Engineering Assessment Summary Report

210-220 Thorndon Quay

DRAFT

1. **Building Information**	
Building Name/ Description	
Street Address	220 Thorndon Quay, Pipitea, Wellington 6011
Territorial Authority	Wellington City Council
No. of Storeys	4 (ground G, carpark mezzanine M, office level 1, apartment level 2)
Area of Typical Floor (approx.)	1200m^2
Year of Design (approx.)	1961
NZ Standards designed to	Unknown.
Structural System including Foundations	Gravity: Concrete slabs supported on RC columns w large corbels (no beams) North/south: RC perimeter frames of columns and spandrels. East/west: Full height RC shear walls. Foundations: RC ground beams spanning between large pile caps, RC piles driven to unknown depth.
Does the building comprise a shared structural form or shares structural elements with any other adjacent titles?	No, however seismic gap between neighbouring buildings is negligible.
Key features of ground profile and identified geohazards	Flat site on original ground (not reclaimed).
Previous strengthening and/ or significant alteration	Significant alteration: 1997. New carpark mezzanine between Ground and L1. New apartment mezzanines at L3. PS4 issued 2002. Strengthening: 2010. In response to 2007 EQP notice. Sprayed RC shear wall in bay of rear perimeter wall.
Heritage Issues/ Status	None.
Other Relevant Information	Precast 150 Hollowcore units used for majority or 1997 carpark mezzanine. Units provided with 50mm of seating on steel UB sections. (Seating value taken from plans – no invasive testing carried out)

2. Assessment Information	
Consulting Practice	King & Dawson Ltd Architects & Engineers
CPEng Responsible, including: • Name • CPEng number • A statement of suitable skills and experience in the seismic assessment of existing buildings[1]	Jared Sullivan, CPEng #209924 Over 10 years of experience compiling DSA assessments and over 20 years practicing as a structural engineer. Myself or my staff have attended all relevant seminars and conferences on EQ assessments and the NZSEE guidelines.
Documentation reviewed, including: • date/ version of drawings/ calculations[2] • previous seismic assessments	1961 Original drawings: *Paper House, Frank Kerslake Consulting Engineer* 1997 Alteration Docs: *New apartments at 210-220 Thorndon Quay for Globe Holdings, Dunning Thornton* [Calculations, details, specifications]. 2010 Strengthening Docs: *Strengthening at 210-220 Thorndon Quay, King & Dawson*
Geotechnical Report(s)	2010 bore logs carried out across road; *Geotechnical Investigation, Bank Stability, 231 Thorndon Quay, Wellington – Abuild consulting Engineers.*
Date(s) Building Inspected and extent of inspection	April 2019, May 2019. Site walk-throughs to confirm information provided by drawings; scanning for rebar in spandrels (confirmed more bars than previously thought), measure of frame elements (confirmed dimensions scaled from original drawings), looking for longitudinal beams in carpark mezzanine slab (none present).
Description of any structural testing undertaken and results summary	None.
Previous Assessment Reports	2007 IEP, resulted in Potentially Earthquake Prone Notice. 2009 DSA, found structure to be 11%NBS. 2010 Strengthening took structure to 44%NBS.
Other Relevant Information	

[1] This should include reference to the engineer's Practice Field being in Structural Engineering, and commentary on experience in seismic assessment and recent relevant training

[2] Or justification of assumptions if no drawings were able to be obtained

3. Summary of Engineering Assessment Methodology and Key Parameters Used	
Occupancy Type(s) and Importance Level	IL2 Structure. Split occupancy: Ground level: commercial Mezzanine: carpark L1: office space L2: residential
Site Subsoil Class	D (deep soil)
For an ISA:	n/a
For a DSA:	
Summary of how Part C was applied, including: • the analysis methodology(s) used from C2 • other sections of Part C applied	Elastic force-based analysis was carried out to determine basic performance and identify yielding elements. This showed that the structure performs well under east/west loading, however frame elements under north/south loading are expected to yield and soften. Simplified pseudo-nonlinear static pushover analysis was carried out to determine effectiveness of secondary load paths. Note that rather that hinge frame elements one by one to determine hierarchy of mechanisms, all frame elements were softened simultaneously to assess worst case scenario. This showed that columns are expected to exceed their curvature capacity at 33% of ULS loading, after frames have fully hinged.
Other Relevant Information	Due to the simplified nature of the pseudo nonlinear analysis, values found represent an absolute lower bound of expected performance. Also note that loss of gravity load capacity of columns under plastic deflections was conservatively assumed rather than calculated, as this assessment is a precursor to a strengthening scheme where this mechanism is to be avoided entirely.

<table>
<tr><th colspan="3">4. Assessment Outcomes</th></tr>
<tr><td>Assessment Status
(Draft or Final)</td><td colspan="2">Final</td></tr>
<tr><td>Assessed %NBS Rating</td><td colspan="2">20 to 30%</td></tr>
<tr><td>Seismic Grade and Relative Risk (from Table A3.1)</td><td colspan="2">Grade D; high risk.</td></tr>
<tr><td><u>For an ISA:</u></td><td colspan="2">n/a</td></tr>
<tr><td><u>For a DSA:</u></td><td colspan="2"></td></tr>
<tr><td>Comment on the nature of Secondary Structural and Non-structural elements/ parts identified and assessed</td><td colspan="2">Precast 150 hollow-core units used in 1997 carpark mezzanine. Adequate seating is provided, and storey deflection is not sufficient to induce significant additional bending moment in the units. These are assessed as acceptable.</td></tr>
<tr><td>Describe the Governing Critical Structural Weakness</td><td colspan="2">Governing CSW relates to the curvature capacity of RC columns being exceeded under plastic loading, likely causing a loss of axial capacity.</td></tr>
<tr><td>If the results of this DSA are being used for earthquake prone decision purposes, <u>and</u> elements rating <34<i>%NBS</i> have been identified (including Parts)[3]:</td><td>Engineering Statement of Structural Weaknesses and Location
Under n/s ULS loading, perimeter frames are expected to yield and hinge. Without a secondary load path, structure will deflect past the curvature capacity of the columns, potentially causing a loss of axial capacity.</td><td><i>Mode of Failure and Physical Consequence</i> Statement(s)
Consequence of columns losing axial capacity is a soft-storey forming and potential collapse of structure.</td></tr>
<tr><td>Recommendations
(optional for EPB purposes)</td><td colspan="2"></td></tr>
</table>

[3] If a building comprises a shared structural form or shares structural elements with other adjacent titles, information about the extent to which the low scoring elements affect, or do not affect the structure.

DRAFT

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King & Dawson Ltd architects & engineers ltd

13th June 2019

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

Seismic Strengthening Design Stage One: DSA Review

210-220 Thorndon Quay

1.0 Executive Summary

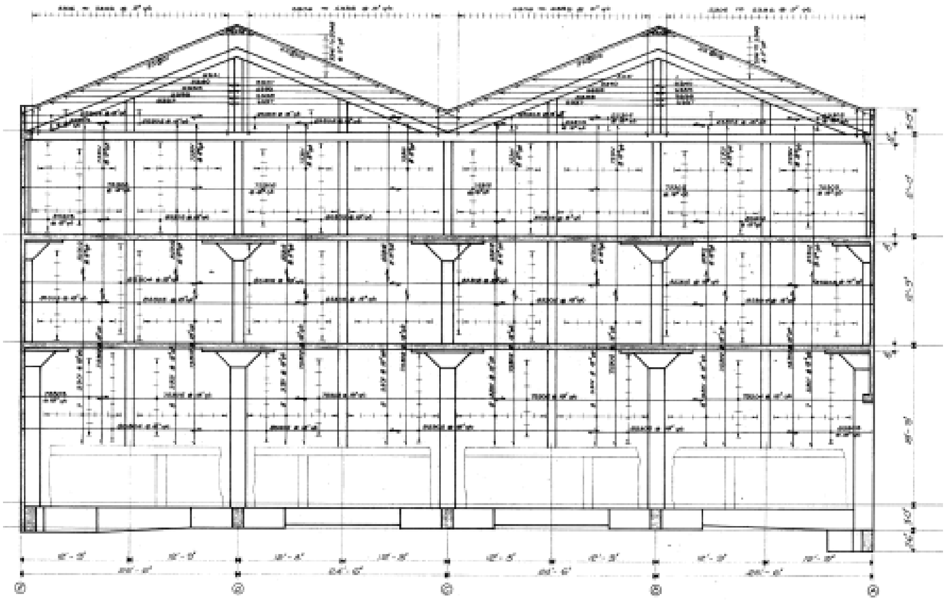
We (King & Dawson) have been engaged to complete a seismic strengthening design for the structure at the above address, to bring it above the earthquake risk threshold of 67% New Building Standard (%NBS), as defined by the *Seismic Assessment of Existing Buildings* (NZSEE et al, July 2017). In order to quantify this, the current rating of the structure needs to be confirmed. The calculations presented here have been carried out to the latest standards and show that the structure at the above address achieves 20-30%NBS.

Image: Google Maps 2019

2.0 Background

2.1 Description of Structure

The building is a 3-storey concrete structure originally constructed circa 1960, with commercial space on the ground floor, office space on the first storey and residential apartments on the second. Between 1997 and 2002, two major alterations have been made; the construction of a concrete mezzanine floor dividing the ground level space, for use as a carpark, and construction of a timber framed mezzanine dividing the top storey to increase floorspace in the apartments.



Above: Cross section showing south wall prior to alterations - 'Paper House' Frank Kerslake, 1961

2.2 Load Resisting Systems

The gravity load resisting system consists of reinforced concrete columns with large corbels framing into reinforced concrete slabs. No intermediate beams support the in-situ slabs.

The lateral load resisting system in the transverse (east/west) direction consists of full height reinforced concrete shear walls on the north and south perimeter faces.
In the longitudinal (north/south) direction, lateral loads are resisted by the gravity columns working with lightly reinforced concrete spandrels to form moment resisting frames along the front and rear walls. This system is supplemented by the sprayed concrete walls on the east perimeter face that were constructed as part of the 2010 strengthening.

2.3 Prior Assessments

In 2007, an assessment was carried out as per the Initial Evaluation Procedure (IEP), finding the building to be likely below 34%NBS, and the structure was issued with a Potentially Earthquake Building notice. In 2009 a DSA was carried out by us finding the building met 11%NBS. A strengthening system was designed and overseen by us in 2010 to bring the building up to 44%, consisting of sprayed concrete shear walls along the rear wall, at ground and mezzanine level.

Since 2010, the guidelines for assessing existing buildings have advanced considerably, culminating in Version 1 (July 2017) of the *Seismic Assessment of Existing Buildings*, including updates to Chapter 5 (C5: Concrete Buildings) in March 2018. In order to verify that the strengthening we have been engaged to design complies with current codes, the calculations presented here seek to review the 2009 DSA, and update critical aspects.

3.0 Assessment Parameters

3.1 General

Site subsoil category	D (deep soil)	
Seismic weight	26800 kN	
Hazard factor	$Z = 0.4$	
Return period factor	$R = 1.0$	
Near fault distance	$D = 2$ km	
Performance factor	$Sp = 0.7$	(except where $1.0 < \mu < 2.0$)
Ductility	$\mu = 2.0$	(north / south direction)
	$\mu = 1.25$	(east / west direction)
Building period	$T < 0.25s$	(both directions)
Seismic Coefficient	$C = 0.54$	(north / south direction)
	$C = 0.74$	(east / west direction)
Base shear	$V_{base} = 14300$ kN	(north / south direction)
	$V_{base} = 19700$ kN	(east / west direction)
Lateral load eccentricity	$e = \pm 10\%$	

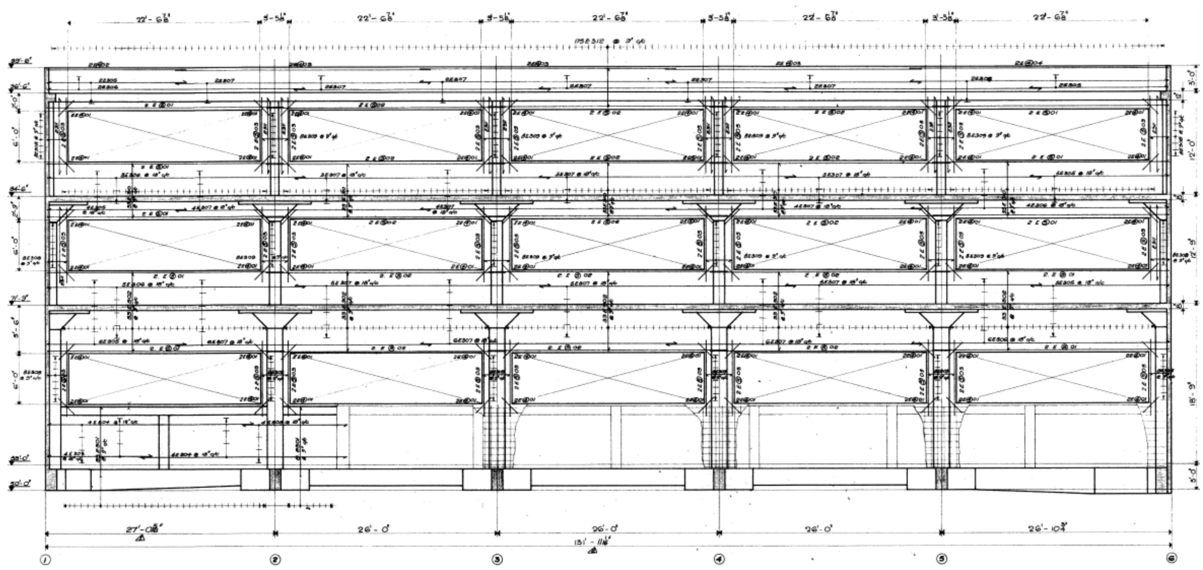
3.2 Ductility

Ductility for the frames resisting north/south actions was conservatively limited to 2.0. This is due to the expected behaviour of the lightly reinforced spandrels, and is consistent with prior assessments of the building. To calculate plastic deflections, ductility has been taken as 1.0.

Ductility for the shear walls resisting east/west actions has been limited to 1.25, as these walls are expected to behave in a nominally ductile manner.

3.3 Assumptions

For the purposes of this assessment it has been conservatively assumed under north/south loading when the perimeter frames (see below) start to exceed their calculated capacity, hinges form at the ends of all frame members. The purpose of this assumption was to check the gravity load resisting system under worst case conditions, rather than attempting to establish a particular hierarchy of frame member failure. The final NBS rating therefore represents an absolute lower bound.



Above: Cross section showing east wall prior to 2009 strengthening - 'Paper House' Frank Kerslake, 1961

4.0 Results

4.1 Prior Results

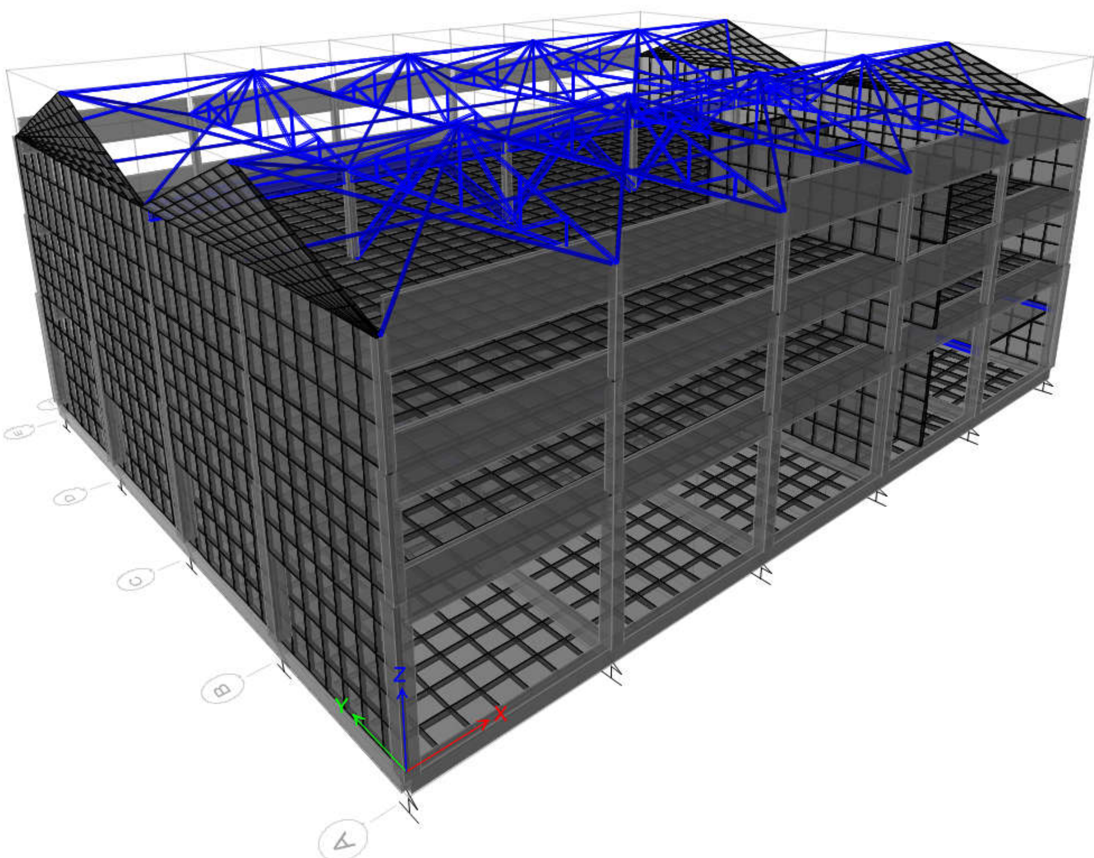
The 2009 strengthening left the building between 34 and 67%NBS, where the elements governing performance were the reinforced concrete spandrels in the front and rear walls, as well as several minor elements also not reaching the 67% threshold.

4.2 Updated Results

During the course of this assessment a more complete copy of the original drawings has been located showing more reinforcement in the perimeter spandrels than previously thought. In addition to this, the calculations presented here have updated the element strengths based on the latest guidelines, and updated the design actions based on an ETABS computer analysis. The ETABS model built for this analysis accounted for previous strengthening work, as well as the effect of the service shaft at the south end of the building.

These show that the structure as it currently stands achieves 20-30% of NBS. This rating is governed primarily by the East wall spandrels in flexure working under north / south loading, but also by the perimeter columns in the same circumstance. Upon the overloading and softening of these members, load is expected to redistribute through other frame members, overloading each in turn until the deflection of the structure as a whole exceeds the curvature capacity of the gravity load resisting columns.

Refer to the summary at the front of the attached calculations for details.



Above: ETABS model of structure

4.4 Precast Hollow-core Floor System

The carpark mezzanine floor between ground level and level 1 has been constructed using precast 150mm hollow-core units, supported by steel beams with 50mm of seating.

Following the 2016 Kaikoura earthquake, problems were identified relating to the use of various precast floor systems, including hollow-core.

However, these issues are not expected to arise until the interstorey drift values approach 2%. Given that the expected drift values of the mezzanine floor under ductility 1.0 loading in the post frame hinging state are less than 0.5%, (ie, worst possible scenario), it is not expected that these units will present an issue.

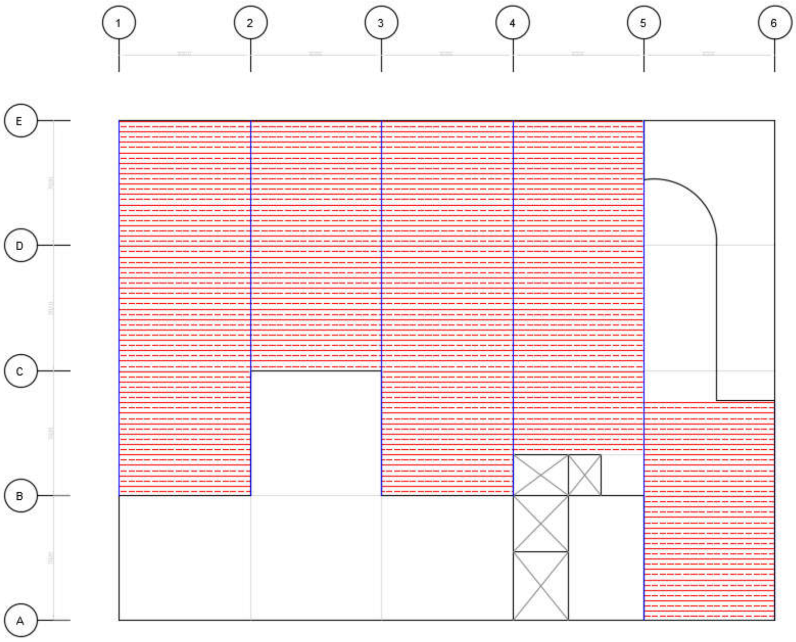


Figure 3: Plan view of carpark mezzanine. Hollowcore units in red, steel support beams in blue.

5.0 Conclusions

Our assessment has found the structure at 210-220 Thorndon Quay achieves 20-30%NBS.
The expected failure mechanism is the overloading of the front and rear perimeter frames, causing plastic hinges to form, allowing the columns to rotate past their allowable limits and losing axial capacity.

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

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02 Design Loadings

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- *05a Storey displacement charts*
- *05b Column curvature capacities*

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

Assessment of hollow-core floors for seismic performance
Richard Fenwick, Des Bull, Debra Gardiner – University of Canterbury, 2010

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

Calculation Summary

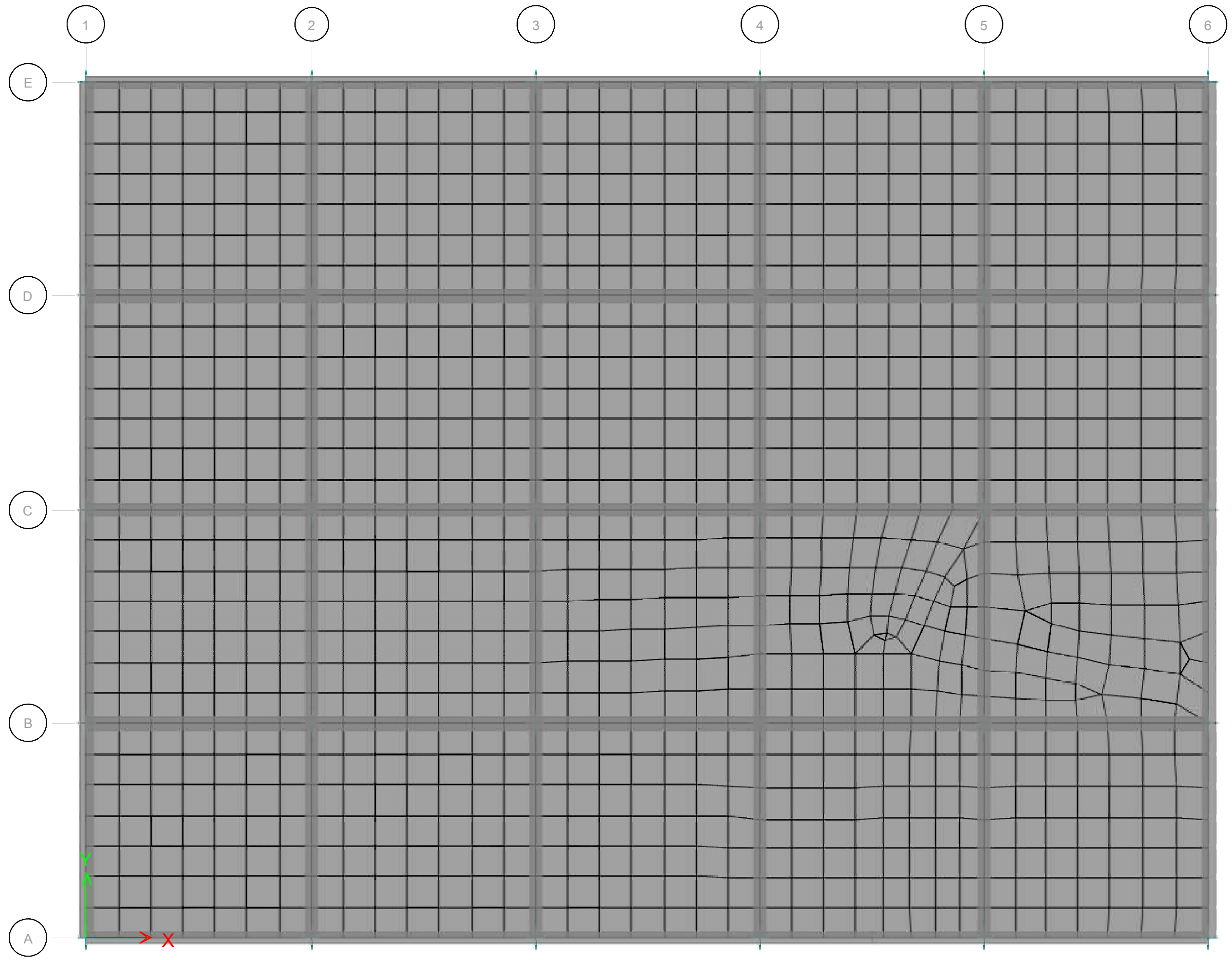
Existing Structure

Element		Action	Strength	Load	Rating	
West Frame Columns	LG-L1	Flexure	615	674	91%	
(Street side, Grid A)		Shear	678	510	100%	
	L1-L2	Flexure	285	452	63%	
		Shear	385	494	78%	
	L2-L3	Flexure	70	201	35%	
		Shear	161	208	77%	
West Frame Spandrels	L1	Flexure	324.2	1421	23%	
(Street side, Grid A)		Shear	548	696	79%	
	L2	Flexure	382.5	1537	25%	
		Shear	606	618	98%	
	L3	Flexure	357.4	902	40%	
		Shear	599	435	100%	
East Frame Columns	LG-L1	Flexure	615	956	64%	
(Rear side, Grid E)		Shear	837	1081	77%	
	L1-L2	Flexure	285	557	51%	
		Shear	375	590	64%	
	L2-L3	Flexure	70	195	36%	
		Shear	162	207	78%	
East Frame Spandrels	L1	Flexure	653.4	4106	16%	
(Rear side, Grid E)		Shear	866	964	90%	
	L2	Flexure	374.1	1449	26%	
		Shear	624	377	100%	
	L3	Flexure	184	456	40%	
		Shear	493	116	100%	
Internal Columns	LG-L1	Flexure	615	557	100%	
N & S Shear Walls		Flexure	82063	12449	100%	E/W Loading
		Shear	41357	10852	100%	E/W Loading
Roof Perimeter Connections		Shear	2.5	2.4	100%	
Gable End Connections		Shear	3.4	5.5	62%	

Min = 16%

Column drift (post frame hinging)	28	142	20%

As frame elements soften and plastic hinges form, load is redistributed through secondary systems if present. Final value is based on post hinging columns exceeding curvature limits and losing probable axial capacity.
These therefore represent an absolute lower bound of %NBS.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2
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A
Y
X

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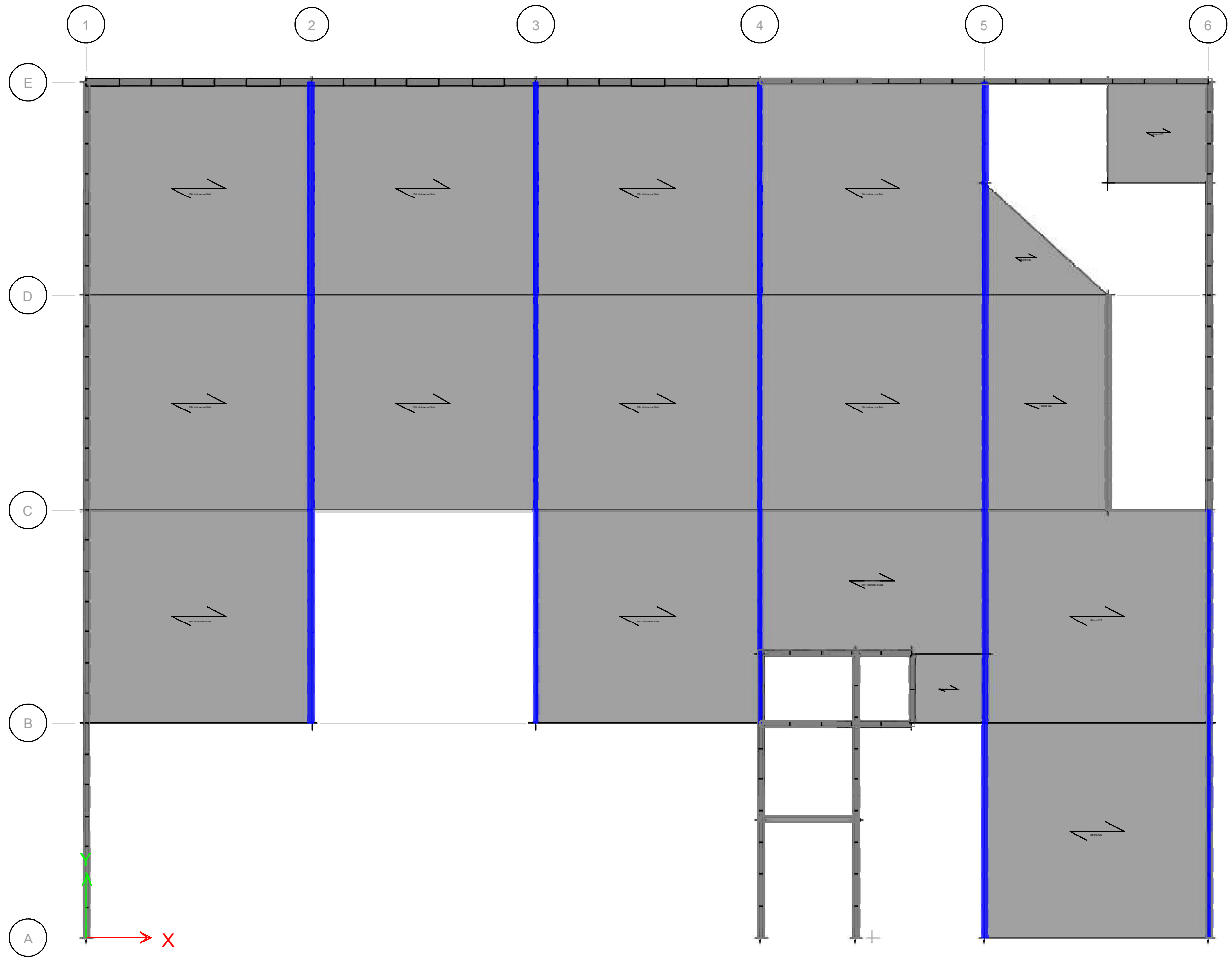

Plan View - Carpark Mezzanine - Z = 3.37 (m)

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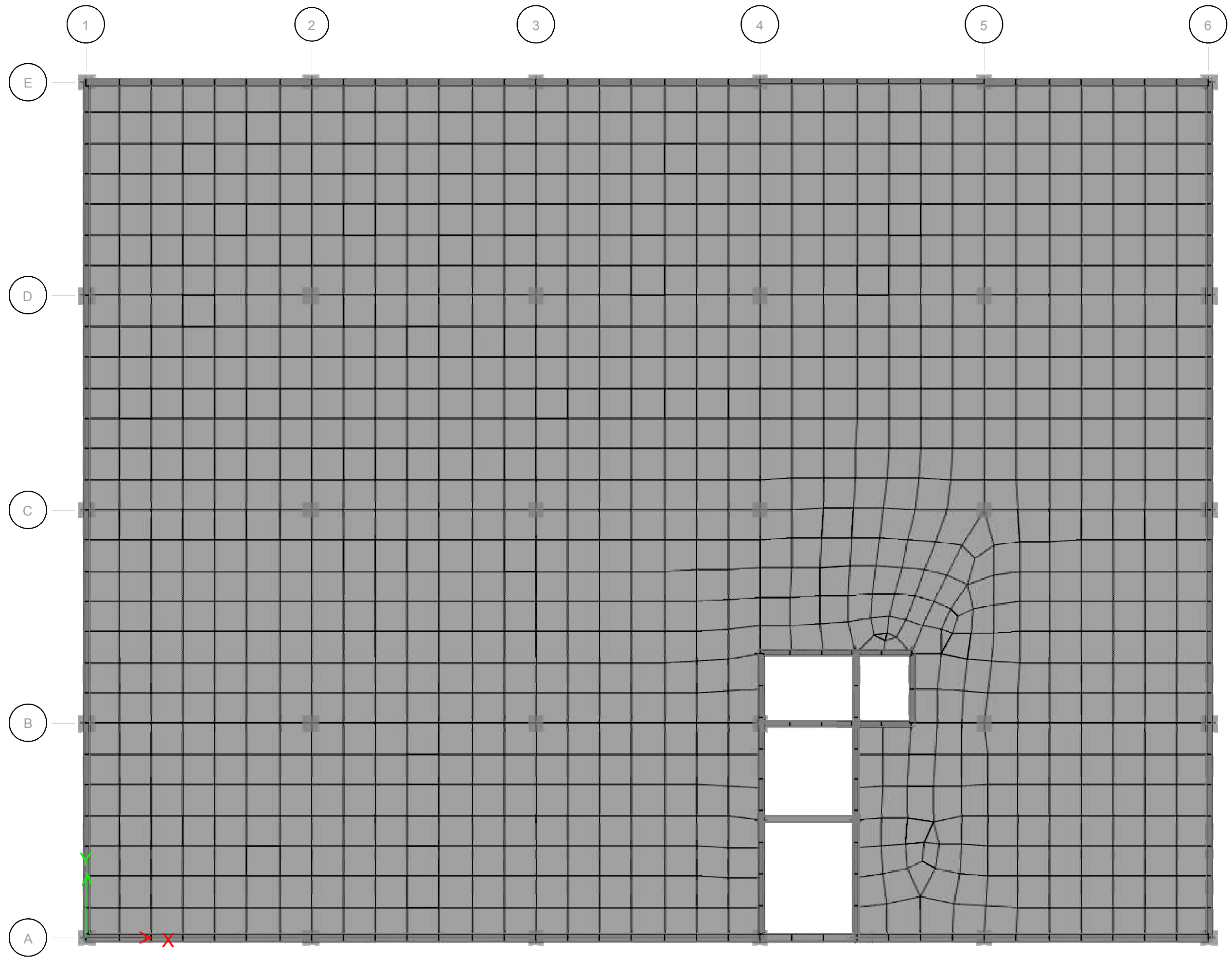

Plan View - Level 1 - Z = 5.715 (m)

