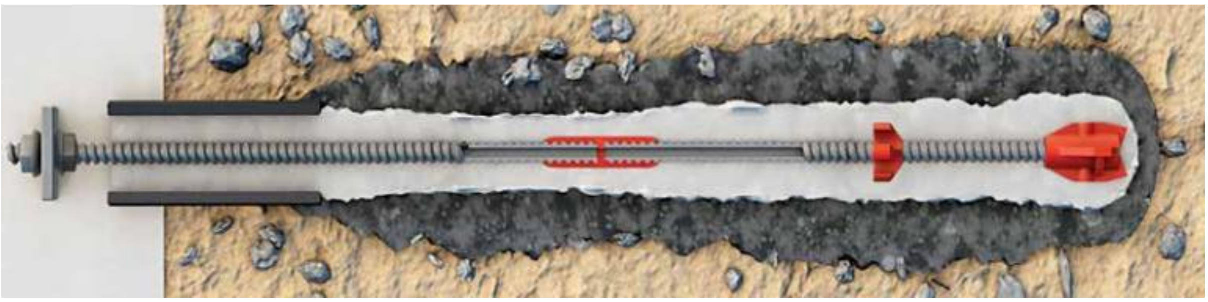
When the proposed shear wall elements are engaged, a significant uplift force is expected at one end, to be transferred through the foundations and into the subsoil. The existing foundations consist of reinforced concrete piles, currently thought to be precast and driven to refusal. Note that the existing plans and specifications do not include details of the pile driving or the depth these piles reached. This means that any assessment existing foundations contains a high degree of uncertainty.



Left: Existing pile cap detail

4.2 Micro-piles

The most likely solution to the above uncertainty is to provide supplementary piles. These would be 'Ischebeck' style micro-piles, ~150mm diameter hollow steel rods using grout injection to encase themselves in concrete, dropped through the pile cap between existing piles. This would provide rated uplift capacity, ensuring that the system performs as designed despite uncertainty around the existing piles. Extent of micro-piling is to be confirmed pending geotechnical advice.



Above: Ischebeck Titan micro-pile cross section

5.0 Conclusions

The draft design presented here will bring the structure at 210-220 Thorndon Quay from 20-30%NBS to 70-75%NBS. It involves four bays of sprayed concrete shear walls, and three bays of steel frames, and some minor works to the upper edge of the north and south walls. *Stage Three: Detailed Design* will finalize this proposal for construction.

~

Yours faithfully	Reviewed	Approved
		DRAFT
Matt Taylor **BEngTech**	**Jared Sullivan,** **BE (hons), CMEngNZ, CPEng**	**J C Wilson** **Director**

6.0 Appendices

6.1 Calculation Index

6.2 References

The following documents have been referenced throughout our calculations:

The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments
New Zealand Society for Earthquake Engineering et al., July 2017 incl. March 2018 Errata

NZS3101:2006 Concrete Structures
Standards New Zealand, incl. Amendments 1-3 2016

NZS3603:1993 Timber Structures
Standards New Zealand, incl. Amendments 1, 2 & 4 2005

NZS3404:1997 Steel Structures
Standards New Zealand, incl. Amendments 1 & 2 2007

Paper House, 1961
Original structural drawings, Frank Kerslake Consulting Engineer

New apartments at 210-220 Thorndon Quay for Globe Holdings Limited, 1997
Alteration structural drawings, Dunning Thornton Consultants Ltd

King & Dawson Ltd architects & engineers				
JOB: 220 Thorndon Quay	JOB No: 10256	DESIGNED: MHT	PAGE No:
	DATE: 2019	CHECKED: JRS	01

Calculation Summary

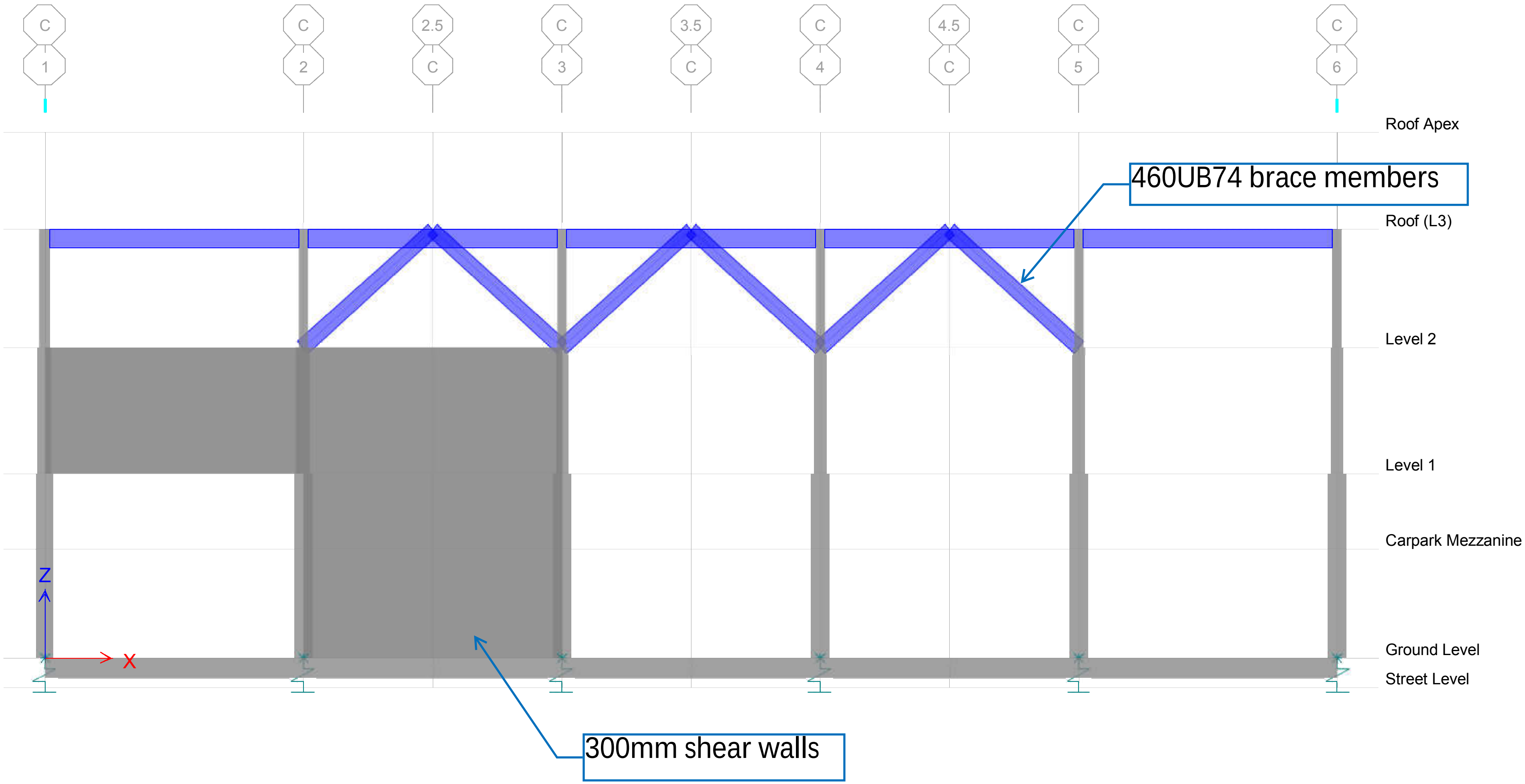
Scheme 14 - Grid C shear walls & CBF's, gable end fixings

Element		Action	Strength	Load	Rating	
West Frame Columns	LG-L1	Flexure	615	213	100%	
(Street side, Grid A)		Shear	678	120	100%	
	L1-L2	Flexure	285	61	100%	
		Shear	385	46	100%	
	L2-L3	Flexure	70	36	100%	
		Shear	161	26	100%	
West Frame Spandrels	L1	Flexure	324.2	640	51%	
(Street side, Grid A)		Shear	548	297	100%	
	L2	Flexure	382.5	604	63%	
		Shear	606	296	100%	
	L3	Flexure	357.4	460	78%	
		Shear	599	221	100%	
East Frame Columns	LG-L1	Flexure	615	550	100%	
(Rear side, Grid E)		Shear	837	580	100%	
	L1-L2	Flexure	285	100	100%	
		Shear	375	81	100%	
	L2-L3	Flexure	70	56	100%	
		Shear	162	33	100%	
East Frame Spandrels	L1	Flexure	653.4	525	100%	
(Rear side, Grid E)		Shear	866	166	100%	
	L2	Flexure	374.1	467	80%	
		Shear	624	137	100%	
	L3	Flexure	184	184	100%	
		Shear	493	49	100%	
Internal Columns	LG-L1	Flexure	615	307	100%	
N & S Shear Walls		Flexure	82063	31907	100%	E/W Loading
		Shear	41357	10453	100%	E/W Loading
Roof Perimeter Connections		Shear	2.5	2.4	100%	
Gable End Connections		Shear	3.4	3.4	100%	

