CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK

Sheet No S/28a of

Project Description:

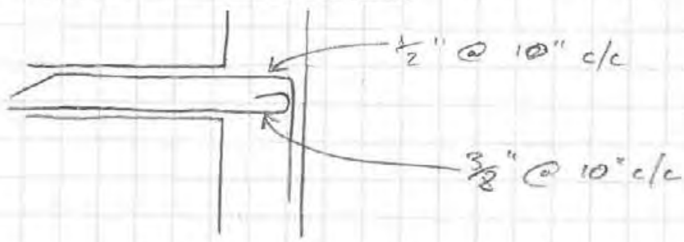
Office:

Computed: PMO 22/09/2015

Check: / /

1ST FLOOR DIAPHRAGM

CONNECTION TO WALLS



Total = (71 + 127) @ 254 mm c/c = 198 mm² @ 254mm

f_y = 245 MPa

Using Shear friction approach: $\frac{198 \times 245}{254} \times 1000$ = 191 kN/m

0.75 x 191 = 143 kN/m

For Typical 8" wall : Length = 8.867m - GL 2

∴ 143 x 8.867 = 1268 kN

↳ Greater than demand. Mass:
GL 2 = 1138 - 290 = 848 kN
GL 14 = 1511 - 549 = 962 kN

∴ O.K.

CHECK SLAB FOR SHEAR

Using wall analogy:

d = 0.8 l = 11.379 x 0.8 = 9.103
Bars = ⅜" @ 10" c/c
V_p = 1275 kN — O.K.

∴ $\boxed{\text{DIAPHRAGM O.K. FOR SHEAR}}$

CHECK SLAB FOR BENDING

Aspect ratio of slab sections:
Worse case = 7 : 5 = 1.4

↳ Shear governed by inspection.

∴ $\boxed{\text{DIAPHRAGM O.K. FOR BENDING}}$

OPUS

CALCULATION SHEET S/28 b

Project/Task/File No: WEBC EAST WING Sheet No of

Project Description: Office:

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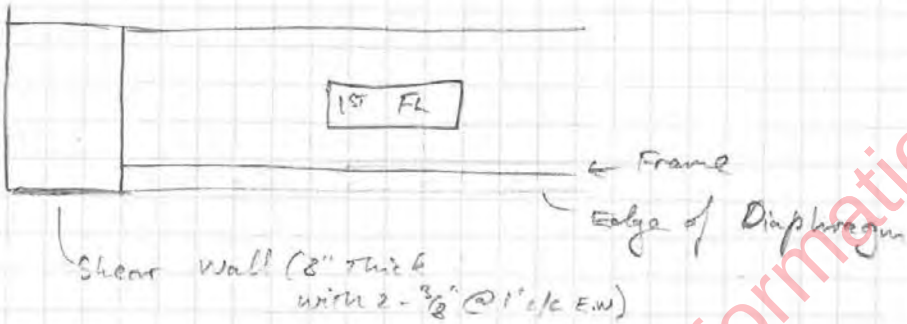
EXISTING CONCRETE FRAME ON NORTH ELEVATION

SHEAR WALL AT EAST END :

Wall not directly connected to diaphragm at 1st Floor :

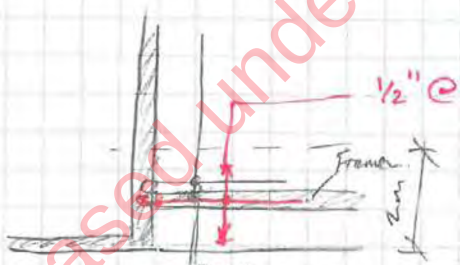


- Some bending in wall expected (out-of-plane) to transfer forces.

CAPACITY OF WALL

8" Thick → $\phi V_c + \phi V_s$ = 792 kN

CAPACITY OF SLAB TO WALL CONNECTION



1/2" @ 10" = 6 × 127 × 245 = 187 kN

3/8" @ 12" = 5 × 71 × 245 = 87 kN

IN SPANDREL :

2 × 1" Bars = 2 × 507 × 245 = 248 kN

5 × 5/8" Bars = 5 × 198 × 245 = 242 kN

764 kN

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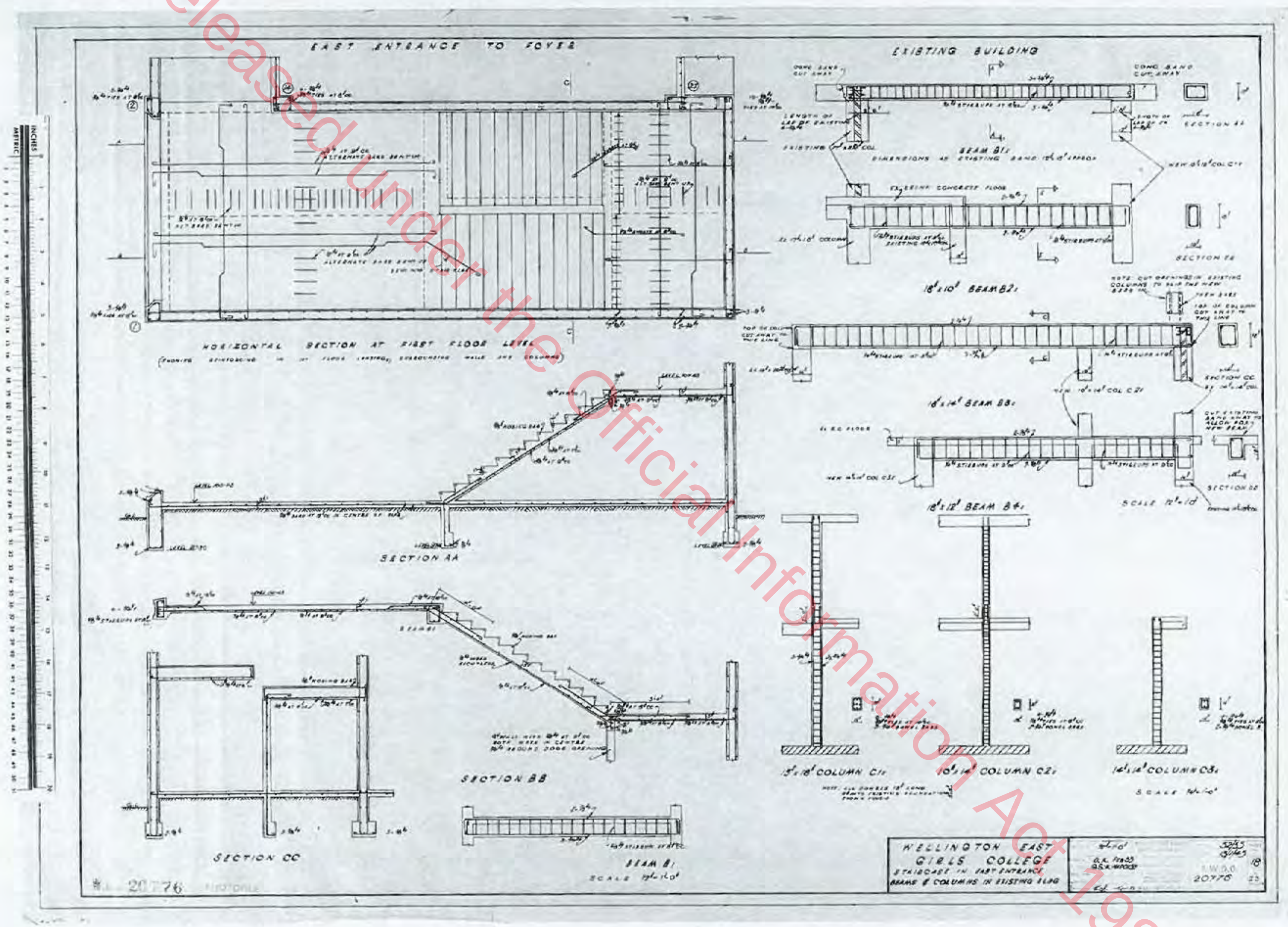
S/28c


EAST ENTRANCE TO FOYER
HORIZONTAL SECTION AT FIRST FLOOR LEVEL
SECTION AA
SECTION BB
SECTION CC
EXISTING BUILDING
BEAM B1
18"x10" BEAM B2
16"x14" BEAM B3
18"x12" BEAM B4
15"x18" COLUMN C1
10"x14" COLUMN C2
14"x14" COLUMN C3
WELLINGTON EAST
GIRLS COLLEGE
STAIRCASE IN EAST ENTRANCE
BEAMS & COLUMNS IN EXISTING BLDG
20776
INCHES
METRIC