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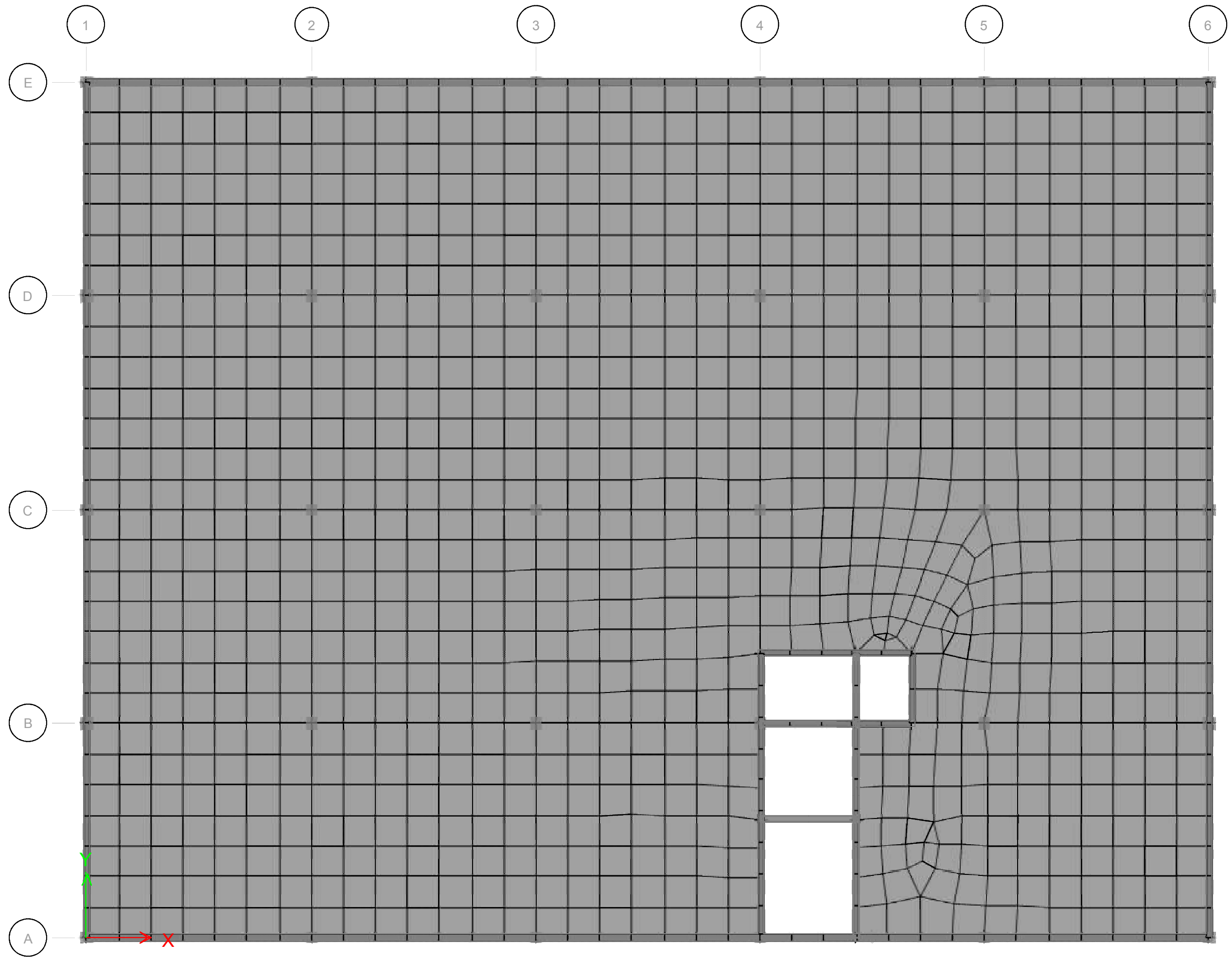
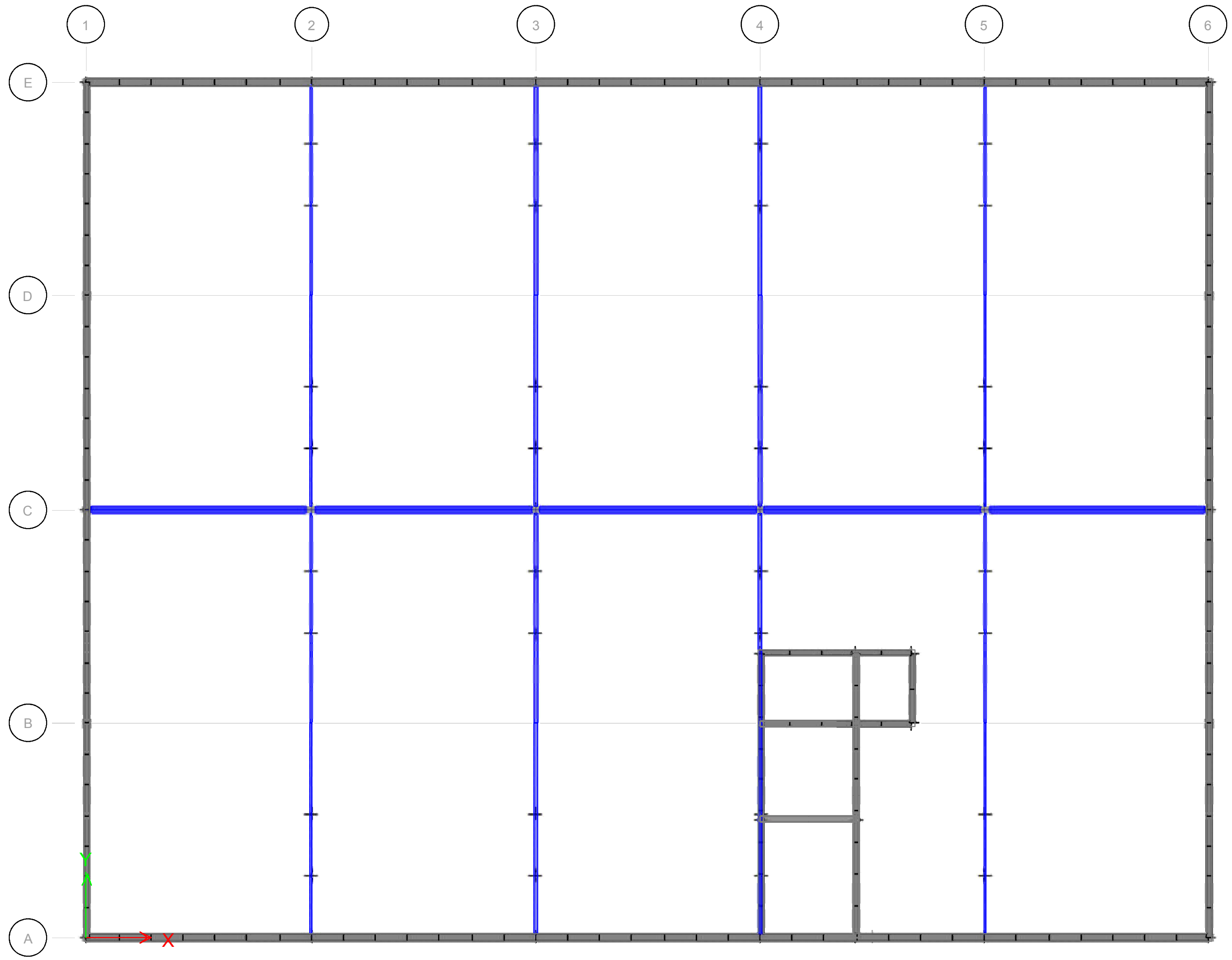
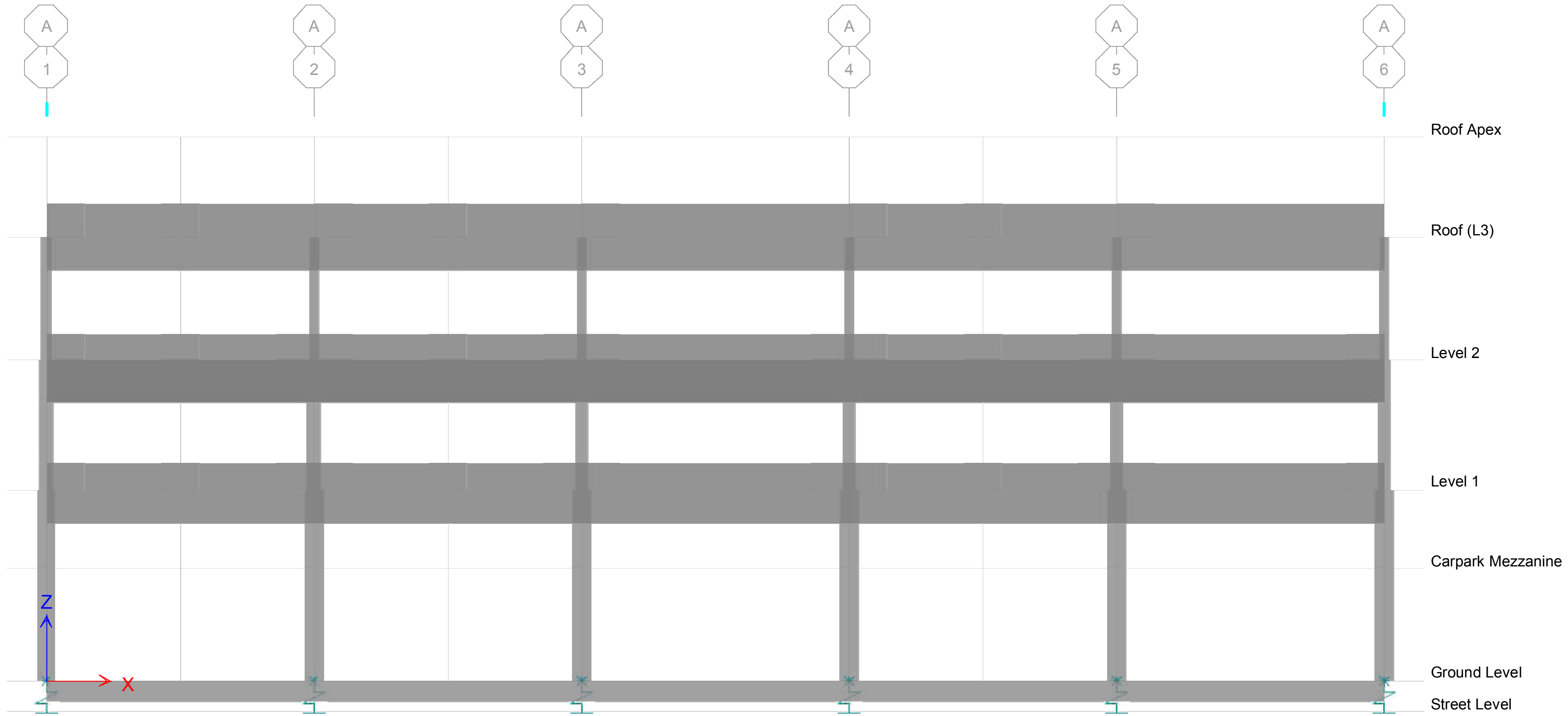
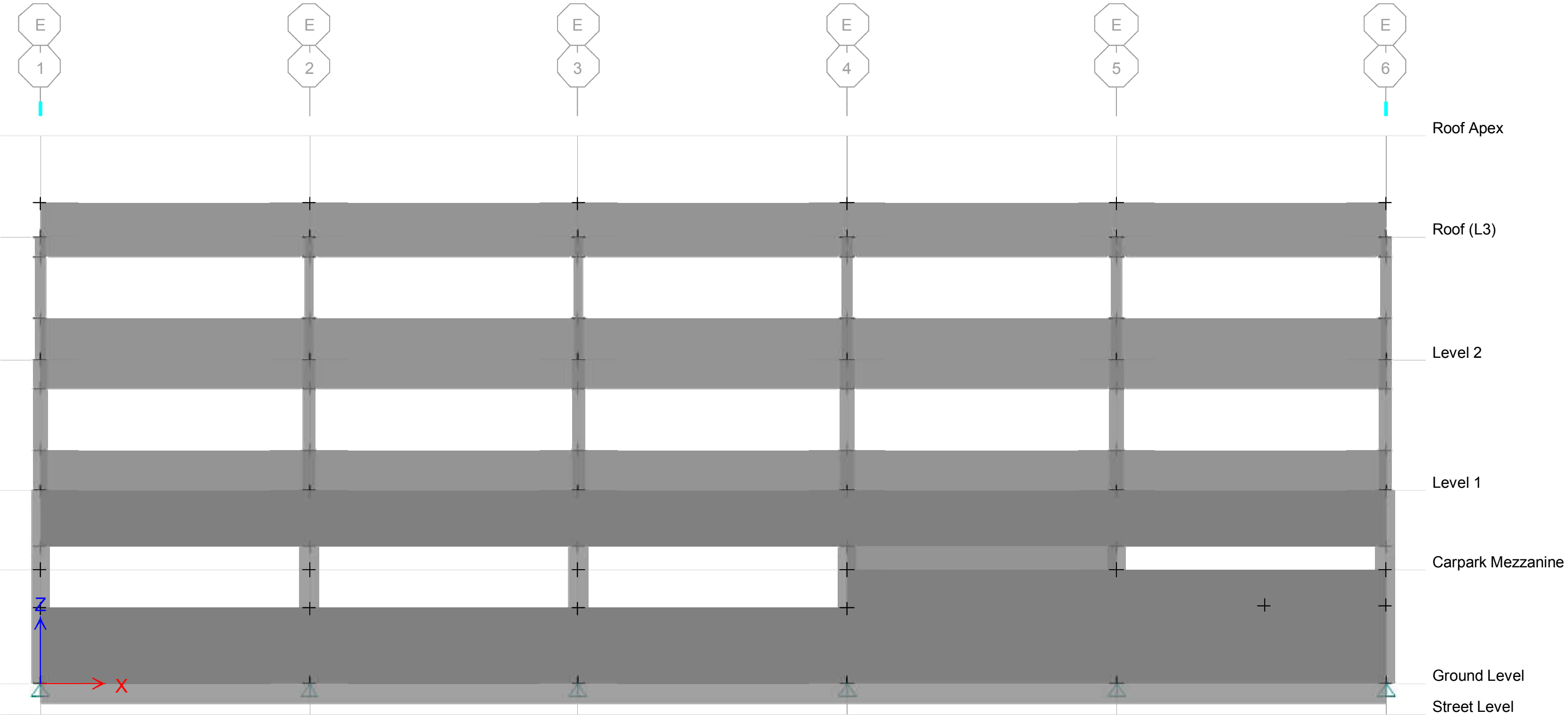


Plan View - Level 2 - Z = 9.601 (m)

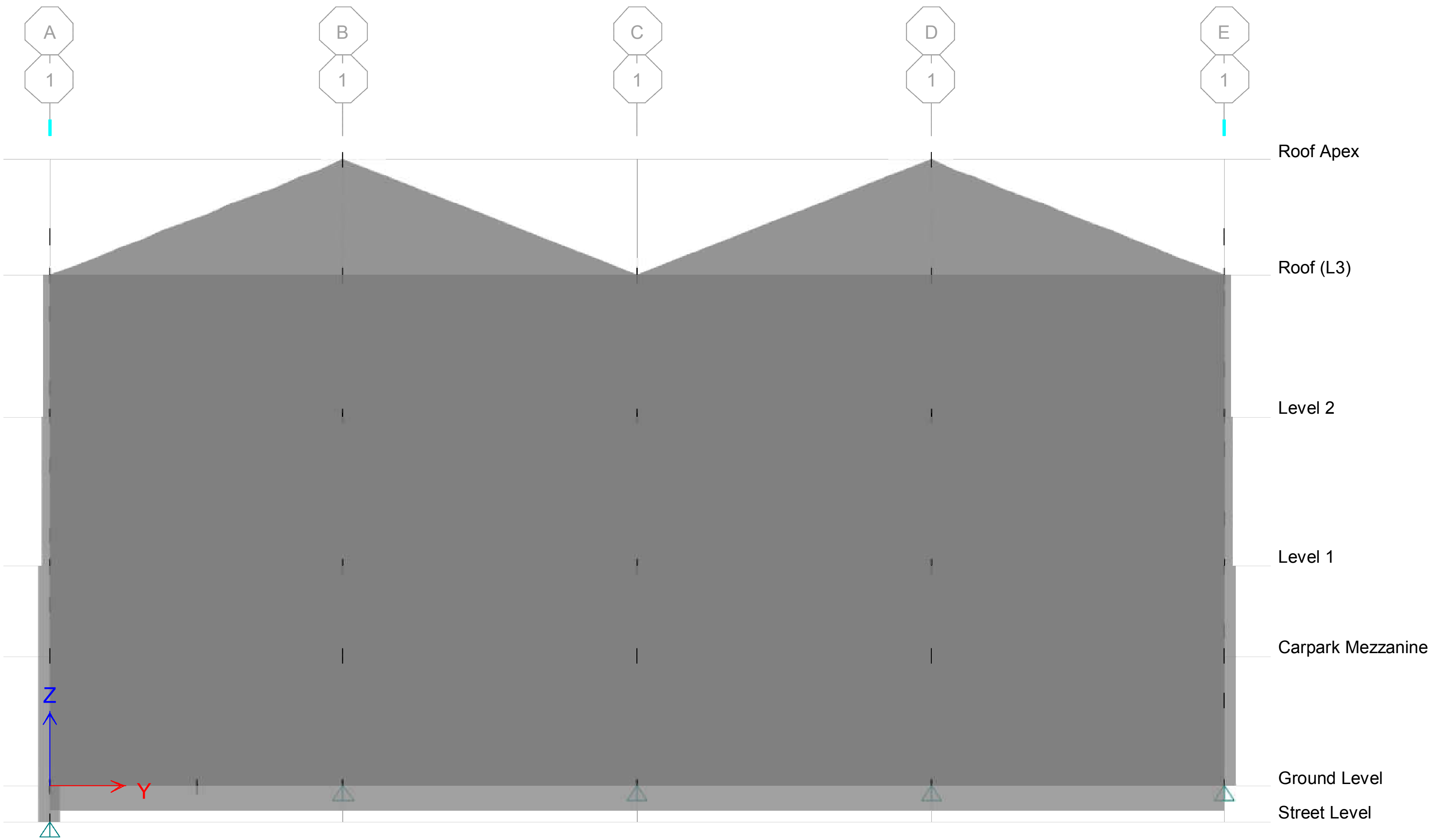


Plan View - Roof (L3) - Z = 13.259 (m)


A
1
A
2
A
3
A
4
A
5
A
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
Z
X



Elevation View - E



Elevation View - 1

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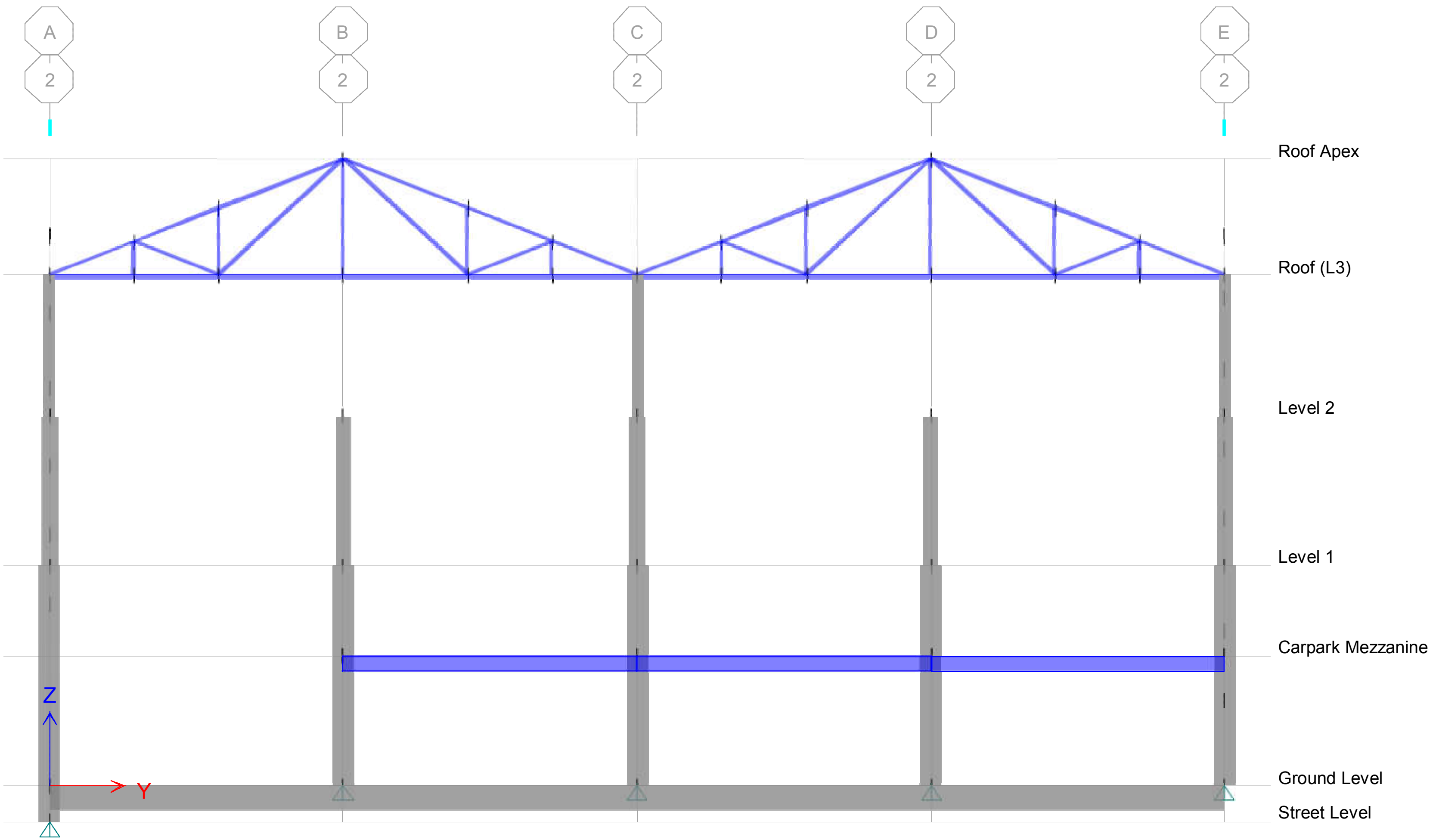


Elevation View - 2

ETABS Modelling Data

Existing Structure

TABLE: Auto Seismic - NZS 1170:2004								
Load Pattern	**Eccentricity**	**Site Class**	**Z**	**R**	**D**	**Sp**	**µ**
	%				km		
Seismic X Pattern	+10	D	0.4	1	2	0.7	2
Seismic X Pattern	-10	D	0.4	1	2	0.7	2
Seismic Y Pattern	+10	D	0.4	1	2	0.925	1.25
Seismic Y Pattern	-10	D	0.4	1	2	0.925	1.25
Ductility 1.0 X Pattern	+10	D	0.4	1	2	0.7	1
Ductility 1.0 X Pattern	-10	D	0.4	1	2	0.7	1
Ductility 1.0 Y Pattern	+10	D	0.4	1	2	0.7	1
Ductility 1.0 Y Pattern	-10	D	0.4	1	2	0.7	1

	Period Used	**Coeff Used**	**Weight Used**	**Base Shear**	**Ft Used**
	sec		kN	kN	kN
Seismic X Pattern	0.282	0.5345	26673	14258	1141
Seismic X Pattern	0.282	0.5345	26673	14258	1141
Seismic Y Pattern	0.168	0.9713	26673	25906	2072
Seismic Y Pattern	0.168	0.9713	26673	25906	2072
Ductility 1.0 X Pattern	0.282	0.84	26673	22405	1792
Ductility 1.0 X Pattern	0.282	0.84	26673	22405	1792
Ductility 1.0 Y Pattern	0.168	0.84	26673	22405	1792
Ductility 1.0 Y Pattern	0.168	0.84	26673	22405	1792

TABLE: Story Forces							
Story	**Load Combo**	**P**	**VX**	**VY**	**T**	**MX**	**MY**
		kN	kN	kN	kN-m	kN-m	kN-m
Roof Apex	Seismic Combo X Max	6	-197	0	3004	91	-119
Roof (L3)	Seismic Combo X Max	887	-2639	0	38651	13514	-19954
Level 2	Seismic Combo X Max	10001	-8588	0	127152	148981	-216744
Level 1	Seismic Combo X Max	19327	-12702	0	186480	286880	-441783
Carpark Mez.	Seismic Combo X Max	9089	-11032	71	151515	146900	-249640

TABLE: Story Forces							
Story	**Load Combo**	**P**	**VX**	**VY**	**T**	**MX**	**MY**
		kN	kN	kN	kN-m	kN-m	kN-m
Roof Apex	Seismic Combo Y Max	6	0	-359	-7207	91	-119
Roof (L3)	Seismic Combo Y Max	887	0	-4795	-97185	17546	-17734
Level 2	Seismic Combo Y Max	10001	0	-15605	-317521	174339	-202788
Level 1	Seismic Combo Y Max	19327	0	-23079	-470446	376713	-392342
Carpark Mez.	Seismic Combo Y Max	8397	1273	-23811	-520462	178259	-194077

Seismic Weights for 210-220 Thorndon Quay

roof	unit	item	x	y	z	prop		density	weight	totals	
			m	m	m	No		kN/m^3	kN	kN	
			or areas								
G		parapet beams	140	0.3	0.23	1	1	24	227		
		parapets	40	1	0.15	2	1	24	288		
		gables	15	3	0.15	4	0.5	24	324		
		columns	0.3	3.6	0.30	30	0.5	24	117		
		walls	30	3.6	0.15	2	0.5	24	389		
		pier walls	0.6	3.6	0.15	4	0.5	24	16		
		lift and stair walls (above)	20	3.6	0.20	1	0.5	24	173		
		roof & partitions	30	1	40.00	1	1	0.6	720	**2253**	**G**
Q		roof no access	30	1	40.00	1	1	0.25	300	**300**	**Q**

2nd floor	unit	item	x	y	z	prop		density	weight	totals	
			m	m	m	No		kN/m^3	kN	kN	
			or areas								
G		conc cols (above)	0.3	3.6	0.30	30	0.5	24	117		
		walls (above)	30	3.6	0.15	2	0.5	24	389		
		pier walls (above)	0.6	3.6	0.15	4	0.5	24	16		
		lift and stair walls (above)	20	3.6	0.20	1	0.5	24	173		
		conc cols (below)	0.4	3.9	0.40	30	0.5	24	225		
		walls (below)	30	3.9	0.15	2	0.5	24	421		
		pier walls (below)	0.6	2	0.15	4	0.5	24	9		
		lift and stair walls (below)	20	3.9	0.20	1	0.5	24	187		
		concrete spandrels (east)	40	2	0.15	1	1	24	288		
		concrete spandrels (west)	40	2	0.15	1	1	24	288		
		concrete plinths	2	0.5	2.00	30	0.5	24	720		
		conc slab	30	0.2	40.00	1	1	24	5760		
		2nd floor, partitions, services (timber)	30	1	40.00	1	1	0.6	720	**9311**	**G**
Q		residential	30	1	40.00	1	1	1.5	1800	**1800**	**Q**

Excerpt from 2009 Assessment
Verification of ETABS seismic weight calculation.

1st floor	unit	item	x	y	z		prop	density	weight	totals	
			m	m	m	No		kN/m^3	kN	kN	
			or areas								
G		conc cols (above)	0.4	3.9	0.40	30	0.5	24	225		
		walls (above)	30	3.9	0.15	2	0.5	24	421		
		pier walls (above)	0.6	3.9	0.15	4	0.5	24	17		
		lift and stair walls (above)	20	3.9	0.20	1	0.5	24	187		
		conc cols (below)	0.55	2.2	0.55	30	0.5	24	240		
		walls (below)	30	2.2	0.15	2	0.5	24	238		
		pier walls (below)	0.6	2	0.15	4	0.5	24	9		
		lift and stair walls (below)	20	2.2	0.20	1	0.5	24	106		
		concrete spandrels (east)	40	2	0.15	1	1	24	288		
		concrete spandrels (west)	40	3	0.15	1	1	24	432		
		concrete plinths	2	0.5	2.00	30	0.5	24	720		
		conc slab	30	0.2	40.00	1	1	24	5760		
		1st floor, partitions, services (timber)	30	1	40.00	1	1	0.6	720	**9361**	**G**
Q		residential	30	1	40.00	1	1	1.5	1800	**1800**	**Q**

Mezz floor	unit	item	x	y	z		prop	density	weight	totals	
			m	m	m	No		kN/m^3	kN	kN	
			or areas								
G		conc cols (above)	0.55	2.2	0.55	30	0.5	24	240		
		walls (above)	30	2.2	0.15	2	0.5	24	238		
		pier walls (above)	0.6	2.2	0.15	4	0.5	24	10		
		lift and stair walls (above)	20	2.2	0.20	1	0.5	24	106		
		conc cols (below)	0.55	3.5	0.55	30	0.5	24	381		
		walls (below)	30	3.5	0.15	2	0.5	24	378		
		lift and stair walls (below)	20	3.5	0.20	1	0.5	24	168		
		conc slab (150 Dycore & topping)	22.5	1	40.00	1	1	3.3	2970		
		steel beams	27	1	1.00	3	1	0.54	44		
		steel beams	20	1	1.00	3	1	0.54	32		
		conc beam	0.15	0.7	40.00	1	1	24	101		
		conc beam 2	15	0.6	0.40	1	1	24	86		
		timber mezzanine floor	7.5	1	8.00	1	1	0.6	36		
		1st floor, partitions, services (timber)	22.5	1	40.00	1	1	0.6	540	**5329**	**G**
Q		vehicle parking	22.5	1	40.00	1	1	2.5	2250		
		restaurant	7.8	1	8.00	1	1	3	187	**2437**	**Q**

Excerpt from 2009 Assessment
Verification of ETABS seismic weight calculation.

1 G Loads

Equivalent Static Seismic Forces: NZS 1170.5

Site Data

Location = Thorndon
Soil Type = D — Deep or soft soil
Nearest Fault = Wellington — Figure 3.5
D (km) = 1 — Table 3.3
$N_{max}(s)$ = 1 — Table 3.7

Building Data

Period T_1 = 0.4 s
Ductility μ = 1
Return Period = 500 yrs
S_p = 1
k_μ = 1

Elastic Site Spectra

$C_h(T)$ = 3.00 — Table 3.1
Z = 0.4 — Table 3.3 or Figure 3.3/Figure 3.4
R_u or R_s = 1.00 — Table 3.5
N(T,D) = 1.00

$C(T_1) = N(T,D)$
= 1.2

Minimum Checks

$C(T_1) >= (Z/20+0.02)R_u$ or $0.03R_u$
= 0.04 = 0.03
OK OK

d= 1 damping

Horizontal design action coefficient

$Cd(T1) = C(T_1)S_p/k_\mu d$
= **1.00**

Seismic Weight Calculation / **Static Lateral Load Distribution**

Level i	G (kN)	Q (kN)	Apply y_a (y/n)	Live Area (m^2)	ψ_c	ψ_a	Q_u (kN)	$G+Q_u$ (kN)	$h_i - h_{i-1}$ (m)	h_i (m)	h_iW_i (MNm)	h_iW_i / S (H_iW_i)	F_i (kN)	V_i (kN)
TYP			y											
ROOF	2253	0	y	1200	0.0	0.39	0	2253	3.6	13.2	29734	0.15	5933	5933
3	9311	1800	y	1200	0.3	0.39	209	9520	3.9	9.6	91394	0.47	11606	17539
2	9361	1800	y	1200	0.3	0.39	209	9570	2.2	5.7	54549	0.28	6927	24466
1	5329	2437	y	900	0.3	0.40	292	5621	3.5	3.5	19674	0.101	2498	26964
GRD														
SUM								26964			195352	1.000	26964	

Seismic Weight, W_t **26964 kN**
Base Shear **26964 kN**

Variation = (26964 / 26673) - 1 = 1.01%
Within acceptable limits

Story Response - Maximum Story Drifts

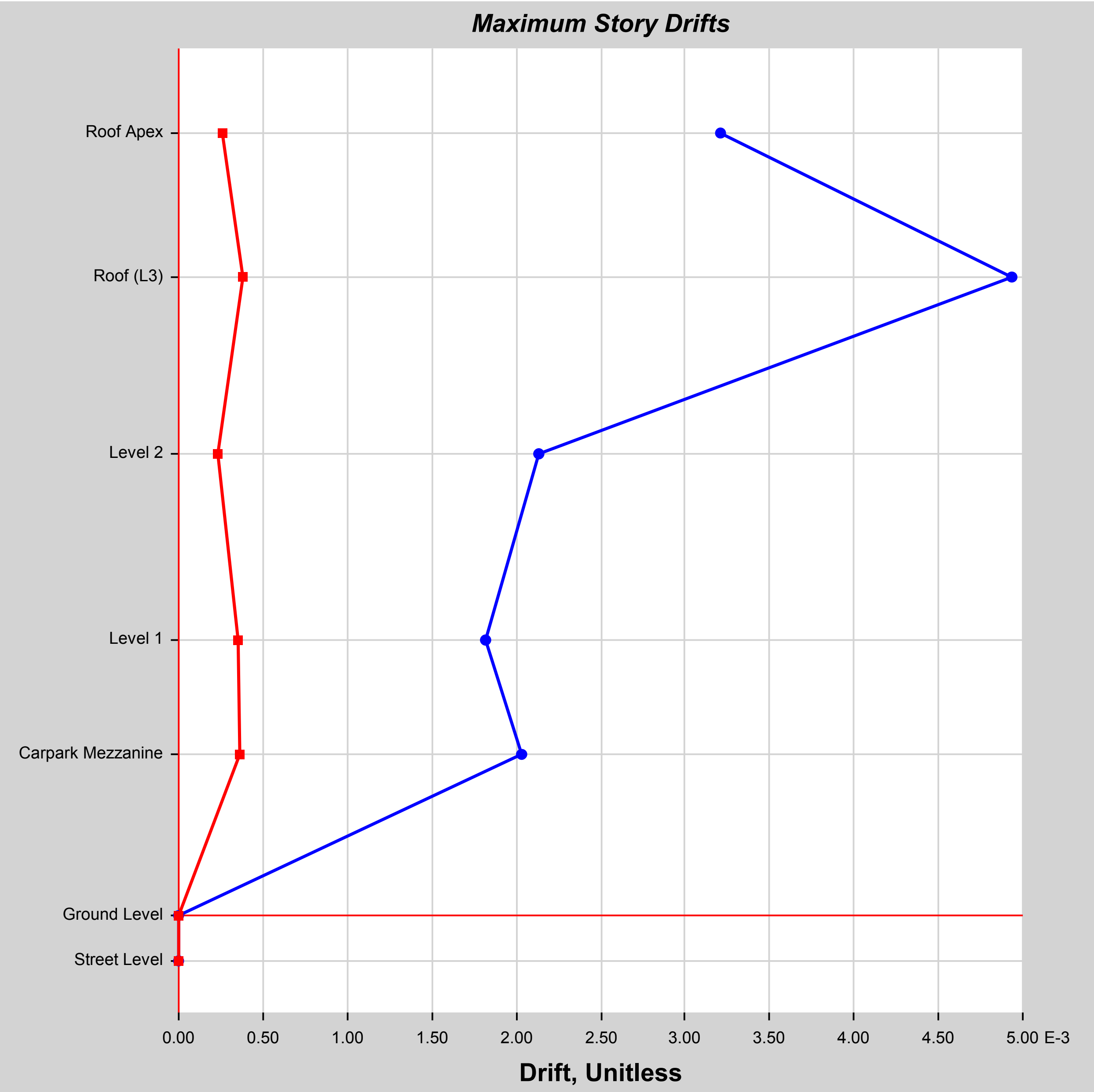
Summary Description

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

Input Data

Name	StoryResp7		
Display Type	Max story drifts	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 Drifts