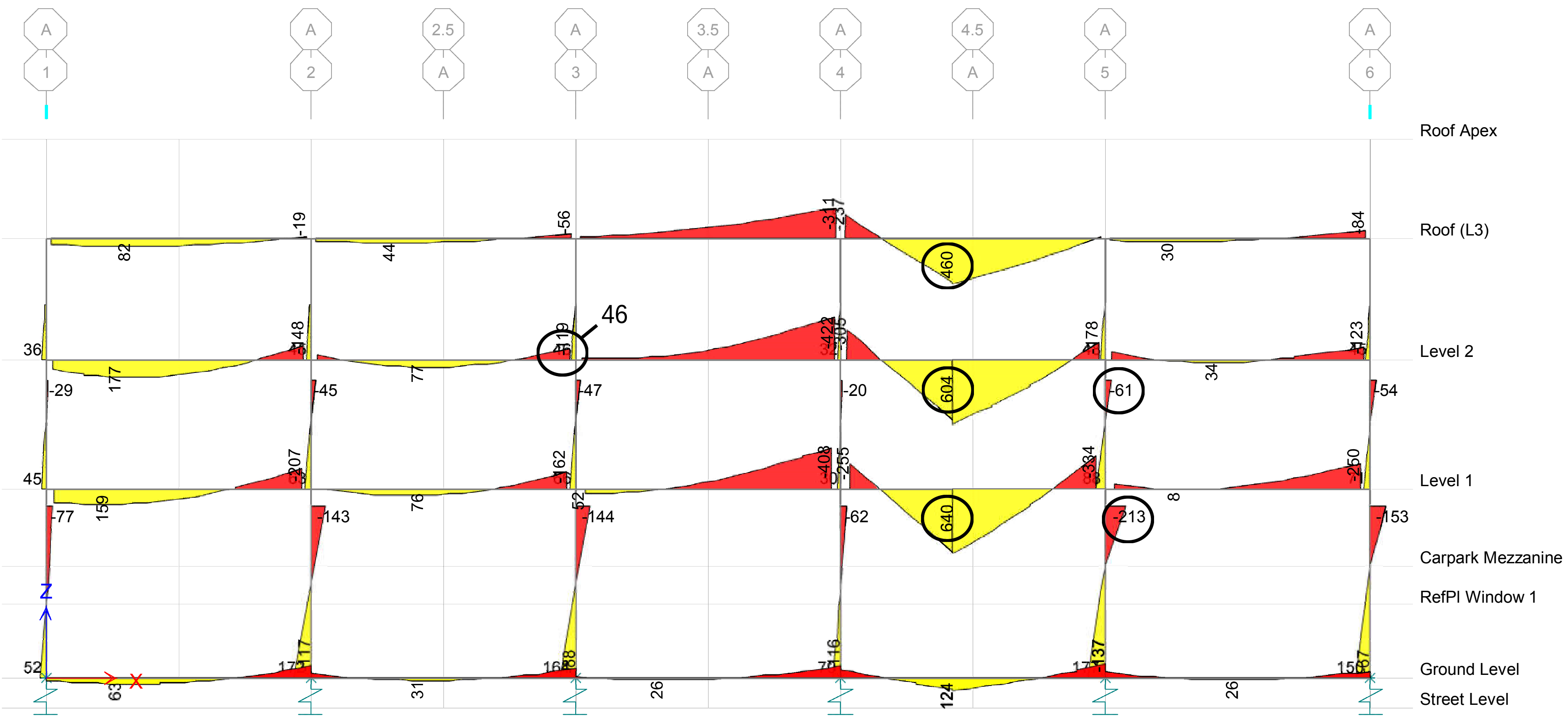
Min = 51%

Column drift (post frame hinging)	28	39	72%

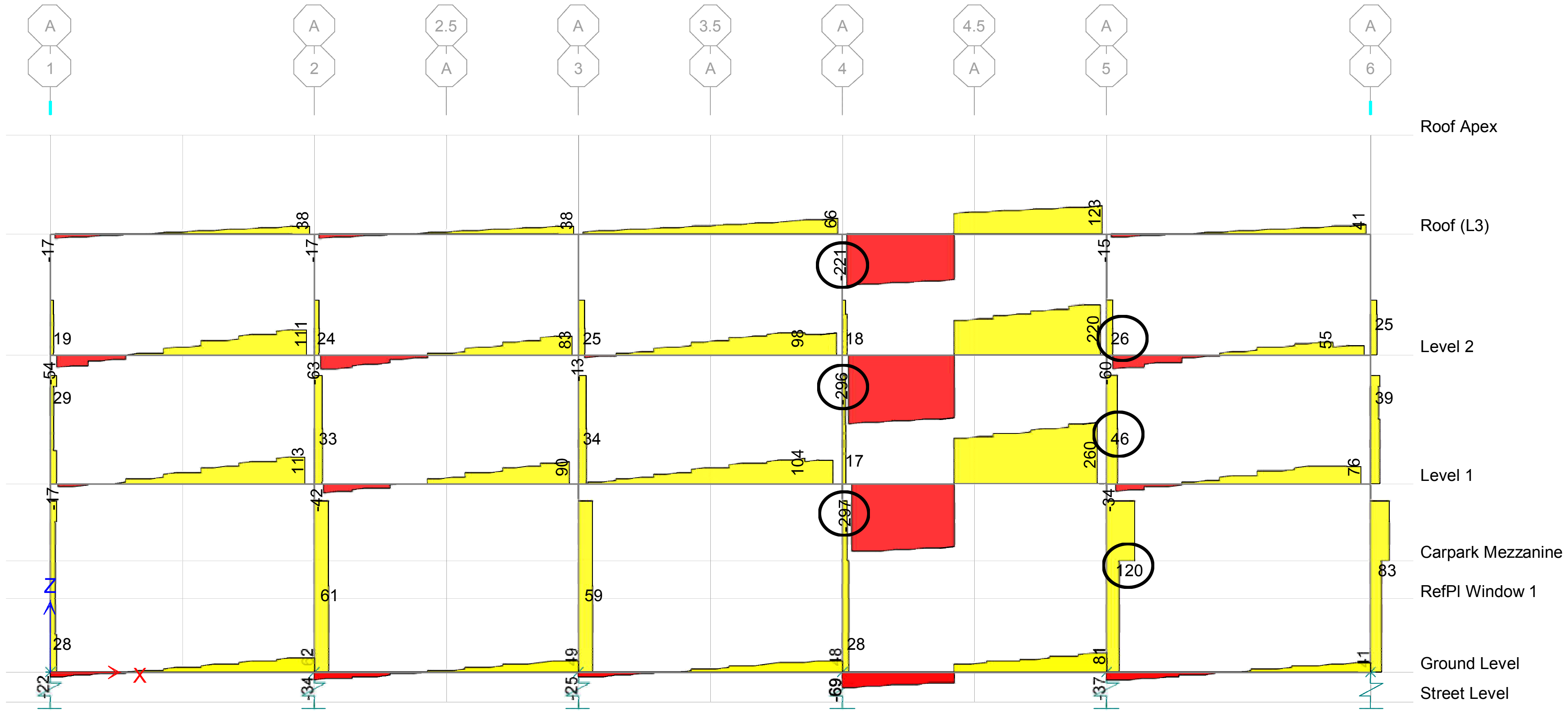
Refer *Strengthening Design Stage One* for full model layout.
All new primary elements are located on Grid C, shown below.



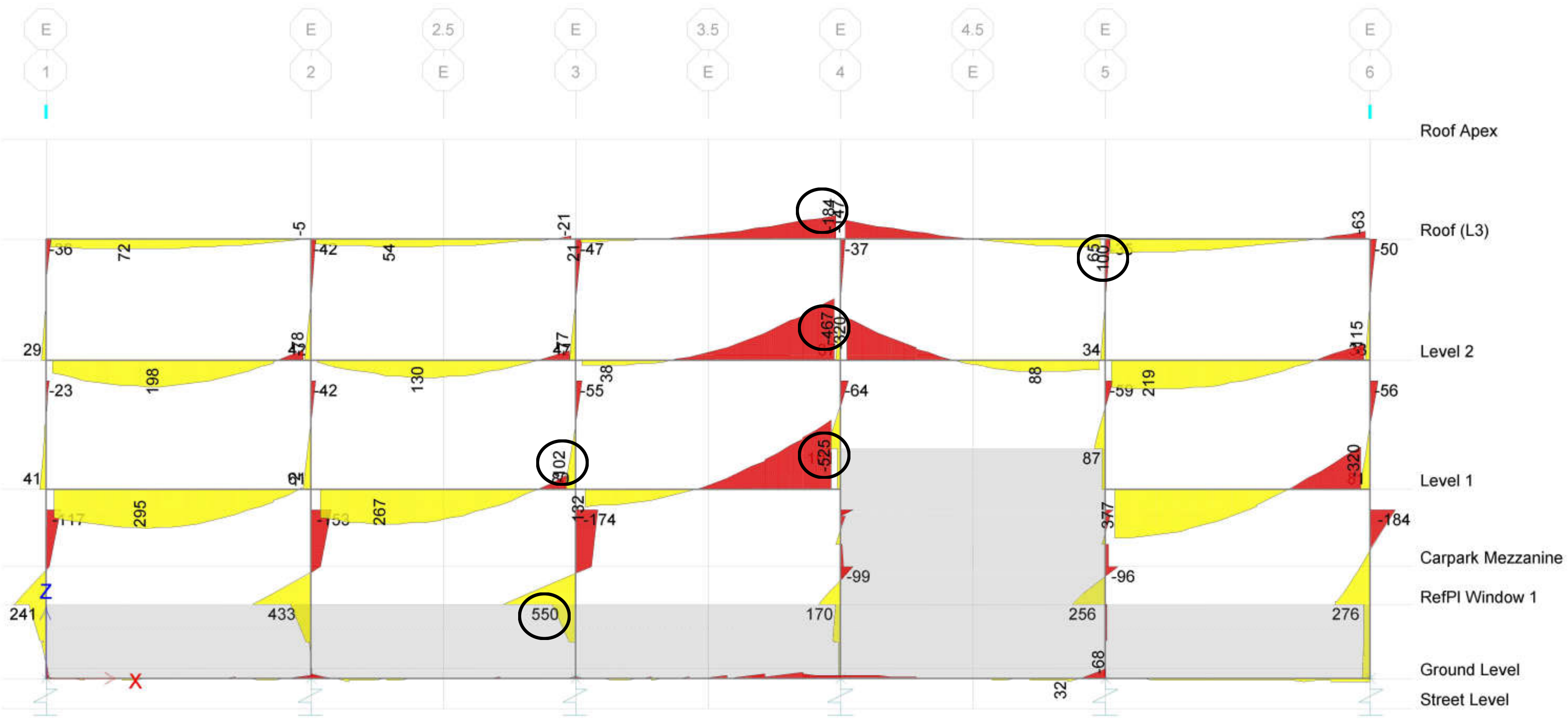
Page 02b.i


Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
46
460
604
640
-61
-213

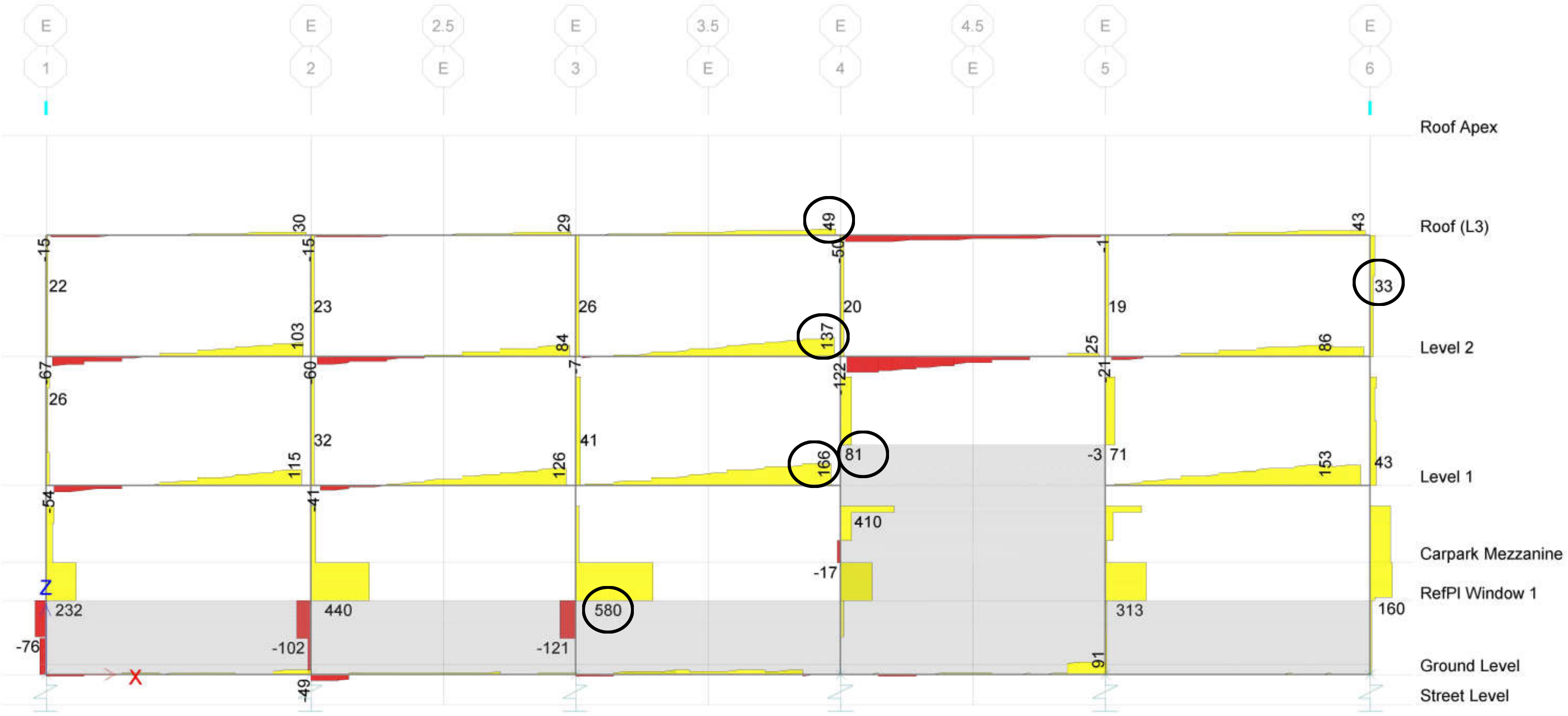
Page 02b.ii


Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
-221
-296
-297
26
46
120
220
260
123