S/28d

CALCULATION SHEET

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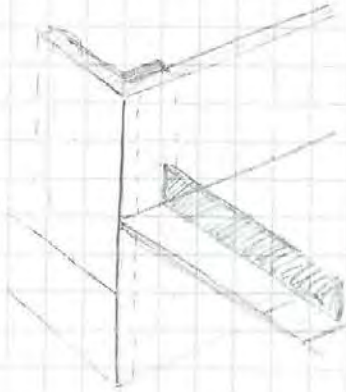
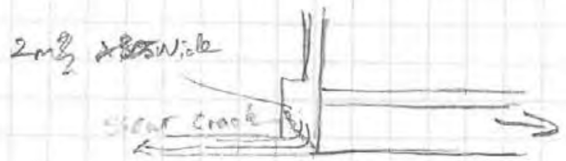
CONNECTION THROUGH SHEAR BETWEEN DIAPHRAGM & SHEAR WALL





12" Thick column

If v_{max} = 6 MPa

$d = \sim 275$mm

$l = 2000$

Shear reinforcement crossing crack = $2 \times 3/8" + 2 \times 5/8"$

$\therefore \phi V_s = 2 \times \frac{71 \times 275}{305} \times 245 \times 0.75 = 24$

$+ 2 \times \frac{198}{254} \times 275 \times 245 \times 0.75 = 79$

$\phi V_c = 0.2 \times \sqrt{30} \times 275 \times 2000 \times 0.75 = 451$

~ 554 kN capacity.

— Possibly some damage around connection between end shear wall and diaphragm.

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CALCULATION SHEET

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Project/Task/File No: WEGC EAST BLOCK

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EXTERNAL CBF ON NORTH ELEVATION

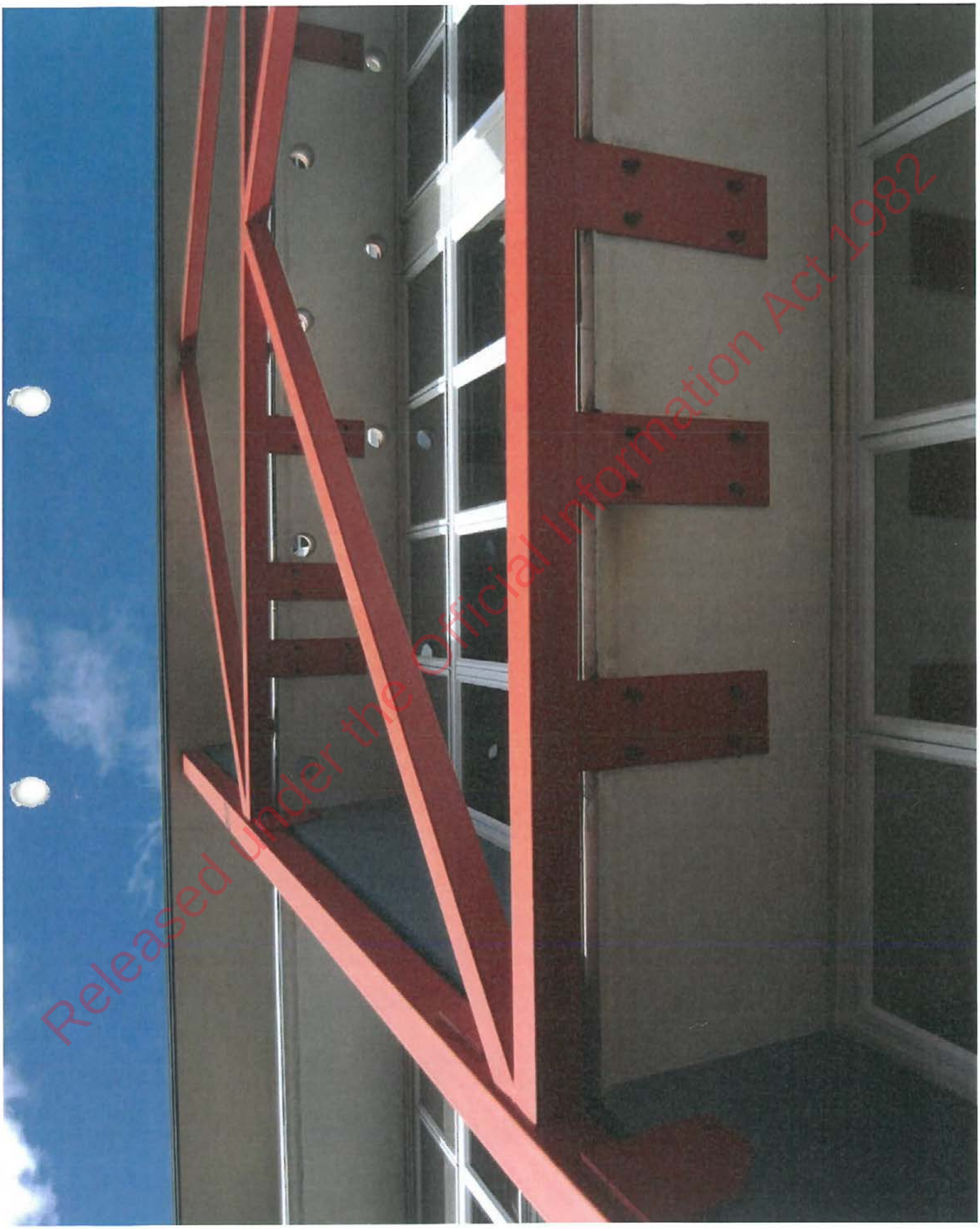
- CBF formed with 200x6 SHS columns and 150x5 SHS diagonals and collector beams.
- Fixed at 1st Fl & 2nd Fl to original part of building + at Roof (Newer part).