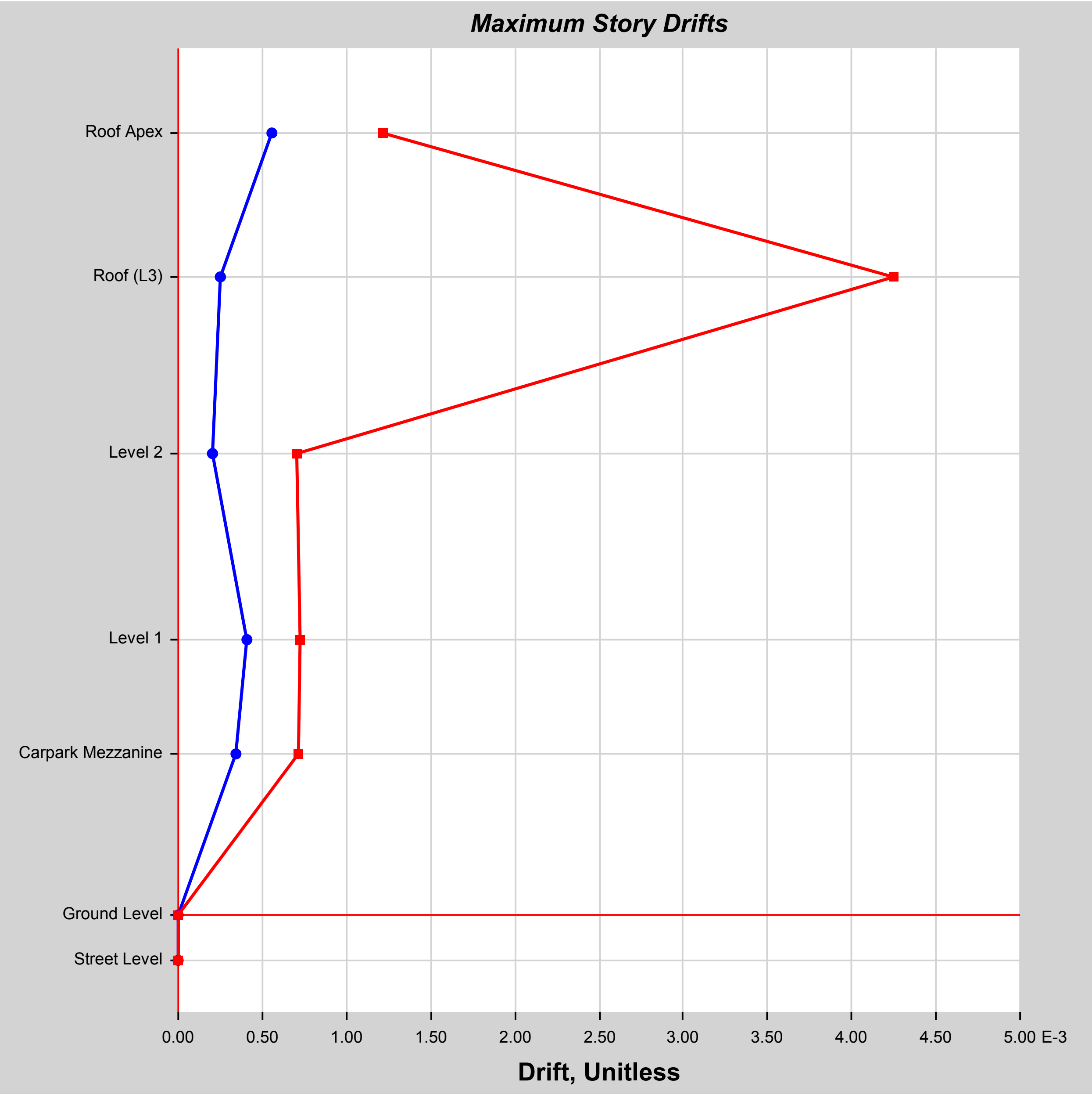
Summary Description

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

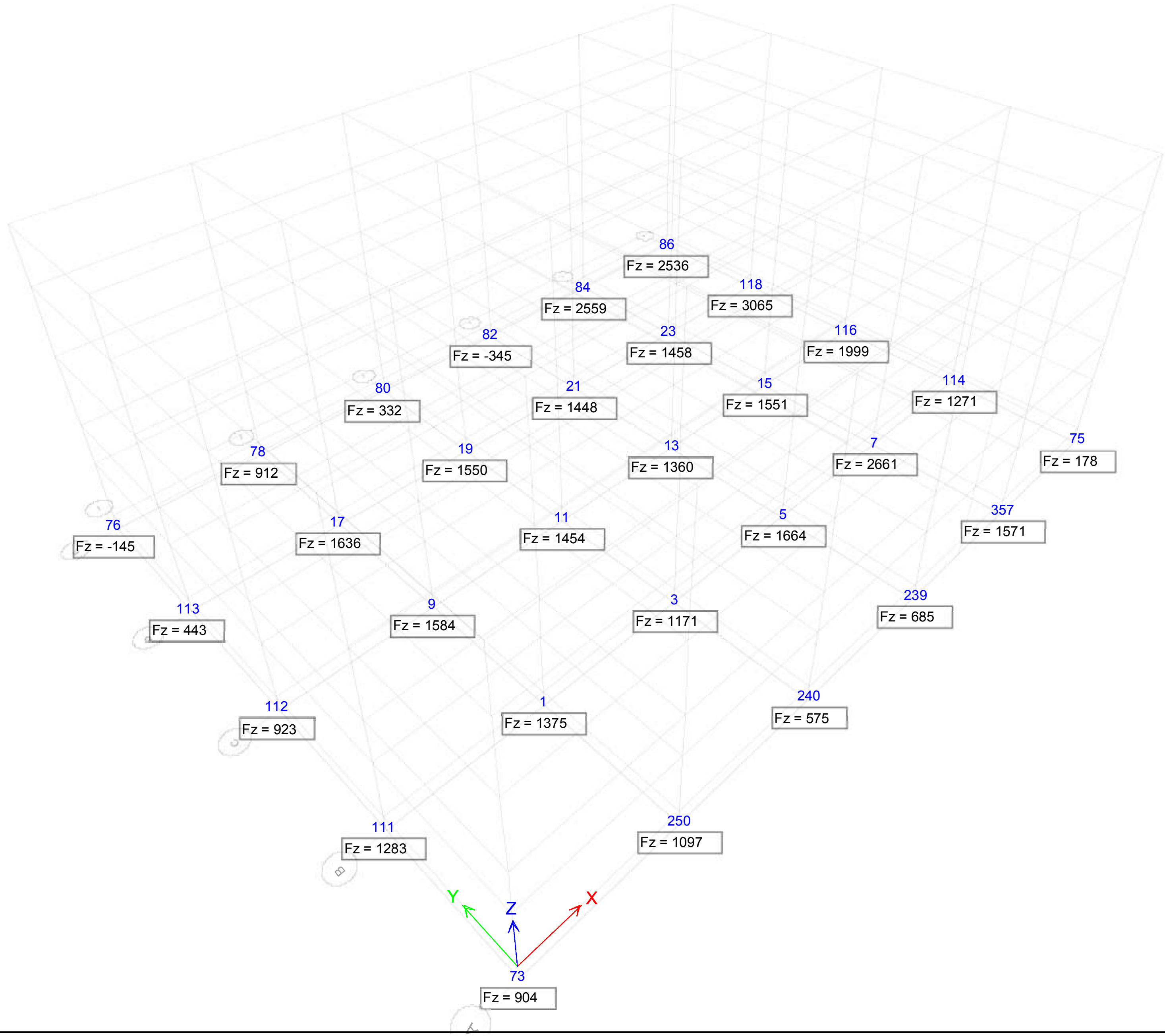
Input Data

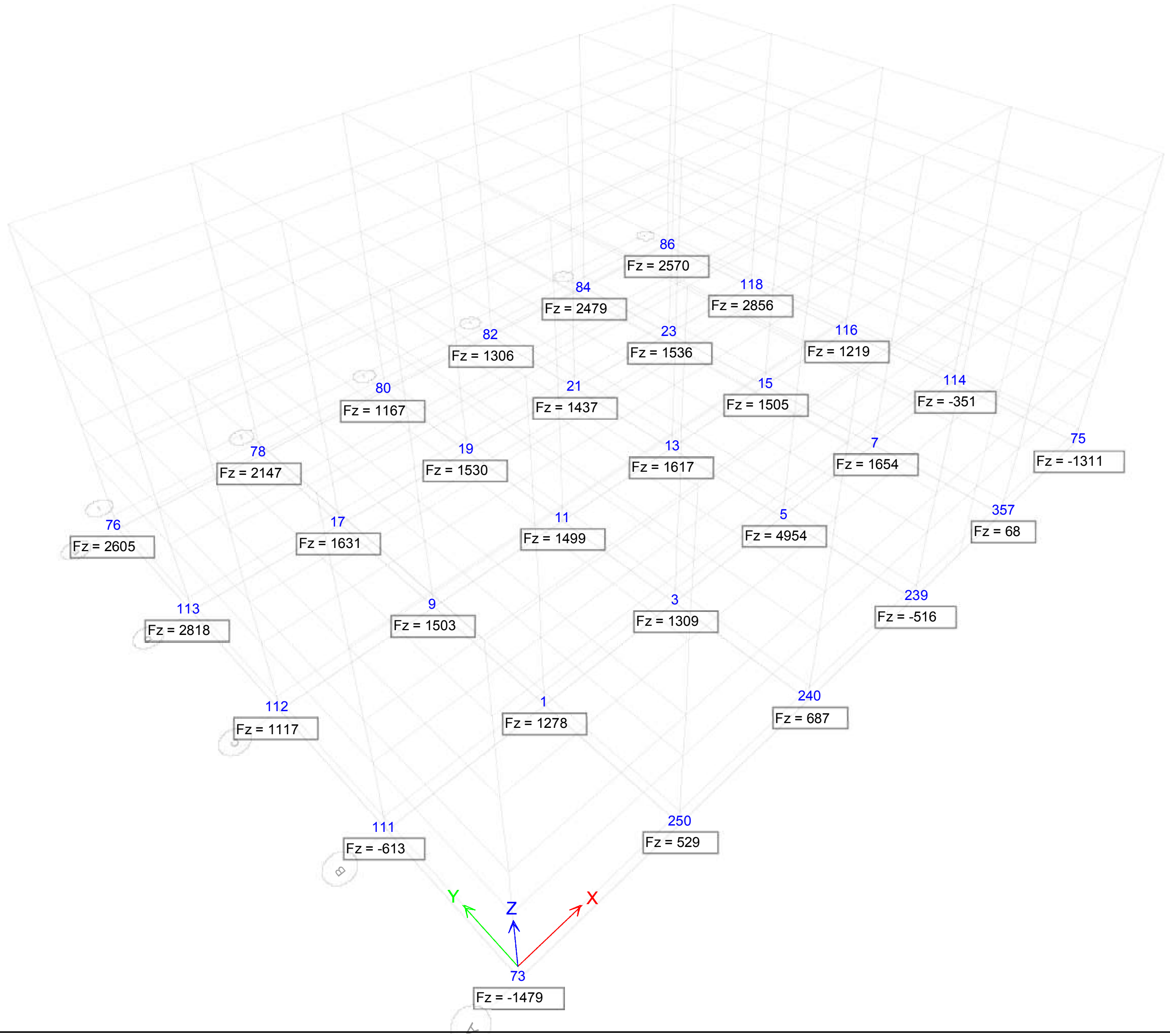
Name	StoryResp7		
Display Type	Max story drifts	Story Range	All Stories
Load Combo	Seismic Combo Y	Top Story	Roof Apex
Output Type	Max	Bottom Story	Street Level

Plot

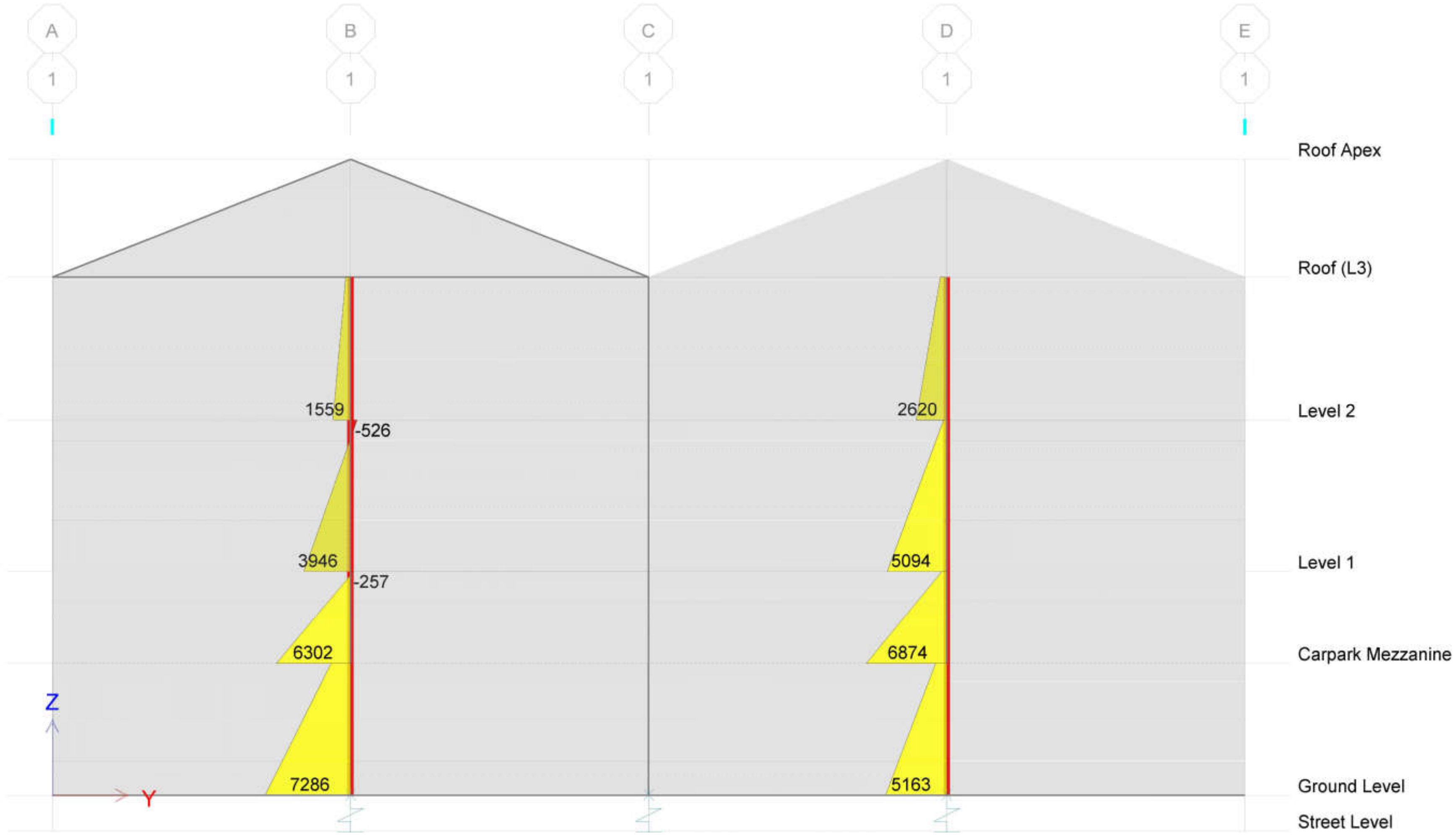


Page 02c.i

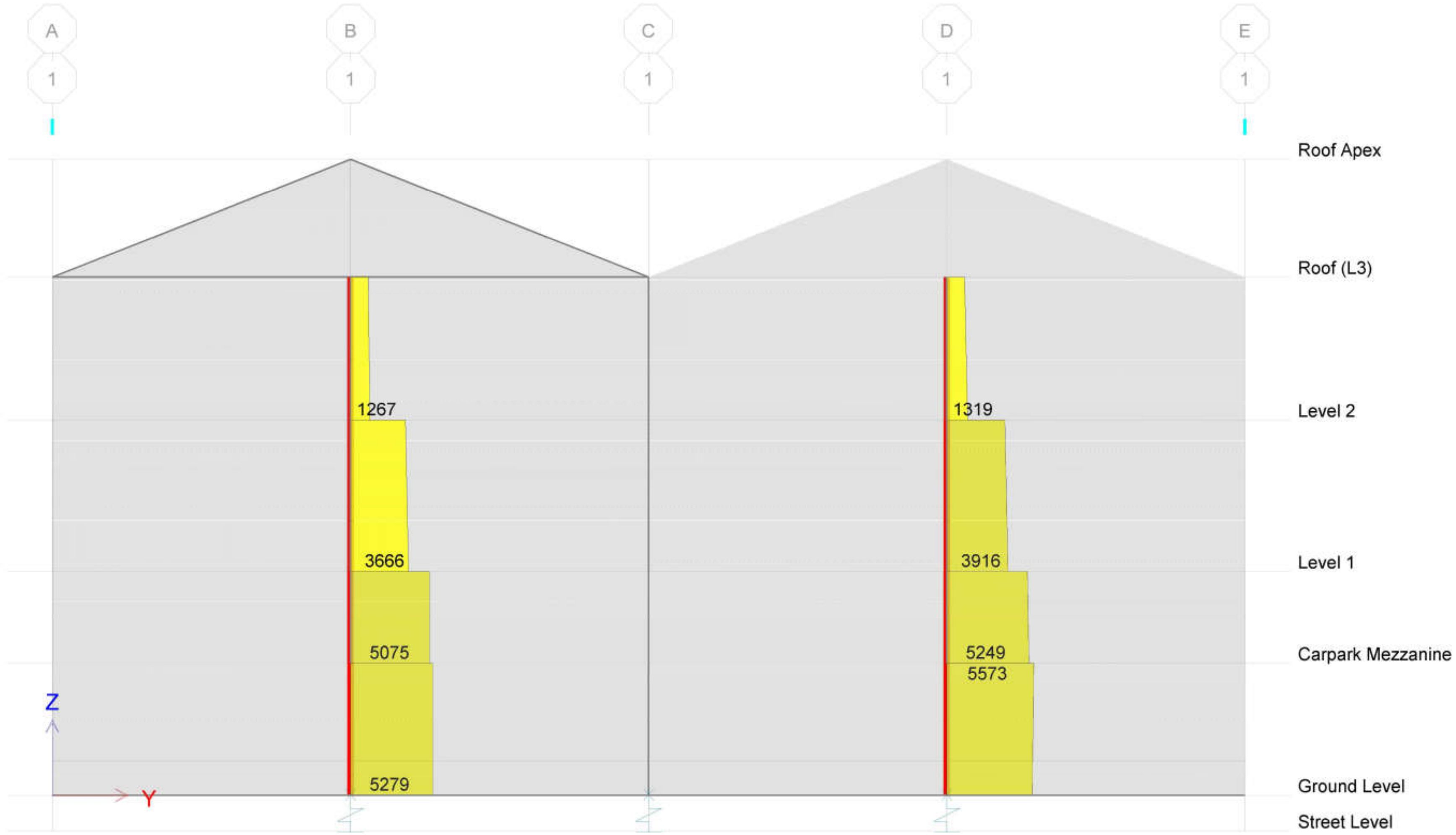

86
Fz = 2536
84
Fz = 2559
118
Fz = 3065
82
Fz = -345
23
Fz = 1458
116
Fz = 1999
80
Fz = 332
21
Fz = 1448
15
Fz = 1551
114
Fz = 1271
78
Fz = 912
19
Fz = 1550
13
Fz = 1360
7
Fz = 2661
75
Fz = 178
76
Fz = -145
17
Fz = 1636
11
Fz = 1454
5
Fz = 1664
357
Fz = 1571
113
Fz = 443
9
Fz = 1584
3
Fz = 1171
239
Fz = 685
112
Fz = 923
1
Fz = 1375
240
Fz = 575
111
Fz = 1283
250
Fz = 1097
Y
Z
X
73
Fz = 904



220 Thorndon Quay - Existing Structure.EDB 3-D View Restraint Reactions (Seismic Combo Y) [kN, kN-m]

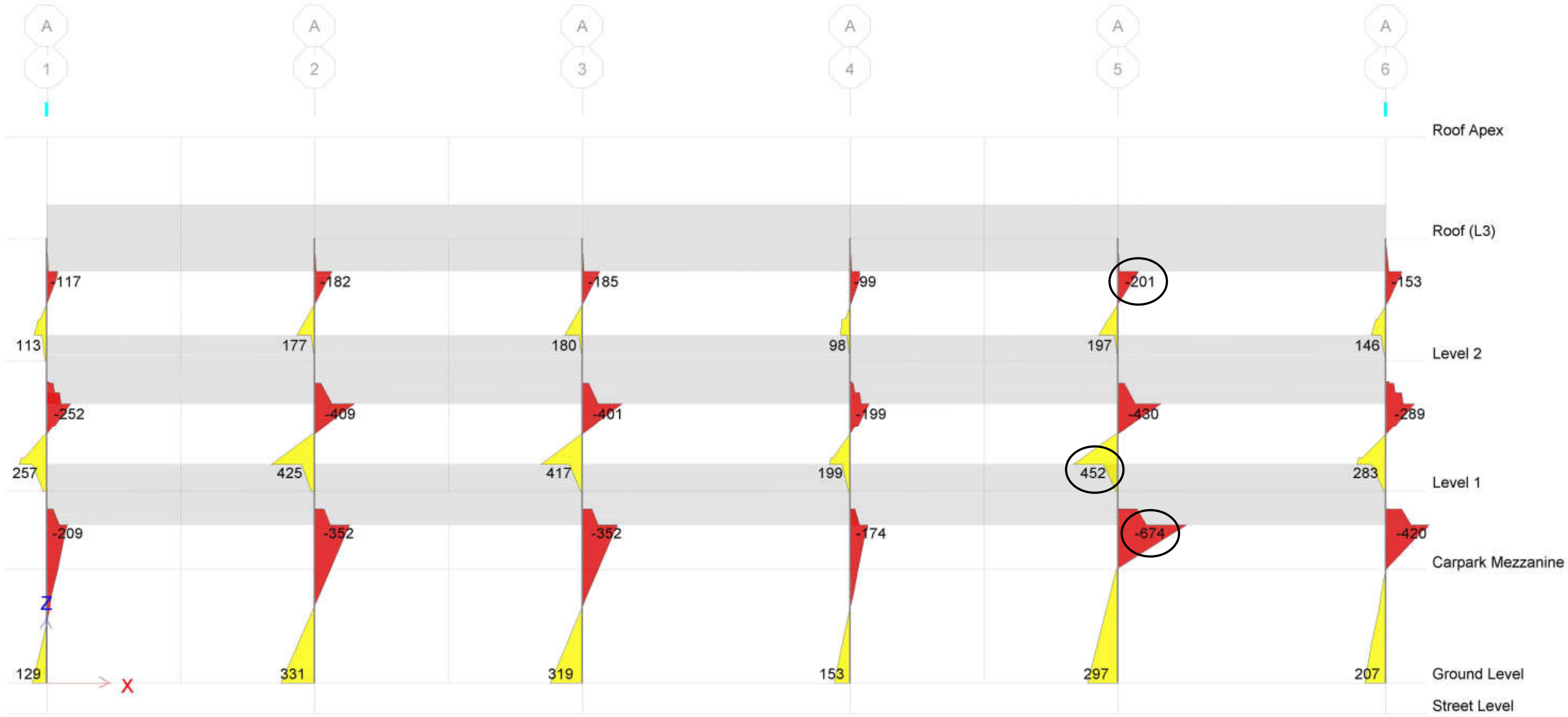


 Elevation View - 1 Moment 3-3 Diagram (Seismic Combo Y) [kN-m]

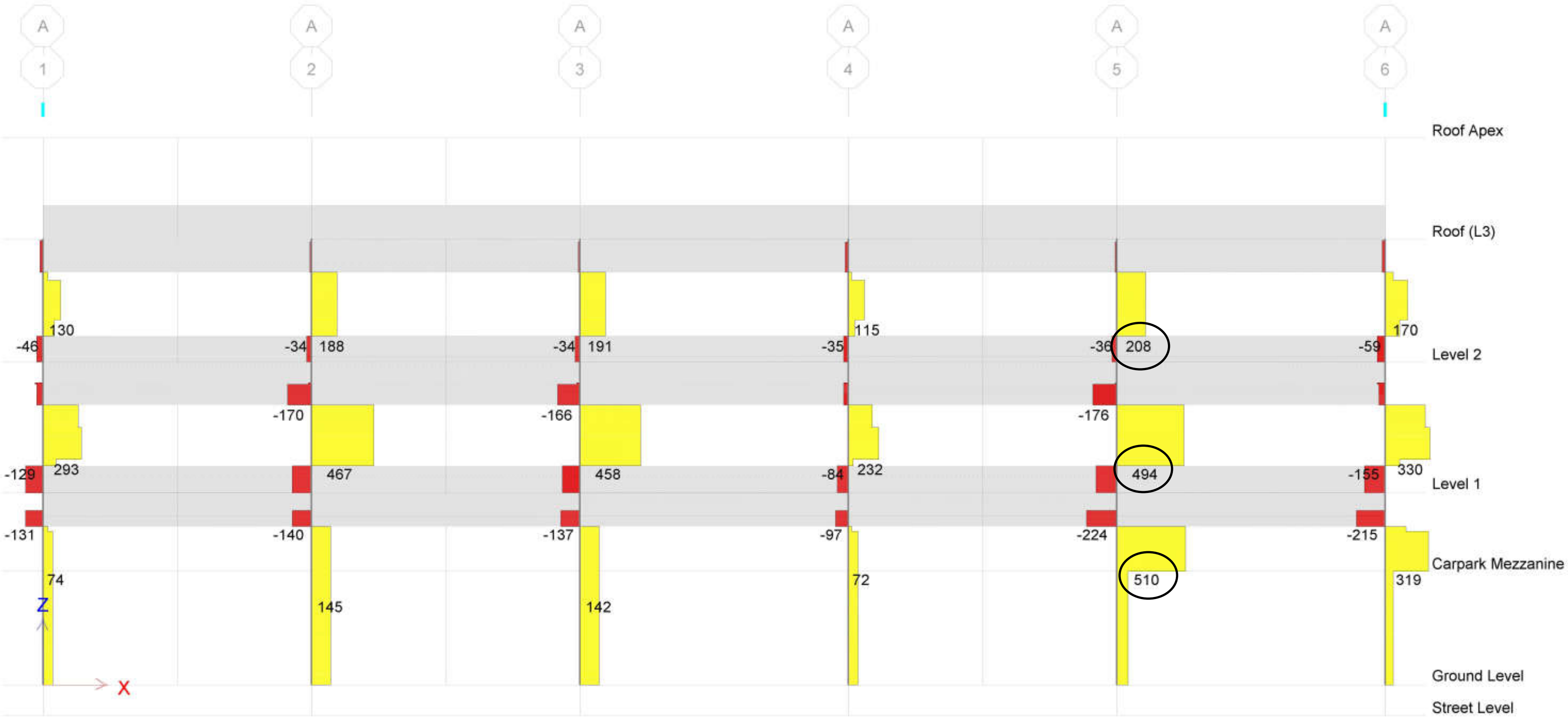


220 Thorndon Quay - Existing Structure.EDB Elevation View - 1 Shear Force 2-2 Diagram (Seismic Combo Y) [kN]

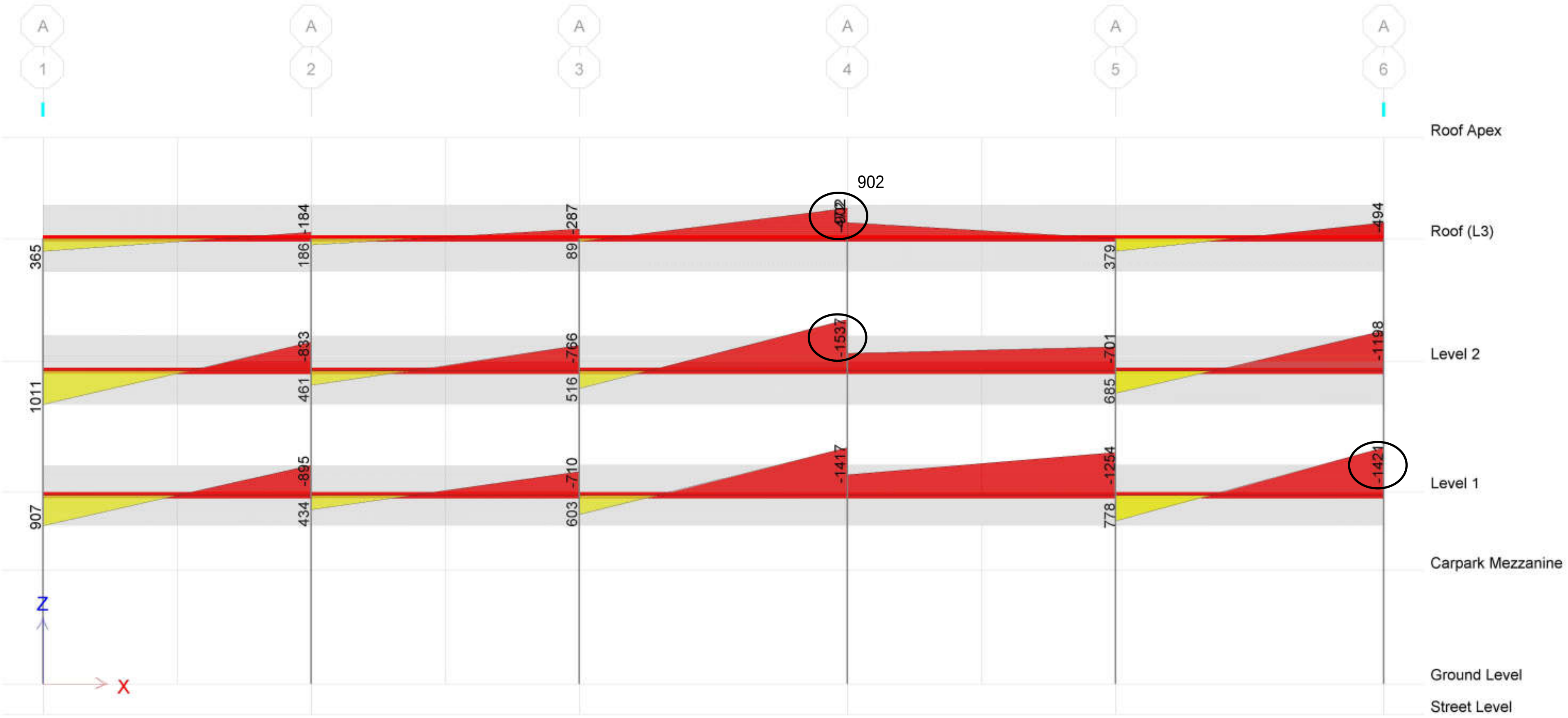
Page 02c.v


A
1
2
3
4
5
6
Roof Apex
Roof (L3)
-117
-182
-185
-99
-201
-153
113
177
180
98
197
146
Level 2
-252
-409
-401
-199
-430
-289
257
425
417
199
452
283
Level 1
-209
-352
-352
-174
-674
-420
Carpark Mezzanine
Z
X
129
331
319
153
297
207
Ground Level
Street Level

Page 02c.vi

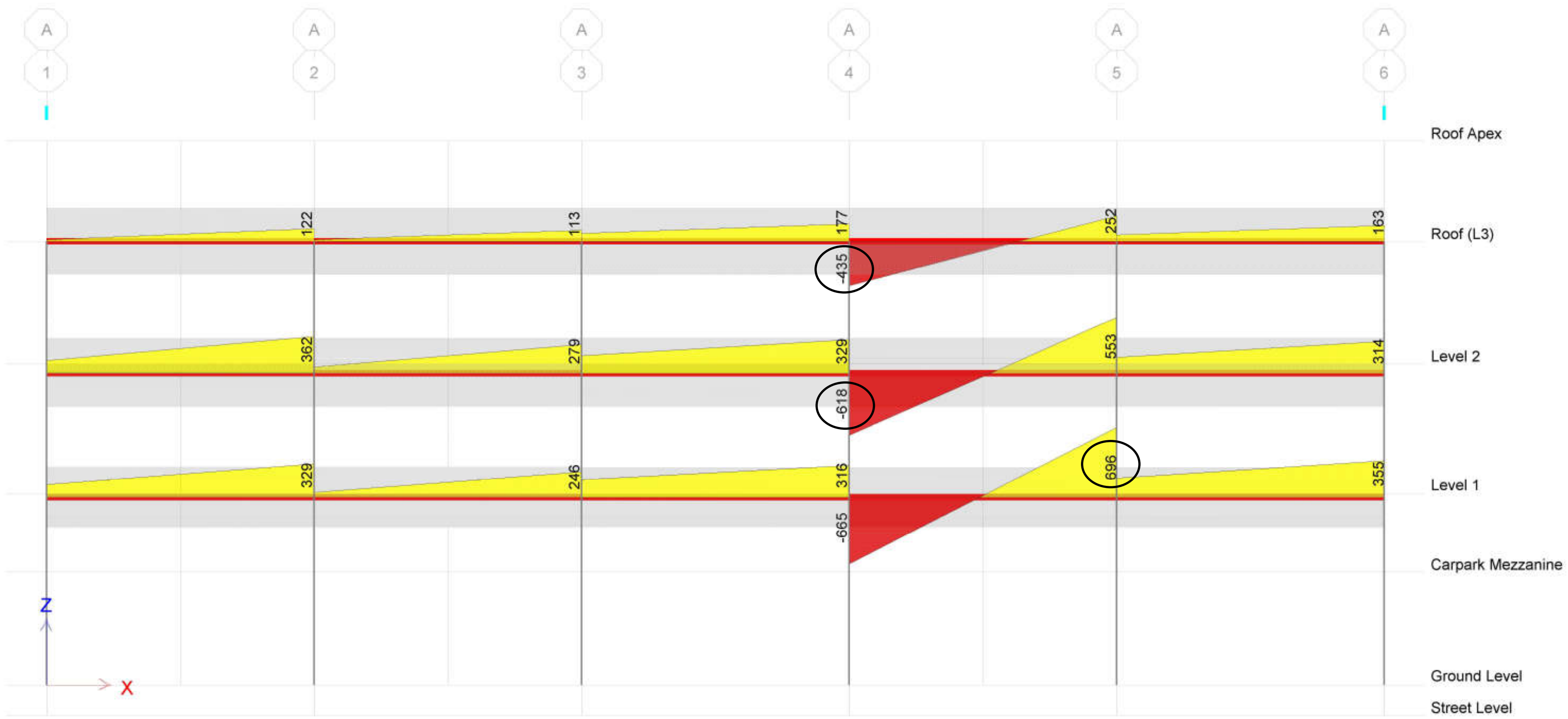

A
1
2
3
4
5
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
130
-46
-34
188
-34
191
115
-35
-36
208
170
-59
-170
-166
-176
-129
293
467
458
-84
232
494
-155
330
-131
-140
-137
-97
-224
-215
74
145
142
72
510
319
Z
X