Page 02b.iii

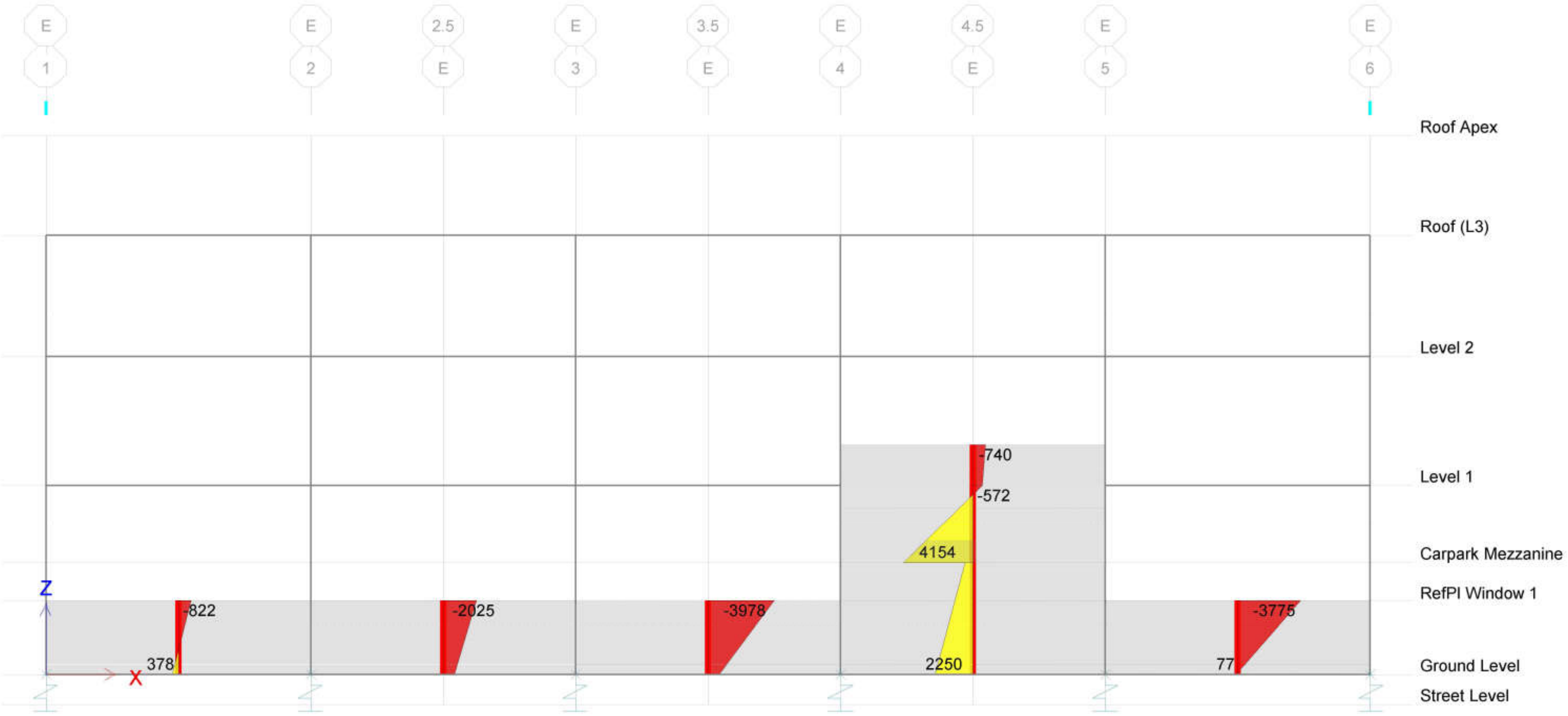

Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
550
-525
-467
-184
241
433
170
256
276

Page 02b.iv



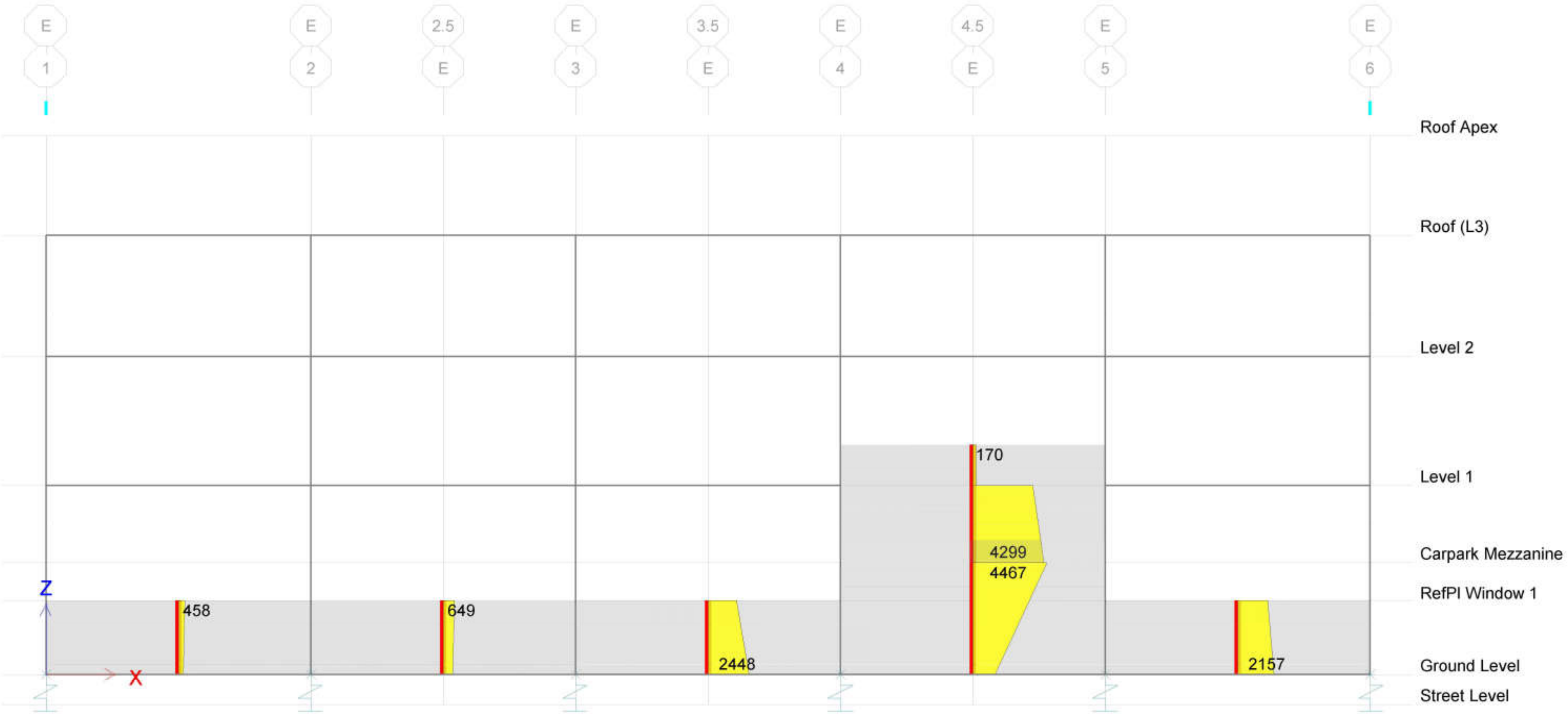
E 1
E 2
2.5 E
E 3
3.5 E
E 4
4.5 E
E 5
E 6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
30
29
49
43
-15
-15
-50
-1
22
23
26
20
19
33
103
84
137
25
86
-67
60
-7
-122
21
26
32
41
115
126
166
81
-3
71
153
43
-54
-41
410
-17
232
440
580
313
160
-76
-102
-121
91
-49
Z
X

Page 02b.v

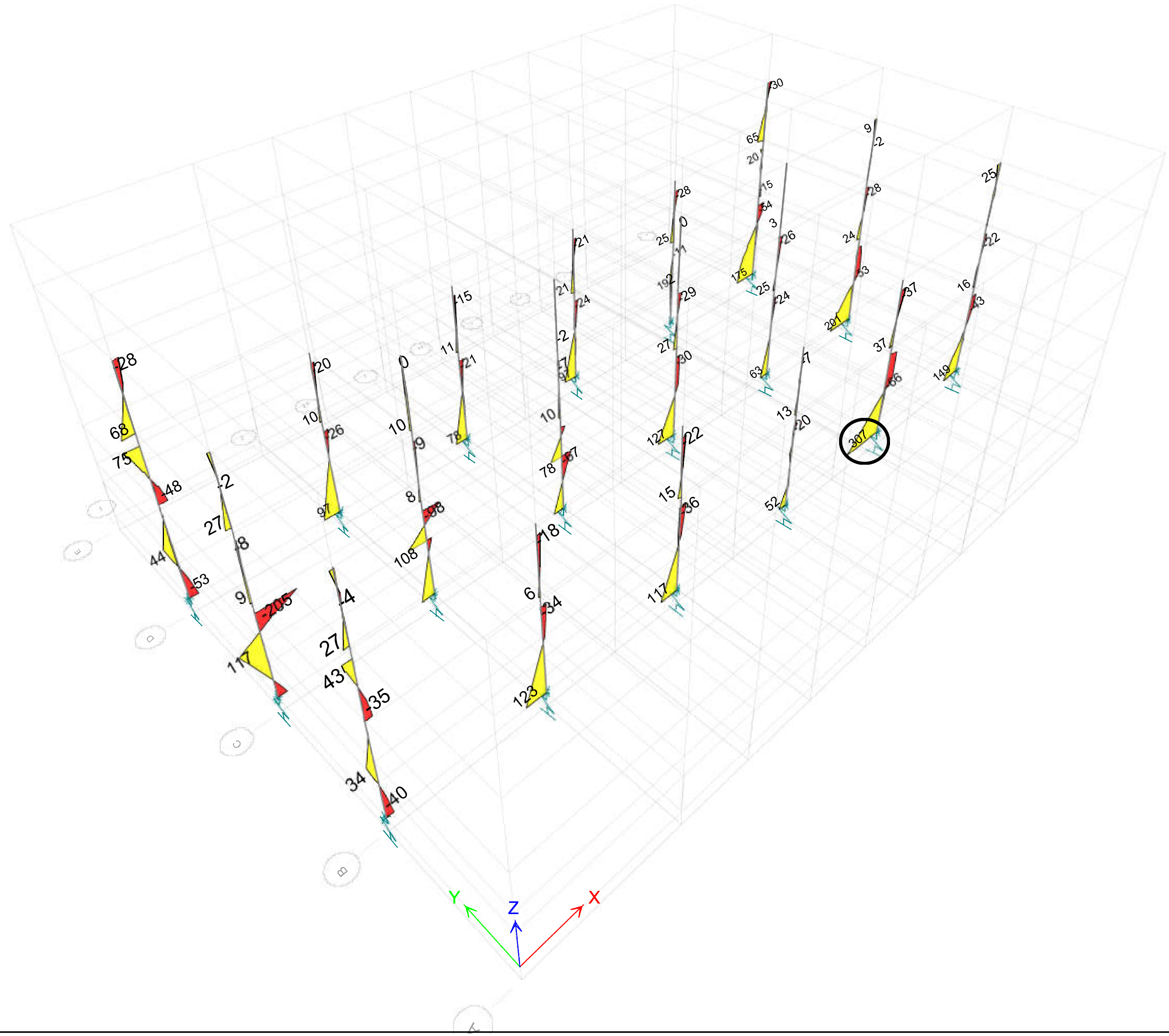


Post Strengthening - Pre hinging.EDB Elevation View - E Moment 3-3 Diagram (Seismic Combo X) [kN-m]