OPUS

5/30

S/31

S/32a



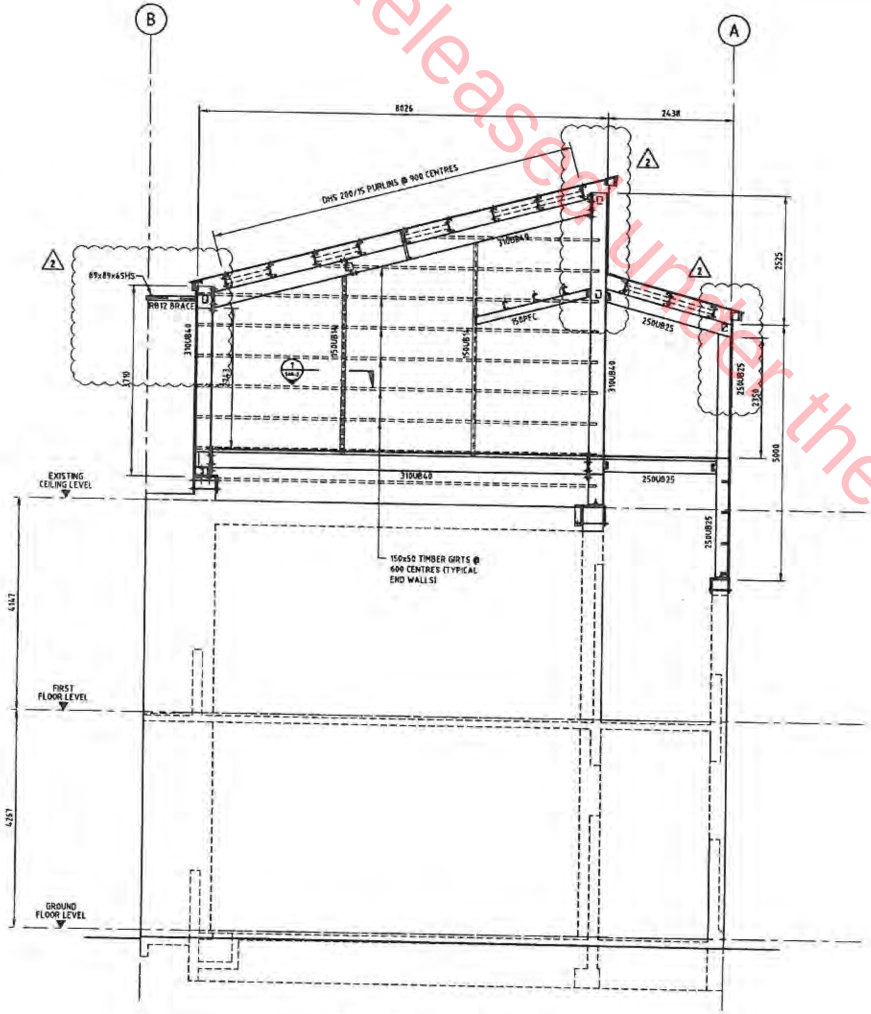
S/326

S/32c

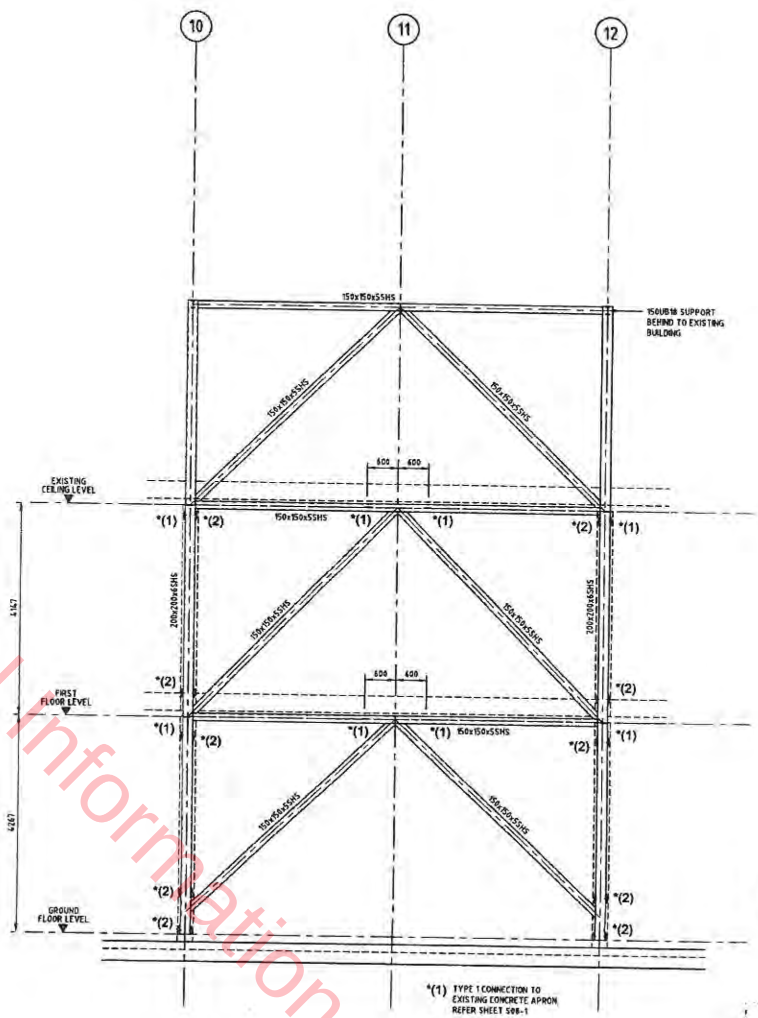
S/32d

S/33

A1



END ELEVATION (WEST)
(VIEWED FROM OUTSIDE - PORTAL SETOUT)
SECTION 5 S01 S02
SCALE 1:50



*(1) TYPE 1 CONNECTION TO EXISTING CONCRETE APRON REFER SHEET S08-1
*(2) TYPE 2 CONNECTION TO EXISTING CONCRETE COLUMN, REFER SHEET S08-1

SECTION 6 S01 S02
SCALE 1:50

W.C.C. RECORDS

CONSENT/TENDER

Rev.	Date	Revision Details	By	Ver.	App.
2	19.08.05	FOR CONSENT/TENDER	CM		
1	05.08.05	PRELIMINARY	ABM		

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Email: cwwell@conwag.com

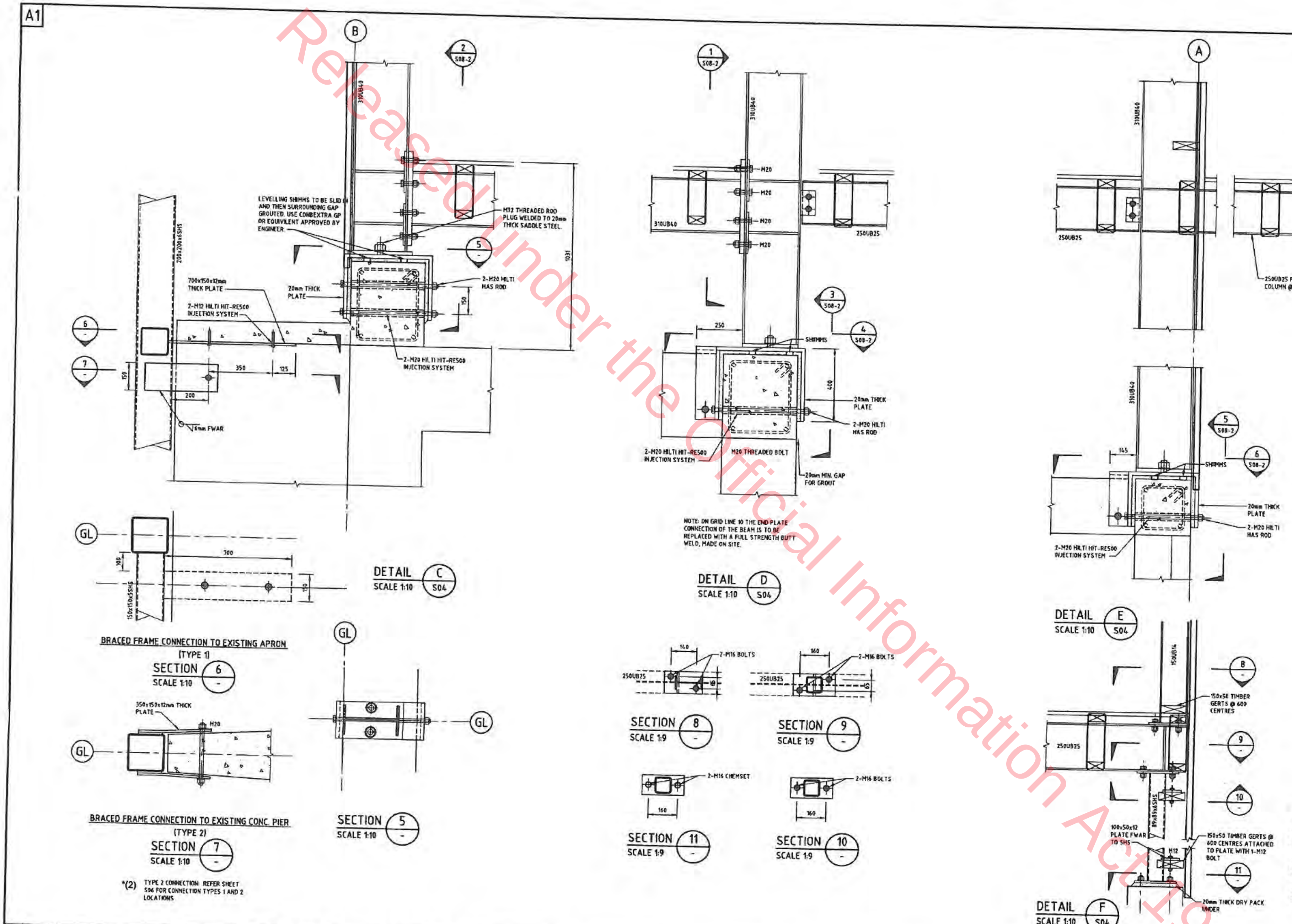
Client: WELLINGTON EAST GIRLS COLLEGE

Project: EXTENSION TO EAST WING BLOCK 7

Drawn	Signed	Date
ABM		00-00-00
Designed TW	Signed	00-00-00
Verified CJM	Signed	Date
Approved MM	Signed	Date

Drawing Title: CROSS SECTIONS SHEET THREE

CW Project No. 7970 12
Scale 1:50 (@ A1) 1:100 (@ A3)
Drawing No. S 06
Rev. 2



NOTES:

1). CONTRACTOR SHALL AVOID DAMAGING ANY EXISTING REINFORCING WHEN DRILLING. METAL DETECTOR MAY BE REQUIRED TO LOCATE EXISTING REINFORCING.

2). THE DIMENSIONS OF THE SADDL SHALL BE BASED ON SITE MEASUREMENTS TAKEN PRIOR TO WORK COMMENCING.