Page 02c.vii

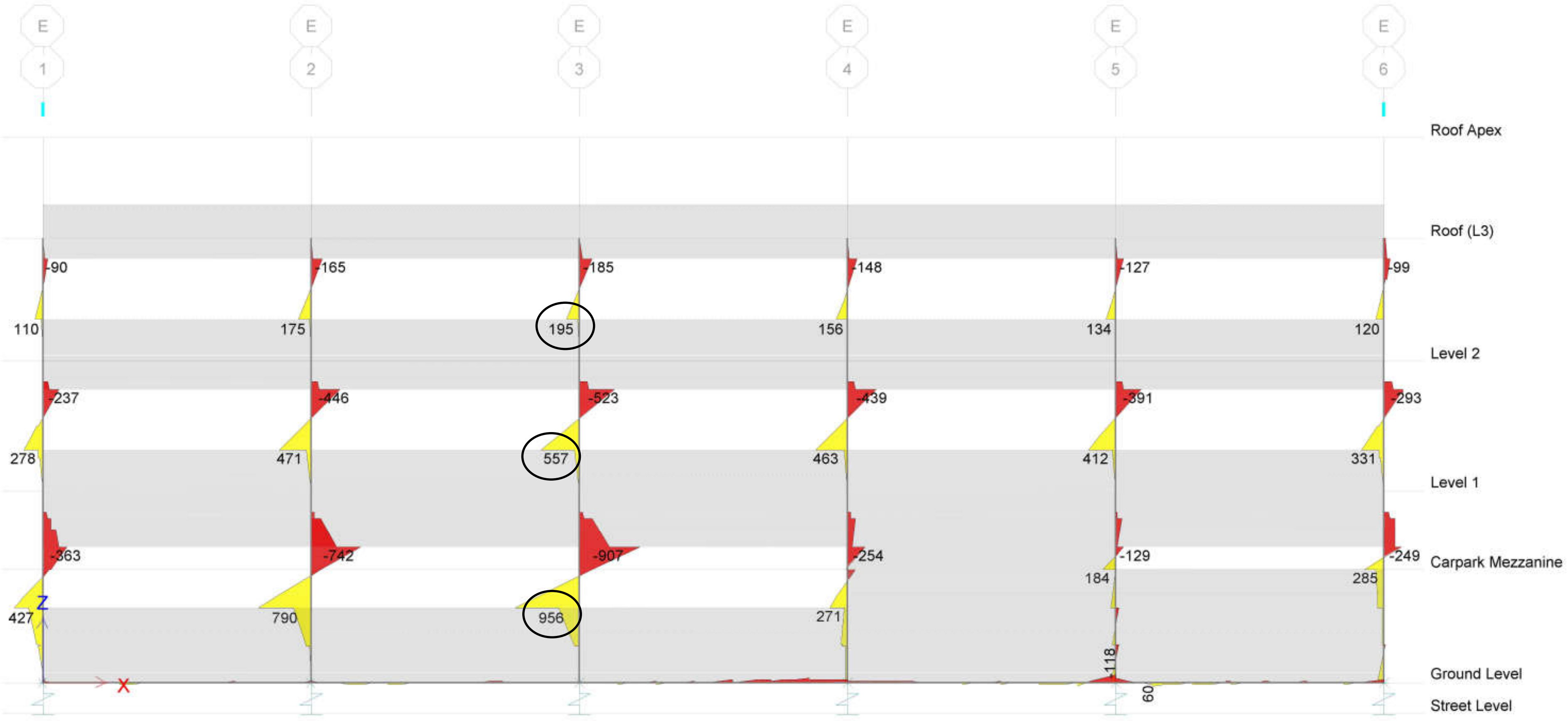


220 Thorndon Quay - Existing Structure.EDB Elevation View - A Moment 3-3 Diagram (Seismic Combo X) [kN-m]

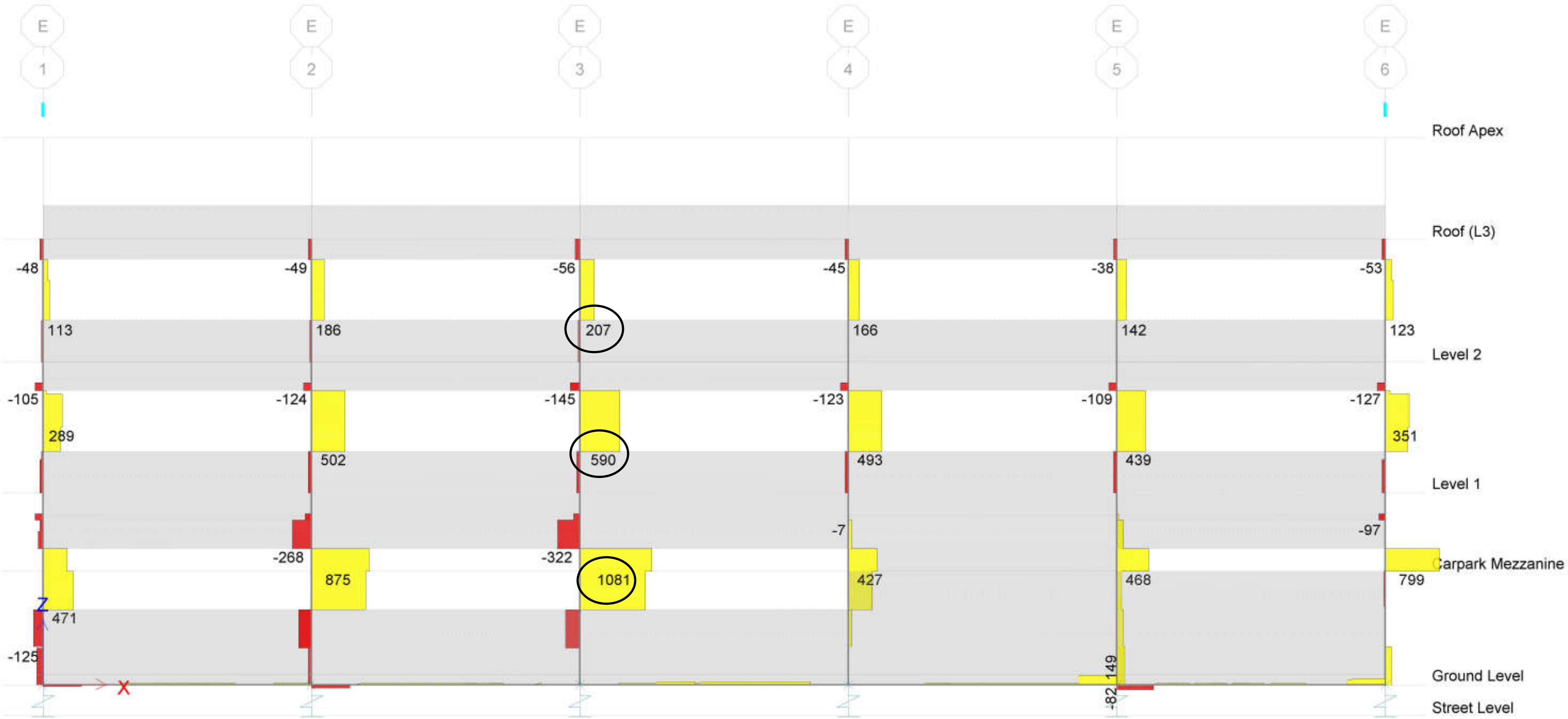
Page 02c.viii

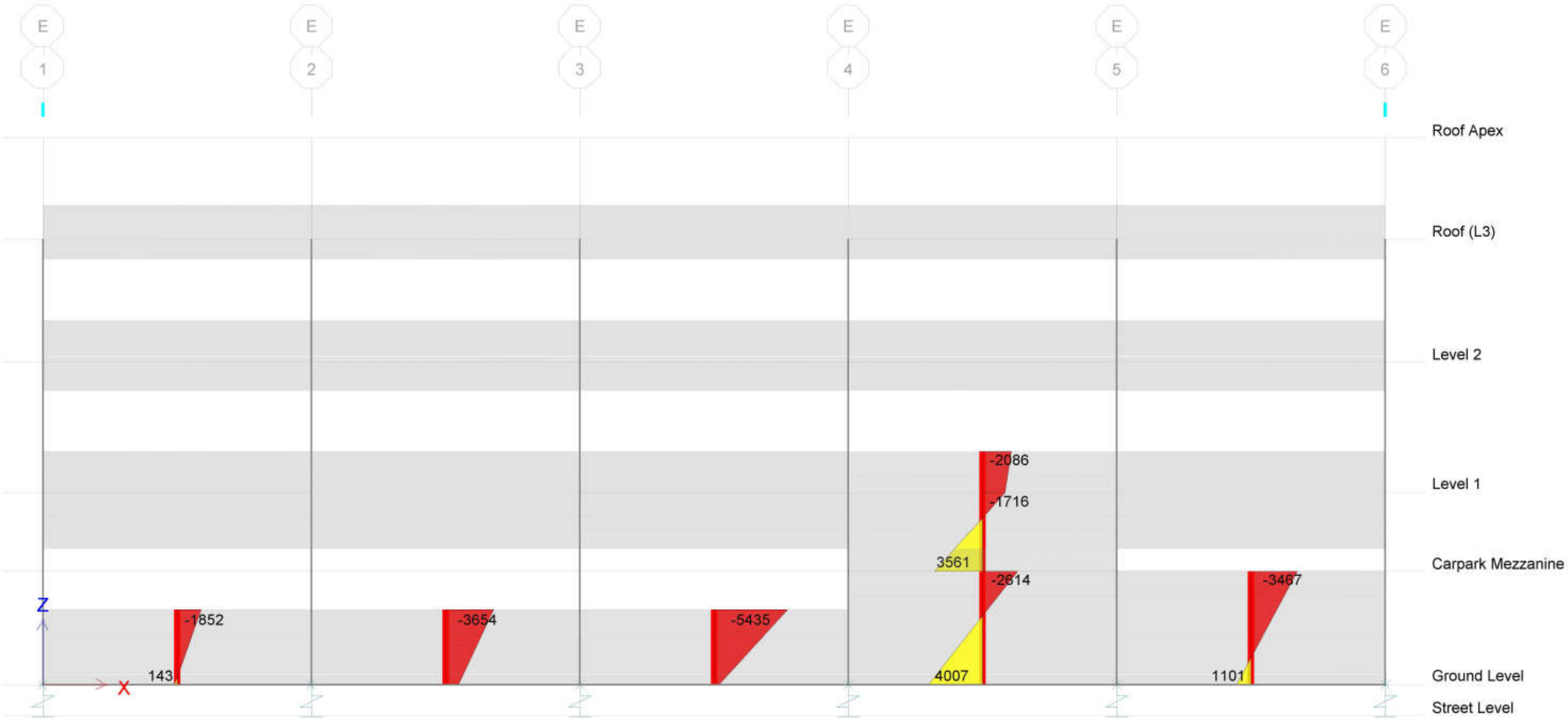


Page 02c.ix


E
1
2
3
4
5
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
-90
-165
-185
-148
-127
-99
110
175
195
156
134
120
-237
-446
-523
-439
-391
-293
278
471
557
463
412
331
-363
-742
-907
-254
-129
-249
184
285
427
790
956
271
118
60

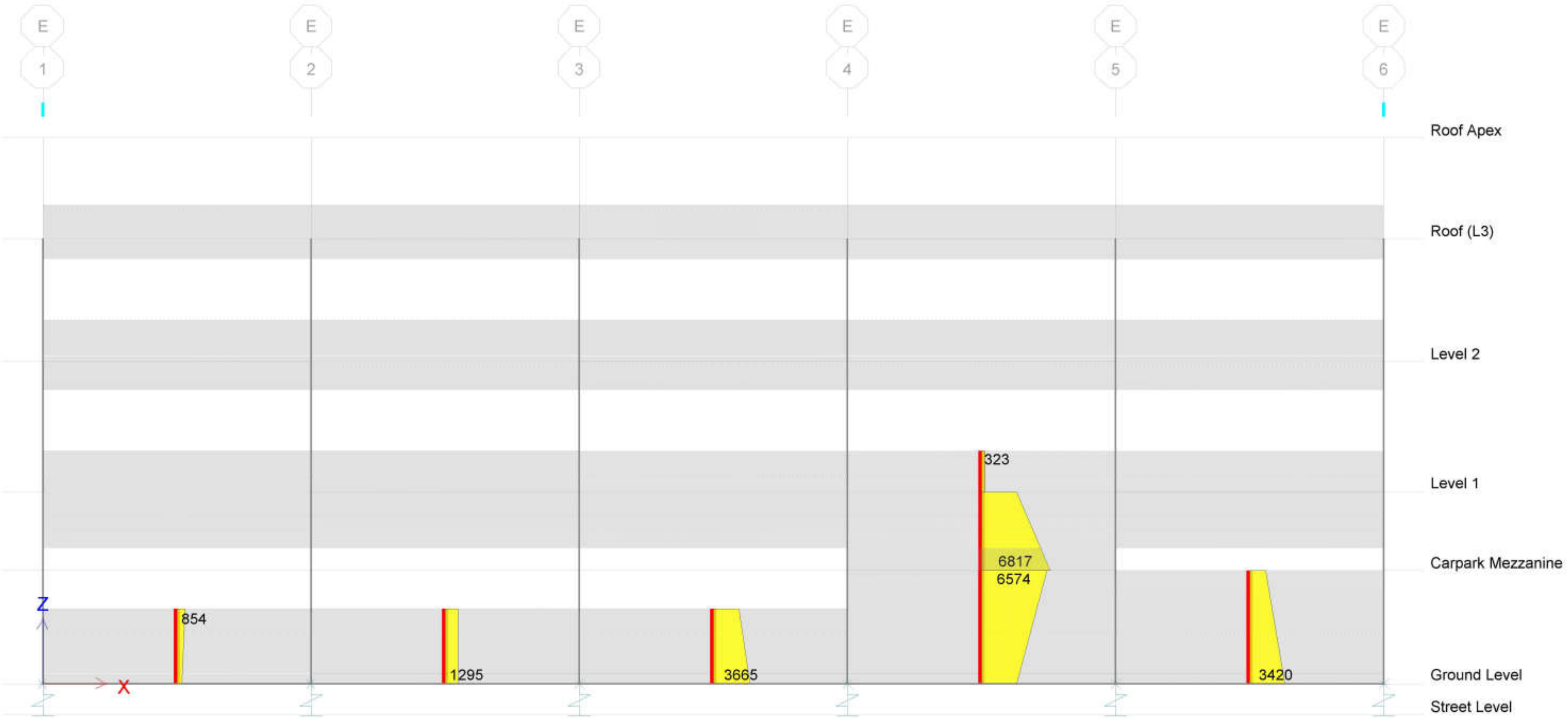
Page 02c.x

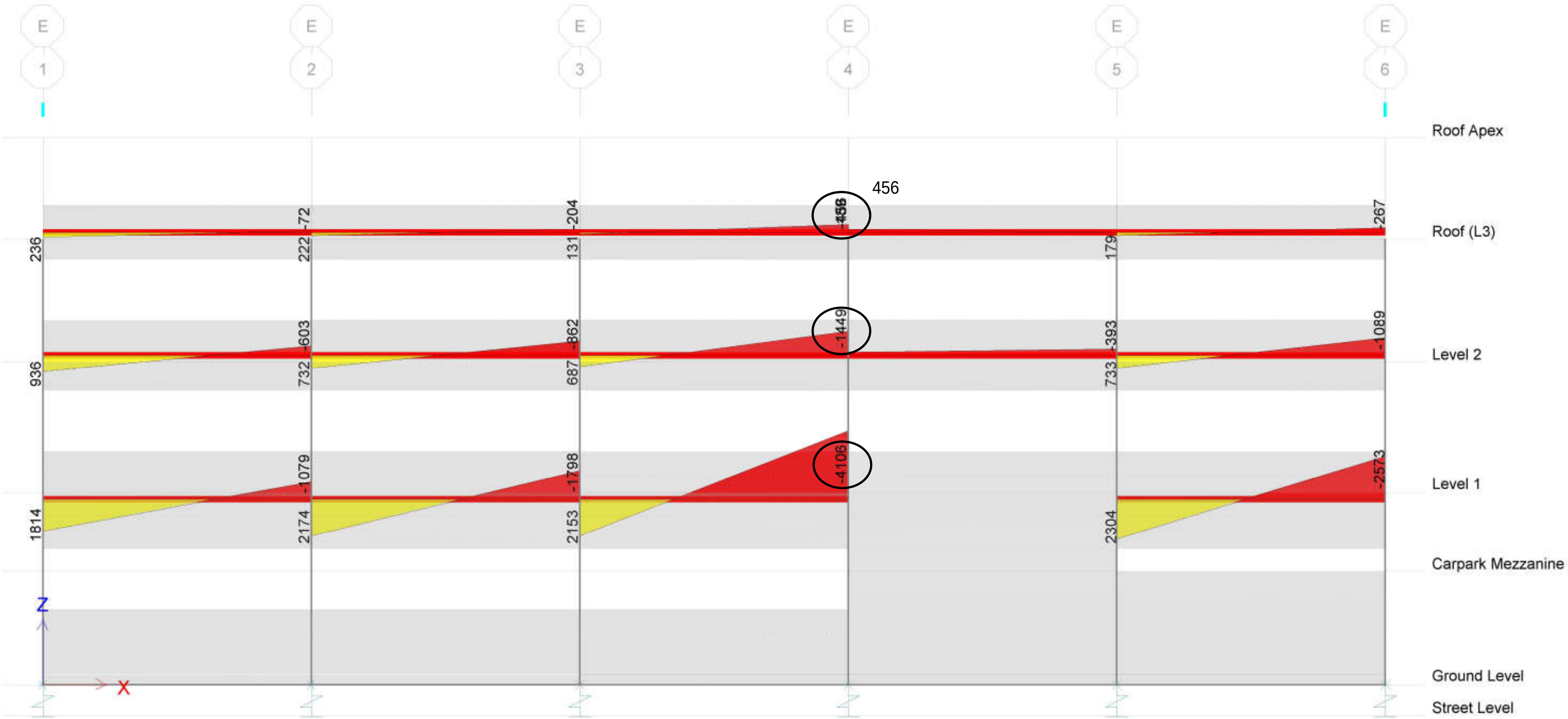

E 1
E 2
E 3
E 4
E 5
E 6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
-48
-49
-56
-45
-38
-53
113
186
207
166
142
123
-105
-124
-145
-123
-109
-127
289
502
590
493
439
351
-7
-97
-268
-322
875
1081
427
468
799
471
-125
149
-82
Z
X



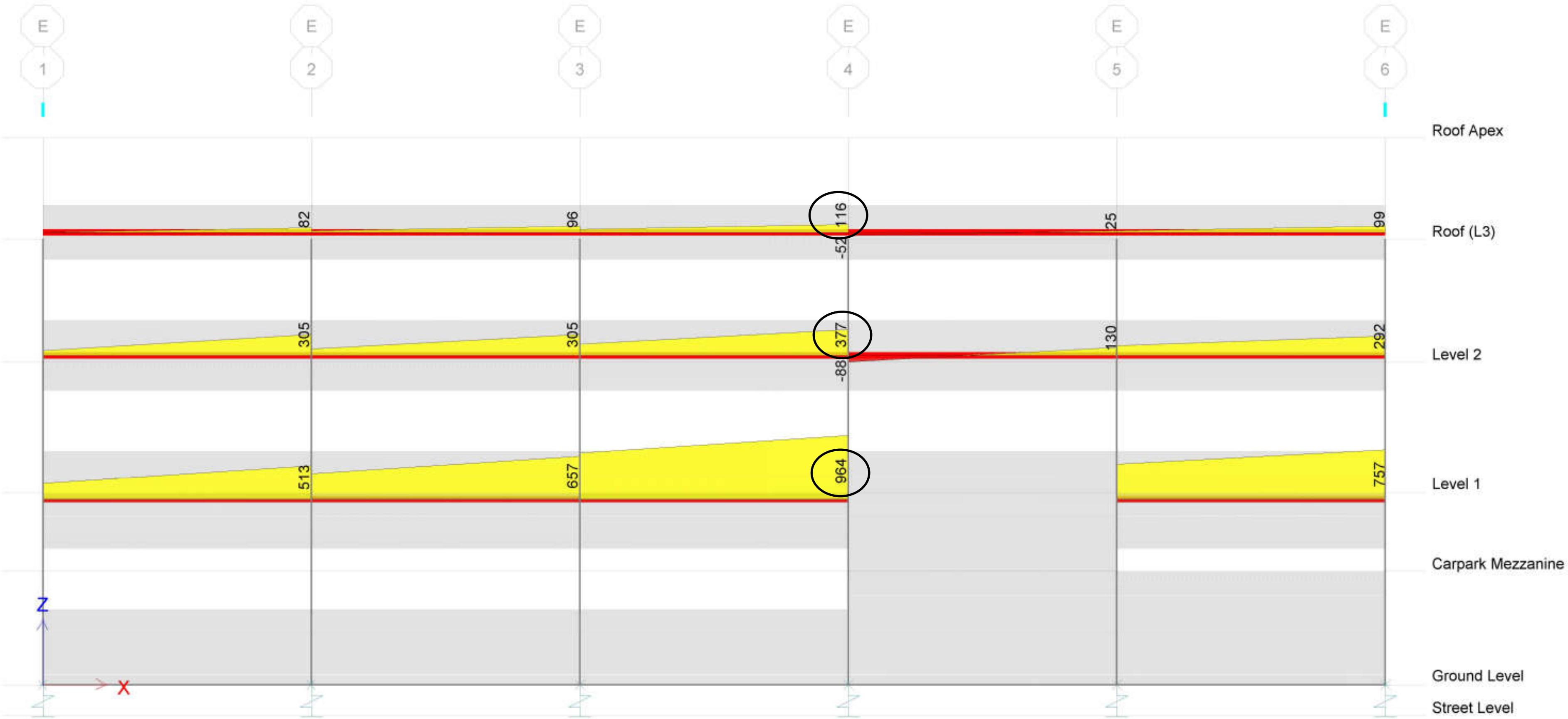
220 Thorndon Quay - Existing Structure.EDB Elevation View - E Moment 3-3 Diagram (Seismic Combo X) [kN-m]

Page 02c.xii


E
1
E
2
E
3
E
4
E
5
E
6
Roof Apex
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
323
6817
6574
854
1295
3665
3420
Z
X

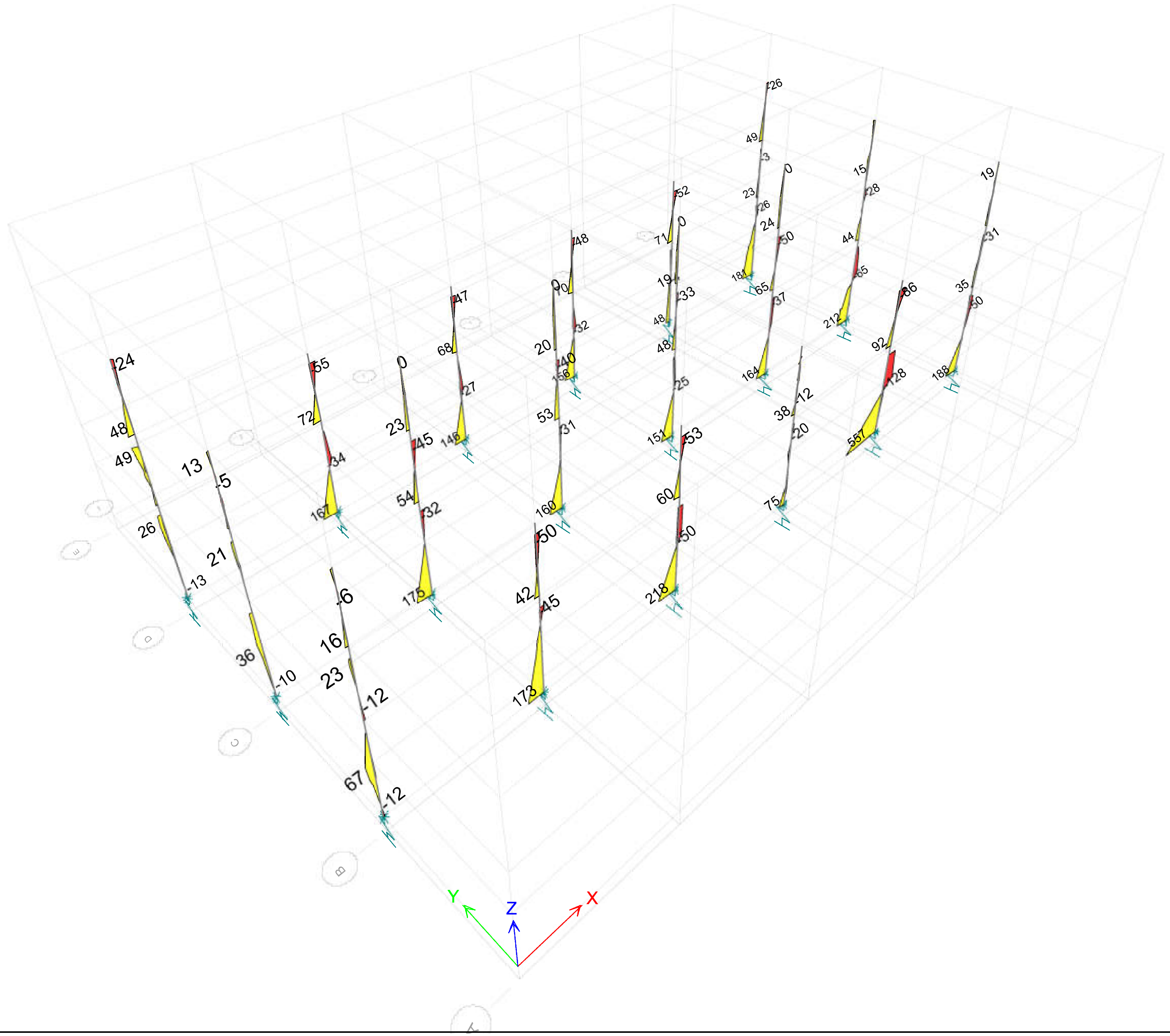


E 1
E 2
E 3
E 4
E 5
E 6
Roof Apex
456
Roof (L3)
Level 2
Level 1
Carpark Mezzanine
Ground Level
Street Level
236
222
-72
131
-204
179
-267
936
732
-603
687
-862
-1449
733
-393
-1089
1814
2174
-1079
2153
-1798
-4106
2304
-2573
Z
X

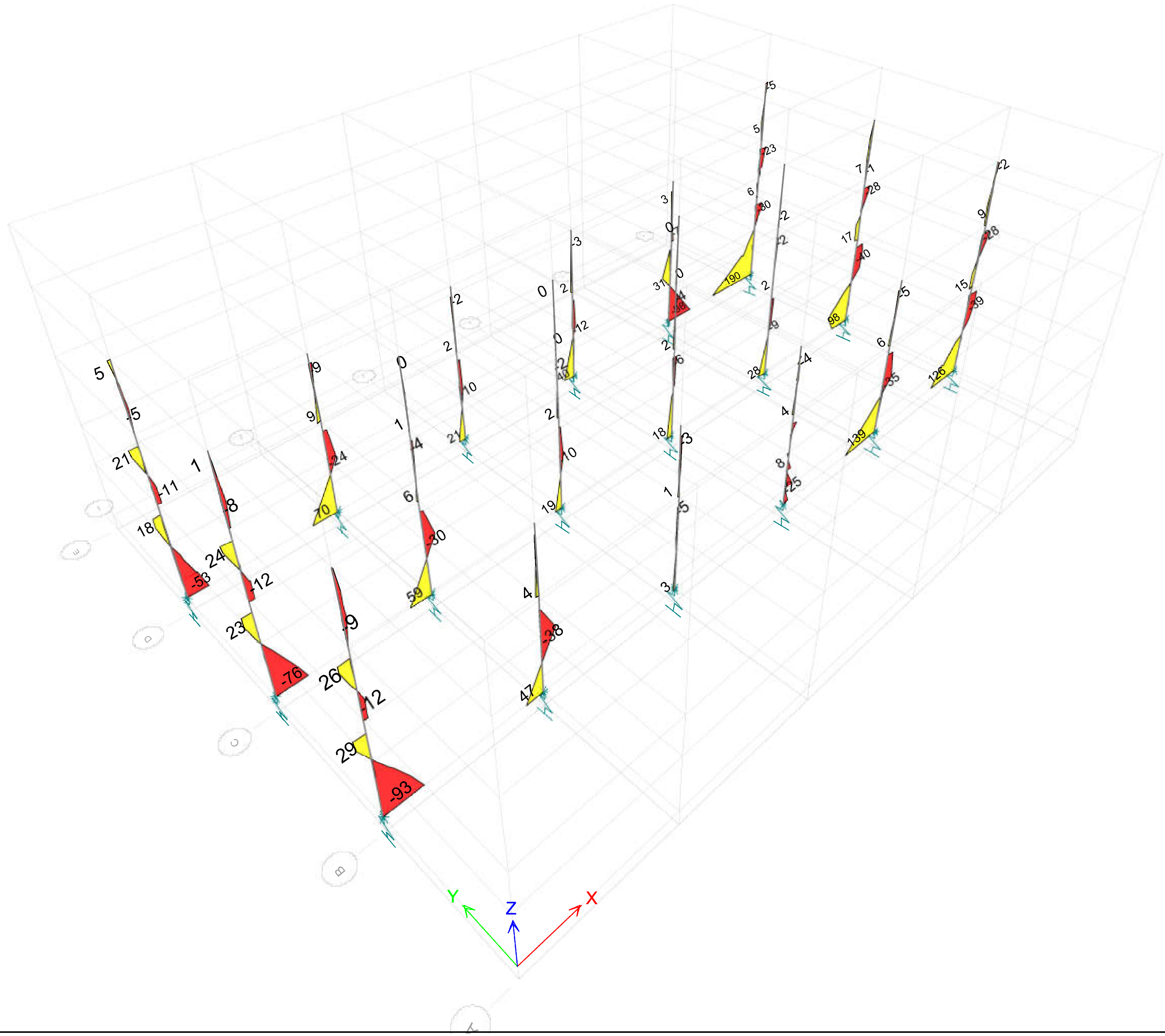


220 Thorndon Quay - Existing Structure.EDB   Elevation View - E   Shear Force 2-2 Diagram   (Seismic Combo X) [kN]

Page 02c.xv



3-D View Moment 3-3 Diagram (Seismic Combo X) [kN-m]



-93
29
-12
26
-9
-76
23
-12
24
-53
18
-11
21
-5
5
-70
-24
-59
-30
47
-38
190
139
126
98
Y
Z
X

MOMENT REDISTRIBUTION CHECKS

NZS3101:2006 Concrete Structures

1.0 - Inelastic Rotation Capacity

Checking whether moment red 3.7

SUPERSEDED
Moment redistribution calc's superseded.
For strengthening purposes, heirarchy of plastic hinge mechanisms has been replaced by worst case scenario. Refer section 5.

Distance from extreme compre
centroid of tension reinforceme

Neutral axis depth

Steel strength multiplier $k := 2.0$

Strain in longitudinal reinforcement closest to tension side of member when compression strain in concrete is 0.003 — $\varepsilon_u := 0.0918$

Yield strain of reinforcement — $\varepsilon_y := 0.0014$

Inelastic rotation capacity — $$\theta_{cap} := \left(\frac{\varepsilon_u - k \cdot \varepsilon_y}{d - c}\right) \cdot \frac{d}{2} = 2.71 \cdot deg$$

2.0 - Rotational Demand

Breadth of section — $b := 150mm$

Depth of section — $h := 2895mm$

Second moment of area of gross section — $$I_g := \frac{b \cdot h^3}{12} = 0.303\,m^4$$

Effective second moment of area (monet of inertia) of cracked section about centroidal axis — $I_{cr} := 0.43 \cdot I_g = 0.13\,m^4$ — Refer table C6.5

Elastic bending moment on section — $M_e := 1988kN \cdot m \cdot 50\%$

Bending moment used to determine ultimate strength — $M_{ULS} := 653.4kN \cdot m$

Increment of redistributed moment on section — $$M_i := \frac{M_e - M_{ULS}}{0.85} = 401 \cdot kN \cdot m$$

Youngs modulus of reinforced concrete — $E := 25GPa$

Radius of curvature — $$R := \frac{E}{\left(\frac{M_i}{I_{cr}}\right)} = 8.137 \times 10^3\,m$$

Arc length - taking as half beam length — $A := 4m$

Rotational demand — $$\theta_{dem} := \frac{180 \cdot A}{R \cdot \pi} = 1.61 \cdot deg$$

$$\text{Check} := \begin{cases} \text{"OK"} & \text{if } \theta_{cap} \geq \theta_{dem} \\ \text{"Distribution not permitted"} & \text{otherwise} \end{cases}$$

$\text{Check} = \text{"OK"}$

2.0 - Deemed to Comply Checks

As per clause 6.3.7.2
(a) Members are of uniform depth
(b) Elastic moments comply with stiffness requirements
(c) Equilibrium is maintained
(d) n/a for assessment of existing structure
(e) Reduction limit

$\Delta_{max} := 30\%$

Refer excel calcs for redistributed values

SUPERSEDED
Moment redistribution calc's superseded.
For strengthening purposes, heirarchy of plastic hinge mechanisms has been replaced by worst case scenario. Refer section 5.

Reinforced Concrete Design: Elastic

Wall Reference: Spandrel analysis for moment distribution assessment
Floor: East face, L1 spandrels

SUPERSEDED
Moment redistribution calc's superseded.
For strengthening purposes, heirarchy of plastic hinge mechanisms has been replaced by worst case scenario. Refer section 5.