Page 02b.vi

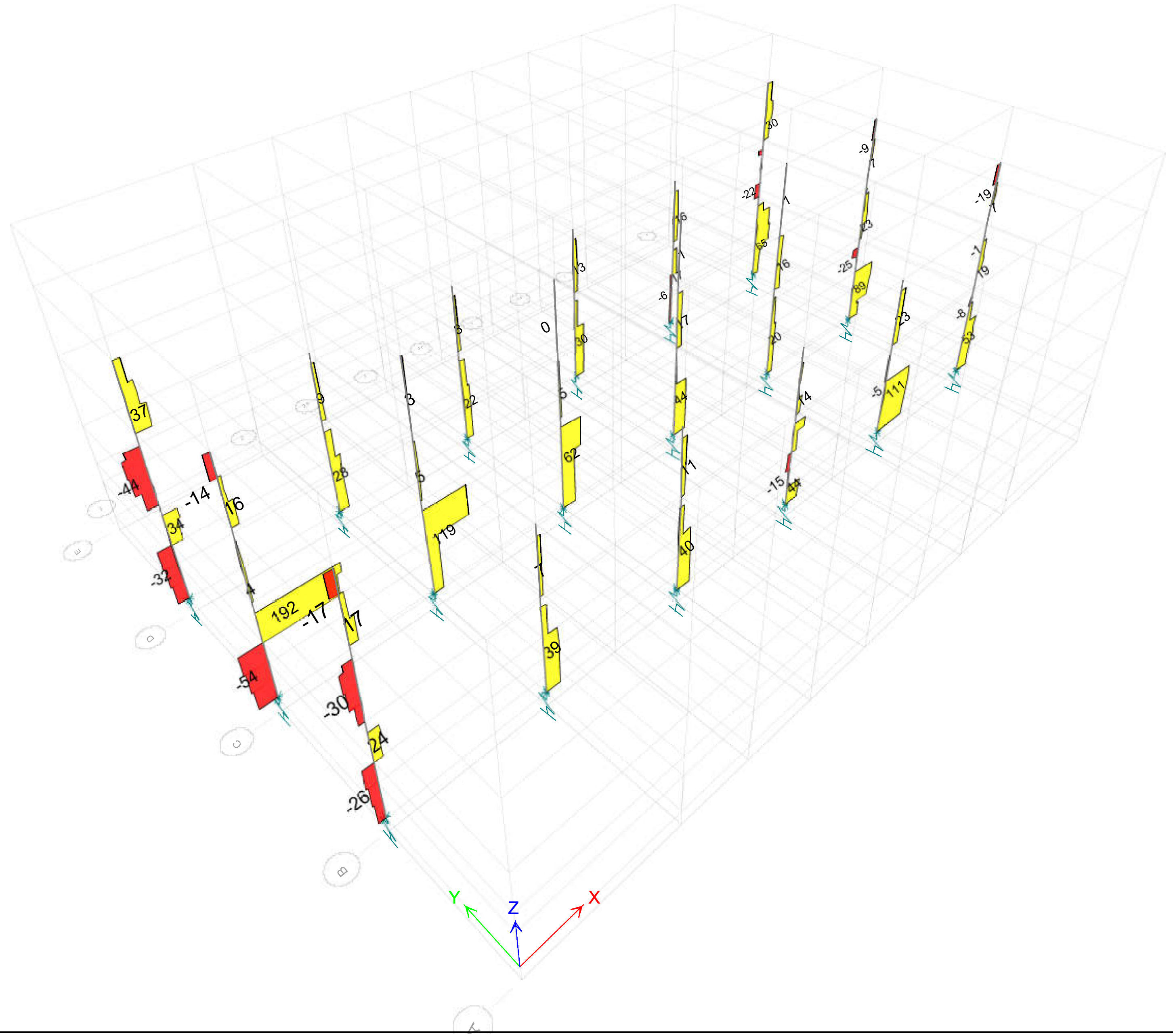

E
1
E
2
2.5
E
E
3
3.5
E
E
4
4.5
E
E
5
E
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
170
4299
4467
458
649
2448
2157
Z
X

Page 02b.vii

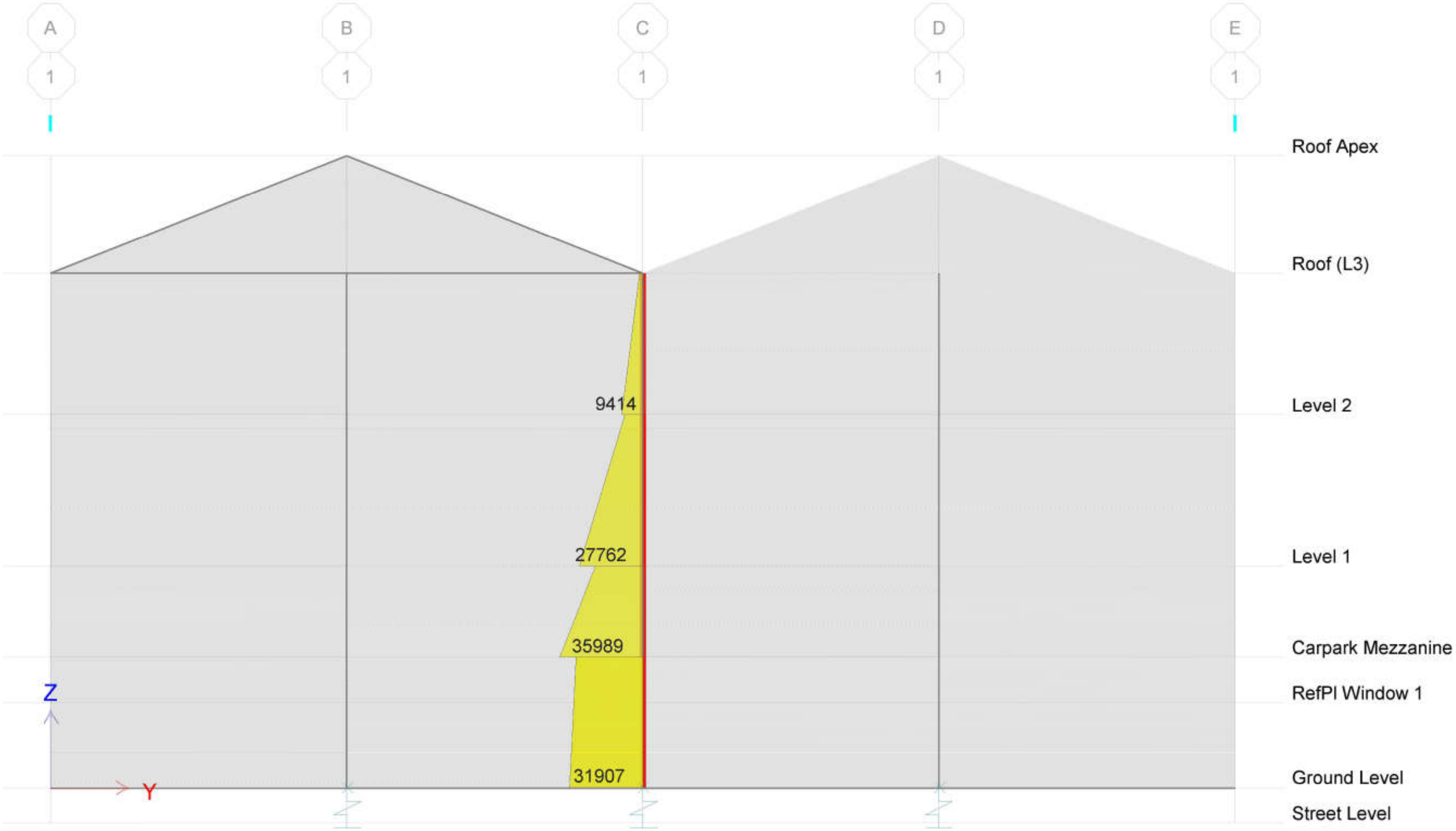


Post Strengthening - Pre hinging.EDB   3-D View   Moment 3-3 Diagram   (Seismic Combo X) [kN-m]

Page 02b.viii



Page 02b.ix


A
B
C
D
E
1
1
1
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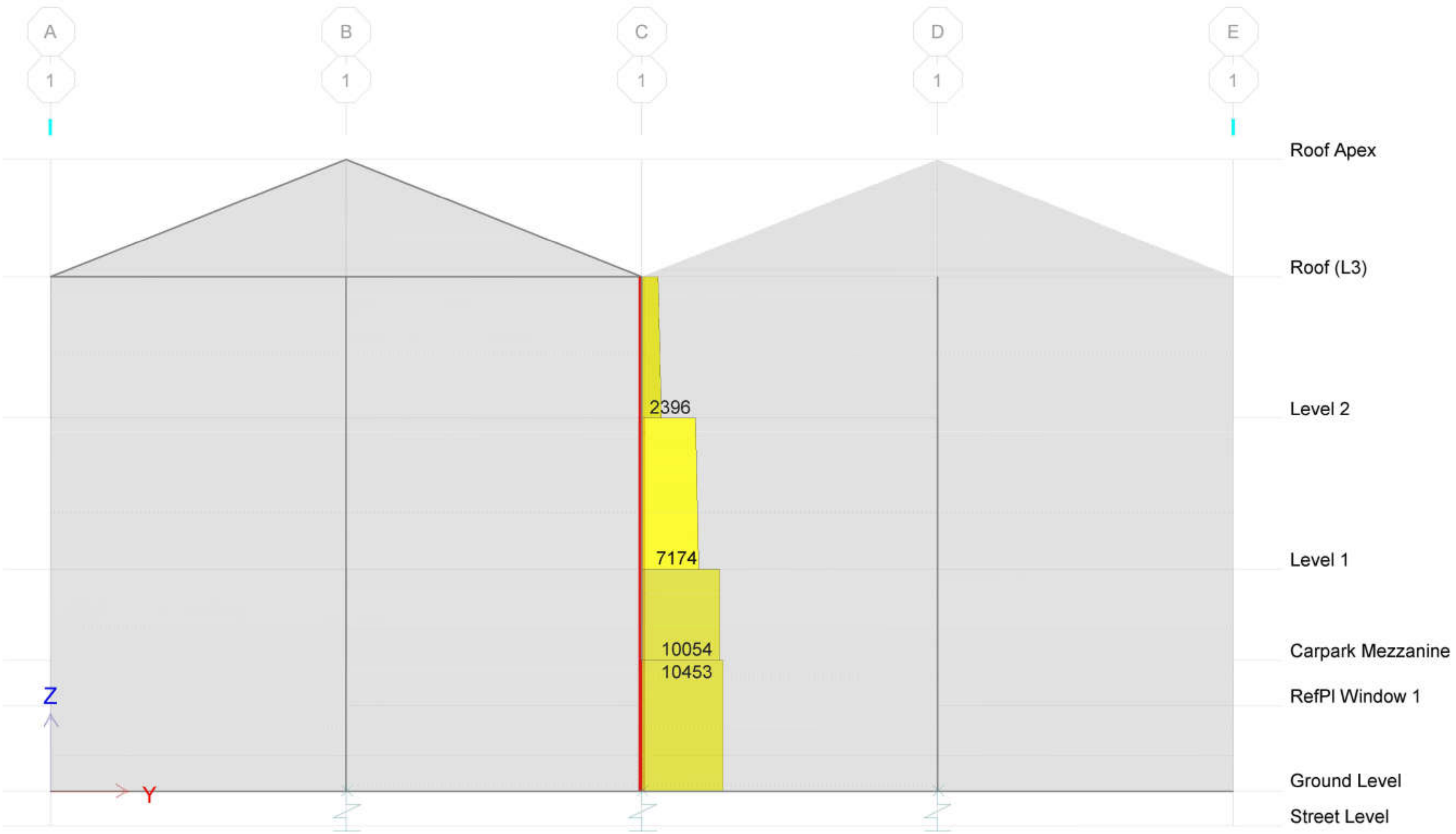
1
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
9414
27762
35989
31907
Z
Y