3). THE SADDLE STEEL SHALL BE PROPERLY SEALED BEFORE GROUTING.

CONSENT/TENDER

Rev.	Date	Revision Details	By	Ver.	App.
2	18.08.05	FOR CONSENT/TENDER	CM	V.	
1	05.08.05	PRELIMINARY	ABM	V.	

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Client: WELLINGTON EAST GIRLS COLLEGE

Project: EXTENSION TO EAST WING BLOCK 7

	Signed	Date
Drawn ABM		00-00-00
Designed TW		00-00-00
Verified CJM		
Approved MM		

Drawing Title: DETAILS SHEET TWO

CW Project No. 7970 12

Scale 1:10 (@ A1) 1:20 (@ A3)

Drawing No. S 08-1 Rev. 2

S/35

Axial Capacity Check

DIAGONAL BRACES

Axial capacity to NZS3404:1997

Force 1 kN

	Major Axis		**Minor Axis**	
	Clear length x axis	5788 mm	Clear length y axis	5788 mm
4.8.3	K_{ex}	1	K_{ey}	1
	$K_{ex}L_x$	5788 mm	$K_{ey}L_y$	5788 mm
	Member	SHS - 350LO 150 x 150 x 5.0 SHS		
	f_y	350 MPa		
	k_f	1.0		
	r_x	58.7 mm	r_y	58.7 mm
	$(k_{ex}L_x/r_x) \times \sqrt{(f_y/250)}$	117	$(k_{ey}L_y/r_y) \times \sqrt{(f_y/250)}$	117
6.2	Ns	984 kN		
Table 3.3	ϕ	0.9		
	ϕNs	885 kN		
6.3.3	λ_n	116.669		
	α_a	15.613		
Table 6.3.3(1)	α_b	-0.5		
	λ	108.862		
	η	0.311		
	ξ	0.948		
	α_c	0.484		
6.3.3	$\alpha_c N_s$	476 kN		
Table 3.3	ϕ	0.9		
	ϕN_c	429 kN		
	Min ϕNs and ϕNc	429 kN	- OK	

150 x 150 x 5.0 SHS

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Axial Capacity Check - COLUMNS

Axial capacity to NZS3404:1997

	Force	1 kN			
	Major Axis		**Minor Axis**		
4.8.3	Clear length x axis	4267 mm	Clear length y axis	4267 mm	
	K_{ex}	1	K_{ey}	1	
	$K_{ex}L_x$	4267 mm	$K_{ey}L_y$	4267 mm	
	Member	SHS - 350LO 200 x 200 x 6.0 SHS			
	f_y	350 MPa			
	k_f	1.0			
	r_x	78.6 mm	r_y	78.6 mm	
	$(k_{ex}L_x/r_x) \times \sqrt{(f_y/250)}$	64	$(k_{ey}L_y/r_y) \times \sqrt{(f_y/250)}$	64	
6.2	Ns	1586 kN			
Table 3.3	ϕ	0.9			
	ϕNs	1427 kN			
6.3.3	λ_n	64.234			
	α_a	20.515			
Table 6.3.3(1)	α_b	-0.5			
	λ	53.976			
	η	0.132			
	ξ	2.074			
	α_c	0.841			
6.3.3	$\alpha_c N_s$	1333 kN			
Table 3.3	ϕ	0.9			
	ϕN_c	1200 kN			
	Min ϕNs and ϕNc	1200 kN - OK			

200 x 200 x 6.0 SHS

CALCULATION SHEET

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Project Description:

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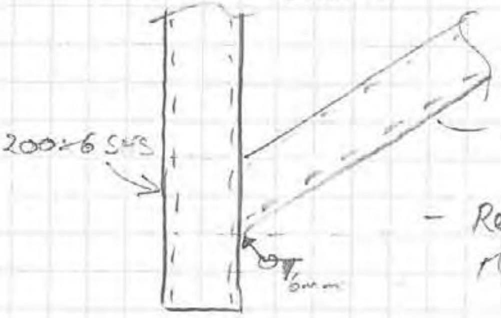
EXTERNAL BRACED FRAME

DIAGONAL - TO - COLUMN CONNECTION AT BASE

– SUPERSEDED - HAS CROSS 150×5 SHS TIE



– Relies on shear through the post.

SHEAR CAPACITY

Shear Capacity of 200×6 SHS

– 350 Grade :

5.11.4 $V_w = 0.6 f_y A_w = 0.6 \times 350 \times 200 \times 6 \times 2 = 504\ kN$

Force from brace at $f_y A_g$ = 984 kN × sine 45 = 696 kN

2 Shear Planes ∴ 504 × 2 = 1008 kN – Probably o.k.

WELD CAPACITY

6mm Fillet all round - E48xx weld :

Throat thickness = $\frac{1}{\sqrt{2}} \times 6 = 4.24\ mm$

$f_{uw} = 480$ MPa

9.7.3.10.3 $V_w = 0.6 f_{uw}\ t_t\ k_r = 0.6 \times 480 \times 4.24 \times 1 = 1.22\ kN/mm$

Length = 4 × 150 = 600 mm

∴ 732 kN

Table 3.3 Assuming SP welds - Safety factor = 0.8

∴ 0.8 × 732 = 586 kN

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CALCULATION SHEET

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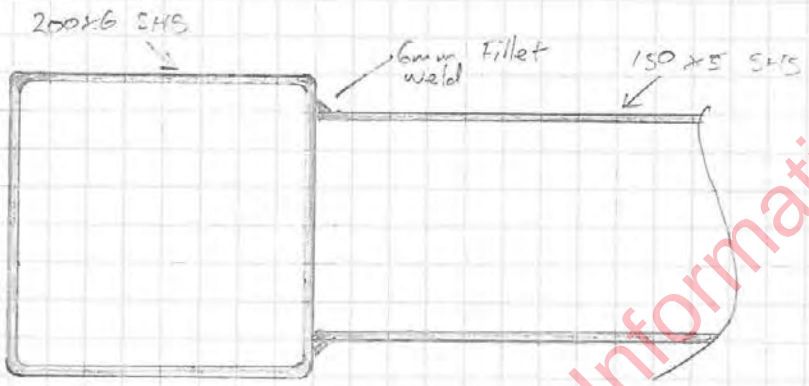
PUNCHING OF DIAGONAL THROUGH VERTICAL:

COLUMN = 200 × 6 SHS
DIAGONALS : 150 × 5 SHS



Rough check : Punching shear :

Perimeter = (150 + 2×6) × 4 = 648 mm

Shear capacity of 6mm thick plate =

0.6 × 6 × 648 × 350 = 816 kN — Better than weld capacity.

CALCULATION SHEET

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CAPACITY OF BRACED BAY

Capacity based on axial compressive strength of diagonal - worse case = 429 kN

Lateral force to generate 429 kN in brace :