Wall data:
b =
L =

Vertical Reinforcing:
E_s =
f_y =

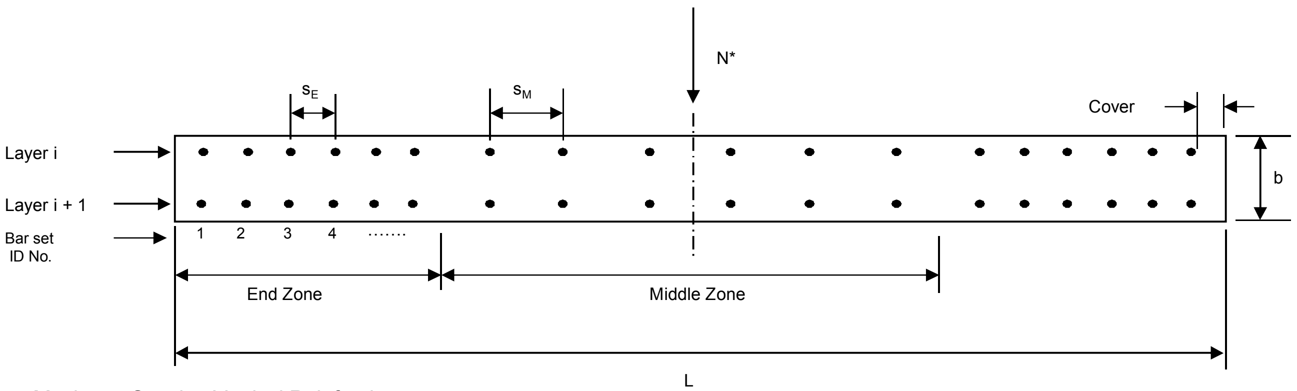
Zone	d_b mm	Number layers	s mm	A/s mm^2/m
End Zone	32	2	200	8042
Middle Zone	10	1	450	175

p_{min} <	p	< p_{max}	Steel ratio check
0.0048	0.0536	0.0563	OK
0.0048	0.0012	0.0563	Outside limits

Note:
Steel ratio's OK as assessing existing structure

$d_{b\ min}$ = 10 mm cl 11.3.11.2 (b)
$d_{b\ max}$ = b / 7 cl 11.3.11.2 (d)
= 21.4 mm

p_{min} = max [$0.7/f_y$, $\sqrt{f_c'}/(4f_y)$] cl 11.3.10.3.8
p_{max} = $16/f_y$ cl 11.3.11.3



Maximum Spacing Vertical Reinforcing

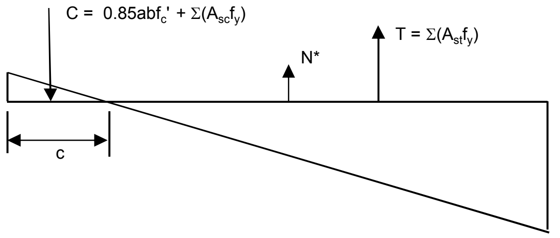
s_{max} = min [L/3 , 3b , 450] mm
= 450 mm

cl 11.3.11.3

Moment Calculation

Sum Tension Forces = $N^* + \Sigma(A_{st} f_y)$
= 568 kN

Sum Compression Forces = $0.85abf_c' + \Sigma(A_{sc} f_y)$
= 568 kN



Equate forces and solve for c:

c = 86 mm

a = 0.85c

Vertical Reinforcing Design

M^* = 0 kNm
ϕ = 1
ϕM_n = 1391 kNm

Check: $M^* < \phi M_n$
OK

Strains:
ε_{conc} = 0.0030
ε_{yield} = 0.0014

Moment Calculation Table

Bar Set ID No.	Force Position mm	Bar Diam mm	No. Bars	Strain	Force kN	Moment kNm
1	60	32	2	-0.0009	-290	-17
2	510	10	1	0.0148	22	11
3	960	10	1	0.0306	22	21
4	1410	10	1	0.0463	22	31
5	1860	10	1	0.0621	22	41
6	2310	10	1	0.0778	22	52
7	2760	32	2	0.0936	457	1261
8	0	0	0	0.0000	0	0
9	0	0	0	0.0000	0	0
10	0	0	0	0.0000	0	0
11	0	0	0	0.0000	0	0
12	0	0	0	0.0000	0	0
13	0	0	0	0.0000	0	0
14	0	0	0	0.0000	0	0
15	0	0	0	0.0000	0	0
16	0	0	0	0.0000	0	0
17	0	0	0	0.0000	0	0
18	0	0	0	0.0000	0	0
19	0	0	0	0.0000	0	0
20	0	0	0	0.0000	0	0
21	0	0	0	0.0000	0	0
22	0	0	0	0.0000	0	0
23	0	0	0	0.0000	0	0
24	0	0	0	0.0000	0	0
25	0	0	0	0.0000	0	0
26	0	0	0	0.0000	0	0

SUPERSEDED
Moment redistribution calc's superseded.
For strengthening purposes, heirarchy of plastic hinge mechanisms has been replaced by worst case scenario. Refer section 5.

Moment Redistribution - Spandrels

Grid A, Existing Structure

SUPERSEDED
Moment redistribution calc's superseded.
For strengthening purposes, heirarchy of plastic hinge mechanisms has been replaced by worst case scenario. Refer section 5.

		1								Max	Final	
3	-									93	493	kN.m
	+	316								21		
2	-									42	842	kN.m
	+	778								45		
1	-		677	524	1049	1097	944	858	768	858	858	kN.m
	+	663	278	461		369		443	464	464		

Grid E, Existing Structure

		1	2	3	4	5	6	μ	Limit	Max	Final	
3	-		171	184	277	109	314	211	220	220	220	kN.m
	+	260	118	118	7	174		135	182	182		
2	-		794	751	1042	48	1262	779	883	883	883	kN.m
	+	971	458	596		775		700	680	700		
1	-		1417	1268	2541		2712	1985	1898	1985	1985	kN.m
	+	1696	1183	1543		2514		1734	1760	1760		

Refer ETABS bending moment diagrams, pages 02c.v and 02c.ix

Taking maximum value of the following:

- mean positive moment,
- mean negative moment,
- maximum poisitve moment with 30% reduction
- maximum negative moment with 30% reduction

Final values represent expected bending moment post spandrel softening due to flexural cracking.

ETABS MODELLING - WINKLER SOIL SPRINGS

Seismic Assessment of Existing Buildings, C5, July 2017 (incl. March 2018 Errata)
Foundation Analysis and Design, J.E. Bowles, 1988

Assessing soil-structure interaction. Existing plans contain limited detail of piles; assessing multiple scenarios for most conservative analysis. Refer ETABS comparison diagrams.

1.0 - Slab Bearing

Tributary areas

$$a_1 := 7.62\mathrm{m}\cdot 8.0\mathrm{m} = 60.96\,\mathrm{m}^2$$

$$a_2 := 0.5\cdot a_1 = 30.48\,\mathrm{m}^2$$

$$a_3 := 0.5\cdot a_2 = 15.24\,\mathrm{m}^2$$

Modulus of subgrade reaction

$$\lambda := 36000\frac{\mathrm{kN}}{\mathrm{m}^3}$$

Refer ABUILD report; Geotechnical Investigation - Bank Stability - 231 Thorndon Quay.
Note that results have been taken from the appropriate depth so as to represent resistance below 220 Thorndon Quay.

Winkler vertical spring stiffnessesInternal

$$S_1 := \lambda\cdot a_1 = 2.2\times 10^6\cdot\frac{\mathrm{kN}}{\mathrm{m}}$$

Perimeter

$$S_2 := \lambda\cdot a_2 = 1.1\times 10^6\cdot\frac{\mathrm{kN}}{\mathrm{m}}$$

Corner

$$S_3 := \lambda\cdot a_3 = 5.5\times 10^5\cdot\frac{\mathrm{kN}}{\mathrm{m}}$$

Soil Type	Modulus of Subgrade Reaction
Loose sand:	4800 - 16000 kN/m³
Medium dense sand:	9600 – 80000 kN/m³
Dense sand:	64000 – 128000 kN/m³
Clayey medium dense sand:	32000 – 80000 kN/m³
Silty medium dense sand:	24000 – 48000 kN/m³
Clayey soil with q_u < 200 kPa:	12000 – 24000 kN/m³
Clayey soil with q_u in range 200 to 400 kPa:	24000 – 48000 kN/m³
Clayey soil with q_u > 800 kPa:	> 48000 kN/m³

Above: guideline values extracted from J.E. Bowles, "Foundation Analysis and Design", McGraw Hill 1988

2.0 - Pile Compression - Stiff

Pile depth to refusal (estimated)

$$d := 4.0\mathrm{m}$$

Elastic Modulus - Concrete (probable)

$$E_c := 200000\mathrm{MPa}$$

Axial shortening

$$\Delta := 1\mathrm{mm}$$

Gross pile area - internals

$$A_{gi} := 5\cdot\frac{\pi\cdot(300\mathrm{mm})^2}{4} = 0.353\,\mathrm{m}^2$$

Gross pile area - perimeter

$$A_{gp} := 3\cdot\frac{\pi\cdot(300\mathrm{mm})^2}{4} = 0.212\,\mathrm{m}^2$$

Winkler vertical spring value - internals $P_{st1} := \frac{\Delta \cdot A_{gi} \cdot E_c}{d \cdot mm} = 1.767 \times 10^7 \frac{kN}{m}$

Winkler vertical spring value - perimeter $P_{st2} := \frac{\Delta \cdot A_{gp} \cdot E_c}{d \cdot mm} = 1.06 \times 10^7 \frac{kN}{m}$

3.0 - Pile Compression - Soft

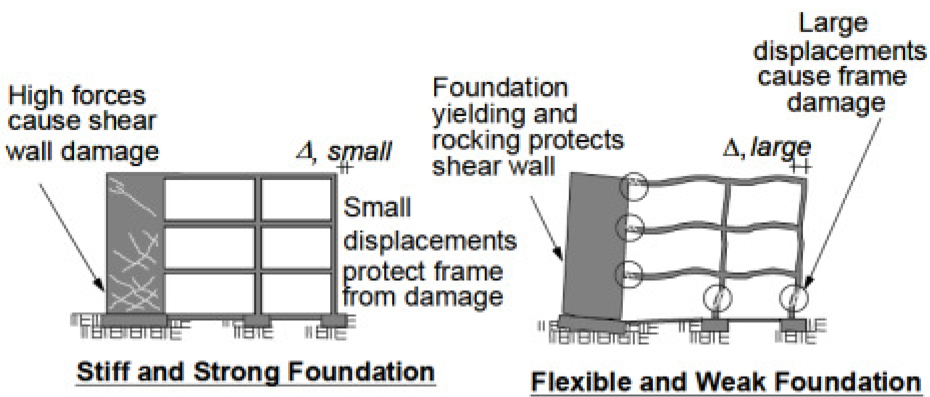
Axial shortening $\Delta := 0.5mm$

Winkler vertical spring value $P_{so1} := \frac{\Delta \cdot A_{gi} \cdot E_c}{d \cdot mm} = 8.836 \times 10^6 \frac{kN}{m}$

Winkler vertical spring value $P_{so2} := \frac{\Delta \cdot A_{gp} \cdot E_c}{d \cdot mm} = 5.301 \times 10^6 \frac{kN}{m}$

4.0 - Final Values

ETABS comparison of the above values show that soil bearing values produce the most severe frame reactions, due to softer springs allowing larger rotation of shear walls, causing frames to take majority of load (refer diagram below). Taking winkler vertical spring values from section 1.0. Note that as this does not account for piles, these values represent a lower bound of expected reactions. Conservative.

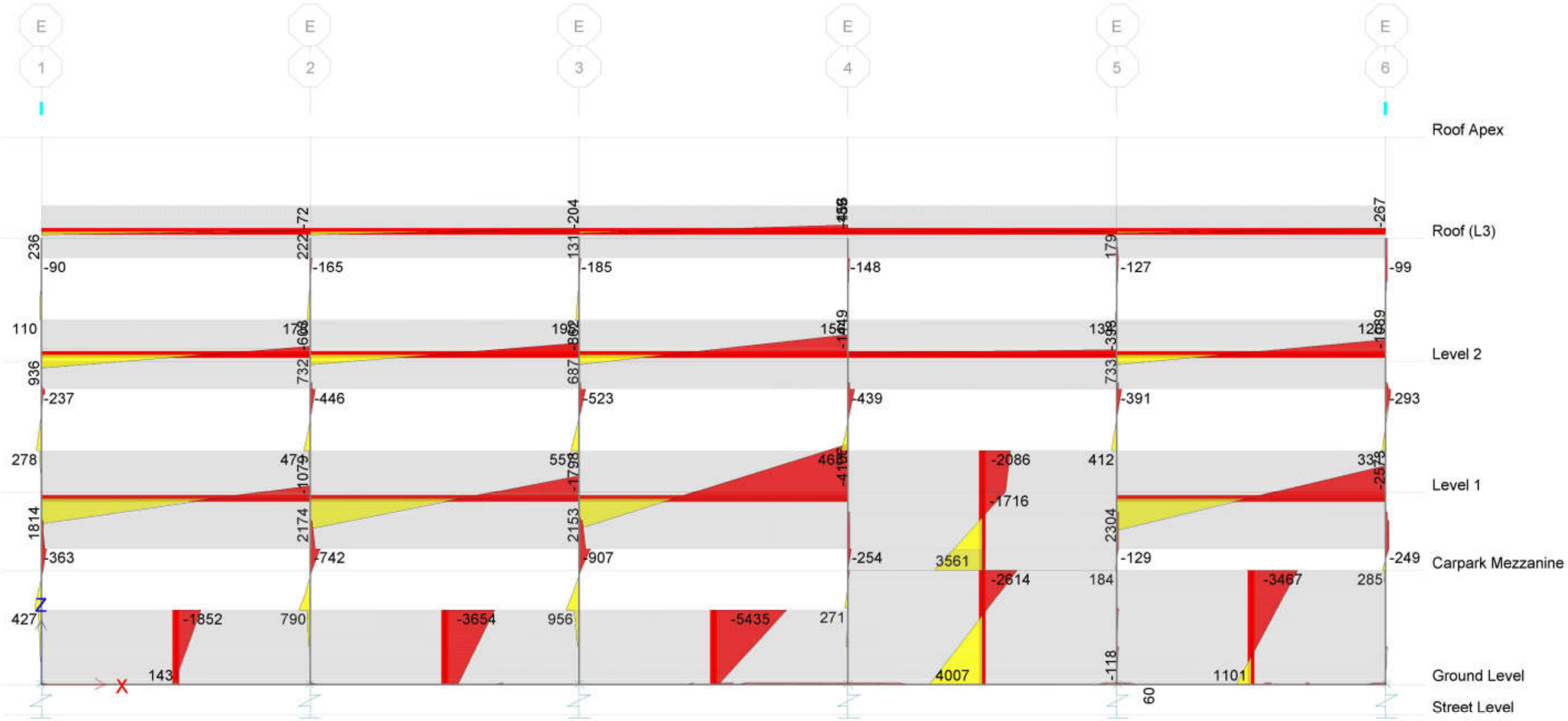


Above: Fig C4.4, Seismic Assessment of Existing Buildings, July 2017

SSI Model: Slab Bearing.
Worst case, governs.

220 Thorndon Quay - Existing Structure - Slab Bearing.EDB Elevation View - A Moment 3-3 Diagram (Seismic Combo X) [kN-m]

SSI Model: Slab Bearing.
Worst case, governs.



220 Thorndon Quay - Existing Structure - Slab Bearing.EDB Elevation View - E Moment 3-3 Diagram (Seismic Combo X) [kN-m]

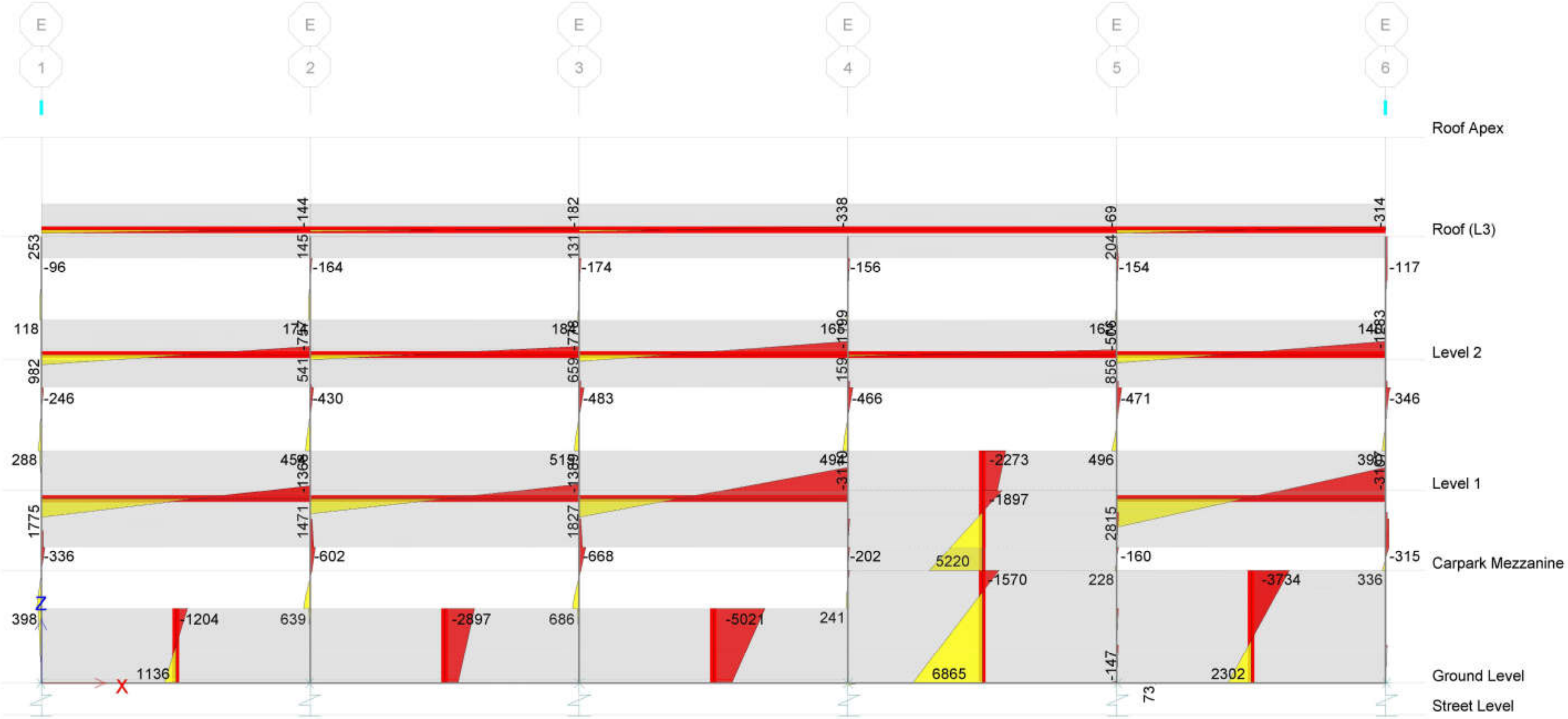
SSI Model: Soft Piles.
Does not govern.

Page 02h.iii

220 Thorndon Quay - Existing Structure - Soft Piles.EDB Elevation View - A Moment 3-3 Diagram (Seismic Combo X) [kN-m]

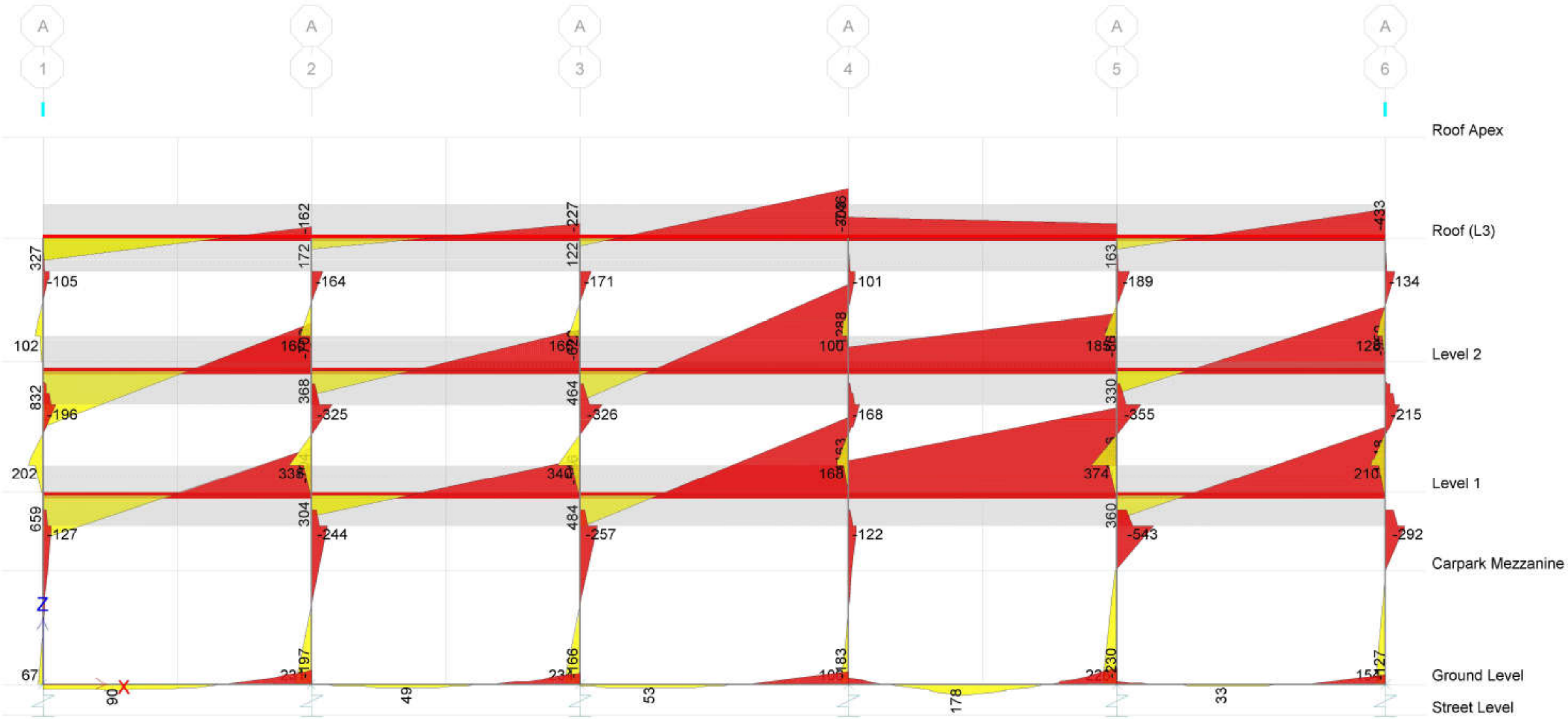
SSI Model: Soft Piles.
Does not govern.

Page 02h.iv



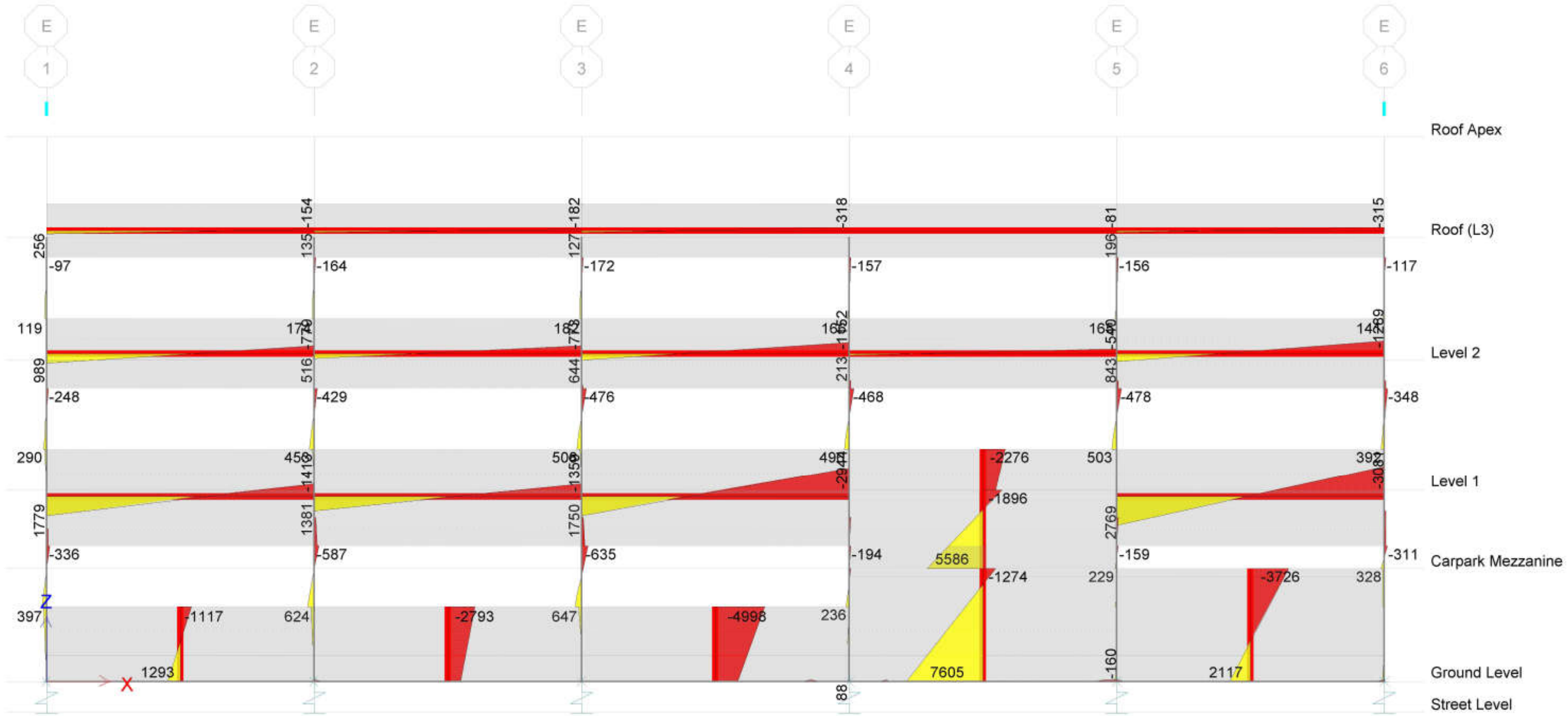
220 Thorndon Quay - Existing Structure - Soft Piles.EDB Elevation View - E Moment 3-3 Diagram (Seismic Combo X) [kN-m]

SSI Model: Stiff Piles.
Does not govern.



SSI Model: Stiff Piles.
Does not govern.

Page 02h.vi

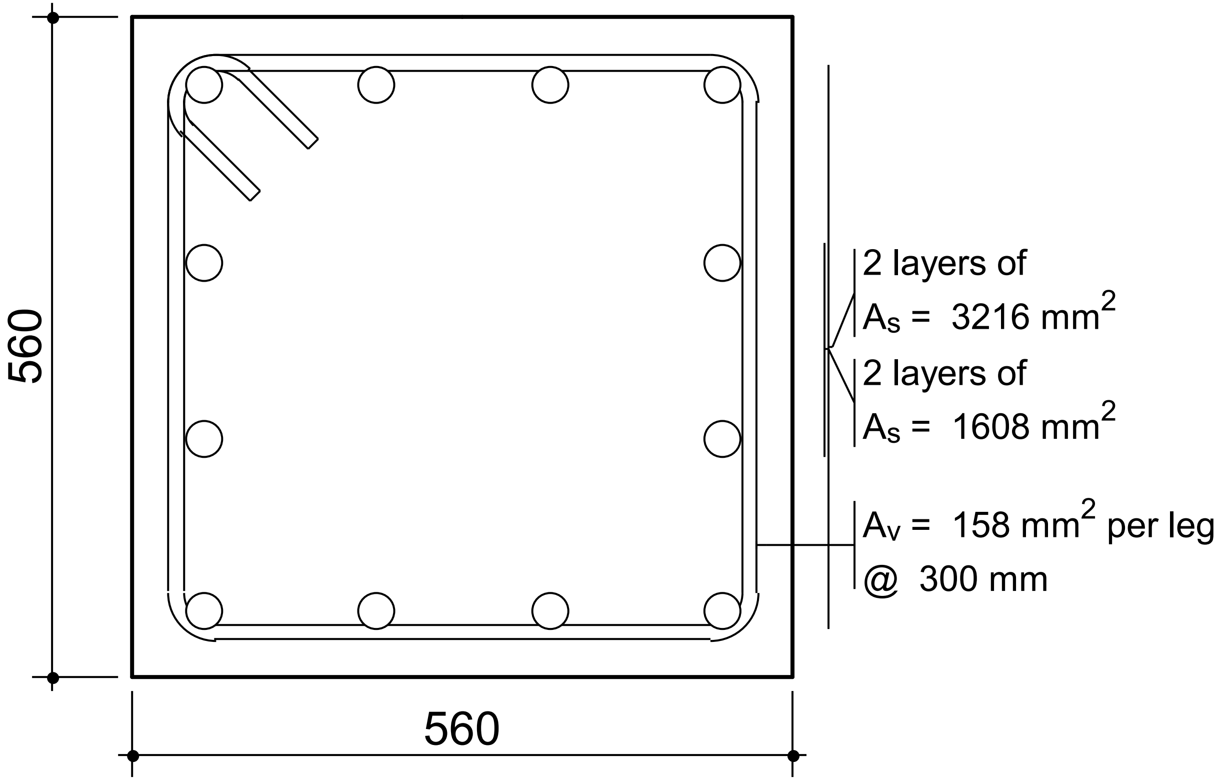


COLUMN FLEXURAL ASSESSMENTS

Refer lower RHS of page for column designation.
Taking axial contributions as zero (conservative simplification).
Multiple shear links have been simplified to single link of equivalent cross section.
Refer over page for interaction diagram and final capacity.
Refer Section 03b for shear assessments.

Geometric Properties

	Gross Conc.	Trans (n=7.97)
Area (mm^2) x 10^3	313.6	380.9
Inertia (mm^4) x 10^6	8195.4	10552.0
y_t (mm)	280	280
y_b (mm)	280	280
S_t (mm^3) x 10^3	29269.3	37670.0
S_b (mm^3) x 10^3	29269.3	37701.7



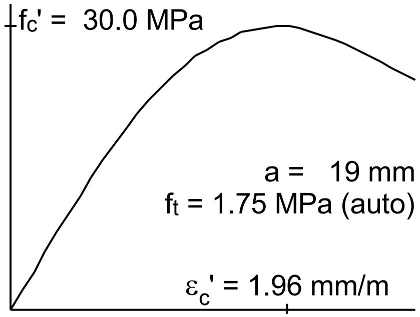
Crack Spacing

2 x dist + 0.1 d_b /ρ

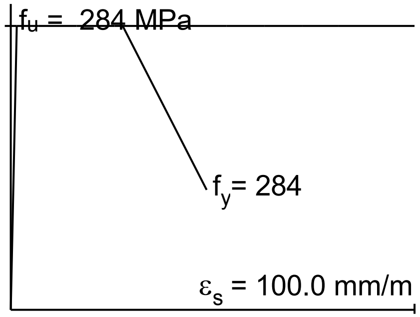
Loading (N,M,V + dN,dM,dV)

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

Concrete



Rebar

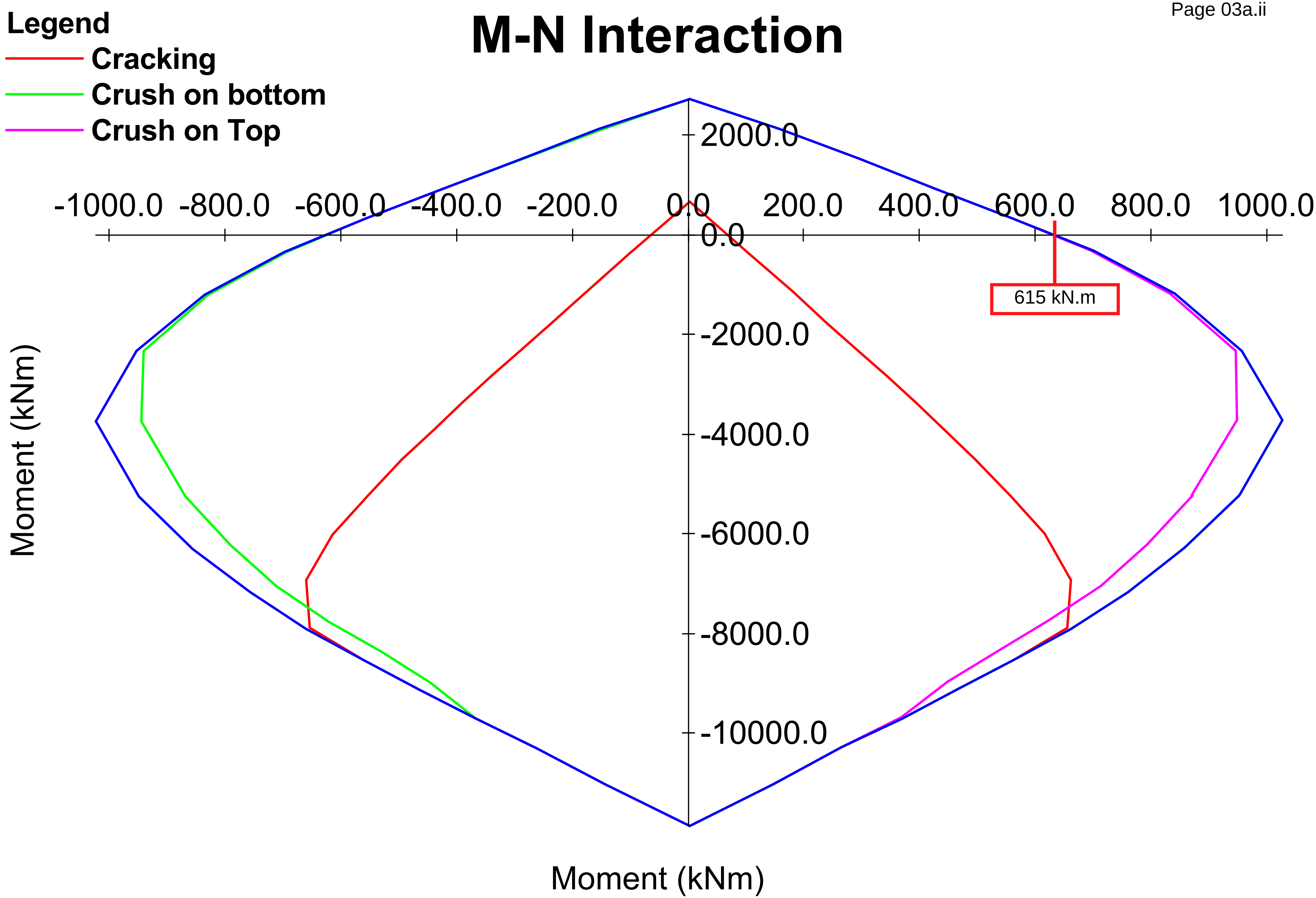


All dimensions in millimetres
Clear cover to transverse reinforcement = 30 mm

LG - L1 Cols

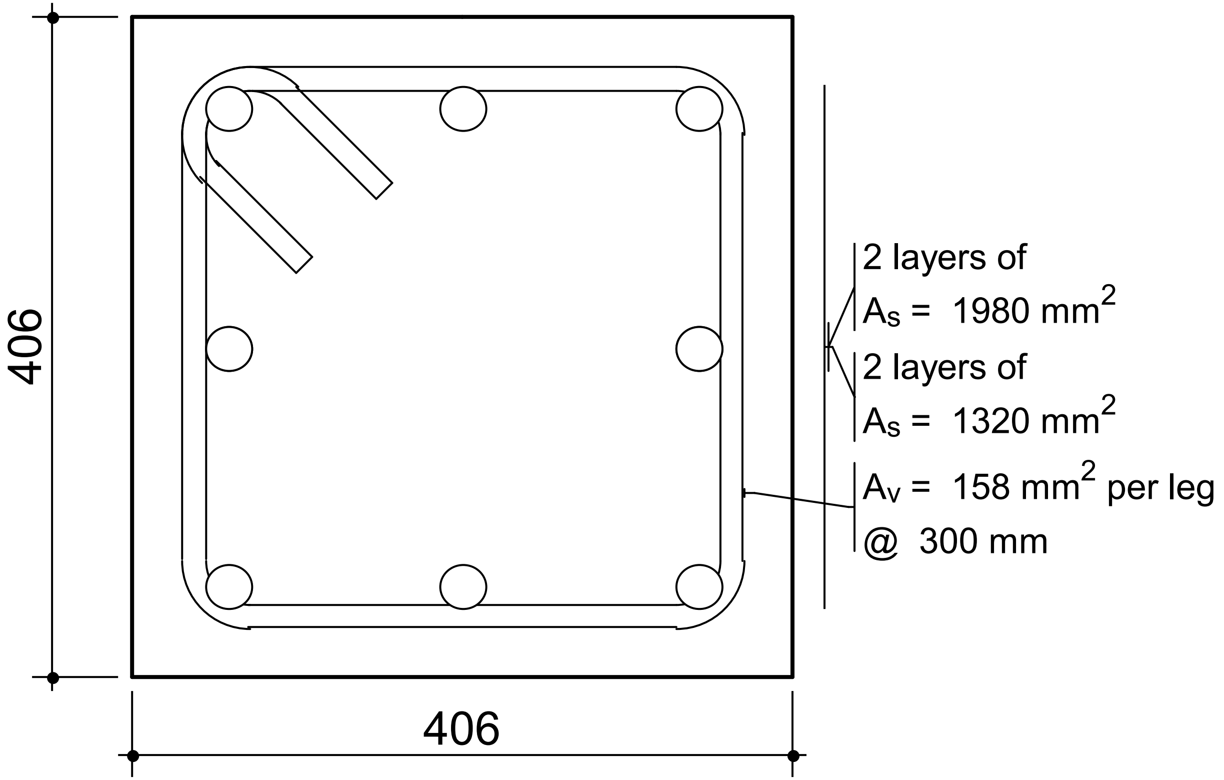
MHT **2019/4/23**

M-N Interaction



Geometric Properties

	Gross Conc.	Trans (n=7.97)
Area (mm^2) x 10^3	164.8	210.9
Inertia (mm^4) x 10^6	2264.2	2860.9
y_t (mm)	203	203
y_b (mm)	203	203
S_t (mm^3) x 10^3	11153.9	14093.3
S_b (mm^3) x 10^3	11153.9	14093.3