Page 02b.x


A
B
C
D
E
1
1
1
1
1
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
RefPl Window 1
Ground Level
Street Level
2396
7174
10054
10453
Z
Y

Story Response - Maximum Story Displacement

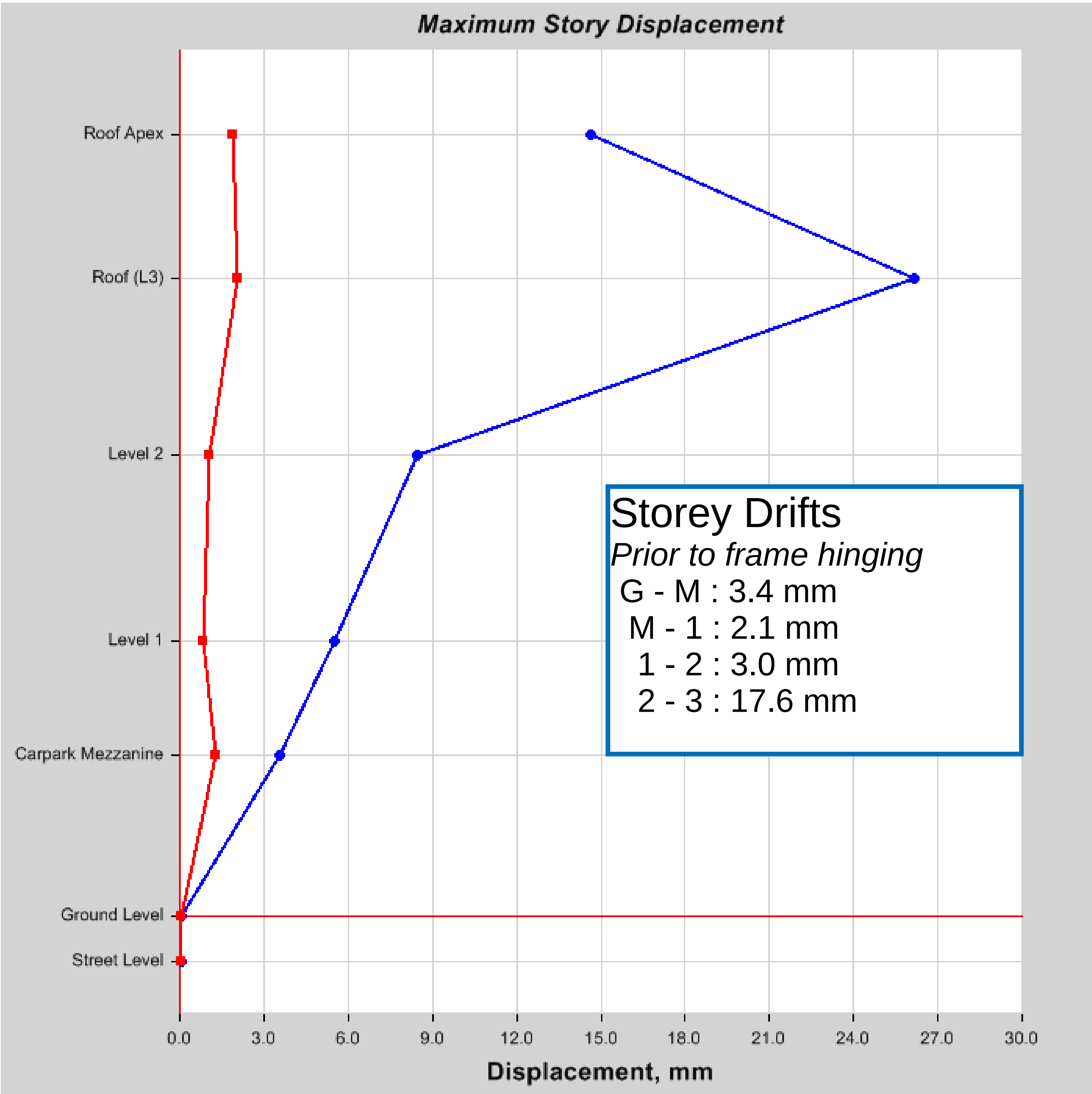
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp8		
Display Type	Max story displ	Story Range	All Stories
Load Combo	Seismic Combo X	Top Story	Roof Apex
Output Type	Max	Bottom Story	Street Level

Plot



Story Response - Maximum Story Displacement

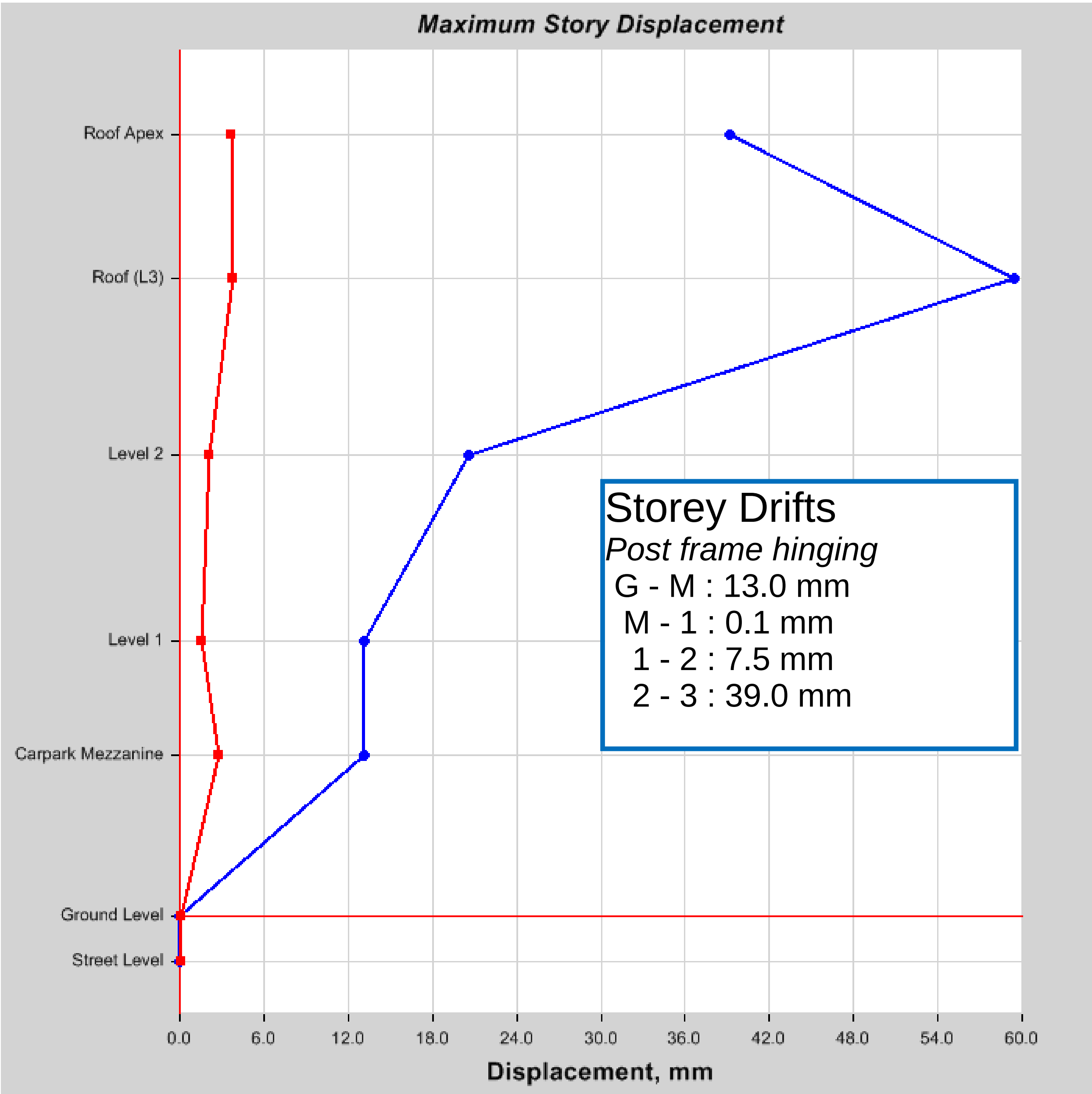
Summary Description

This is story response output for a specified range of stories and a selected load case or load combination.

Input Data

Name	StoryResp9		
Display Type	Max story displ	Story Range	All Stories
Load Combo	Ductility 1.0 Combo X	Top Story	Roof Apex
Output Type	Not Applicable	Bottom Story	Street Level

Plot



INTERSTOREY DRIFT CHECKS - STRENGTHENED STATE

Seismic Assessment of Existing Buildings, Technical Proposal to Revise C5, 30/11/2018

Refer DSA Review for derivation of drift limits.
Refer ETABS displacement charts for drift values.

1.0 - Prior to frame hinging

Displacements (refer Etabs charts, Page 05a.i)

$\Delta^*_0 := 3.4\text{mm}$ $\qquad \frac{\Delta_0}{\Delta^*_0} = 728 \cdot \%$

$\Delta^*_{0.5} := 2.1\text{mm}$ $\qquad \frac{\Delta_{0.5}}{\Delta^*_{0.5}} = 173 \cdot \%$

$\Delta^*_1 := 3.0\text{mm}$ $\qquad \frac{\Delta_1}{\Delta^*_1} = 649 \cdot \%$

$\Delta^*_2 := 17.6\text{mm}$ $\qquad \frac{\Delta_2}{\Delta^*_2} = 161 \cdot \%$

2.0 - Post frame hinging

Displacements (refer Etabs charts, Page 05a.ii)

$\Delta^*_0 := 13\text{mm}$ $\qquad \frac{\Delta_0}{\Delta^*_0} = 190 \cdot \%$

$\Delta^*_{0.5} := 0.1\text{mm}$ $\qquad \frac{\Delta_{0.5}}{\Delta^*_{0.5}} = 3624 \cdot \%$