Brace = 45°

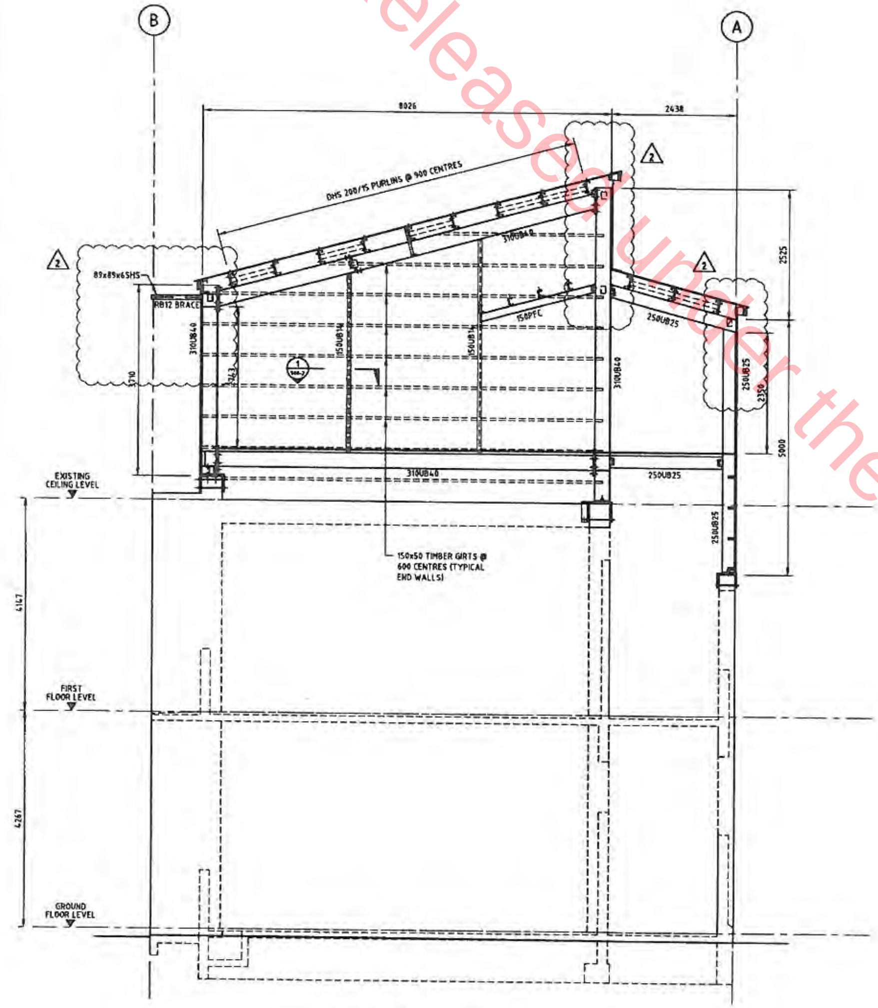
∴ 429 x Sin 45 = 303 kN

2 Braces (1 Tension, 1 compression)

∴ Capacity = 2 x 303 = 606 kN

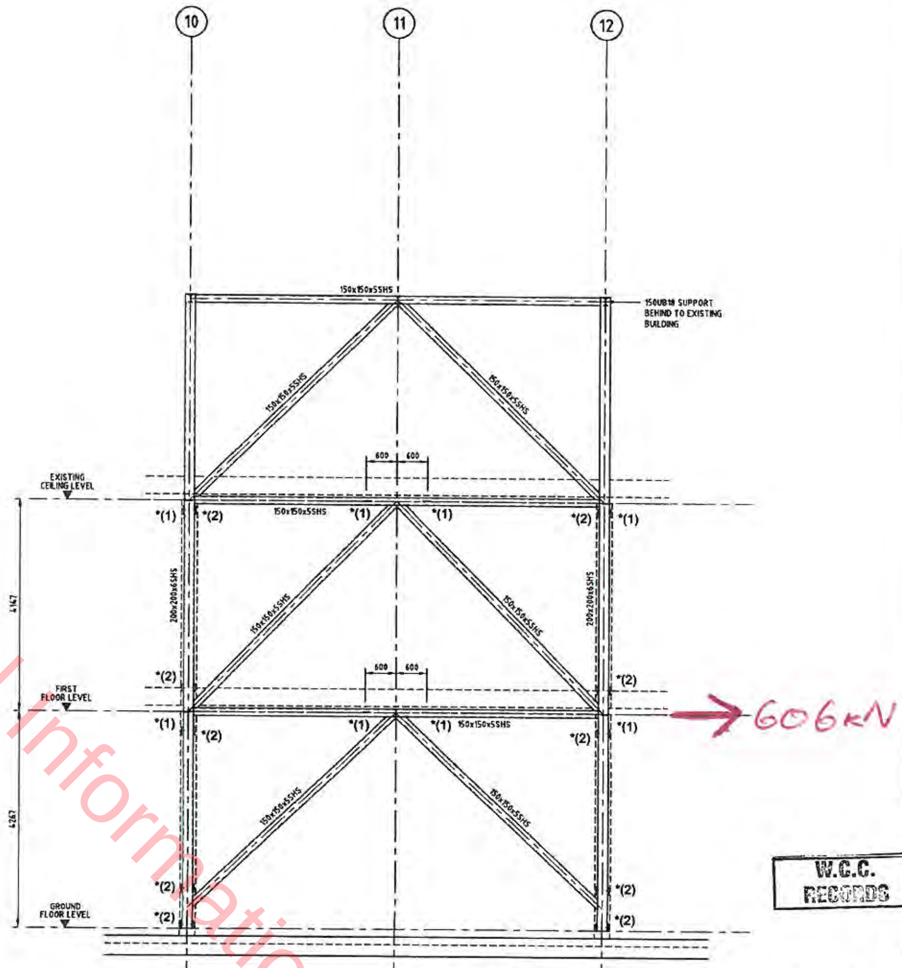
S/40

A1



END ELEVATION (WEST)
(VIEWED FROM OUTSIDE - PORTAL SETOUT)

SECTION 5 S01 S02
SCALE 1:50



*(1) TYPE 1 CONNECTION TO EXISTING CONCRETE APRON, REFER SHEET S08-1

*(2) TYPE 2 CONNECTION TO EXISTING CONCRETE COLUMN, REFER SHEET S08-1

SECTION 6 S01 S02
SCALE 1:50

W.C.C. RECORDS

CONSENT/TENDER

Rev.	Date	Revision Details	By	Ver.	App.
2	10.08.05	FOR CONSENT/TENDER	CM		
1	05.08.05	PRELIMINARY	ABM		

Connell Mott MacDonald ...building the future

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Client: WELLINGTON EAST GIRLS COLLEGE

Project: EXTENSION TO EAST WING BLOCK 7

Drawn	Signed	Date
ABM		00-00-00
Designed TW	Signed	Date 00-00-00
Verified CJM	Signed	Date
Approved MM	Signed	Date

Drawing Title: CROSS SECTIONS SHEET THREE

CW Project No. 7970 12

Scale 1:50 (@ A1) 1:100 (@ A3)

Drawing No. S 06 Rev. 2

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CALCULATION SHEET

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CONNECTION OF CBF TO BUILDING

Lateral loads are transferred via connections between floor slabs / roof slab to collector beams.

Connection = S 08-1 SECTION 6 :

Number of connections = 6

Capacity of anchor : 15.8 kN each in shear

Total number of fixings : 6×2 = 12 Per floor =

= 12 × 15.8 = 202 kN / Floor

If higher Grade anchors : 21 kN / Anchor – Ramset G 5.8

12 × 21 = 252 kN

If Stainless = 27 kN / Anchor

= 324 kN

If Grade 8.8 Hilti = 27.2 kN / Anchor

= 326 kN

∴ Total capacity of anchors in shear = 182 kN – 326 kN

However, cleat connection unlikely to prevent twist and the connection is very eccentric.....

∴ Anchors will not be loaded equally and unlikely to achive overall transfer of shear from floor.

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CALCULATION SHEET

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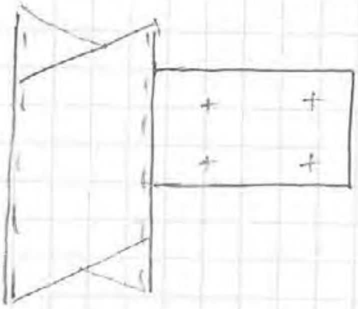
RE-CHECK FOR REVISED BOLT GROUP



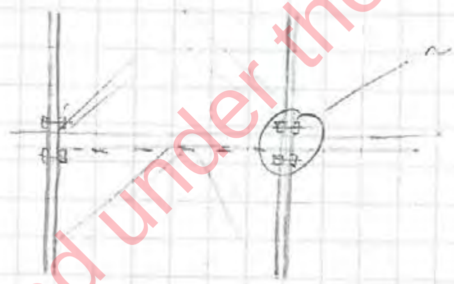
Connection capacity ≈ 25 - 33 kN per fixing point. - See next page.