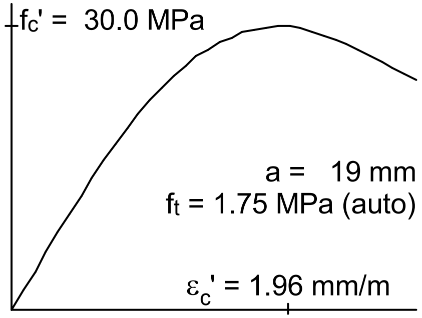
Crack Spacing

2 x dist + 0.1 d_b / ρ

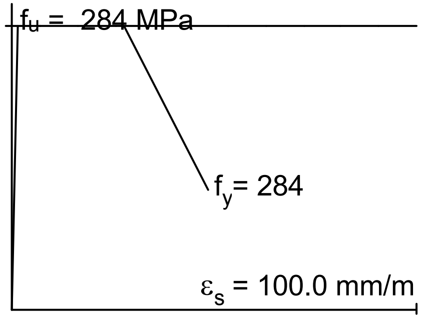
Loading (N,M,V + dN,dM,dV)

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

Concrete



Rebar



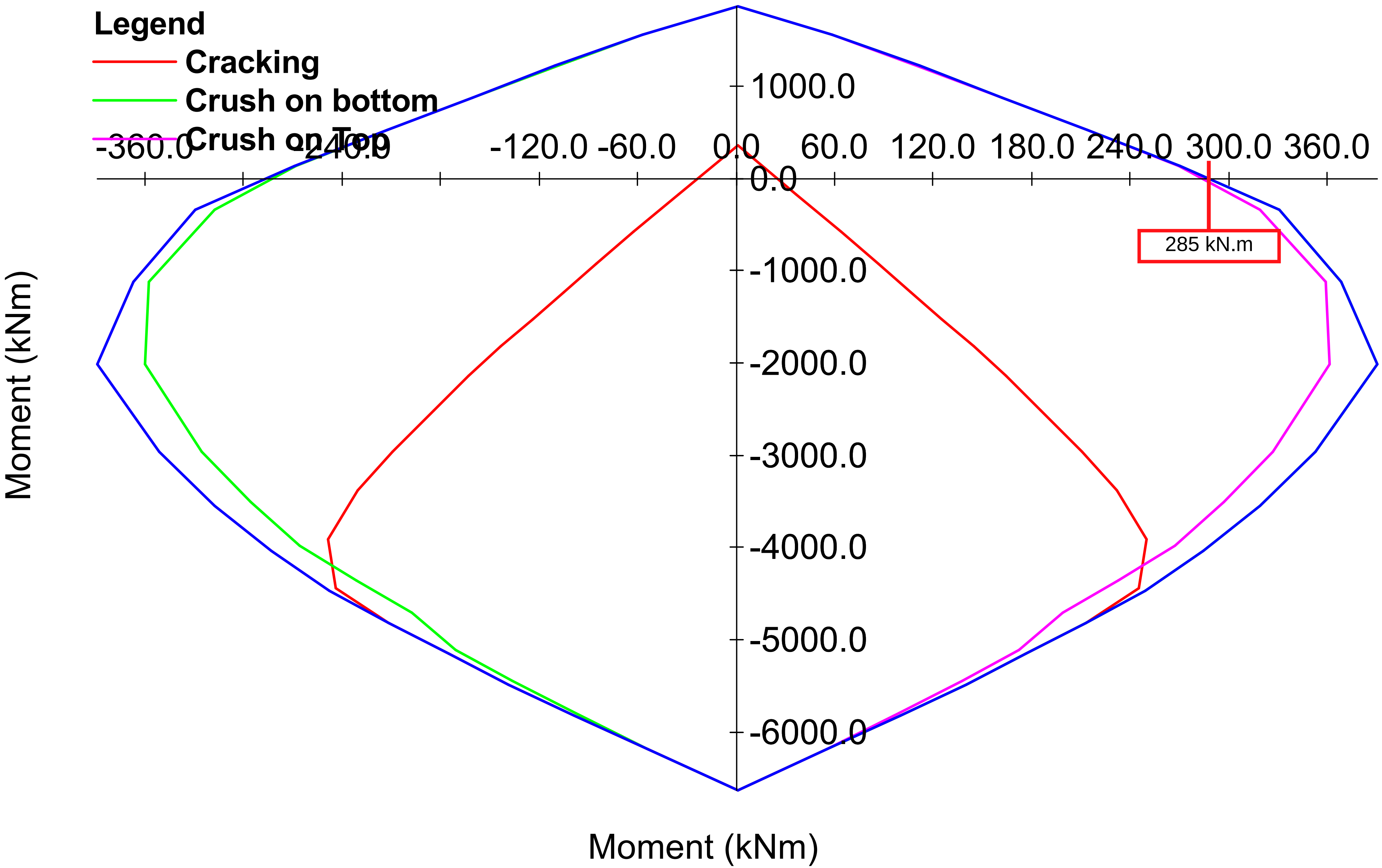
All dimensions in millimetres

Clear cover to transverse reinforcement = 30 mm

L1 - L2 Cols

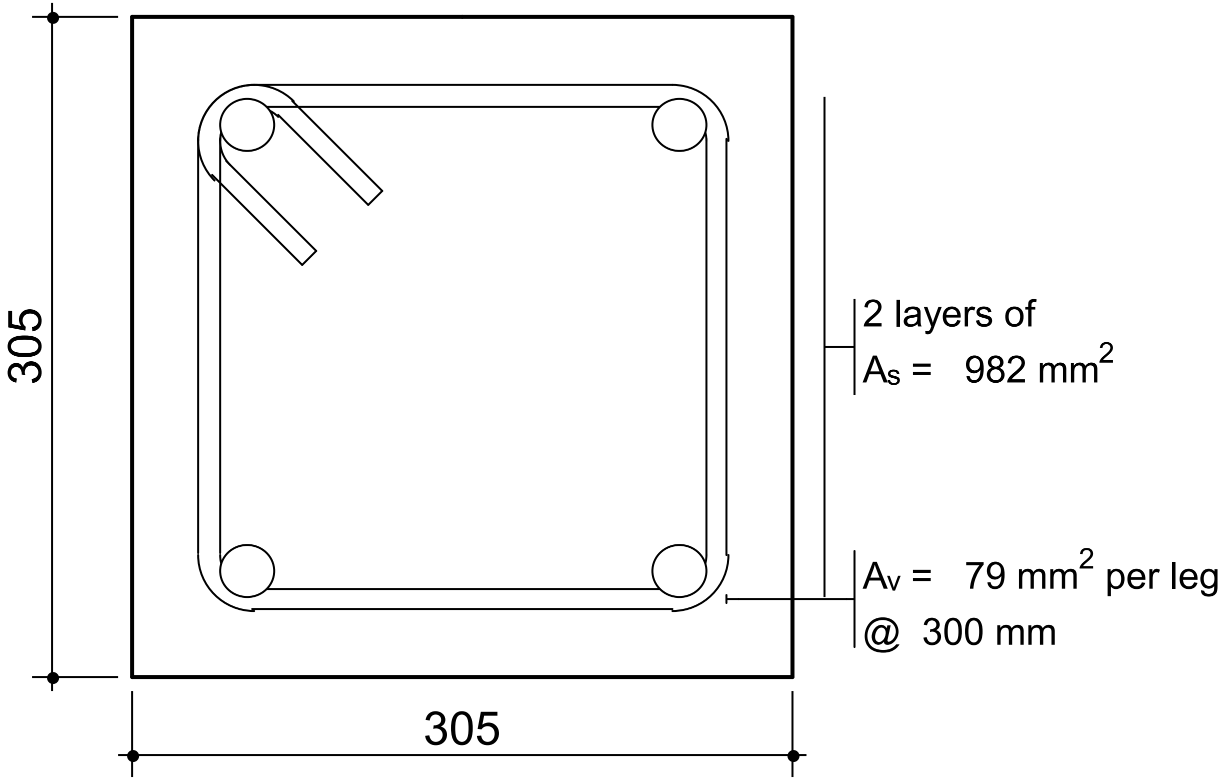
2019/4/23

M-N Interaction



Geometric Properties

	Gross Conc.	Trans (n=7.97)
Area (mm^2) x 10^3	93.0	106.7
Inertia (mm^4) x 10^6	721.1	867.8
y_t (mm)	153	153
y_b (mm)	153	153
S_t (mm^3) x 10^3	4728.8	5690.8
S_b (mm^3) x 10^3	4728.8	5690.8



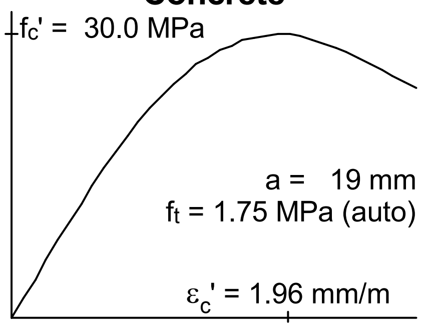
Crack Spacing

2 x dist + 0.1 d_b /ρ

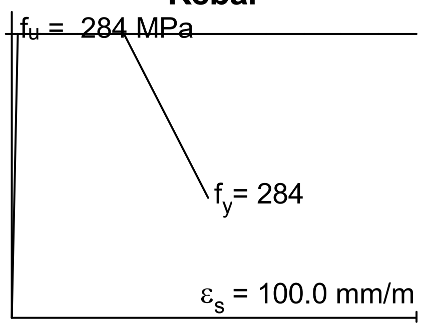
Loading (N,M,V + dN,dM,dV)

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

Concrete



Rebar



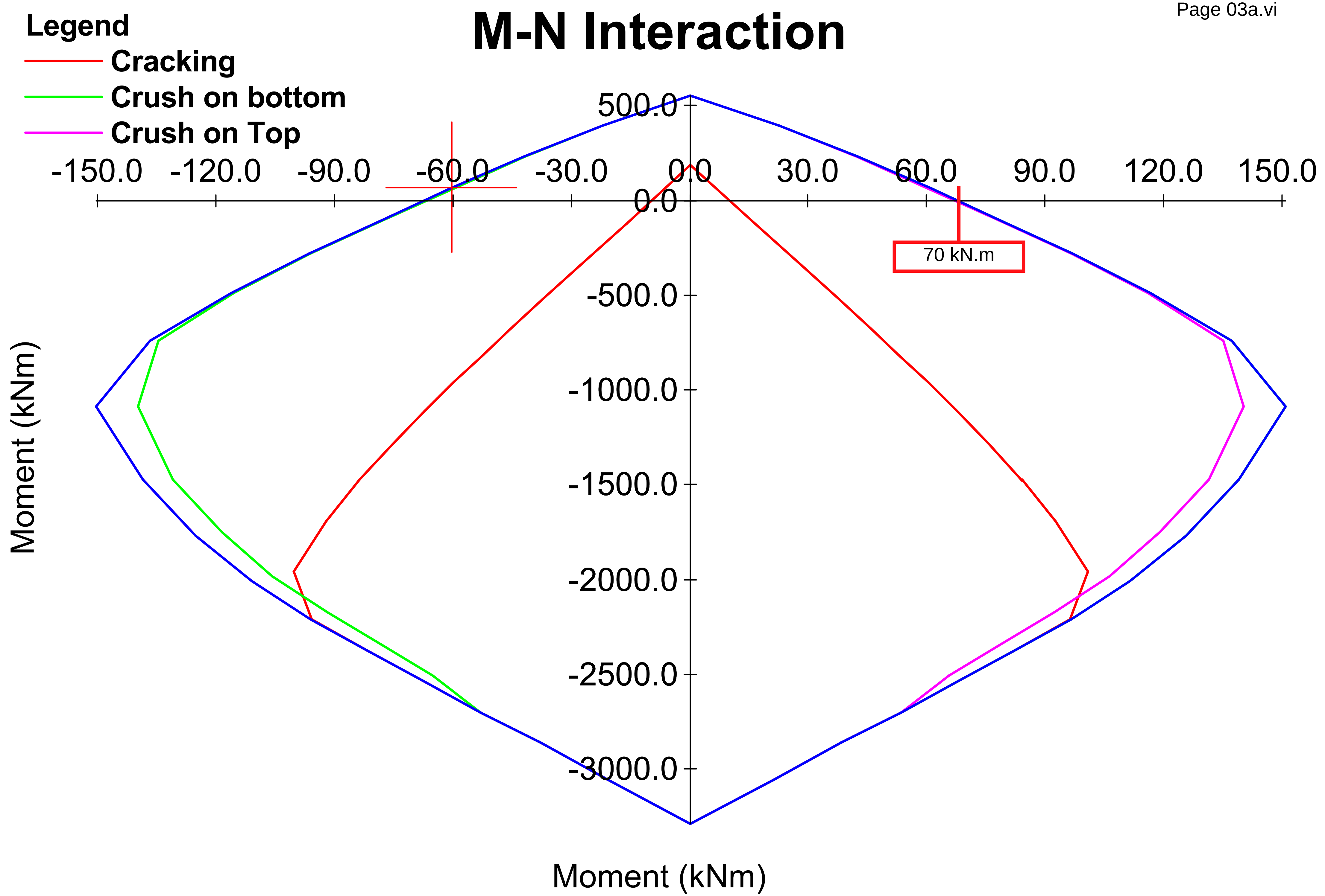
All dimensions in millimetres

Clear cover to transverse reinforcement = 30 mm

L2 - L3 Cols

2019/4/23

M-N Interaction



COLUMN SHEAR CAPACITY : EAST FRAME, LG-L1, 6E

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

1.0 - General

1.1 - Section Properties

Breadth $b := 560\text{mm}$

Depth $d := 560\text{mm}$

Height $h := 2.345\text{m}$

Probable concrete strength $f_c := 30\text{MPa}$

Probable yield strength of steel $f_y := 284\text{MPa}$

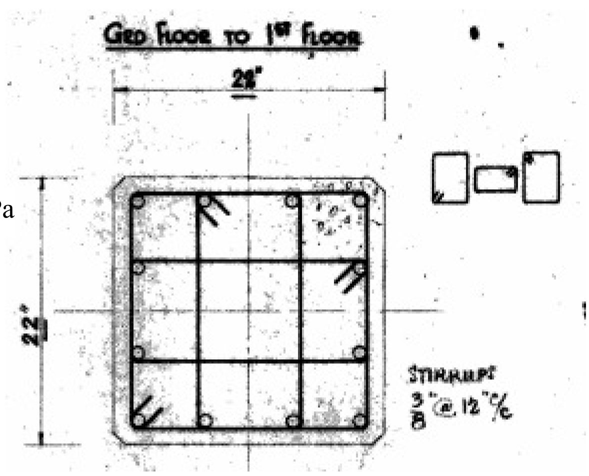
Longitudinal bar diameter $d_1 := 32\text{mm}$

Number of longitudinal bars $n_1 := 12$

Stirrup diameter $d_2 := 10\text{mm}$

Number of stirrup legs per set $n_2 := 4$

Stirrup spacing $s := 300\text{mm}$



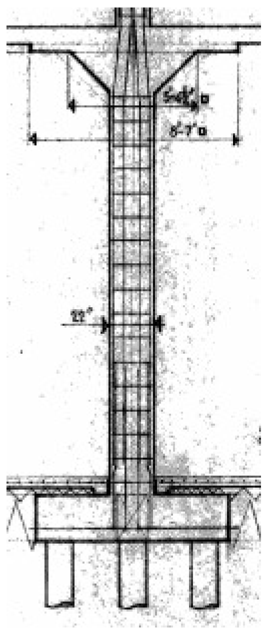
Concrete cover $c_{vr} := 30\text{mm}$

Number of column ends with rotational fixity (refer Fig C5.13) $\text{fixity} := 2.0$

Gross area of section $A_g := b \cdot d = 0.314\,\text{m}^2$

Gross area of longitudinal steel $A_s := n_1 \dfrac{\pi \cdot d_1^2}{4} = 9651 \cdot \text{mm}^2$

Gross area of stirrup steel per set $A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 314 \cdot \text{mm}^2$



1.2 - Design Loads

Design moment $M^* := 237\text{kN} \cdot \text{m}$

Design shear $V^* := 664\text{kN}$

Design axial load $N^* := 572\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha $\alpha' := 3 - \dfrac{M^*}{V^* \cdot d}$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \text{ and } \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1.5$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 0.031$

Beta

$\beta' := 0.5 + 20\rho_1 = 1.115$

$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 1$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete

$V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 598 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column

$d'' := d - 2 \cdot c_{vr} = 500 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal.

$V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 258 \cdot kN$

4.0 - Shear Contribution from Axial Load

Alpha

$\alpha_i := \mathrm{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 6.4 \cdot deg$

$\alpha_{ii} := \mathrm{atan}\left[\frac{(d - c_{vr})}{h}\right] = 12.7 \cdot deg$

$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 12.7 \cdot deg$

$V_n := N^* \cdot \tan(\alpha) = 129.3 \cdot kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3

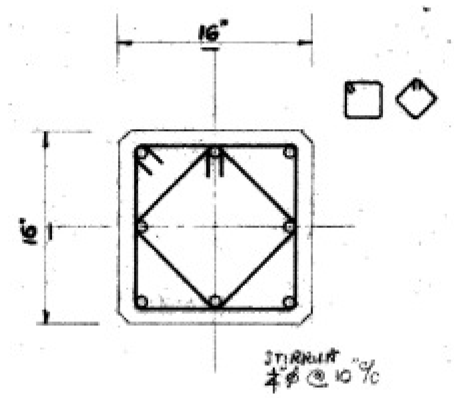
$V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 837 \cdot kN$

COLUMN SHEAR CAPACITY : EAST FRAME, L1-L2, 5E

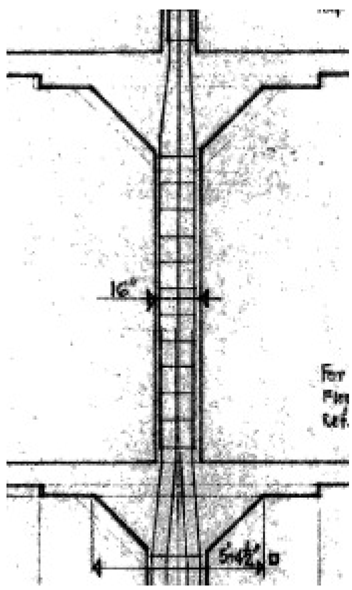
Seismic Assessment of Existing Buildings, C5, July 2017 (incl. March 2018 Errata)

1.0 - General

1.1 - Section Properties



Breadth	$b := 406\,mm$
Depth	$d := 406\,mm$
Height	$h := 3.886\,m$
Probable concrete strength	$f_c := 30\,MPa$
Probable yield strength of steel	$f_y := 284\,MPa$
Longitudinal bar diameter	$d_1 := 29\,mm$
Number of longitudinal bars	$n_1 := 9$
Stirrup diameter	$d_2 := 10\,mm$
Number of stirrup legs per set	$n_2 := 4$
Stirrup spacing	$s := 254\,mm$
Concrete cover	$c_{vr} := 30\,mm$
Number of column ends with rotational fixity (refer Fig C5.13)	$fixity := 2.0$
Gross area of section	$A_g := b \cdot d = 0.165\,m^2$
Gross area of longitudinal steel	$A_s := n_1 \dfrac{\pi \cdot d_1^2}{4} = 5945 \cdot mm^2$
Gross area of stirrup steel per set	$A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 314 \cdot mm^2$



1.2 - Design Loads

Design moment	$M^* := 439\,kN \cdot m$
Design shear	$V^* := 467\,kN$
Design axial load	$N^* := 225\,kN$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 0.036$

Beta $\beta' := 0.5 + 20\rho_1 = 1.221$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 1$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 209 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 346 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 211 \cdot kN$

4.0 - Shear Contribution from Axial Load

Alpha

$$\alpha_i := \operatorname{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 2.8 \cdot deg$$

$$\alpha_{ii} := \operatorname{atan}\left[\frac{(d - c_{vr})}{h}\right] = 5.5 \cdot deg$$

$$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 5.5 \cdot deg$$

$$V_n := N^* \cdot \tan(\alpha) = 21.8 \cdot kN$$

5.0 - Probable Shear Capacity

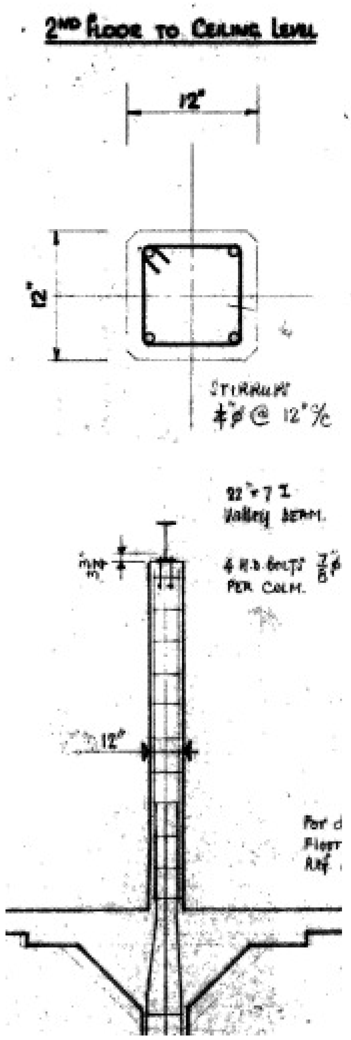
Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 375 \cdot kN$

COLUMN SHEAR CAPACITY : EAST FRAME, L2-L3, 3E

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

1.0 - General

1.1 - Section Properties



Breadth $b := 305\text{mm}$

Depth $d := 305\text{mm}$

Height $h := 3.658\text{m}$

Probable concrete strength $f_c := 30\text{MPa}$

Probable yield strength of steel $f_y := 284\text{MPa}$

Longitudinal bar diameter $d_1 := 25\text{mm}$

Number of longitudinal bars $n_1 := 4$

Stirrup diameter $d_2 := 10\text{mm}$

Number of stirrup legs per set $n_2 := 2$

Stirrup spacing $s := 254\text{mm}$

Concrete cover $c_{vr} := 30\text{mm}$

Number of column ends with rotational fixity (refer Fig C5.13) $\text{fixity} := 2.0$

Gross area of section $A_g := b \cdot d = 0.093\,\text{m}^2$

Gross area of longitudinal steel $A_s := n_1 \dfrac{\pi \cdot d_1^{\,2}}{4} = 1963 \cdot \text{mm}^2$

Gross area of stirrup steel per set $A_v := n_2 \dfrac{\pi \cdot d_2^{\,2}}{4} = 157 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment $M^* := 182\text{kN} \cdot \text{m}$

Design shear $V^* := 193\text{kN}$

Design axial load $N^* := 93\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha $\alpha' := 3 - \dfrac{M^*}{V^* \cdot d}$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 0.021$

Beta $\beta' := 0.5 + 20\rho_1 = 0.922$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.922$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 109 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 245 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal.

$$V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 75 \cdot kN$$

4.0 - Shear Contribution from Axial Load

Alpha

$$\alpha_i := \operatorname{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 2.2 \cdot deg$$

$$\alpha_{ii} := \operatorname{atan}\left[\frac{(d - c_{vr})}{h}\right] = 4.3 \cdot deg$$

$$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 4.3 \cdot deg$$

$$V_n := N^* \cdot \tan(\alpha) = 7 \cdot kN$$

5.0 - Probable Shear Capacity

Refer C5.5.2.3

$$V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 162 \cdot kN$$

King & Dawson Ltd architects & engineers

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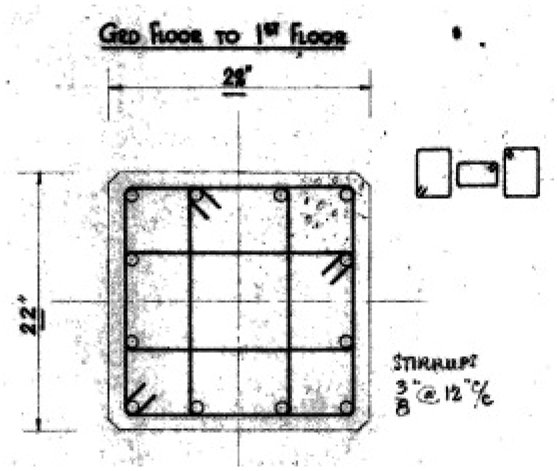
	DATE:	CHECKED:	03b.vii

COLUMN SHEAR CAPACITY : WEST FRAME, LG-L1, 5A

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

1.0 - General



1.1 - Section Properties

Breadth $b := 560\text{mm}$

Depth $d := 560\text{mm}$

Height $h := 2.345\text{m}$

Probable concrete strength $f_c := 30\text{MPa}$

Probable yield strength of steel $f_y := 284\text{MPa}$

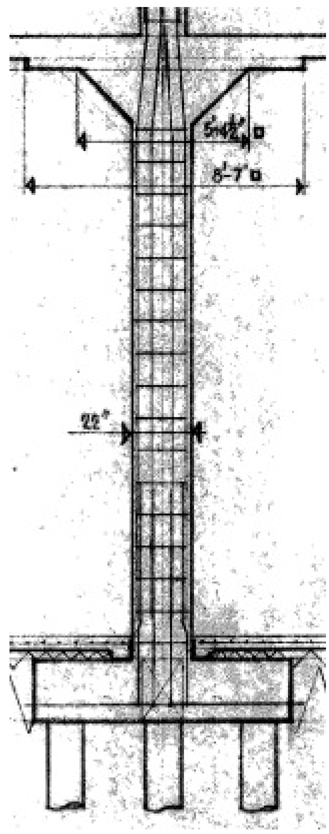
Longitudinal bar diameter $d_1 := 32\text{mm}$

Number of longitudinal bars $n_1 := 12$

Stirrup diameter $d_2 := 10\text{mm}$

Number of stirrup legs per set $n_2 := 4$

Stirrup spacing $s := 300\text{mm}$



Concrete cover $c_{vr} := 30\text{mm}$

Number of column ends with rotational fixity (refer Fig C5.13) $\text{fixity} := 2.0$

Gross area of section $A_g := b \cdot d = 0.314\,\text{m}^2$

Gross area of longitudinal steel $A_s := n_1 \dfrac{\pi \cdot d_1^{\,2}}{4} = 9651 \cdot \text{mm}^2$

Gross area of stirrup steel per set $A_v := n_2 \dfrac{\pi \cdot d_2^{\,2}}{4} = 314 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment $M^* := 266\text{kN}\cdot\text{m}$

Design shear $V^* := 231\text{kN}$

Design axial load $N^* := 627\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha $\alpha' := 3 - \dfrac{M^*}{V^* \cdot d}$

$$\alpha := \begin{cases} \alpha' \text{ if } \begin{cases} \alpha' \ge 1.0 \\ \alpha' \le 1.5 \end{cases} \\ 1.0 \text{ if } \alpha' \le 1.0 \\ 1.5 \text{ if } \alpha' \ge 1.5 \end{cases} = 1$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 0.031$

Beta $\beta' := 0.5 + 20\rho_1 = 1.115$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 1$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 398 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 500 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal.

$$V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 258 \cdot kN$$

4.0 - Shear Contribution from Axial Load

Alpha

$$\alpha_i := \operatorname{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 6.4 \cdot deg$$

$$\alpha_{ii} := \operatorname{atan}\left[\frac{(d - c_{vr})}{h}\right] = 12.7 \cdot deg$$

$$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 12.7 \cdot deg$$

$$V_n := N^* \cdot \tan(\alpha) = 141.7 \cdot kN$$

5.0 - Probable Shear Capacity

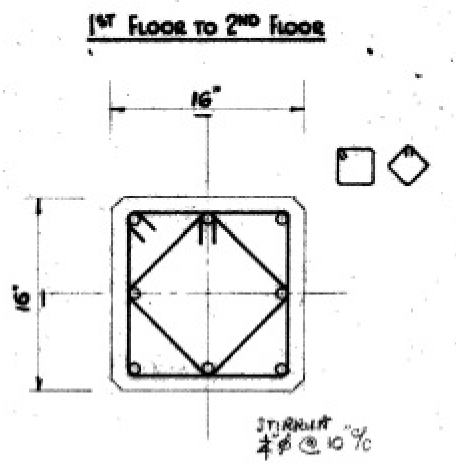
Refer C5.5.2.3

$$V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 678 \cdot kN$$

COLUMN SHEAR CAPACITY : WEST FRAME, L1-L2, 5A

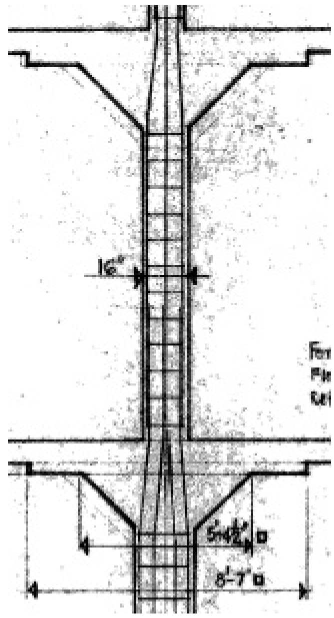
Seismic Assessment of Existing Buildings, C5, July 2017 (incl. March 2018 Errata)

1.0 - General



1.1 - Section Properties

Breadth	$b := 406mm$
Depth	$d := 406mm$
Height	$h := 3.886m$
Probable concrete strength	$f_c := 30MPa$
Probable yield strength of steel	$f_y := 284MPa$
Longitudinal bar diameter	$d_1 := 29mm$
Number of longitudinal bars	$n_1 := 9$
Stirrup diameter	$d_2 := 10mm$
Number of stirrup legs per set	$n_2 := 4$
Stirrup spacing	$s := 254mm$
Concrete cover	$c_{vr} := 30mm$
Number of column ends with rotational fixity (refer Fig C5.13)	$fixity := 2.0$
Gross area of section	$A_g := b \cdot d = 0.165\,m^2$
Gross area of longitudinal steel	$A_s := n_1 \dfrac{\pi \cdot d_1^{\,2}}{4} = 5945 \cdot mm^2$
Gross area of stirrup steel per set	$A_v := n_2 \dfrac{\pi \cdot d_2^{\,2}}{4} = 314 \cdot mm^2$



1.2 - Design Loads

Design moment	$M^* := 297kN \cdot m$
Design shear	$V^* := 327kN$
Design axial load	$N^* := 335kN$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 0.036$

Beta $\beta' := 0.5 + 20\rho_1 = 1.221$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 1$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 209 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 346 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal.

$$V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 211 \cdot kN$$

4.0 - Shear Contribution from Axial Load

Alpha

$$\alpha_i := \operatorname{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 2.8 \cdot deg$$

$$\alpha_{ii} := \operatorname{atan}\left[\frac{(d - c_{vr})}{h}\right] = 5.5 \cdot deg$$

$$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 5.5 \cdot deg$$

$$V_n := N^* \cdot \tan(\alpha) = 32.4 \cdot kN$$

5.0 - Probable Shear Capacity

Refer C5.5.2.3

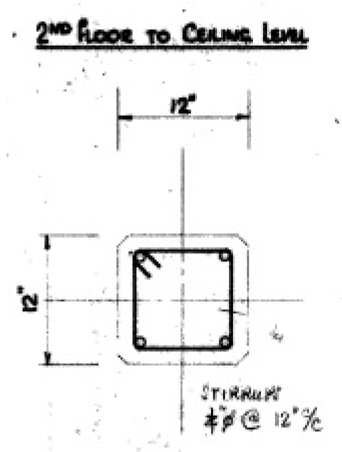
$$V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 385 \cdot kN$$

COLUMN SHEAR CAPACITY : WEST FRAME, L2-L3, 3A

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

1.0 - General

1.1 - Section Properties



Breadth $b := 305\text{mm}$

Depth $d := 305\text{mm}$

Height $h := 3.658\text{m}$

Probable concrete strength $f_c := 30\text{MPa}$

Probable yield strength of steel $f_y := 284\text{MPa}$

Longitudinal bar diameter $d_1 := 25\text{mm}$

Number of longitudinal bars $n_1 := 4$

Stirrup diameter $d_2 := 10\text{mm}$

Number of stirrup legs per set $n_2 := 2$

Stirrup spacing $s := 254\text{mm}$

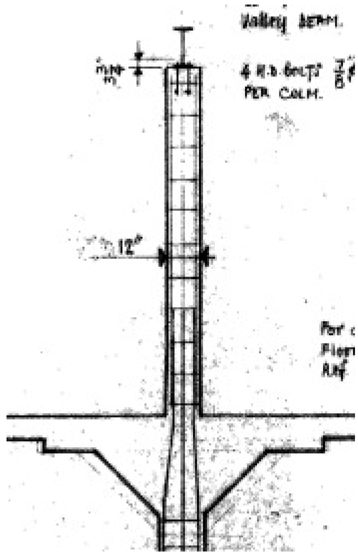
Concrete cover $c_{vr} := 30\text{mm}$

Number of column ends with rotational fixity (refer Fig C5.13) $\text{fixity} := 2.0$