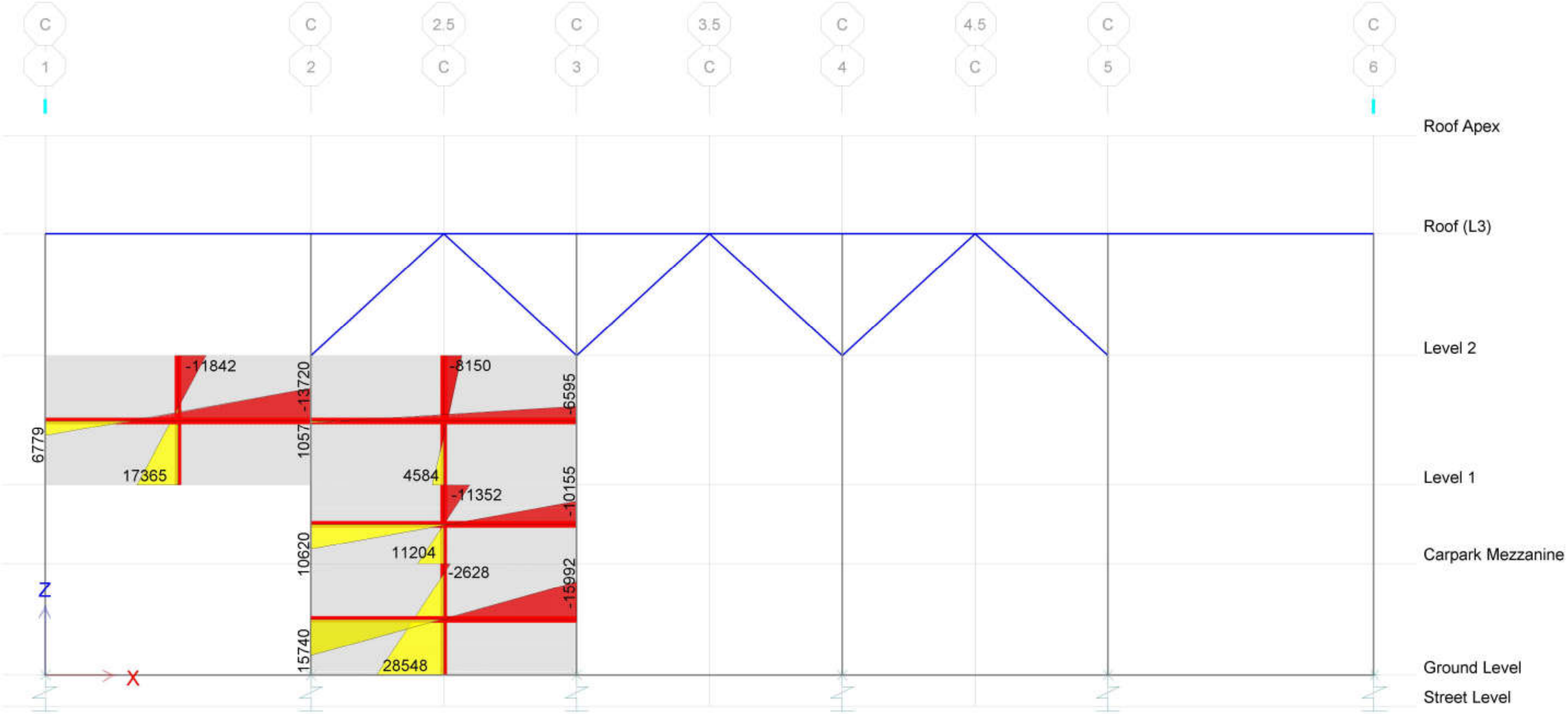
$\Delta^*_1 := 7.5\text{mm}$ $\qquad \frac{\Delta_1}{\Delta^*_1} = 260 \cdot \%$

$\Delta^*_2 := 39\text{mm}$ $\qquad \frac{\Delta_2}{\Delta^*_2} = 73 \cdot \%$

3.0 - Conclusions

Frame plastic hinging is expected at ~51% ULS shaking (refer prior calc's), at which point load is transfered to new elements.
Columns are expected to exceed load bearing capacity at a minimum of 73% ULS shaking.
Final rating of structure after proposed strengthening is 73%NBS.

Page 02d.i



C
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C
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4.5
C
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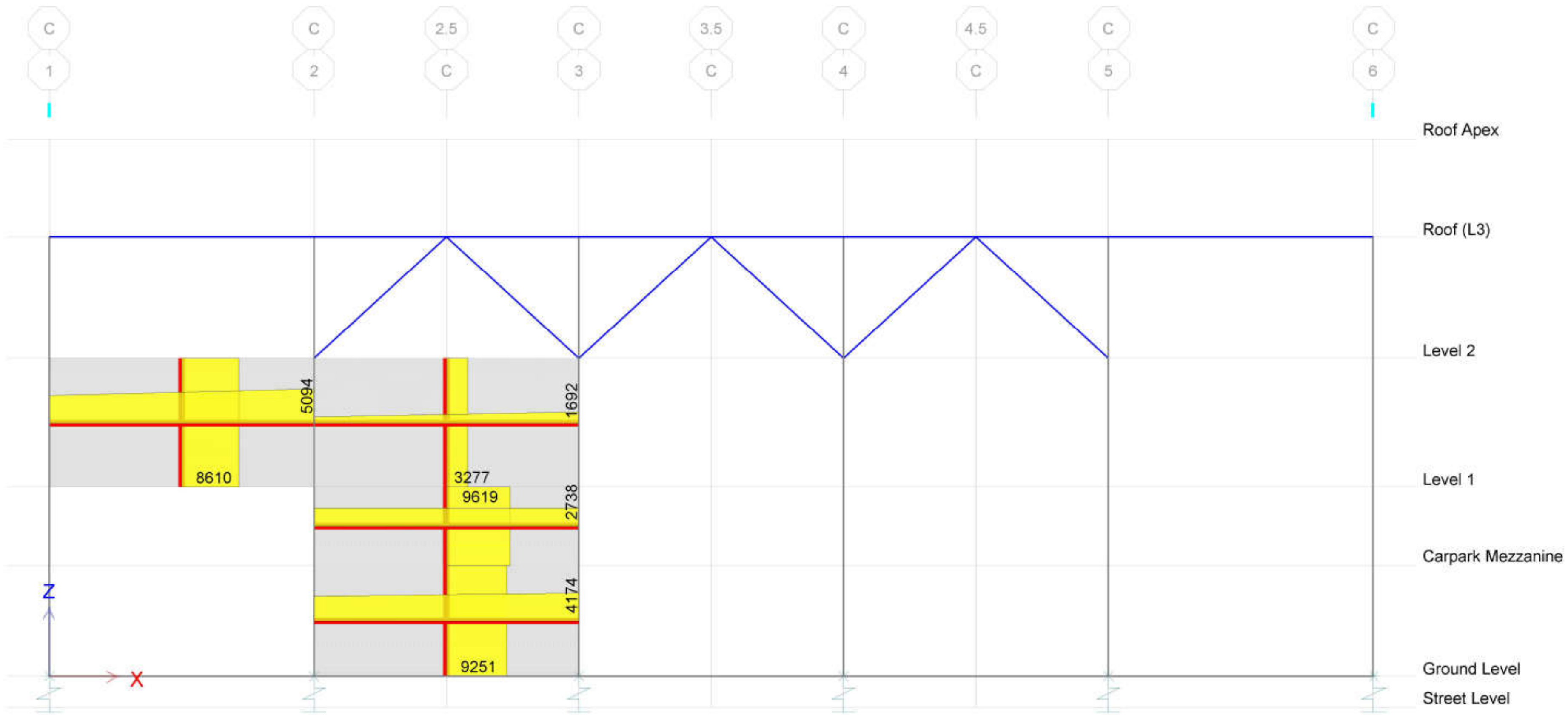
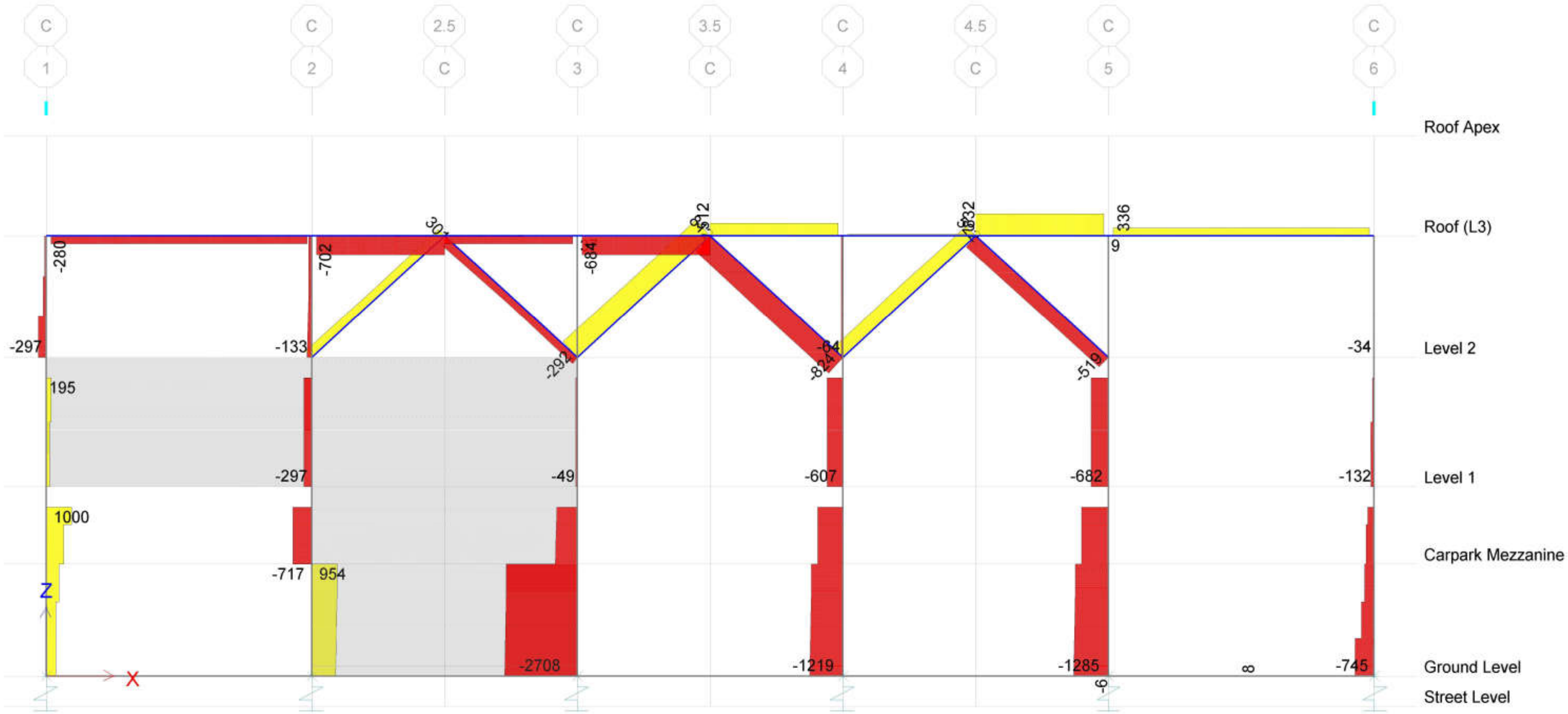
5
C
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
-11842
-13720
-8150
-6595
6779
1057
17365
4584
-11352
-10155
10620
11204
-2628
-15992
15740
28548
Z
X

Page 02d.ii



Post Strengthening - Post hinging.EDB Elevation View - C Shear Force 2-2 Diagram (Ductility 1.25 Combo X) [kN]