Number of fixings per level = 6

6 x 25 = 150 150 kN / Floor

6 x 33 = 198 → 200 kN / Floor

Including post connections :



Total = 2 x 600 + 200 = 1400 kN

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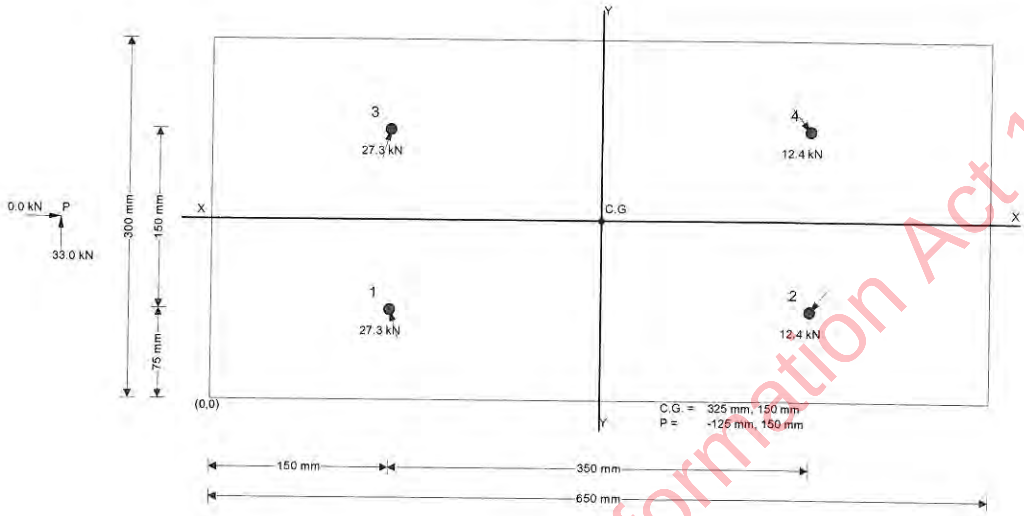
	Calcs for			Start page no./Revision 1	
	Calcs by PMO	Calcs date 21/09/2015	Checked by	Checked date	Approved by	Approved date

BOLT GROUP ANALYSIS

TEDDS calculation version 1.0.00



Geometry of bolt group

Number of rows	R = **2**
Number of columns	C = **2**
Pitch distance	S_x = **350** mm
Gauge distance	S_y = **150** mm
Edge distance in vertical direction	d_y = **75** mm
Edge distance in horizontal direction	d_x = **150** mm

Load data

Vertical load applied on bolt group	P_y = **33.000** kN
Horizontal load applied on bolt group	P_x = **0.000** kN
X coordinate of vertical force	X = **-125** mm
Y coordinate of horizontal force	Y = **150** mm

Center of gravity of bolt group

X distance of center of bolt group	$X_c = ((C - 1) \times S_x) / 2 + d_x$ = **325** mm
Y distance of center of bolt group	$Y_c = ((R - 1) \times S_y) / 2 + d_y$ = **150** mm

Load eccentricity from center of gravity of bolt group

Eccentricity of vertical load from C.G.	$e_x = \text{abs}(X - X_c)$ = **450** mm
Eccentricity of horizontal load from C.G.	$e_y = \text{abs}(Y - Y_c)$ = **0** mm
Moment about center of gravity	$M = P_x \times (Y - Y_c) - P_y \times (X - X_c)$ = **14.850** kNm

Bolt number	**Bolt distance from centre of gravity**		**Direct shear**		**Torsional shear**		**Total force**
	X_i (mm)	Y_i (mm)	P_{dx} (kN)	P_{dy} (kN)	P_{tx} (kN)	P_{ty} (kN)	(kN)
1	-175	-75	0.0	-8.2	7.7	-17.9	27.3
2	175	-75	0.0	-8.2	7.7	17.9	12.4
3	-175	75	0.0	-8.2	-7.7	-17.9	27.3
4	175	75	0.0	-8.2	-7.7	17.9	12.4

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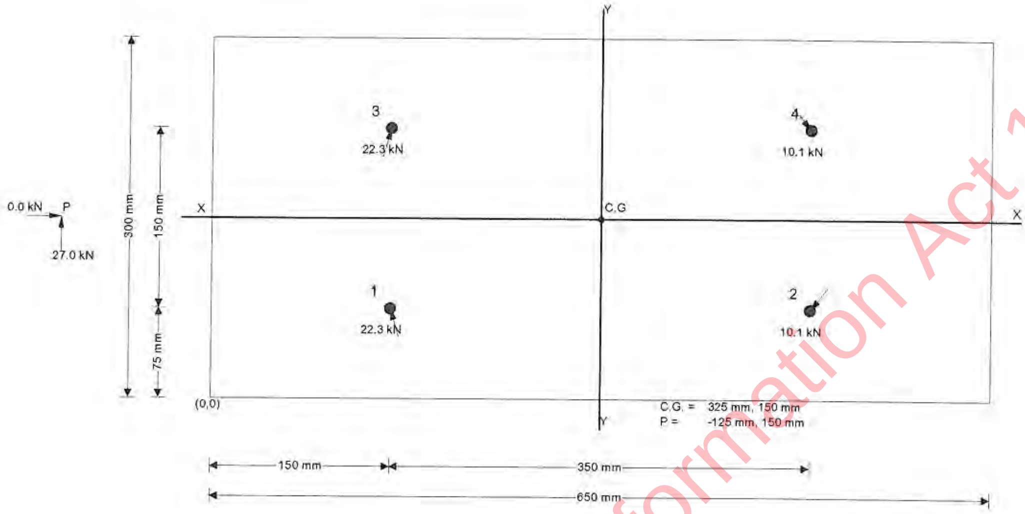
	Calcs for				Start page no./Revision 1	
	Calcs by PMO	Calcs date 21/09/2015	Checked by	Checked date	Approved by	Approved date

BOLT GROUP ANALYSIS

TEDDS calculation version 1.0.00



Geometry of bolt group

Number of rows	R = **2**
Number of columns	C = **2**
Pitch distance	S_x = **350** mm
Gauge distance	S_y = **150** mm
Edge distance in vertical direction	d_y = **75** mm
Edge distance in horizontal direction	d_x = **150** mm

Load data

Vertical load applied on bolt group	P_y = **27.000** kN
Horizontal load applied on bolt group	P_x = **0.000** kN
X coordinate of vertical force	X = **-125** mm
Y coordinate of horizontal force	Y = **150** mm

Center of gravity of bolt group

X distance of center of bolt group	$X_c = ((C - 1) \times S_x) / 2 + d_x$ = **325** mm
Y distance of center of bolt group	$Y_c = ((R - 1) \times S_y) / 2 + d_y$ = **150** mm

Load eccentricity from center of gravity of bolt group

Eccentricity of vertical load from C.G.	$e_x = abs(X - X_c)$ = **450** mm
Eccentricity of horizontal load from C.G.	$e_y = abs(Y - Y_c)$ = **0** mm
Moment about center of gravity	$M = P_x \times (Y - Y_c) - P_y \times (X - X_c)$ = **12.150** kNm

Bolt number	**Bolt distance from centre of gravity**		**Direct shear**		**Torsional shear**		**Total force**
	X_i (mm)	Y_i (mm)	P_{dx} (kN)	P_{dy} (kN)	P_{tx} (kN)	P_{ty} (kN)	(kN)
1	-175	-75	0.0	-6.7	6.3	-14.7	22.3
2	175	-75	0.0	-6.7	6.3	14.7	10.1
3	-175	75	0.0	-6.7	-6.3	-14.7	22.3
4	175	75	0.0	-6.7	-6.3	14.7	10.1

CALCULATION SHEET

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FIXINGS BETWEEN FLOOR SLAB AND HORIZONTAL

Relying only on the anchors closest to the CBF:

Total = 12

Capacity of each anchor - M12 Hilti HIT = 16.8 kN Factored

If $\phi = 1.0$ → $\frac{16.8}{0.8} = 21$ kN

∴ 12 × 21 = 252 kN

This is based on having little moment in the bolt group, so that the fixing cleats and 150 SHS would need to be relatively stiff and not deform significantly.

DEMAND

Case 1. If X-Braces at top floor are effective = 306 kN × 2 = 612 kN

2. " " " " " " not " = 538 kN × 2 = 1076 kN

TOTAL CAPACITY = 552 kN

∴ Case 1 = $\frac{552}{612}$ = 90%

$\frac{552}{1076}$ = 51%

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CALCULATION SHEET

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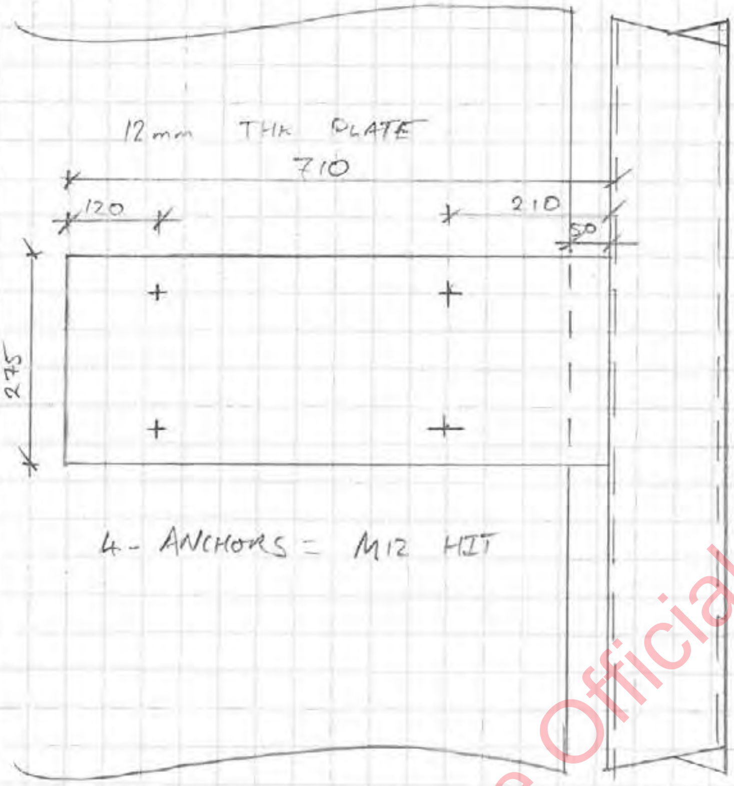
CAPACITY OF CONNECTIONS

- Based on site measure - November 2015



Applying load to edge of plate (210mm from first row of bolts), and assuming no resistance to rotation of plate:

Capacity = 31 kN.