Gross area of section $A_g := b \cdot d = 0.093\,\text{m}^2$

Gross area of longitudinal steel $A_s := n_1 \dfrac{\pi \cdot d_1^2}{4} = 1963 \cdot \text{mm}^2$

Gross area of stirrup steel per set $A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 157 \cdot \text{mm}^2$



1.2 - Design Loads

Design moment $M^* := 178\text{kN}\cdot\text{m}$

Design shear $V^* := 184\text{kN}$

Design axial load $N^* := 75\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha $\alpha' := 3 - \dfrac{M^*}{V^* \cdot d}$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 0.021$

Beta $\beta' := 0.5 + 20\rho_1 = 0.922$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.922$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 109 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 245 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal.

$$V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 75 \cdot kN$$

4.0 - Shear Contribution from Axial Load

Alpha

$$\alpha_i := \mathrm{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 2.2 \cdot deg$$

$$\alpha_{ii} := \mathrm{atan}\left[\frac{(d - c_{vr})}{h}\right] = 4.3 \cdot deg$$

$$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 4.3 \cdot deg$$

$$V_n := N^* \cdot \tan(\alpha) = 5.6 \cdot kN$$

5.0 - Probable Shear Capacity

Refer C5.5.2.3

$$V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 161 \cdot kN$$

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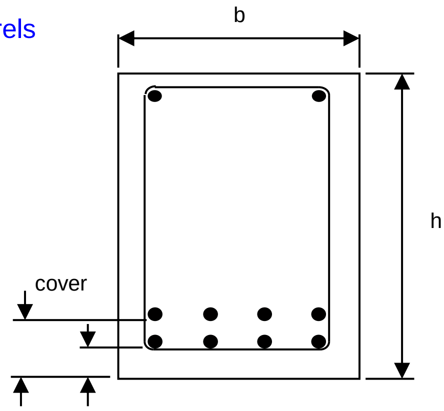
JOB: 220 Thorndon Quay	JOB No	DESIGNED: MHT	PAGE No: 03c.i
	DATE: 13/05/2019		

Rectangular Reinforced Concrete Beam: Elastic Design

Beam Reference: East Wall Spandrels
Floor: Level 1



α = 0.85
β = 0.85
ε_c = 0.003

$p_{min} = \dfrac{\sqrt{f'_c}}{4f_y}$ (cl. 9.3.8.2.1)
= 0.0049

$p_{max} = 0.75p_{bal}$ (cl. 9.3.8.1)
$= \dfrac{0.75\alpha f'_c\beta\varepsilon_c E_s}{f_y(\varepsilon_c E_s+f_y)}$
= 0.0389

Beam Data

b = 150 mm
h = 2895 mm
f_y = 284 MPa
E_s = 200 GPa
f'_c = 30 MPa
ϕ = 1

Moment Capacity

d_{bi} mm	No. Bars	Cover mm	A_{bi} mm^2	d_i mm	$A_{bi}*d_i$
16	2	60	402	2827	1136804
10	11	1447.5	864	1443	1246231
16	2	2835	402	52	20910
0	0	0	0	2895	0
		Neutral Axis	1668		2403945

$d = \dfrac{\Sigma(A_{bi}d_i)}{\Sigma A_{bi}}$
= 1441 mm

$a = \dfrac{\Sigma A_{bi} f_y}{\alpha f'_c b}$
= ##### mm

$p = \dfrac{\Sigma A_{bi}}{bd}$ **O.K.**
= 0.0077

ϕM_n = 653.4 kNm

Shear Reinforcing

f_{yt} = 284 MPa
Legs per stirrup set = 1
d_t = 10 mm

Stirrup angle α = 0 Deg
ϕ = 1
Max Aggregate = 19 mm

Spacing mm	v_s MPa	v_n MPa	ϕV_n kN
229	0.65	1.23	265
204	0.73	1.31	282
179	0.83	1.41	304
154	0.97	1.54	333
129	1.15	1.73	374
104	1.43	2.01	434
79	1.88	2.46	531
54	2.75	3.33	720
29	5.13	5.70	1233
4	37.18	37.75	8160
-21	-7.08	-6.50	-1406
-46	-3.23	-2.66	-574
-71	-2.09	-1.52	-328
-96	-1.55	-0.97	-210
-121	-1.23	-0.65	-141

$v_c = k_a k_d v_b$
= 0.58 MPa

k_d = 1   d <= 400 mm
= $(400/d)^{0.25}$ d > 400 mm
= 0.73

k_a = 1.00   Agg. >= 20 mm
= 0.85   Agg. < 10 mm
= 0.99

$v_b = (0.07 + 10p)\sqrt{f'_c}$ 0.08 < (0.07 + 10p) < 0.20
= 0.81 MPa f'_c <= 50 MPa

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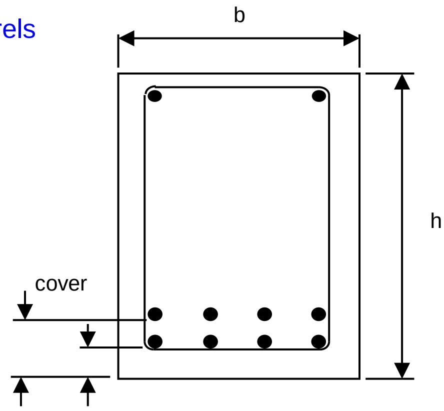
JOB: Thorndon Quay	JOB No	DESIGNED: MHT	PAGE No: 03c.ii
	DATE: 13/05/2019		

Rectangular Reinforced Concrete Beam: Elastic Design

Beam Reference: East Wall Spandrels
Floor: Level 2



α = 0.85
β = 0.85
ε_c = 0.003

$p_{min} = \dfrac{\sqrt{f'_c}}{4f_y}$ (cl. 9.3.8.2.1)
= 0.0049

$p_{max} = 0.75p_{bal}$ (cl. 9.3.8.1)
$= \dfrac{0.75\alpha f'_c\beta\varepsilon_c E_s}{f_y(\varepsilon_c E_s+f_y)}$
= 0.0389

Beam Data

b =	150	mm
h =	2060	mm
f_y =	284	MPa
E_s =	200	GPa
f'_c =	30	MPa
ϕ =	1	

Moment Capacity

d_{bi} mm	No. Bars	Cover mm	A_{bi} mm^2	d_i mm	$A_{bi}*d_i$
16	2	60	402	1992	801031
10	7	1030	550	1025	563523
16	2	2000	402	52	20910
0	0	0	0	2060	0
		Neutral Axis	1354		1385464

$d = \dfrac{\Sigma(A_{bi}d_i)}{\Sigma A_{bi}}$
= 1023 mm

$a = \dfrac{\Sigma A_{bi} f_y}{\alpha f'_c b}$
= ##### mm

$p = \dfrac{\Sigma A_{bi}}{bd}$ **O.K.**
= 0.0088

ϕM_n = 374.1 kNm

Shear Reinforcing

f_{yt} = 284 MPa
Legs per stirrup set = 1
d_t = 10 mm

Stirrup angle α = 0 Deg
ϕ = 1
Max Aggregate = 19 mm

Spacing mm	v_s MPa	v_n MPa	ϕV_n kN
229	0.65	1.32	203
204	0.73	1.40	215
179	0.83	1.51	231
154	0.97	1.64	252
129	1.15	1.83	281
104	1.43	2.10	323
79	1.88	2.56	392
54	2.75	3.43	526
29	5.13	5.80	891
4	37.18	37.85	5809
-21	-7.08	-6.41	-983
-46	-3.23	-2.56	-393
-71	-2.09	-1.42	-218
-96	-1.55	-0.87	-134
-121	-1.23	-0.55	-85

$v_c = k_a k_d v_b$
= 0.67 MPa

k_d = 1   d <= 400 mm
= $(400/d)^{0.25}$ d > 400 mm
= 0.79

k_a = 1.00   Agg. >= 20 mm
= 0.85   Agg. < 10 mm
= 0.99

$v_b = (0.07 + 10p)\sqrt{f'_c}$ 0.08 < (0.07 + 10p) < 0.20
= 0.87 MPa f'_c <= 50 MPa

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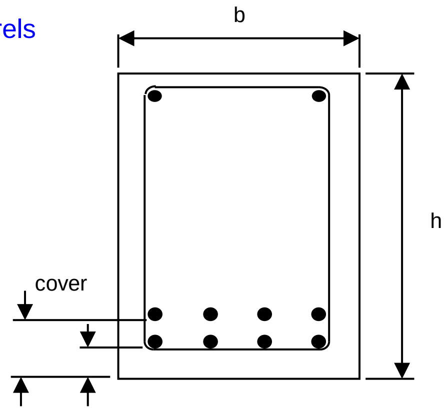
JOB: Thorndon Quay	JOB No	DESIGNED: MHT	PAGE No: 03c.iii
	DATE: 13/05/2019		

Rectangular Reinforced Concrete Beam: Elastic Design

Beam Reference: East Wall Spandrels
Floor: Level 3



α = 0.85
β = 0.85
ε_c = 0.003

$p_{min} = \dfrac{\sqrt{f'_c}}{4f_y}$ (cl. 9.3.8.2.1)
= 0.0049

$p_{max} = 0.75p_{bal}$ (cl. 9.3.8.1)
$= \dfrac{0.75\alpha f'_c\beta\varepsilon_c E_s}{f_y(\varepsilon_c E_s+f_y)}$
= 0.0389

Beam Data

b = 150 mm
h = 1610 mm
f_y = 284 MPa
E_s = 200 GPa
f'_c = 30 MPa
ϕ = 1

Moment Capacity

d_{bi} mm	No. Bars	Cover mm	A_{bi} mm^2	d_i mm	$A_{bi}*d_i$
13	2	60	265	1544	409745
10	4	805	314	800	251327
16	2	1550	402	52	20910
0	0	0	0	1610	0
		Neutral Axis	982		681982

$d = \dfrac{\Sigma (A_{bi}d_i)}{\Sigma A_{bi}}$
= 695 mm

$a = \dfrac{\Sigma A_{bi} f_y}{\alpha f'_c b}$
= 72.9 mm

$p = \dfrac{\Sigma A_{bi}}{bd}$ **O.K.**
= 0.0094

ϕM_n = 183.5 kNm

Shear Reinforcing

f_{yt} = 284 MPa
Legs per stirrup set = 1
d_t = 10 mm

Stirrup angle α = 0 Deg
ϕ = 1
Max Aggregate = 19 mm

Spacing mm	v_s MPa	v_n MPa	ϕV_n kN
229	0.65	1.42	148
204	0.73	1.50	156
179	0.83	1.60	167
154	0.97	1.74	181
129	1.15	1.92	201
104	1.43	2.20	229
79	1.88	2.65	277
54	2.75	3.53	367
29	5.13	5.90	615
4	37.18	37.95	3954
-21	-7.08	-6.31	-657
-46	-3.23	-2.46	-256
-71	-2.09	-1.32	-138
-96	-1.55	-0.78	-81
-121	-1.23	-0.46	-48

$v_c = k_a k_d v_b$
= 0.77 MPa

k_d = 1   d <= 400 mm
$= (400/d)^{0.25}$ d > 400 mm
= 0.87

k_a = 1.00   Agg. >= 20 mm
= 0.85   Agg. < 10 mm
= 0.99

$v_b = (0.07 + 10p)\sqrt{f'_c}$ $0.08 < (0.07 + 10p) < 0.20$
= 0.90 MPa $f'_c <=$ 50 MPa

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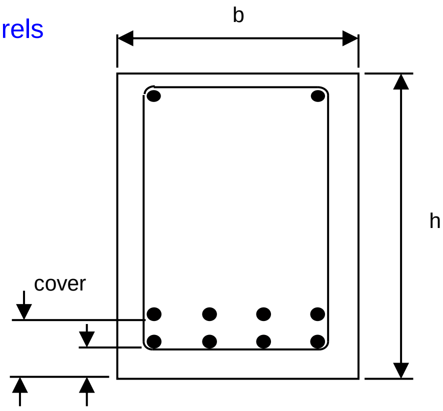
JOB: Thorndon Quay	JOB No	DESIGNED: MHT	PAGE No: 03c.iv
	DATE: 13/05/2019		

Rectangular Reinforced Concrete Beam: Elastic Design

Beam Reference: West Wall Spandrels
Floor: Level 1



α = 0.85
β = 0.85
ε_c = 0.003

$p_{min} = \dfrac{\sqrt{f'_c}}{4f_y}$ (cl. 9.3.8.2.1)
= 0.0049

$p_{max} = 0.75p_{bal}$ (cl. 9.3.8.1)
$= \dfrac{0.75\alpha f'_c\beta\varepsilon_c E_s}{f_y(\varepsilon_c E_s + f_y)}$
= 0.0389

Beam Data

b =	150	mm
h =	1800	mm
f_y =	284	MPa
E_s =	200	GPa
f'_c =	30	MPa
ϕ =	1	

Moment Capacity

d_{bi} mm	No. Bars	Cover mm	A_{bi} mm^2	d_i mm	$A_{bi}*d_i$
16	2	60	402	1732	696479
10	7	900	550	895	492052
16	2	1740	402	52	20910
0	0	0	0	1800	0
		Neutral Axis	1354		1209441

$d = \dfrac{\Sigma(A_{bi}d_i)}{\Sigma A_{bi}}$
= 893 mm

$a = \dfrac{\Sigma A_{bi} f_y}{\alpha f'_c b}$
= ##### mm

$p = \dfrac{\Sigma A_{bi}}{bd}$ O.K.
= 0.0101

ϕM_n = 324.2 kNm

Shear Reinforcing

f_{yt} = 284 MPa
Legs per stirrup set = 1
d_t = 10 mm

Stirrup angle α = 0 Deg
ϕ = 1
Max Aggregate = 19 mm

Spacing mm	v_s MPa	v_n MPa	ϕV_n kN
229	0.65	1.40	188
204	0.73	1.48	199
179	0.83	1.59	212
154	0.97	1.72	231
129	1.15	1.91	256
104	1.43	2.18	293
79	1.88	2.64	353
54	2.75	3.51	470
29	5.13	5.88	788
4	37.18	37.93	5082
-21	-7.08	-6.33	-848
-46	-3.23	-2.48	-332
-71	-2.09	-1.34	-179
-96	-1.55	-0.79	-106
-121	-1.23	-0.47	-64

$v_c = k_a k_d v_b$
= 0.75 MPa

k_d = 1 d <= 400 mm
$= (400/d)^{0.25}$ d > 400 mm
= 0.82

k_a = 1.00 Agg. >= 20 mm
= 0.85 Agg. < 10 mm
= 0.99

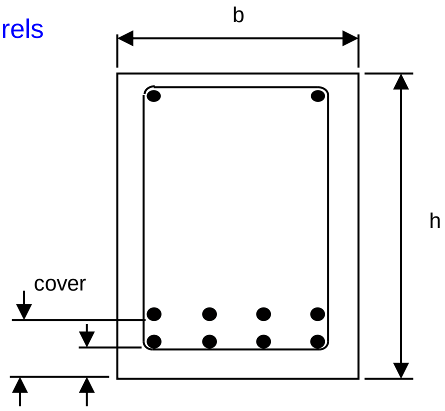
$v_b = (0.07 + 10p)\sqrt{f'_c}$ 0.08 < (0.07 + 10p) < 0.20
= 0.94 MPa f'_c <= 50 MPa

King & Dawson Ltd architects & engineers

JOB: Thorndon Quay	JOB No	DESIGNED: MHT	PAGE No: 03c.v
	DATE: 13/05/2019		

Rectangular Reinforced Concrete Beam: Elastic Design

Beam Reference: West Wall Spandrels
Floor: Level 2



α = 0.85
β = 0.85
ε_c = 0.003

$p_{min} = \dfrac{\sqrt{f'_c}}{4f_y}$ (cl. 9.3.8.2.1)
= 0.0049

p_{max} = 0.75p_{bal} (cl. 9.3.8.1)
$= \dfrac{0.75\alpha f'_c \beta \varepsilon_c E_s}{f_y(\varepsilon_c E_s + f_y)}$
= 0.0389

Beam Data

b =	150	mm
h =	2000	mm
f_y =	284	MPa
E_s =	200	GPa
f'_c =	30	MPa
ϕ =	1	

Moment Capacity

d_{bi} mm	No. Bars	Cover mm	A_{bi} mm^2	d_i mm	$A_{bi}*d_i$
16	2	60	402	1932	776903
10	8	1000	628	995	625177
16	2	1940	402	52	20910
0	0	0	0	2000	0
		Neutral Axis	1433		1422991

$d = \dfrac{\Sigma (A_{bi} d_i)}{\Sigma A_{bi}}$
= 993 mm

$a = \dfrac{\Sigma A_{bi} f_y}{\alpha f'_c b}$
= ##### mm

$p = \dfrac{\Sigma A_{bi}}{bd}$ **O.K.**
= 0.0096

ϕM_n = 382.5 kNm

Shear Reinforcing

f_{yt} = 284 MPa
Legs per stirrup set = 1
d_t = 10 mm

Stirrup angle α = 0 Deg
ϕ = 1
Max Aggregate = 19 mm

Spacing mm	v_s MPa	v_n MPa	ϕV_n kN
229	0.65	1.36	203
204	0.73	1.44	215
179	0.83	1.54	230
154	0.97	1.68	250
129	1.15	1.87	278
104	1.43	2.14	319
79	1.88	2.60	387
54	2.75	3.47	517
29	5.13	5.84	870
4	37.18	37.89	5645
-21	-7.08	-6.37	-949
-46	-3.23	-2.52	-375
-71	-2.09	-1.38	-206
-96	-1.55	-0.83	-124
-121	-1.23	-0.51	-77

$v_c = k_a k_d v_b$
= 0.71 MPa

k_d = 1   d <= 400 mm
= $(400/d)^{0.25}$ d > 400 mm
= 0.80

k_a = 1.00   Agg. >= 20 mm
= 0.85   Agg. < 10 mm
= 0.99

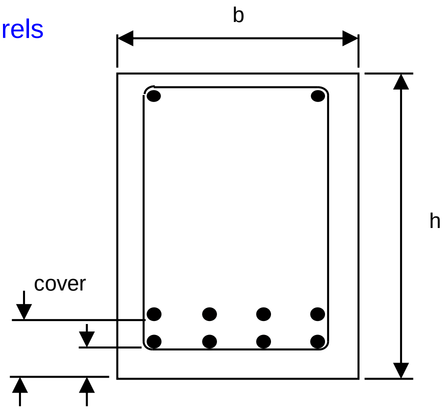
$v_b = (0.07 + 10p)\sqrt{f'_c}$ 0.08 < (0.07 + 10p) < 0.20
= 0.91 MPa f'_c <= 50 MPa

King & Dawson Ltd architects & engineers

JOB: Thorndon Quay	JOB No	DESIGNED: MHT	PAGE No: 03c.vi
	DATE: 13/05/2019		

Rectangular Reinforced Concrete Beam: Elastic Design

Beam Reference: West Wall Spandrels
Floor: Level 3



α = 0.85
β = 0.85
ε_c = 0.003

$p_{min} = \frac{\sqrt{f'_c}}{4f_y}$ (cl. 9.3.8.2.1)
= 0.0049

$p_{max} = 0.75p_{bal}$ (cl. 9.3.8.1)
$= \frac{0.75\alpha f'_c \beta \varepsilon_c E_s}{f_y(\varepsilon_c E_s + f_y)}$
= 0.0389

Beam Data

b =	150	mm
h =	2000	mm
f_y =	284	MPa
E_s =	200	GPa
f'_c =	30	MPa
ϕ =	1	

Moment Capacity

d_{bi} mm	No. Bars	Cover mm	A_{bi} mm^2	d_i mm	A_{bi}*d_i
16	2	60	402	1932	776903
10	6	1000	471	995	468883
12	2	1740	226	254	57453
0	0	0	0	2000	0
			1100		1303239

$d = \frac{\Sigma (A_{bi} d_i)}{\Sigma A_{bi}}$
= 1185 mm

$a = \frac{\Sigma A_{bi} f_y}{\alpha f'_c b}$
= 81.6 mm

$p = \frac{\Sigma A_{bi}}{bd}$ **O.K.**
= 0.0062

ϕM_n = 357.4 kNm

Shear Reinforcing

f_{yt} = 284 MPa
Legs per stirrup set = 1
d_t = 10 mm

Stirrup angle α = 0 Deg
ϕ = 0.75
Max Aggregate = 19 mm

Spacing mm	v_s MPa	v_n MPa	ϕV_n kN
229	0.65	1.19	159
204	0.73	1.27	169
179	0.83	1.37	183
154	0.97	1.51	201
129	1.15	1.69	226
104	1.43	1.97	263
79	1.88	2.42	323
54	2.75	3.30	439
29	5.13	5.67	756
4	37.18	37.72	5029
-21	-7.08	-6.54	-872
-46	-3.23	-2.69	-359
-71	-2.09	-1.55	-207
-96	-1.55	-1.01	-134
-121	-1.23	-0.69	-92

$v_c = k_a k_d v_b$
= 0.54 MPa

k_d = 1   d <= 400 mm
= $(400/d)^{0.25}$ d > 400 mm
= 0.76

k_a = 1.00   Agg. >= 20 mm
= 0.85   Agg. < 10 mm
= 0.99

$v_b = (0.07 + 10p)\sqrt{f'_c}$ 0.08 < (0.07 + 10p) < 0.20
= 0.72 MPa f'_c <= 50 MPa

SPANDREL SHEAR CAPACITY - EAST FACE LEVEL 1

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

SAEB contain updated guidance on beam shear capacity. Revising prior values.

1.1 - Section Properties

Breadth	$b := 150\text{mm}$
Depth	$d := 2895\text{mm}$
Probable concrete strength	$f_c := 30\text{MPa}$
Probable yield strength of steel	$f_y := 284\text{MPa}$
Longitudinal bar diameter	$d_{1a} := 16\text{mm}$ $\quad d_{1b} := 10\text{mm}$
Number of longitudinal bars	$n_{1a} := 4$ $\quad n_{1b} := 4$
Stirrup diameter	$d_2 := 10\text{mm}$
Number of stirrup legs per set	$n_2 := 2$
Stirrup spacing	$s := 225\text{mm}$
Concrete cover	$c_{vr} := 30\text{mm}$
Number of column ends with rotational fixity (refer Fig C5.13)	$\text{fixity} := 2.0$
Gross area of section	$A_g := b \cdot d = 0.434\,\text{m}^2$
Gross area of longitudinal steel	$A_s := n_{1a}\dfrac{\pi \cdot d_{1a}^2}{4} + n_{1b}\dfrac{\pi \cdot d_{1b}^2}{4} = 1118 \cdot \text{mm}^2$
Gross area of stirrup steel per set	$A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 157 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment	$M^* := 760\text{kN}\cdot\text{m}$
Design shear	$V^* := 640\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \begin{cases} \alpha' & \text{if } \begin{cases} \alpha' \ge 1.0 \\ \alpha' \le 1.5 \end{cases} \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1.5$$

Ratio of total longitudinal reinforcement

$$\rho_1 := A_s \div A_g = 2.575 \times 10^{-3}$$

Beta $\beta' := 0.5 + 20\rho_1 = 0.552$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.552$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 456 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 3 \times 10^3 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \dfrac{A_v \cdot f_y \cdot d''}{s} = 562 \cdot kN$

4.0 - Shear Contribution from Axial Load

Not applicable to beams. $V_n := 0kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 866 \cdot kN$

SPANDREL SHEAR CAPACITY - EAST FACE LEVEL 2

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

SAEB contain updated guidance on beam shear capacity. Revising prior values.

1.1 - Section Properties

Breadth	$b := 150\text{mm}$
Depth	$d := 2060\text{mm}$
Probable concrete strength	$f_c := 30\text{MPa}$
Probable yield strength of steel	$f_y := 284\text{MPa}$
Longitudinal bar diameter	$d_{1a} := 16\text{mm}$ $\quad d_{1b} := 10\text{mm}$
Number of longitudinal bars	$n_{1a} := 4$ $\quad n_{1b} := 4$
Stirrup diameter	$d_2 := 10\text{mm}$
Number of stirrup legs per set	$n_2 := 2$
Stirrup spacing	$s := 225\text{mm}$
Concrete cover	$c_{vr} := 30\text{mm}$
Number of column ends with rotational fixity (refer Fig C5.13)	$\text{fixity} := 2.0$
Gross area of section	$A_g := b \cdot d = 0.309\,\text{m}^2$
Gross area of longitudinal steel	$A_s := n_{1a}\dfrac{\pi \cdot d_{1a}^2}{4} + n_{1b}\dfrac{\pi \cdot d_{1b}^2}{4} = 1118 \cdot \text{mm}^2$
Gross area of stirrup steel per set	$A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 157 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment	$M^* := 800\text{kN}\cdot\text{m}$
Design shear	$V^* := 300\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \begin{array}{|l} \alpha' \ \text{ if } \begin{array}{|l} \alpha' \geq 1.0 \\ \alpha' \leq 1.5 \end{array} \\ 1.0 \ \text{ if } \ \alpha' \leq 1.0 \\ 1.5 \ \text{ if } \ \alpha' \geq 1.5 \end{array} = 1.5$$

Ratio of total longitudinal reinforcement $\quad \rho_1 := A_s \div A_g = 3.619 \times 10^{-3}$

Beta $\beta' := 0.5 + 20\rho_1 = 0.572$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.572$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 337 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 2 \times 10^3 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \dfrac{A_v \cdot f_y \cdot d''}{s} = 397 \cdot kN$

4.0 - Shear Contribution from Axial Load

Not applicable to beams. $V_n := 0kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 624 \cdot kN$

SPANDREL SHEAR CAPACITY - EAST FACE LEVEL 3

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

SAEB contain updated guidance on beam shear capacity. Revising prior values.

1.1 - Section Properties

Breadth	$b := 150\text{mm}$
Depth	$d := 1610\text{mm}$
Probable concrete strength	$f_c := 30\text{MPa}$
Probable yield strength of steel	$f_y := 284\text{MPa}$
Longitudinal bar diameter	$d_{1a} := 16\text{mm}$ $d_{1b} := 10\text{mm}$
Number of longitudinal bars	$n_{1a} := 4$ $n_{1b} := 4$
Stirrup diameter	$d_2 := 10\text{mm}$
Number of stirrup legs per set	$n_2 := 2$
Stirrup spacing	$s := 225\text{mm}$
Concrete cover	$c_{vr} := 30\text{mm}$
Number of column ends with rotational fixity (refer Fig C5.13)	$\text{fixity} := 2.0$
Gross area of section	$A_g := b \cdot d = 0.242\,\text{m}^2$
Gross area of longitudinal steel	$A_s := n_{1a}\dfrac{\pi \cdot {d_{1a}}^2}{4} + n_{1b}\dfrac{\pi \cdot {d_{1b}}^2}{4} = 1118 \cdot \text{mm}^2$
Gross area of stirrup steel per set	$A_v := n_2 \dfrac{\pi \cdot {d_2}^2}{4} = 157 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment	$M^* := 200\text{kN} \cdot \text{m}$
Design shear	$V^* := 100\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \left| \begin{array}{l} \alpha' \ \text{ if } \left| \begin{array}{l} \alpha' \geq 1.0 \\ \alpha' \leq 1.5 \end{array} \right. \\ 1.0 \ \text{ if } \ \alpha' \leq 1.0 \\ 1.5 \ \text{ if } \ \alpha' \geq 1.5 \end{array} \right. = 1.5$$

Ratio of total longitudinal reinforcement

$$\rho_1 := A_s \div A_g = 4.631 \times 10^{-3}$$

Beta $\beta' := 0.5 + 20\rho_1 = 0.593$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.593$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 273 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 2 \times 10^3 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \frac{A_v \cdot f_y \cdot d''}{s} = 307 \cdot kN$

4.0 - Shear Contribution from Axial Load

Not applicable to beams. $V_n := 0kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 493 \cdot kN$

SPANDREL SHEAR CAPACITY - WEST FACE LEVEL 1

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

SAEB contain updated guidance on beam shear capacity. Revising prior values.

1.1 - Section Properties

Breadth $b := 150\text{mm}$

Depth $d := 1800\text{mm}$

Probable concrete strength $f_c := 30\text{MPa}$

Probable yield strength of steel $f_y := 284\text{MPa}$

Longitudinal bar diameter $d_{1a} := 16\text{mm}$ $d_{1b} := 10\text{mm}$

Number of longitudinal bars $n_{1a} := 4$ $n_{1b} := 4$

Stirrup diameter $d_2 := 10\text{mm}$

Number of stirrup legs per set $n_2 := 2$

Stirrup spacing $s := 225\text{mm}$

Concrete cover $c_{vr} := 30\text{mm}$

Number of column ends with rotational fixity (refer Fig C5.13) $\text{fixity} := 2.0$

Gross area of section $A_g := b \cdot d = 0.27\,\text{m}^2$

Gross area of longitudinal steel $A_s := n_{1a}\dfrac{\pi \cdot d_{1a}^2}{4} + n_{1b}\dfrac{\pi \cdot d_{1b}^2}{4} = 1118 \cdot \text{mm}^2$

Gross area of stirrup steel per set $A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 157 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment $M^* := 750\text{kN} \cdot \text{m}$

Design shear $V^* := 300\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha $\alpha' := 3 - \dfrac{M^*}{V^* \cdot d}$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1.5$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 4.142 \times 10^{-3}$

Beta $\beta' := 0.5 + 20\rho_1 = 0.583$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.583$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 300 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 2 \times 10^3 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \frac{A_v \cdot f_y \cdot d''}{s} = 345 \cdot kN$

4.0 - Shear Contribution from Axial Load

Not applicable to beams. $V_n := 0kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 548 \cdot kN$

SPANDREL SHEAR CAPACITY - WEST FACE LEVEL 1

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

SAEB contain updated guidance on beam shear capacity. Revising prior values.

1.1 - Section Properties

Breadth	$b := 150\text{mm}$
Depth	$d := 2000\text{mm}$
Probable concrete strength	$f_c := 30\text{MPa}$
Probable yield strength of steel	$f_y := 284\text{MPa}$
Longitudinal bar diameter	$d_{1a} := 16\text{mm} \quad d_{1b} := 10\text{mm}$
Number of longitudinal bars	$n_{1a} := 4 \quad n_{1b} := 4$
Stirrup diameter	$d_2 := 10\text{mm}$
Number of stirrup legs per set	$n_2 := 2$
Stirrup spacing	$s := 225\text{mm}$
Concrete cover	$c_{vr} := 30\text{mm}$
Number of column ends with rotational fixity (refer Fig C5.13)	$\text{fixity} := 2.0$
Gross area of section	$A_g := b \cdot d = 0.3\,\text{m}^2$
Gross area of longitudinal steel	$A_s := n_{1a}\dfrac{\pi \cdot d_{1a}^2}{4} + n_{1b}\dfrac{\pi \cdot d_{1b}^2}{4} = 1118 \cdot \text{mm}^2$
Gross area of stirrup steel per set	$A_v := n_2 \dfrac{\pi \cdot d_2^2}{4} = 157 \cdot \text{mm}^2$

1.2 - Design Loads

Design moment	$M^* := 750\text{kN} \cdot \text{m}$
Design shear	$V^* := 800\text{kN}$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1.5$$

Ratio of total longitudinal reinforcement

$$\rho_1 := A_s \div A_g = 3.728 \times 10^{-3}$$

Beta $\beta' := 0.5 + 20\rho_1 = 0.575$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.575$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 329 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 2 \times 10^3 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \dfrac{A_v \cdot f_y \cdot d''}{s} = 385 \cdot kN$

4.0 - Shear Contribution from Axial Load

Not applicable to beams. $V_n := 0kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 606 \cdot kN$

SPANDREL SHEAR CAPACITY - WEST FACE LEVEL 3

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

SAEB contain updated guidance on beam shear capacity. Revising prior values.