Page 02d.iii


C
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C
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3.5
C
C
4
4.5
C
C
5
C
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
-280
-702
301
-684
512
336
9
-297
-133
-292
-64
-824
-519
-34
195
-297
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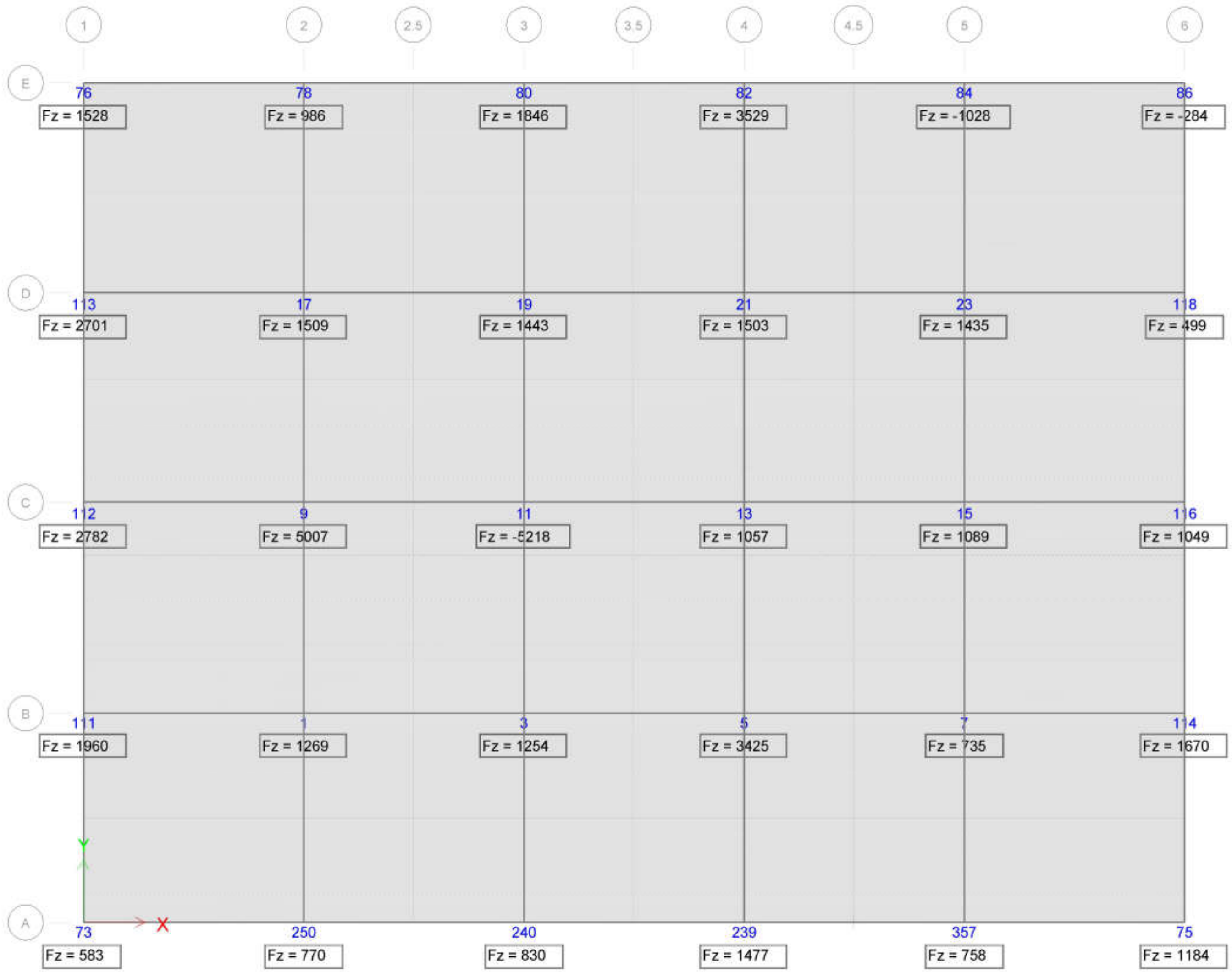
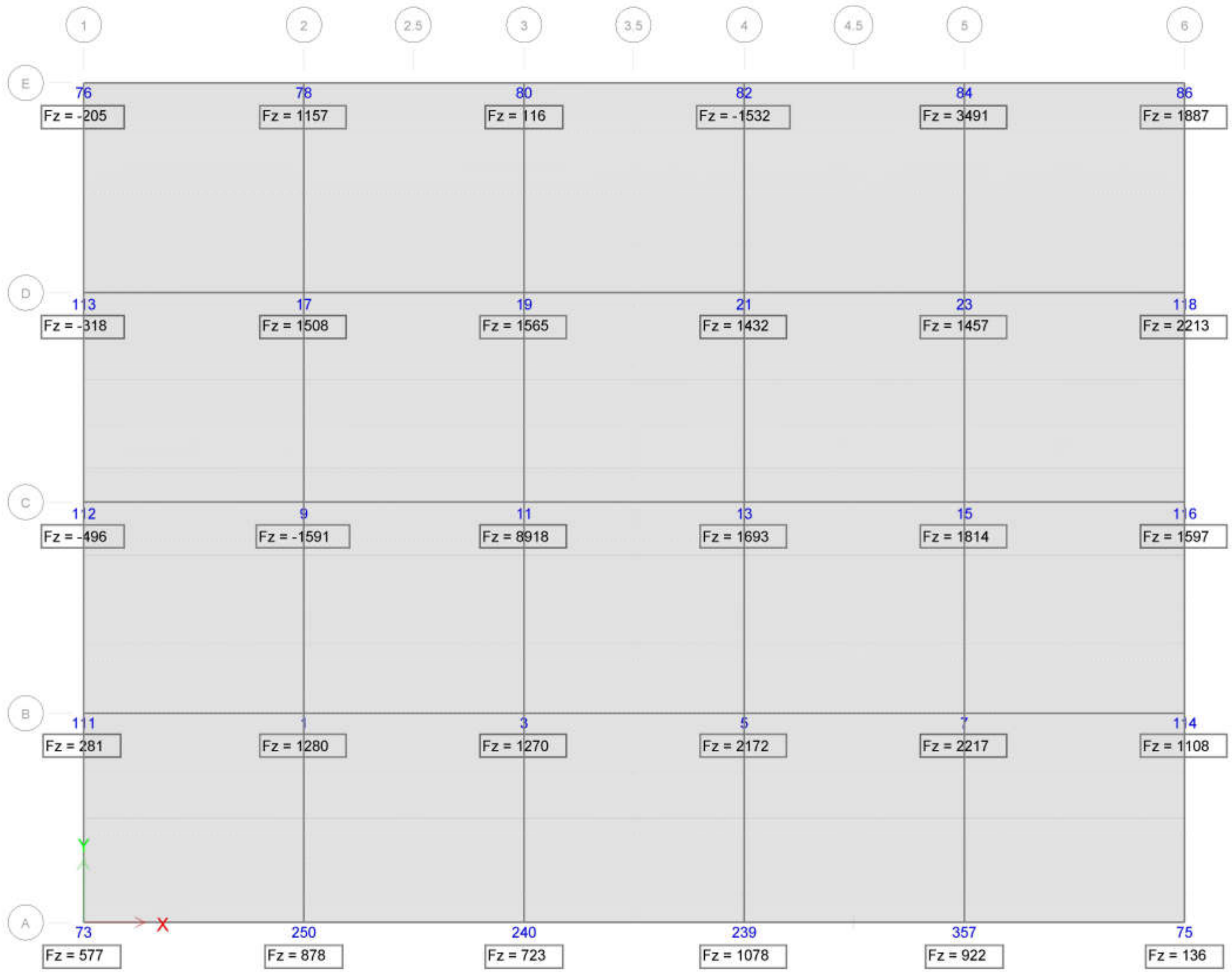
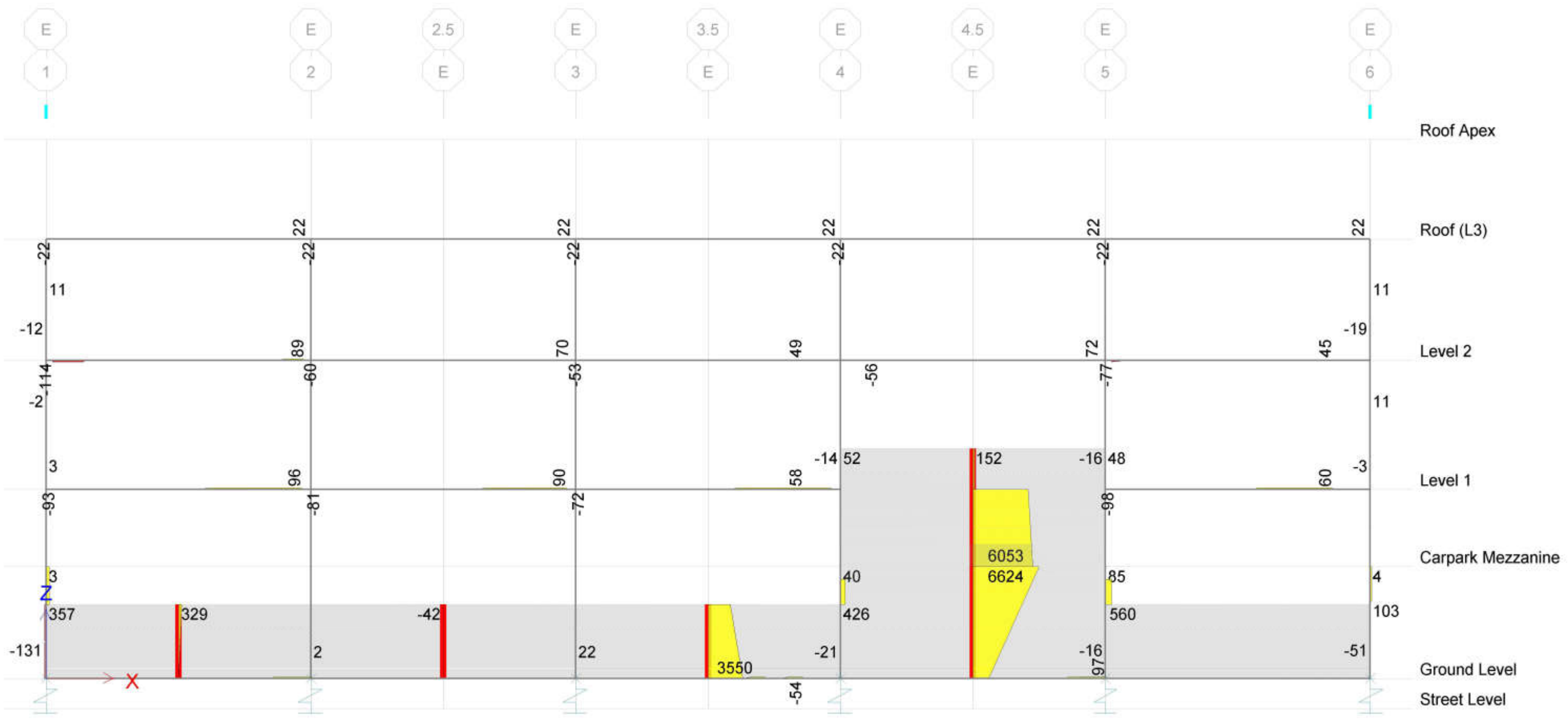
-6
8
-745
Z
X



Post Strengthening - Post hinging.EDB Plan View - Ground Level - Z = 0 (m) Restraint Reactions (Ductility 1.25 Combo X Reverse) [kN, kN-m]



Post Strengthening - Post hinging.EDB Plan View - Ground Level - Z = 0 (m) Restraint Reactions (Ductility 1.25 Combo X) [kN, kN-m]


E
1
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2
2.5
E
E
3
3.5
E
E
4
4.5
E
E
5
E
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
22
-22
11
-12
-114
-2
3
-93
89
-60
96
-81
70
-53
90
-72
49
58
-56
-14
52
152
-16
48
72
-77
-98
45
-19
11
-3
60
6053
6624
3
357
329
-131
2
-42
22
3550
-54
40
426
-21
85
560
-16
97
4
103
-51
Z
X

King & Dawson Ltd architects & engineers

JOB: 210-220 Thorndon Q.	JOB No.	DESIGNED. JBS	PAGE No. 10b
	DATE. 1/6/10	CHECKED.	