But, some resistance to rotation of plate will be provided by the SHS, in which case, absolute upper bound would be 4 × 21 = 84 kN.

Likely capacity between these two bounds:

$31\,kN \leq Cap \leq 84\,kN$

Likely capacity = 2 × 21 = 42 kN - still nearer the conservative end of the scale:

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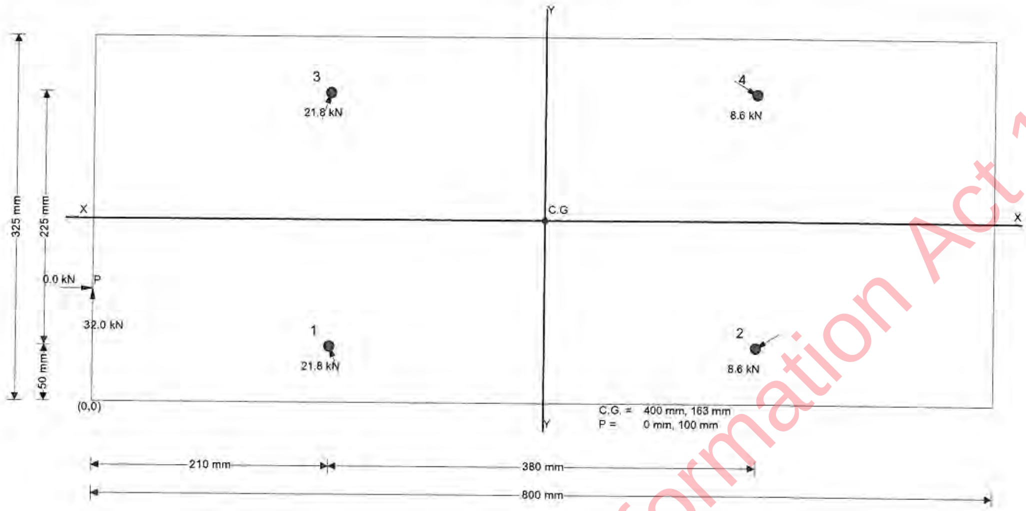
Tedds Opus International Consultants Ltd Majestic Centre, 100 Willis Street PO Box 12003, Wellington 6144 New Zealand	Project WEGC CBF Connections				Job no. S/44 C	
	Calcs for				Start page no./Revision 1	
	Calcs by PMO	Calcs date 30/11/2015	Checked by	Checked date	Approved by	Approved date

BOLT GROUP ANALYSIS

TEDDS calculation version 1.0.00



Geometry of bolt group

Number of rows	R = **2**
Number of columns	C = **2**
Pitch distance	S_x = **380** mm
Gauge distance	S_y = **225** mm
Edge distance in vertical direction	d_y = **50** mm
Edge distance in horizontal direction	d_x = **210** mm

Load data

Vertical load applied on bolt group	P_y = **32.000** kN
Horizontal load applied on bolt group	P_x = **0.000** kN
X coordinate of vertical force	X = **0** mm
Y coordinate of horizontal force	Y = **100** mm

Center of gravity of bolt group

X distance of center of bolt group	$X_c = ((C - 1) \times S_x) / 2 + d_x$ = **400** mm
Y distance of center of bolt group	$Y_c = ((R - 1) \times S_y) / 2 + d_y$ = **163** mm

Load eccentricity from center of gravity of bolt group

Eccentricity of vertical load from C.G.	$e_x = abs(X - X_c)$ = **400** mm
Eccentricity of horizontal load from C.G.	$e_y = abs(Y - Y_c)$ = **63** mm
Moment about center of gravity	$M = P_x \times (Y - Y_c) - P_y \times (X - X_c)$ = **12.800** kNm

Bolt number	**Bolt distance from centre of gravity**		**Direct shear**		**Torsional shear**		**Total force**
	X_i (mm)	Y_i (mm)	P_{dx} (kN)	P_{dy} (kN)	P_{tx} (kN)	P_{ty} (kN)	(kN)
1	-190	-113	0.0	-8.0	7.4	-12.5	21.8
2	190	-113	0.0	-8.0	7.4	12.5	8.6
3	-190	113	0.0	-8.0	-7.4	-12.5	21.8
4	190	113	0.0	-8.0	-7.4	12.5	8.6

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CALCULATION SHEET

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COLUMN TO CBF COLUMN CONNECTIONS

200×6 SHS column connected above/below floor to conc. piers
- S 02-1 Section 7:

CAPACITY OF TAB PLATES IN BENDING:

f_y = 310 MPa

$M_p = \frac{bd^2}{4} f_y = \frac{150 \times 12^2}{4} \times 320 = 1.73$ kNm / Plate

2 Plates ∴ 3.5 kNm

IF:

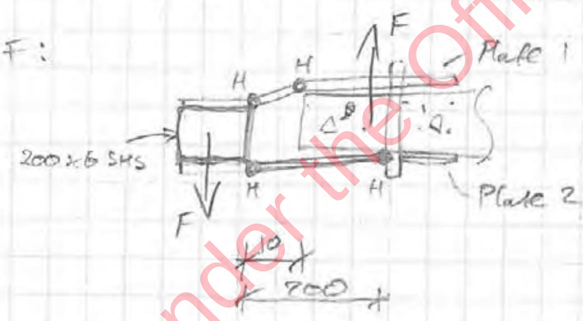

PLATE 1

Number of hinges = 2

Lever arm = 10 mm

$F_1 = \frac{2H}{10mm} = 2 \times \frac{1.73}{0.01} = 346$ kN

PLATE 2

$F_2 = \frac{2H}{0.2} = 2 \times \frac{1.73}{0.2} = 17$ kN - Small.

Check bolt:

M20 through bolt.

Lever arm: ≈ 190mm

∴ Tension in bolt: $\frac{1.73}{0.19} = 9$ kN - O.k.

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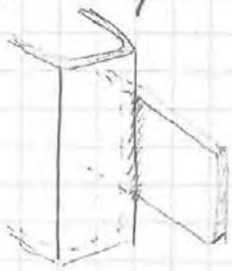
CHECK WELDS

6mm fillet weld



F = 346 kN

Capacity of weld = $\frac{6}{\sqrt{2}} \times 0.6 f_{uw} k_r = \frac{6}{\sqrt{2}} \times 0.6 \times 480 \times 1 = 1.22 kN/mm$

Grade SP. $\Rightarrow \phi = 0.8$

$\therefore 0.8 \times 150 \times 1.22 = 147 kN$ - Critical

$\therefore$ Weld failure on back of plate to column connection likely critical.

Capacity of connection around 150 kN

SHEAR STRENGTH OF PLATE

$0.6 \times 150 \times 12 \times 320 = 346 kN$

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SHEAR IN 200x6 SHS

Connection between column & conc. pier would rely on shear in box section.

Capacity = 504 kN

504 x 2 = 1008 kN – O.k.