1.1 - Section Properties

Breadth	$b := 150mm$
Depth	$d := 2000mm$
Probable concrete strength	$f_c := 30MPa$
Probable yield strength of steel	$f_y := 284MPa$
Longitudinal bar diameter	$d_{1a} := 16mm$ $\quad d_{1b} := 10mm$
Number of longitudinal bars	$n_{1a} := 4$ $\quad n_{1b} := 4$
Stirrup diameter	$d_2 := 10mm$
Number of stirrup legs per set	$n_2 := 2$
Stirrup spacing	$s := 225mm$
Concrete cover	$c_{vr} := 30mm$
Number of column ends with rotational fixity (refer Fig C5.13)	$fixity := 2.0$
Gross area of section	$A_g := b \cdot d = 0.3\,m^2$
Gross area of longitudinal steel	$A_s := n_{1a}\dfrac{\pi \cdot d_{1a}^{\,2}}{4} + n_{1b}\dfrac{\pi \cdot d_{1b}^{\,2}}{4} = 1118 \cdot mm^2$
Gross area of stirrup steel per set	$A_v := n_2\dfrac{\pi \cdot d_2^{\,2}}{4} = 157 \cdot mm^2$

1.2 - Design Loads

Design moment	$M^* := 400kN \cdot m$
Design shear	$V^* := 130kN$

2.0 - Shear Contribution from Concrete

Alpha

$$\alpha' := 3 - \frac{M^*}{V^* \cdot d}$$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \wedge \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1.462$$

Ratio of total longitudinal reinforcement

$$\rho_1 := A_s \div A_g = 3.728 \times 10^{-3}$$

Beta $\beta' := 0.5 + 20\rho_1 = 0.575$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.575$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete $V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 320 \cdot kN$

3.0 - Shear Contribution from Steel

Depth of concrete core of column $d'' := d - 2 \cdot c_{vr} = 2 \times 10^3 \cdot mm$

Assuming tension crack is inclined at 30deg to horizontal. $V_s := \dfrac{A_v \cdot f_y \cdot d''}{s} = 385 \cdot kN$

4.0 - Shear Contribution from Axial Load

Not applicable to beams. $V_n := 0kN$

5.0 - Probable Shear Capacity

Refer C5.5.2.3 $V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 599 \cdot kN$

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CALCULATION SHEET

Job Name:	220 Thorndon Quay		
Job Number:	21716	Designe	MCW
Date:	May 2019	Page No:	03e

Reinforced Concrete Wall Design: Elastic

Wall Reference: North and south shear walls
Floor: G Floor

Wall data:

$b =$ 150 mm
$L =$ 30480 mm

$N^* =$ 2906 kN
$f_c' =$ 30 MPa

Vertical Reinforcing:

$E_s =$ 200 GPa
$f_y =$ 227 MPa

End Zone = 500 mm
Cover to last bar = 75 mm

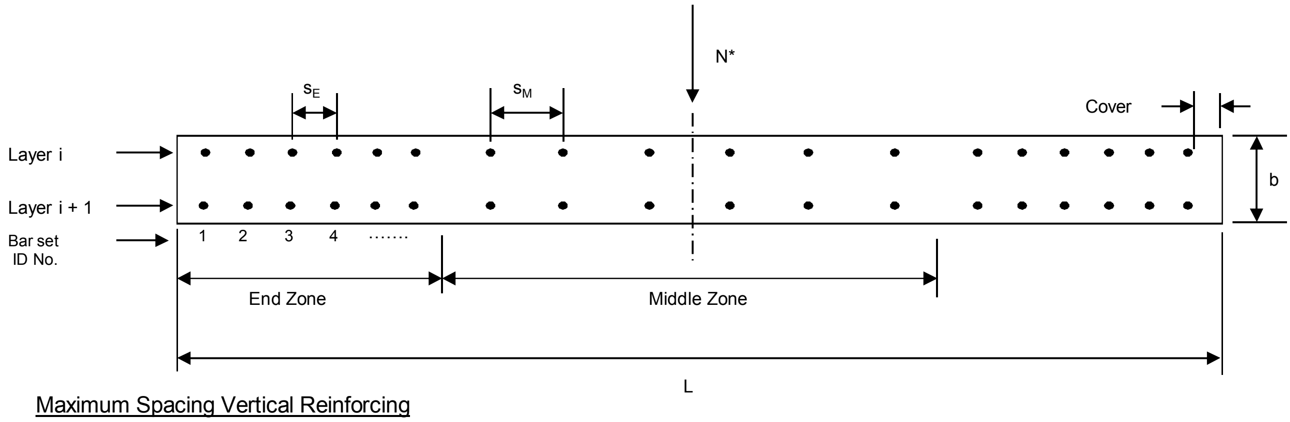
Zone	d_b mm	Number layers	s mm	A/s mm^2/m
End Zone	32	2	200	8042
Middle Zone	10	1	450	175

p_{min} <	p	< p_{max}	Steel ratio check
0.0060	0.0536	0.0705	OK
0.0060	0.0012	0.0705	Outside limits

Note
Steel ratio's OK as assessing existing struicture

$d_{b\ min} =$ 10 mm cl 11.3.11.2 (b)
$d_{b\ max} =$ b / 7 cl 11.3.11.2 (d)
= 21.4 mm

$p_{min} = \max [\ 0.7/f_y\ ,\ \sqrt{f_c'}/(4f_y)\ ]$ cl 11.3.10.3.8
$p_{max} = 16/f_y$ cl 11.3.11.3



Maximum Spacing Vertical Reinforcing

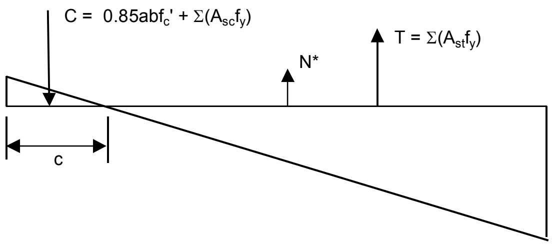
$s_{max} = \min [\ L/3\ ,\ 3b\ ,\ 450\ ]$ mm
= 450 mm

cl 11.3.11.3

Moment Calculation

Sum Tension Forces = $N^* + \Sigma(A_{st} f_y)$
= 4770 kN

Sum Compression Forces = $0.85abf_c' + \Sigma(A_{sc} f_y)$
= 4770 kN



Equate forces and solve for c:

c = 1128 mm

a = 0.85c

Vertical Reinforcing Design

$M^* =$ 48906 kNm
$\phi =$ 1
$\phi M_n =$ 82063 kNm

Check: $M^* < \phi M_n$
OK

Strains:
$\varepsilon_{conc} =$ 0.0030
$\varepsilon_{yield} =$ 0.0011

SHEAR WALL CAPACITY : NORTH AND SOUTH WALLS

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

1.0 - General

1.1 - Section Properties

Breadth $b := 150\mathrm{mm}$

Depth $d := 30480\mathrm{mm}$

Height $h := 2.345\mathrm{m}$

Probable concrete strength $f_c := 30\mathrm{MPa}$

Probable yield strength of steel $f_y := 284\mathrm{MPa}$

Longitudinal bar diameter $d_1 := 10\mathrm{mm}$

Number of longitudinal bars $n_1 := d \div 450\mathrm{mm} = 68$

Stirrup diameter $d_2 := 10\mathrm{mm}$

Number of stirrup legs per set $n_2 := 1$

Stirrup spacing $s := 450\mathrm{mm}$

Concrete cover $c_{vr} := 75\mathrm{mm}$

Number of column ends with rotational fixity (refer Fig C5.13) $\mathrm{fixity} := 2.0$

Gross area of section $A_g := b \cdot d = 4.572\,\mathrm{m}^2$

Gross area of longitudinal steel $A_s := n_1 \frac{\pi \cdot d_1^{\,2}}{4} = 5320 \cdot \mathrm{mm}^2$

Gross area of stirrup steel per set $A_v := n_2 \frac{\pi \cdot d_2^{\,2}}{4} = 79 \cdot \mathrm{mm}^2$

1.2 - Design Loads

Design moment $M^* := 48906\mathrm{kN \cdot m}$

Design shear $V^* := 10500\mathrm{kN}$

Design axial load $N^* := 3200\mathrm{kN}$

2.0 - Shear Contribution from Concrete

Alpha $\alpha' := 3 - \frac{M^*}{V^* \cdot d}$

$$\alpha := \begin{cases} \alpha' & \text{if } \alpha' \ge 1.0 \text{ and } \alpha' \le 1.5 \\ 1.0 & \text{if } \alpha' \le 1.0 \\ 1.5 & \text{if } \alpha' \ge 1.5 \end{cases} = 1.5$$

Ratio of total longitudinal reinforcement $\rho_1 := A_s \div A_g = 1.164 \times 10^{-3}$

Beta

$$\beta' := 0.5 + 20\rho_1 = 0.523$$

$$\beta := \begin{cases} \beta' & \text{if } \beta' \le 1.0 \\ 1.0 & \text{otherwise} \end{cases} = 0.523$$

Shear strength degradation factor (refer Fig C5.12) $\gamma := 0.29$

Shear contribution from concrete

$$V_c := \alpha \cdot \beta \cdot \gamma \cdot \sqrt{f_c} \cdot \sqrt{MPa} \cdot (0.8 \cdot A_g) = 5 \times 10^3 \cdot kN$$

3.0 - Shear Contribution from Steel

Depth of concrete core of column

$$d'' := d - 2 \cdot c_{vr} = 3 \times 10^4 \cdot mm$$

Assuming tension crack is inclined at 30deg to horizontal.

$$V_s := \frac{A_v \cdot f_y \cdot d''}{s} \cdot \cot(30deg) = 3 \times 10^3 \cdot kN$$

4.0 - Shear Contribution from Axial Load

Alpha

$$\alpha_i := \operatorname{atan}\left[\frac{0.5(d - c_{vr})}{h}\right] = 81.2 \cdot deg$$

$$\alpha_{ii} := \operatorname{atan}\left[\frac{(d - c_{vr})}{h}\right] = 85.6 \cdot deg$$

$$\alpha := \begin{cases} \alpha_i & \text{if fixity} \le 1.0 \\ \alpha_{ii} & \text{if fixity} \ge 2.0 \end{cases} = 85.6 \cdot deg$$

$$V_n := N^* \cdot \tan(\alpha) = 4.1 \times 10^4 \cdot kN$$

5.0 - Probable Shear Capacity

Refer C5.5.2.3

$$V_{prob} := 0.85 \cdot (V_c + V_s + V_n) = 41357 \cdot kN$$

ROOF LEVEL LOAD TRANSFER

NZS1170.5:2004 Earthquake Actions
NZS3101:2006 Concrete Structures
NZS3603:1997 Timber Structures

Checking gable ends for face loading and internal partition walls for bracing capacity.
Assessing to NZS1170.5 'Parts and Components'.

1.0 - Gable Ends

1.1 - Design Loading

Concrete gables $w_2 := 24\frac{\mathrm{kN}}{\mathrm{m}^3}\cdot 30\mathrm{m}\cdot 150\mathrm{mm}\cdot\frac{3.0\mathrm{m}}{2} = 162\cdot\mathrm{kN}$

Seismic coefficient (refer excel calc) $C := 1.09$

Lateral load total $F := w_2\cdot C = 176.6\cdot\mathrm{kN}$

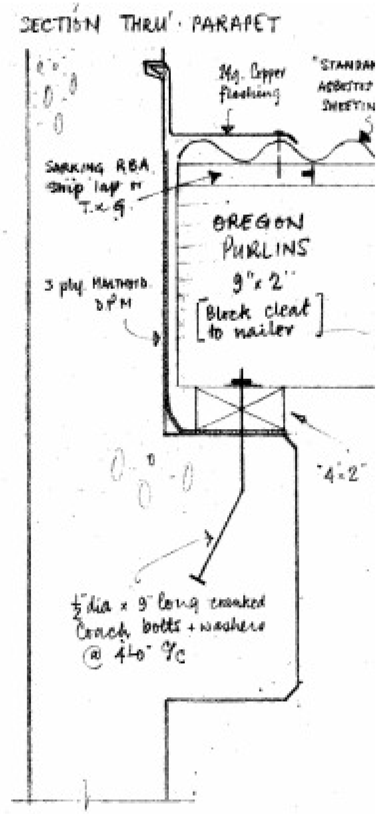
Roof slope $\alpha := \mathrm{atan}\left(\frac{40.8}{101.6}\right) = 22\cdot\mathrm{deg}$

Sloped roof length $l_s := 30\mathrm{m} \div \cos(\alpha) = 32.3\,\mathrm{m}$

Lateral load per metre $F_\omega := \frac{F}{l_s} = 5.5\cdot\frac{\mathrm{kN}}{\mathrm{m}}$

1.2 - Connection checks

1.2.1 - Embedded anchors, shear loading.



Spacing of anchors $s_p := 4\mathrm{ft} = 1.219\,\mathrm{m}$

Strength reduction factor $\phi := 1.0$ (Existing building)

Number of anchors per metre $n := \frac{1}{s_p} = 0.82\frac{1}{\mathrm{m}}$

Diameter of anchors $d_{ia} := \frac{1}{2}\mathrm{in} = 12.7\cdot\mathrm{mm}$

Effective cross sectional area $A_{se} := \frac{\pi\cdot d_{ia}^2}{4} = 127\cdot\mathrm{mm}^2$

Yield strength of anchor $f_y := 270\mathrm{MPa}$

Ultimate strength of steel $f_{ut} := \min\left(1.9\cdot f_y, 860\mathrm{MPa}\right) = 513\cdot\mathrm{MPa}$

Lower characteristic strength of anchor in shear $\phi V_s := \phi\cdot n\cdot 0.6\cdot A_{se}\cdot f_{ut} = 32\cdot\frac{\mathrm{kN}}{\mathrm{m}}$

$$\frac{\phi V_s}{F_\omega} = 586\cdot\%$$

1.2.2 - Timber rupture

Specificaton calls out 'HT Rimu or Totara BA'. Taking as J5 for joint design as per Table 4.1

K factor as per table 4.11 $k_{11} := 14.9$

Stress as per table 4.11 $f_{pj} := 12.9\text{MPa}$

Bolt diameter $d_a := d_{ia} = 12.7 \cdot \text{mm}$

Effective timber thickness $b_e := 2\text{in} = 50.8 \cdot \text{mm}$

Characteristic strength perpindicular to the grain $Q_{kp} := \min\left(k_{11} \cdot f_{pj} \cdot \frac{d_a^{1.5}}{\text{mm}^{1.5}} \cdot \text{mm}^2, 0.5 \cdot b_e \cdot f_{pj} \cdot d_a \right) = 4.2 \cdot \text{kN}$

System characteristic strength $Q_{sk} := Q_{kp}$

Load duration factor $k_1 := 1.0$

Green timber factor $k_{12} := 1.0$

Multiple fastener reduction $k_{13} := 1.0$

Nominal joint strength $\phi Q_n := \phi \cdot n \cdot k_1 \cdot k_{12} \cdot k_{13} \cdot Q_{sk} = 3.41 \cdot \frac{\text{kN}}{\text{m}}$ $\frac{\phi Q_n}{F_\omega} = 62 \cdot \%$

Timber strength is not sufficient to tie gable to diaphragm. Gable will need to cantilever from floor below.

1.2.3 - Cantilever Strength

Design load $F_{mn} := F_\omega - \phi Q_n = 2 \cdot \frac{\text{kN}}{\text{m}}$

Design moment $M^* := F_{mn} \cdot 3.0\text{m} = 6 \cdot \frac{\text{kN} \cdot \text{m}}{\text{m}}$

Bending strength cantilevered gable (refer excel calc) $\phi M_n := 5.6 \frac{\text{kN} \cdot \text{m}}{\text{m}}$ $\frac{\phi M_n}{M^*} = 91 \cdot \%$

2.0 - Internal Partition Bracing

Level three apartments have fire rated seperations. Checking how much of the roof load can be transferred to slab via these walls.

2.1 - Design Loading

Roof mass $w_3 := 0.4kPa \cdot l_s \cdot 40m = 517.3 \cdot kN$

Define bracing units $BU's := \frac{1kN}{20}$

Design lateral load $F_{br} := w_3 \cdot C = 11276 \cdot BU's$

2.2 - Bracing provided

Reccomended bracing rating (standard plasterboard two sides - refer BRANZ Build 141) $Br_{EQ} := \frac{41BU's}{m}$

Length of longitudinal wall per units (end units and internal)

$l_{w.e} := 2.71m + 1.25m + 1.40m + 1.47m = 6.83\,m$

$l_{w.i} := 1.60m + 2.59m + 4.88m = 9.07\,m$

Total length of longitudinal wall $l_{total} := 4 \cdot l_{w.e} + 17 \cdot l_{w.i} = 181.51\,m$

Bracing provided $Br_{pr} := Br_{EQ} \cdot l_{total} = 7442 \cdot BU's$

Remaining load $F_{rem} := F_{br} - Br_{pr} = 192 \cdot kN$

3.0 - Connection to Level 3 Frames

3.1 - Design Loading

Uniformly distributed loading (ignoring contribution of centreline columns) $V_\omega := \frac{0.5F_{rem}}{40m} = 2.396 \cdot \frac{kN}{m}$

3.2 - Perimeter Connection

Refer excel calc for nailed connection strength.

Connection strength $\phi Q_n := 2.5 \frac{kN}{m}$

$$\frac{\phi Q_n}{V_\omega} = 104 \cdot \%$$

Design Actions on Parts (Section 8 NZS1170.5)

Site Characteristics

Soil Type = D — Deep or Soft Soil

Location = Wellington

Z = 0.4 — Table 3.3 or Fig. 3.3 / 3.4

Building Characteristics

Importance Level = 2

ULS / SLS = ULS

Design Life = 50 yrs

Return Period = 500 yrs

Part Characteristics

T_p = 0.6 s

Category = P.3 — Table 8.1

R_p = 0.9

μ_p = 3.00 — Table 8.2

Elastic Site Hazard Coefficient

$C_h(T=0)$ = 1.12 — Table 3.1

Z = 0.4 — Table 3.3

R_u or R_s = 1.00 — Table 3.5

$N_{max}(T=0)$ = 1.00 — Table 3.7

$N(T=0,D)$ = 1.00 — cl. 3.1.6

Maximum Checks (cl. 3.1.1)

$ZR \leq 0.7$ — ZR = 0.40 — OK

$C(0) = C_h(T=0)\ Z\ R\ N(T=0,D)$

$= 0.448$

Part Spectral Shape Coefficient

$C_i(T_p)$ = 2.00

$= 2.00$	$T_p \leq 0.75$ s
$= 2(1.75 - T_p)$	$0.75 < T_p < 1.5$ s
$= 0.50$	$T_p \geq 1.5$ s

= 2.00

Part Response Factor

μ_p = 3.00

C_{ph} = 0.45 — Table 8.2

Level i	Storey Height (m)	h_i (m)	C_{Hi}	$C_p(T_p) = C(0)\ C_{Hi}\ C_i(T_p)$	$C_p(T_p)\ C_{ph}\ R_p$
TYP	**3.0**				
ROOF	3.7	13.3	3.00	2.69	1.09
3	3.9	9.6	2.60	2.33	0.94
2	2.2	5.7	1.95	1.75	0.71
1	3.5	3.5	1.58	1.42	0.57
GND					

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		DATE:	CHECKED:	

Timber roof diaphragm - Shear Flow

V^* = 96 (= F.rem, refer 04a.iii)
L = 40 m

$q^* = V^* / L$
= 2.4 kN/m

$\phi Q_n = \phi n k Q_k$

ϕ = 1
n = 6.7
k = 0.60
Q_k = 0.631 kN

ϕQ_n = 2.5 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 =	60	mm	k_{f1} =	1.00
Thickness 1st member t_1 =	25	mm	k_{f2} =	0.59
First Member Type =	Timber		k_f =	0.59
Total Number Nails =	6.7		k_g =	1.02

$k = k_a\, k_b\, k_c\, k_d\, k_e\, k_f\, k_g$ = 0.60

Nail Design Summary

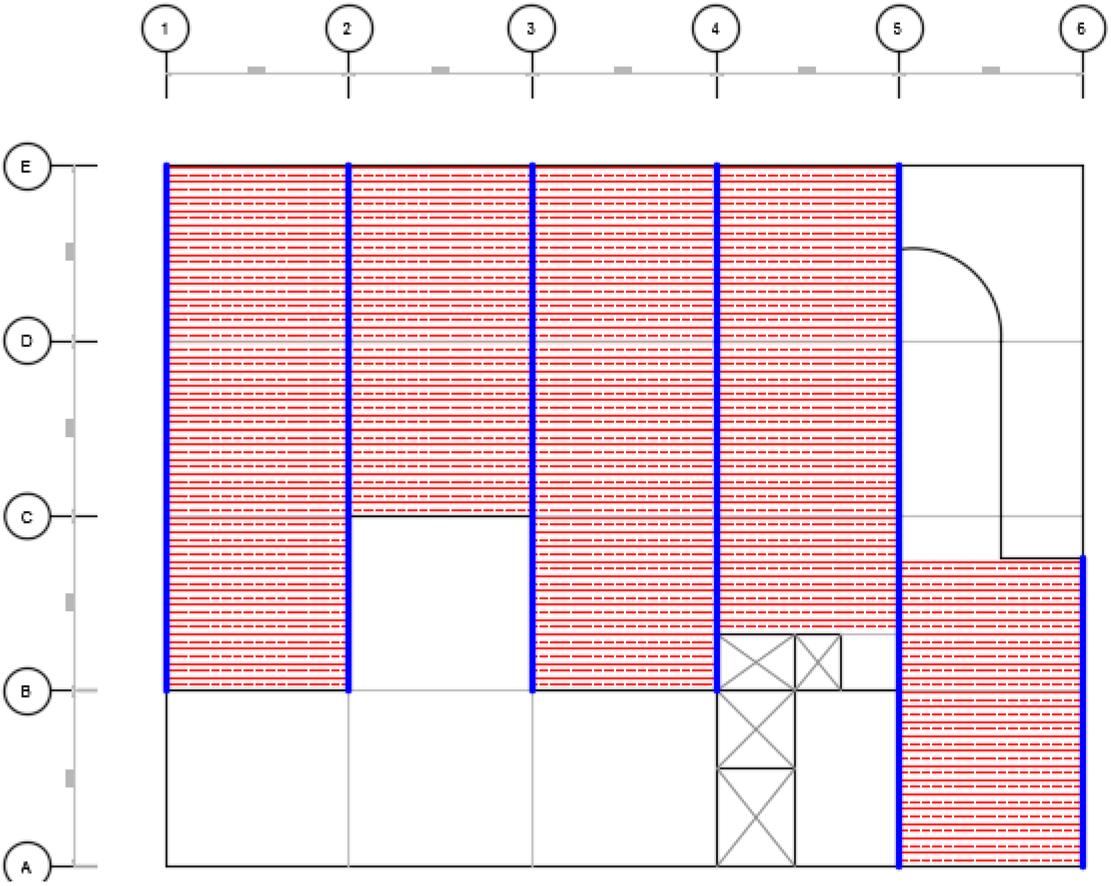
Diameter = 3.15 mm
Minimum Length = 85 mm
Spacing = 149 mm

%NBS = 105%

HOLLOWCORE SLAB ASSESSMENT

Seismic Assessment of Existing Buildings, Technical Proposal to Revise the Engineering Assessment Guidelines (30/11/2018)
NZS3101:2006 Concrete Structures

1997 carpark mezzanine consists of 150 hollowcore units, 65mm topping slab w/ 665 mesh, continuous over whole slab. Hollowcore spans over 410UB54 beams w/ M20 nelson studs @ 400 centres. 50mm seating provided as per drawings.
Assessing using technical proposal as structure has already been strengthened (K&D, 2009) to above 34%NBS.



Above: Mezzanine plan view.
Shaded area indicartes hollowcore.
Sections excluded are hibond flooring,
(ie, between grids 4 and 6)

1.0 - Beam Rotation Demand

1.1 - Beam elongation

Spandrel 'beams' are not at same level as hollowcore units; integral with slab above.
Note also that site inspection shows internal slab thickenings perpindicular to steel beams specified on 1997 are not present. Beam elongation not significant.

1.2 - Rotation of support beam

Ledge height (vertical distance from top of slab) $h_L := 150\text{mm}$

Beam rotation (typically estimated as column drift ratio, for cases where majority of frame drift is accommodated by elastic and plastic deformation of beams) $\theta := \frac{6\text{mm}}{3.37\text{m}} = 0.178 \cdot \%$

Height of beam $h_b := 410\text{mm} + h_L$ (410UB)

Movement between precast unit and support ledge due to rotationof the support beam - **within** elongation zone $\delta_{r1} := \left(\frac{h_b}{2} - h_L\right) \cdot \theta = 0.23 \cdot \text{mm}$

Note that equations are written for RC support beams with integral ledges, not UB sections with seating on top of section.

Movement between precast unit and support ledge due to rotationof the support beam - **outside** elongation zone $\delta_{r2} := h_L \cdot \theta = 0.27 \cdot \text{mm}$

1.3 - Unit movement due to plastic strain in starter bars

Elastic drift $\delta_e := \frac{7\text{mm}}{3.37\text{m}}$

Total drift $\delta := \frac{12\text{mm}}{3.37\text{m}}$

Unit length $L := 8.0\text{m}$

Distance from column CL to ledge face $s := 0.5 \cdot 178\text{mm} - 50\text{mm} = 39 \cdot \text{mm}$

Depth from seating to top of beam $h_L := 50\text{mm}$

Cover to CL of starter bars $d' := 30\text{mm}$

$$\theta_p := (\delta - \delta_e) \cdot \frac{L}{(L - 2s)} = 1.498 \times 10^{-3} \cdot \text{rad}$$

$$\delta_{el\_unit} := \min\left[1.3 \cdot \frac{\theta_p}{2} \cdot (h_L - d'), 0.018 \cdot h_L\right] = 0.02 \cdot \text{mm}$$

1.4 - Total displacement

Sum of beam elongations $\Sigma\delta_{el} := 0$

Total movement of precast floor units relative to the ledge providing support due to elongation and rotation of support beams, for units supported **within** the elongation zone

$$\delta_{tot.1} := \max\left(\Sigma\delta_{el} + \delta_{r1}, \delta_{r2} + \delta_{el\_unit}\right) = 0.287 \cdot mm$$

Total movement of precast floor units relative to the ledge providing support due to elongation and rotation of support beams, for units supported **outside** the elongation zone

$$\delta_{tot.2} := \delta_{r2} + \delta_{el\_unit} = 0.287 \cdot mm$$

2.0 - Interstorey Drift Capacity of Hollowcore System

2.1 - Loss of seating

2.1.1 - Seating provided

Seating provided on plans $L_1 := 50mm$

Construction tolerance reduction $L_2 := 20mm$

Creep / shrinkage reduction $L_3 := \frac{0.6mm}{m} \cdot 8.0m = 4.8 \cdot mm$

Seating provided - final $L_{seat} := L_1 - L_2 - L_3 - \max\left(\delta_{tot.1}, \delta_{tot.2}\right) = 24.9 \cdot mm$

2.1.2 - Seating required

Note that seating requirements are based on bearing failure. As ledge is steel not concrete, bearing of the hollowcore units themselves will govern.
Refer NZS3101:2006, cl. 16.3.

Strength reduction factor $\phi := 0.75$

Probable compressive concrete strength $f'_c := 30MPa$

Maximum bearing strength of concrete $fc_{max} := \phi \cdot \left(0.74 \cdot f'_c\right) = 16.6 \cdot MPa$

Loaded area $A_1 := 31mm \cdot 30mm = 930 \cdot mm^2$

$$A_2 := 89mm \cdot L_{seat} = 2217 \cdot mm^2$$

Bearing strength $N_c := A_1 \cdot fc_{max} = 15 \cdot kN$

Gravity loads $1.2G{+}1.5Q := \frac{8.0m}{2} \cdot 140mm \cdot \left(1.2 \cdot 24 \frac{kN}{m^3} \cdot 215mm + 1.5 \cdot 2.5kPa\right) = 5.57 \cdot kN$

3.0 - Moment failure

Insufficient information is available regarding the reinforcing of the hollowcore units to establish moment capacities. Note however that total plastic drifts (Ductility = 1.0, full frame hinging) at mezzanine height are below 0.5%. Therefore additional bending moment through units is not expected to be significant. Units OK.

Story Response - Maximum Story Displacement

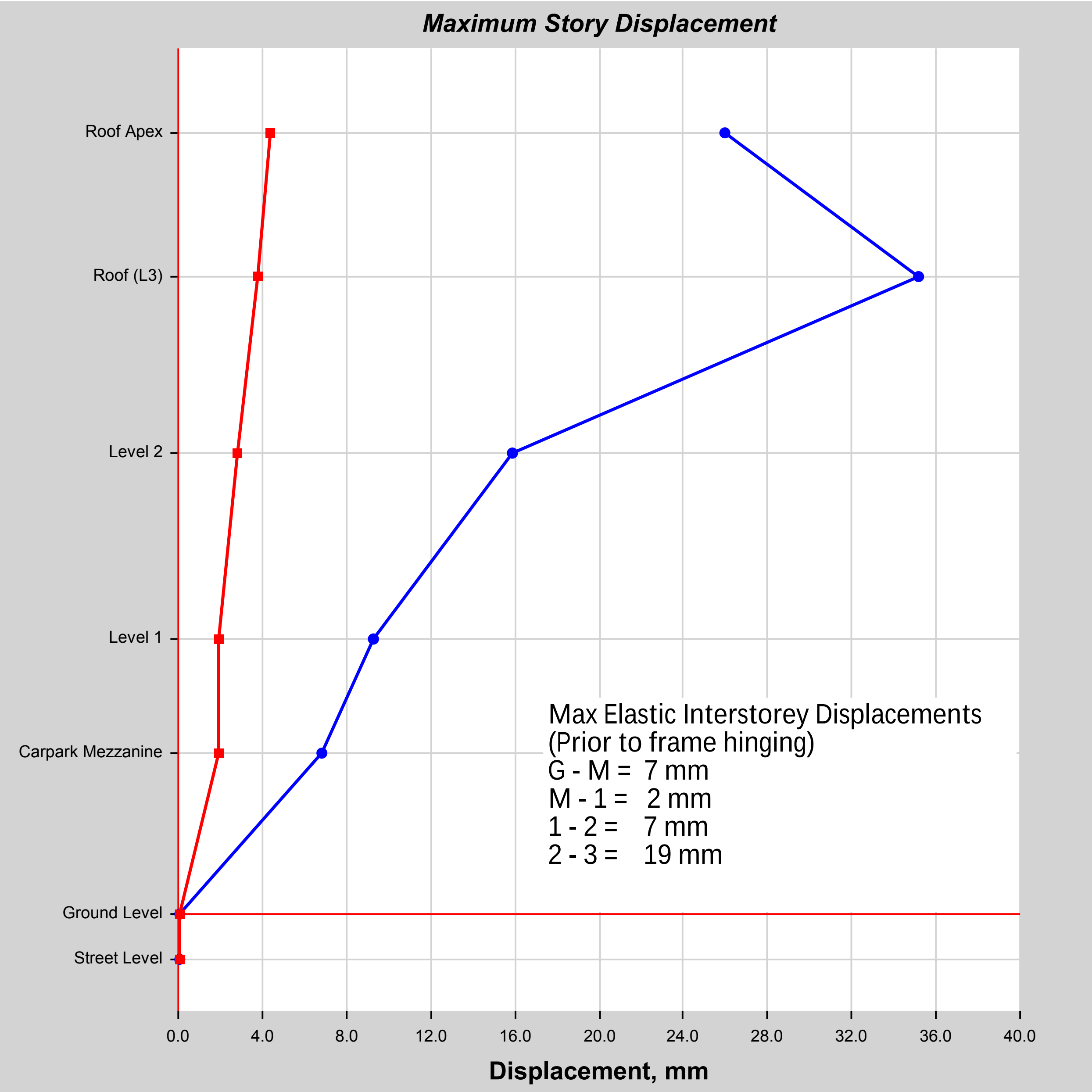
Summary Description

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

Input Data

Name	StoryResp2		
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	StoryResp7		
Display Type	Max story displ	Story Range	All Stories
Load Combo	Ductility 1.0 Combo X	Top Story	Roof Apex
Output Type	Max	Bottom Story	Street Level

Plot

COLUMN CURVATURE CAPACITY

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

1.0 - Ground Level Columns

1.1 - General

Steel yield strength $f_y := 270\text{MPa}$

Youngs modulus of steel $E_s := 200\text{GPa}$

Strain at point of probable yield of longitudinal tension reinforcement $\varepsilon_y := \dfrac{f_y}{E_s} = 1.35 \times 10^{-3}$

1.2 - Ground Level Columns (Full height)

Column depth $H_c := 22\text{in}$

Length of shear span $L_c := 4870\text{mm}$

Equivalent yield curvature $\phi_y := \dfrac{2.0 \cdot \varepsilon_y}{H_c} = 4.83 \times 10^{-6} \cdot \text{mm}^{-1}$

Beta V (dimensionless parameter) $\beta_V := 1.0$

Effective yield drift ratio $\lambda_G := \beta_V \cdot \phi_y \cdot \left(\dfrac{L_c}{3}\right) = 0.784 \cdot \%$

Drift limit $\Delta_G := \lambda_G \cdot L_c = 38 \cdot \text{mm}$

1.3 - Level 0 Columns (Partial height, ground to mezz)

Column depth $H_c := 22\text{in}$

Length of shear span $L_c := 3155\text{mm}$

Equivalent yield curvature $\phi_y := \dfrac{2.00 \cdot \varepsilon_y}{H_c} = 4.83 \times 10^{-6} \cdot \text{mm}^{-1}$

Beta V (dimensionless parameter) $\beta_V := 1.0$

Effective yield drift ratio $\lambda_0 := \beta_V \cdot \phi_y \cdot \left(\dfrac{L_c}{3}\right) = 0.508 \cdot \%$

Drift limit $\Delta_0 := \lambda_G \cdot L_c = 25 \cdot \text{mm}$

1.4 - Level 0.5 Columns (Partial height, mezz to L1)

Column depth $H_c := 22\text{in}$

Length of shear span $L_c := 1500\text{mm}$

Equivalent yield curvature $\phi_y := \dfrac{2.00 \cdot \varepsilon_y}{H_c} = 4.83 \times 10^{-6} \cdot \text{mm}^{-1}$

Beta V (dimensionless parameter) $\beta_v := 1.0$

Effective yield drift ratio $\lambda_{0.5} := \beta_v \cdot \phi_y \cdot \left(\frac{L_c}{3}\right) = 0.242 \cdot \%$

Drift limit $\Delta_{0.5} := \lambda_{0.5} \cdot L_c = 4 \cdot \mathrm{mm}$

1.5 - Level 1 Columns

Column depth $H_c := 16\mathrm{in}$

Length of shear span $L_c := 2965\mathrm{mm}$

Equivalent yield curvature $\phi_y := \frac{2.00 \cdot \varepsilon_y}{H_c} = 6.64 \times 10^{-6} \cdot \mathrm{mm}^{-1}$

Beta V (dimensionless parameter) $\beta_v := 1.0$

Effective yield drift ratio $\lambda_1 := \beta_v \cdot \phi_y \cdot \left(\frac{L_c}{3}\right) = 0.657 \cdot \%$

Drift limit $\Delta_1 := \lambda_1 \cdot L_c = 19 \cdot \mathrm{mm}$

1.6 - Level 2 Columns

Column depth $H_c := 12\mathrm{in}$

Length of shear span $L_c := 3100\mathrm{mm}$

Equivalent yield curvature $\phi_y := \frac{2.00 \cdot \varepsilon_y}{H_c} = 8.86 \times 10^{-6} \cdot \mathrm{mm}^{-1}$

Beta V (dimensionless parameter) $\beta_v := 1.0$

Effective yield drift ratio $\lambda_2 := \beta_v \cdot \phi_y \cdot \left(\frac{L_c}{3}\right) = 0.915 \cdot \%$

Drift limit $\Delta_2 := \lambda_2 \cdot L_c = 28 \cdot \mathrm{mm}$

1.7 - Summary

$\Delta_G = 38 \cdot \mathrm{mm}$

$\Delta_0 = 25 \cdot \mathrm{mm}$

$\Delta_{0.5} = 4 \cdot \mathrm{mm}$

$\Delta_1 = 19 \cdot \mathrm{mm}$

$\Delta_2 = 28 \cdot \mathrm{mm}$

2.0 - Interstorey Displacement Checks

2.1 - Prior to frame hinging

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

$\Delta^*_0 := 7\text{mm}$ $\qquad \dfrac{\Delta_0}{\Delta^*_0} = 354 \cdot \%$

$\Delta^*_{0.5} := 2\text{mm}$ $\qquad \dfrac{\Delta_{0.5}}{\Delta^*_{0.5}} = 181 \cdot \%$

$\Delta^*_1 := 7\text{mm}$ $\qquad \dfrac{\Delta_1}{\Delta^*_1} = 278 \cdot \%$

$\Delta^*_2 := 19\text{mm}$ $\qquad \dfrac{\Delta_2}{\Delta^*_2} = 149 \cdot \%$

2.2 - Post frame hinging (ductility = 1.0)

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

$\Delta^*_0 := 20\text{mm}$ $\qquad \dfrac{\Delta_0}{\Delta^*_0} = 124 \cdot \%$

$\Delta^*_{0.5} := 5\text{mm}$ $\qquad \dfrac{\Delta_{0.5}}{\Delta^*_{0.5}} = 72 \cdot \%$

$\Delta^*_1 := 38\text{mm}$ $\qquad \dfrac{\Delta_1}{\Delta^*_1} = 51 \cdot \%$

$\Delta^*_2 := 142\text{mm}$ $\qquad \dfrac{\Delta_2}{\Delta^*_2} = 20 \cdot \%$

2.3 - Conclusions

Under initial seismic load, elastic deflections are within acceptable limits. However, as calculated in sections 2 and 3, design forces exceed frame element strengths (front and rear perimeter frames, Grids A and E) and plastic hinging will occur.

In order to calculate worst case scenario effects, all modelled frame elements on grids A and E have been released at both ends. While this has redistributed design forces, the above calc shows post hinging displacements well in excess of what the columns can sustain, likely causing loss of axial load bearing capacity and a collapse mechanism.