Excerpt from 2010 Strengthening calculations, shear strength of new sprayed RC shear wall on Grid E

shear reinforcing

try H16-100

$$V_s = \frac{(201)(500)}{(150)(100)} = 6.7\,MPa$$

this is greater than V_{max}

$$V_{n(max)} = 0.2(30) = 6.0\,MPa$$

try 200 thick

try 2 layers H16-200

$$V_s = \frac{(2\times 201)(500)}{(200)(200)} = 5.03\,MPa.$$

$$\phi V_{n(spray)} = 0.75(\overset{V_c}{1.48} + \overset{V_s}{5.03})(200)(0.8)(7600)$$

$$= 5930\,kN.$$

$$\%NBS = \frac{1800 + 5930}{15,100} = \boxed{51\%NBS}$$

use H16-200
2 layers.

flexure

check H16-300.

only account for 50% of tension capacity of column due to ties

refer to 10c.

$$\%NBS = \frac{18,600}{33,500} = \boxed{56\%NBS}$$

Revised rating

V* = 6624 kN (Shear wall only)

φVn / V* = 5930 / 6624 = 90%NBS

Element OK.

SHEAR WALL DESIGN SUMMARY

Seismic Assessment of Existing Buildings, C5, July 2017 (incl. March 2018 Errata)

1.0 - Ground floor

1.1 - Design Loads

Taking design forcess from ductility 1.25 load combination.

Etabs calculated design shear (horizontal) Includes demand on adjacent columns — $V^*_{etabs} := 9251\mathrm{kN} + 50\mathrm{kN} + 65\mathrm{kN}$

Shear capacity of adjacent columns — $Vn_{col} := 837\mathrm{kN}$

Final design shear — $V^*_G := V^*_{etabs} - 2\cdot Vn_{col} = 7692\cdot\mathrm{kN}$

Design moment — $M^*_G := 28548\mathrm{kN}$

1.2 - Design Summary

Wall thickness	300mm
Vertical steel	2 layers of D24's at 250 centres
Horizontal steel	2 layers of HD20's at 200 centres

2.0 - Carpark Mezzanine

2.1 - Design Loads

Taking design forcess from ductility 1.25 load combination.

Etabs calculated design shear (horizontal) Includes demand on adjacent columns — $V^*_{etabs} := 9616\mathrm{kN} + 185\mathrm{kN} + 97\mathrm{kN}$

Shear capacity of adjacent columns — $Vn_{col} := 837\mathrm{kN}$

Final design shear — $V^*_M := V^*_{etabs} - 2\cdot Vn_{col} = 8224\cdot\mathrm{kN}$

Design moment — $M^*_M := 11352\mathrm{kN}$

2.2 - Design Summary

Wall thickness	300mm
Vertical steel	2 layers of D20's at 300 centres
Horizontal steel	2 layers of HD20's at 200 centres

3.0 - Level 1

3.1 - Design Loads

Taking design forcess from ductility 1.25 load combination.

Etabs calculated design shear (horizontal) Includes demand on adjacent columns — $V^*_{etabs} := 8610\mathrm{kN} + 6\mathrm{kN} + 7\mathrm{kN}$

Shear capacity of adjacent columns — $Vn_{col} := 375\mathrm{kN}$

Final design shear — $V^*_M := V^*_{etabs} - 1.5\cdot Vn_{col} = 8061\cdot\mathrm{kN}$

Design moment — $M^*_M := 17365\mathrm{kN}$

3.2 - Design Summary

Wall thickness	300mm
Vertical steel	2 layers of D20's at 300 centres
Horizontal steel	2 layers of HD20's at 200 centres

CONCENTRICALLY BRACED FRAMES

NZS3404:1997 Steel Structures

Preliminary sizing of frame members used for modelling purposes is 460UB74

Element length (centreline) $L := 5236\mathrm{mm}$

Design axial load $N^* := 824\mathrm{kN}$

Section compressive capacity $\phi N_s := 2437\mathrm{kN}$

Effective length factors $k_t := 1.0$

$k_l := 1.0$

$k_r := 1.0$

$L_e := L \cdot k_t \cdot k_l \cdot k_r = 5.236\,\mathrm{m}$

Slenderness reduction factor (x axis buckling) $\alpha_c := 0.928$ Conservative rounding

Member compressive capacity $\phi N_c := \phi N_s \cdot \alpha_c = 2262\,\mathrm{kN}$

Slenderness reduction factor (y axis buckling) $\alpha_c := 0.359$

Member compressive capacity $\phi N_c := \phi N_s \cdot \alpha_c = 875\,\mathrm{kN}$ Interpolation

460UB57 OK in compression.
Tesnion and flexural demand OK by inspection.

GABLE END CONNECTIONS

1.0 - Design Load

Design load on connection (refer DSA review calcs) $F_\omega := 5.5\frac{kN}{m}$

Strength provided by existing connection $V_n := 62\%$

Design load to be taken by supplementary fixings $V* := F_\omega \cdot V_n = 3.41 \cdot \frac{kN}{m}$

2.0 - New Fixings

Assuming new epoxy bolts fixing concrete to stringer, new brackets fixing stringer to purlins. Taking M12 Bolts and CPC80's. Refer excel calc for element strengths.

M12 epoxy bolt shear strength $Vn_{chemset} := 15.1kN$

Lateral capacity of CPC80 nailed connection $Vn_{nails} := 4.1kN$

Lateral capacity of CPC80 screwed connection: $\phi := 0.7$

$n := 4$

$k := 1 \cdot 1 \cdot 1 \cdot 1.25 \cdot 1$

$Q_k := 2.663kN$

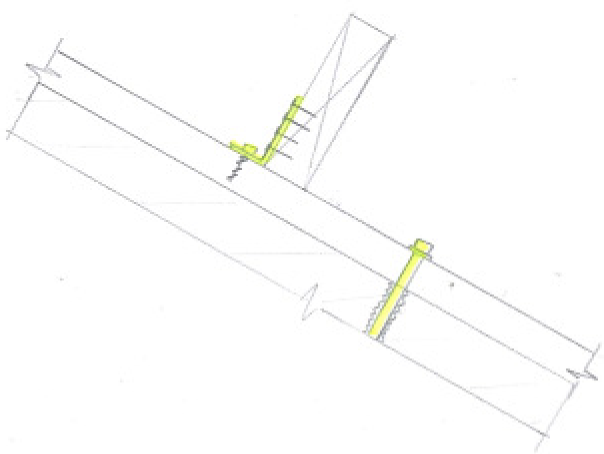
$Vn_{screws} := \phi \cdot n \cdot k \cdot Q_k = 9.32 \cdot kN$

Maximum required bolt spacing $s_{chemset} := \frac{Vn_{chemset}}{V*} = 4.43\,m$

Round down to 2.40m to ensure load distribution.

Maximum required bracket spacing $s_{chemset} := \frac{Vn_{nails}}{V*} = 1.20\,m$

Purlin spacing actual $s_{purlin} := 950mm$ OK.



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Nail Design: Lateral Loading of Joint

CPC80 nailed connection

V^* = 0 kN
L = 1 m

q^* = V^* / L
= 0.0 kN/m

ϕQ_n = $\phi n k Q_k$

ϕ = 0.8
n = 8
k = 1.03
Q_k = 0.631 kN

ϕQ_n = 4.1 kN **OK**

Timber Type = J5
Nail Diameter = 3.15 mm

Modification Factors

Dry/Green =	Dry		k_a =	1.00
Load Duration (k_1) =	Short		k_b =	1.00
Nails End Grain =	No		k_c =	1.00
Nails in Double Shear =	No		k_d =	1.00
Flat Head Nail into Plywood =	No		k_e =	1.00
Penetration 2nd member p =	45	mm	k_{f1} =	1.00
Thickness 1st member t_1 =	45	mm	k_{f2} =	1.00
First Member Type =	Timber		k_f =	1.00
Total Number Nails =	8		k_g =	1.03

$k = k_a\, k_b\, k_c\, k_d\, k_e\, k_f\, k_g$ = 1.03

Nail Design Summary

Diameter = 3.15 mm
Minimum Length = 90 mm
Spacing = 125 mm

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		DATE: 20/06/2019	CHECKED: JRS	

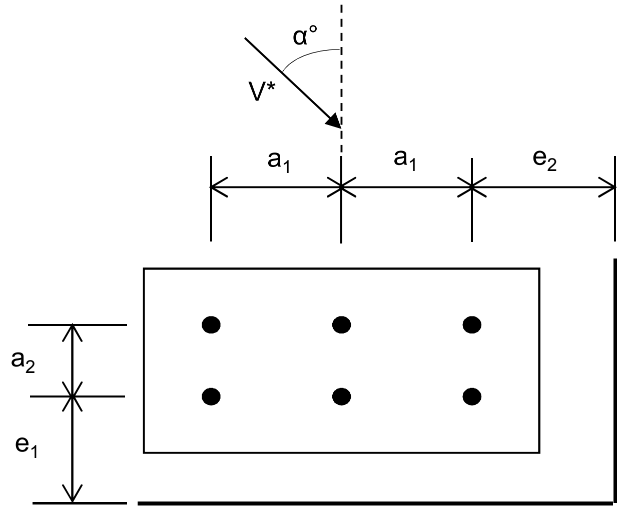
Ramset Chemical Anchoring: Chemset Injection 100 Series

CPC screwed connection

Calculations and data based on Ramset Specifiers Chemical Anchoring Resource Book (3.1of 4)

Anchor Details

Anchor Type:		Threaded Rod Grade 4.6		
Anchor Size:	d_b =	12	mm	
Effective Depth:	h =	150	mm	OK
Edge Distance 1:	e_1 =	400	mm	OK
Edge Distance 2:	e_2 =	75	mm	OK
Concrete Strength:	f_c' =	30	MPa	
Spacing 1:	a_1 =	400	mm	OK
Spacing 2:	a_2 =	400	mm	OK
Number Anchors:	n =	1		
Shear Force Load Direction:	α =	90	°	
Anchor Confined Inside Reinforcing Cage:		No		
Anchor Location in Row:		End		



Design Loading

Tension Load per Anchor:	0.0	kN
Shear Load per Anchor:	0.0	kN

Tension Capacity

$ØN_{uc}$ = 19.7 kN

X_{nc} = 0.98

X_{ne} = 1.00

X_{nae} = 1.00

X_{nai} = 1.00

$ØN_{urc} = ØN_{uc}\, X_{nc}\, X_{ne}\, (X_{nae} \text{ or } X_{nai})$
= 19.3 kN

$ØN_{us}$ = 27.0 kN

$ØN_{ur} = \text{MIN}[ØN_{urc}, ØN_{us}]$
= 19.3 kN

$N^* / ØN_{ur}$ = 0.00 N/A

Shear Capacity

$ØV_{uc}$ = 80.5 kN

d_h / h = 10.7 OK

X_{vc} = 0.97

X_{vd} = 2.00

X_{va} = 0.70

X_{vs} = 0.41

X_{vn} = 1.00

$ØV_{urc} = ØV_{uc}\, X_{vc}\, X_{vd}\, X_{va}\, X_{vs}\, X_{vn}$
= 44.1 kN

$ØV_{us}$ = 15.1 kN

$ØV_{ur} = \text{MIN}[ØV_{urc}, ØV_{us}]$
= 15.1 kN

$V^* / ØV_{ur}$ = 0.00 N/A

Combined Loading Check

Check: $N^* / ØN_{ur} + V^* / ØV_{ur} <=$ 1.20

$N^* / ØN_{ur} + V^* / ØV_{ur}$ = 0.00 + 0.00 = 0.00 OK