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Project Description: Office:

Computed: PMO 27/11/2015

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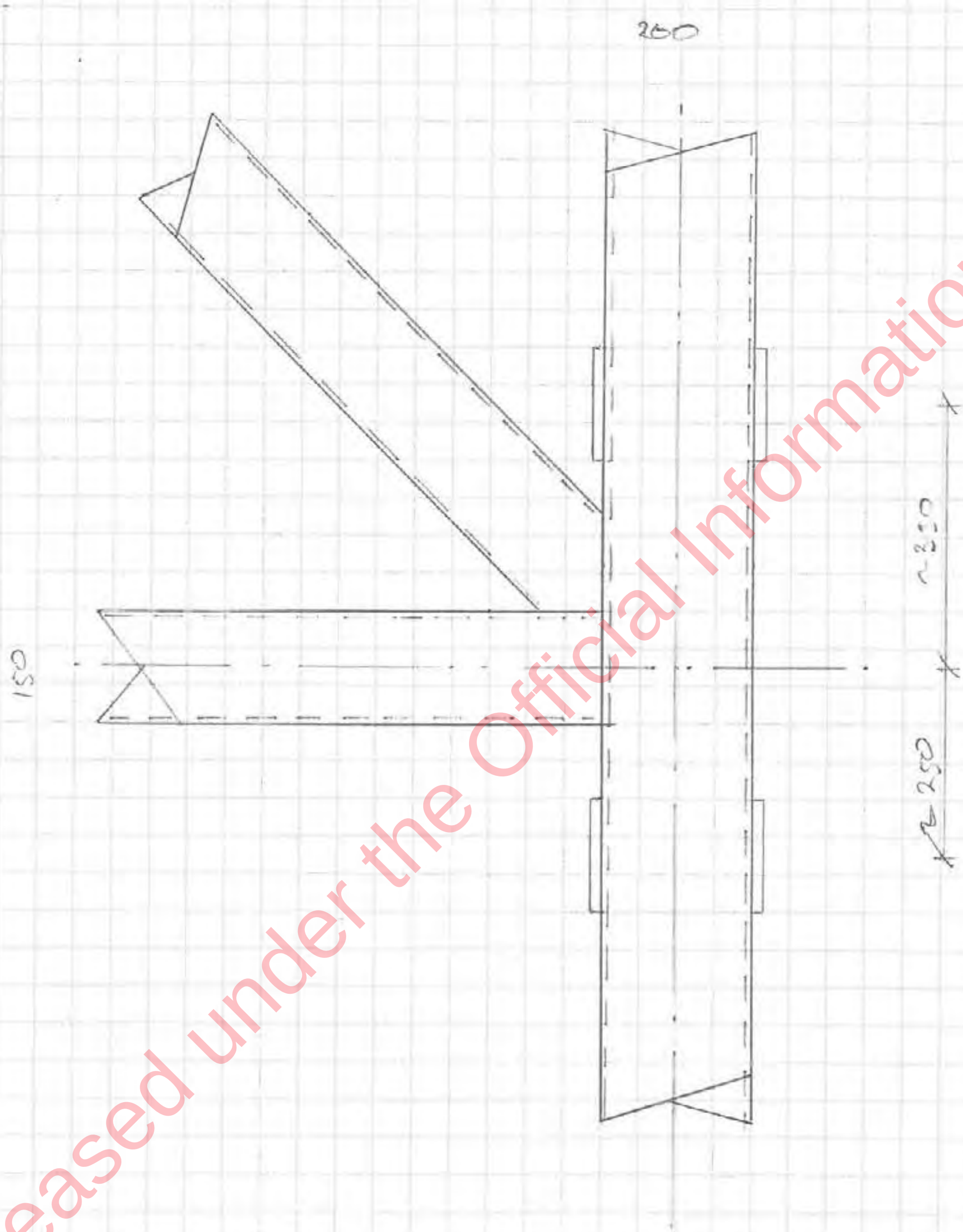
CAPACITY OF CORNER CONNECTION



Each plate connection has about 150 kN capacity

Reduce by 50% to account for some flexibility

∴ 2 × 150 × $\frac{1}{2}$ = 150 kN

Added to 252 kN, and connections at other end

= 252 + 2 × 150 = 552 kN

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OVERALL CONNECTION CAPACITY

Maximum at each floor = 2 x 150 + 100 = 400 kN

However, the anchors fixed along the collector beam will attract most of the force in the first instance, since this loadpath is generally stiffer and more direct.

Since failure of the anchors will be in shear and likely quite brittle, an 'unzipping' mechanism would likely develop through these connections first.

∴ Minimum Capacity = 200 kN/Floor - More likely

Maximum = 400 kN/Floor - Less likely.

Probably capacity towards lower end - Perhaps 225 kN?

REV. CAPACITY AT A2 2 L1 = 252 kN
This excludes the contribution of the column fixings.

COLUMN FIXINGS - Resist vertical load

5 x 2 x 45 kN = 450 kN
Double-shear

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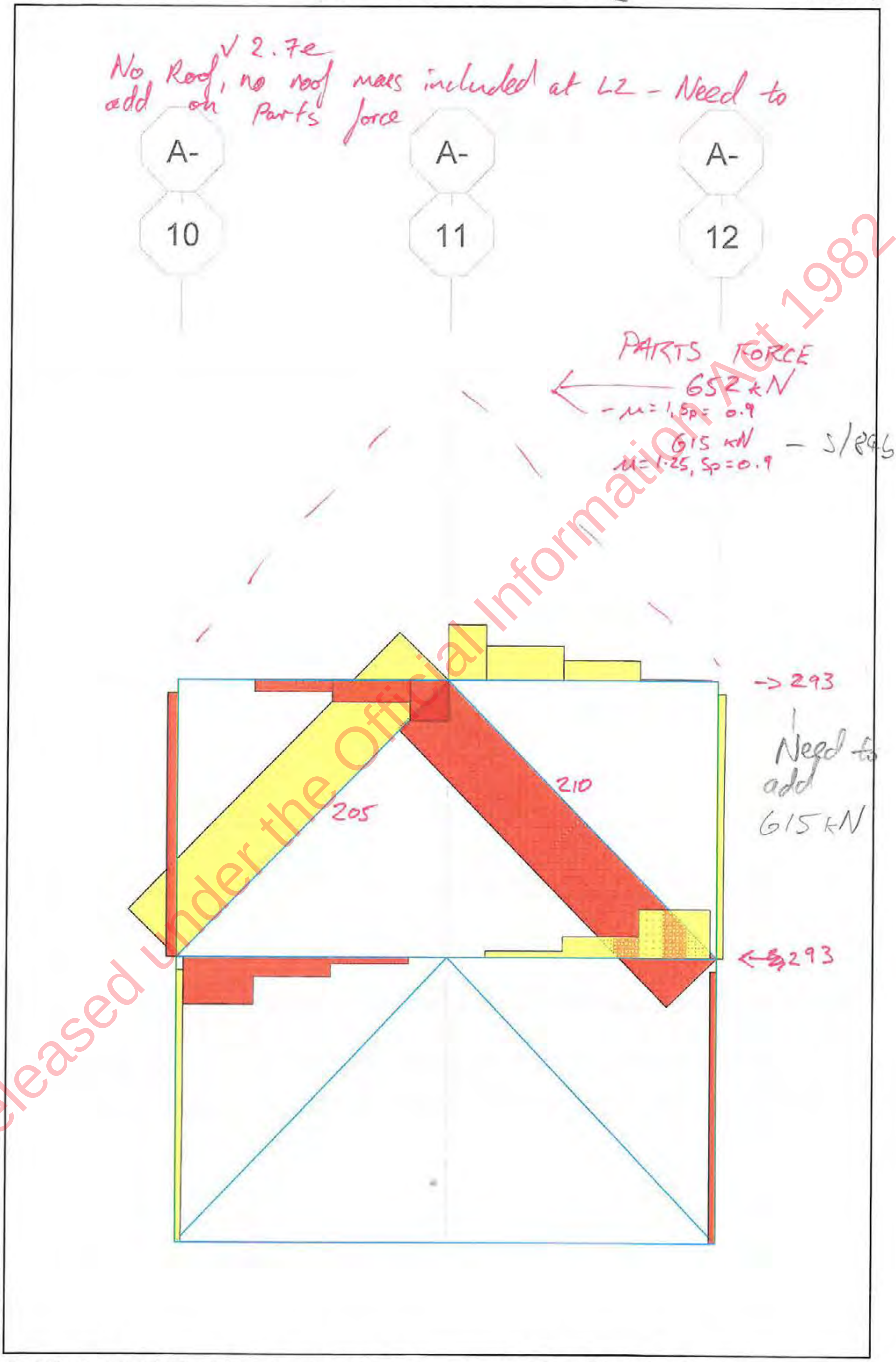
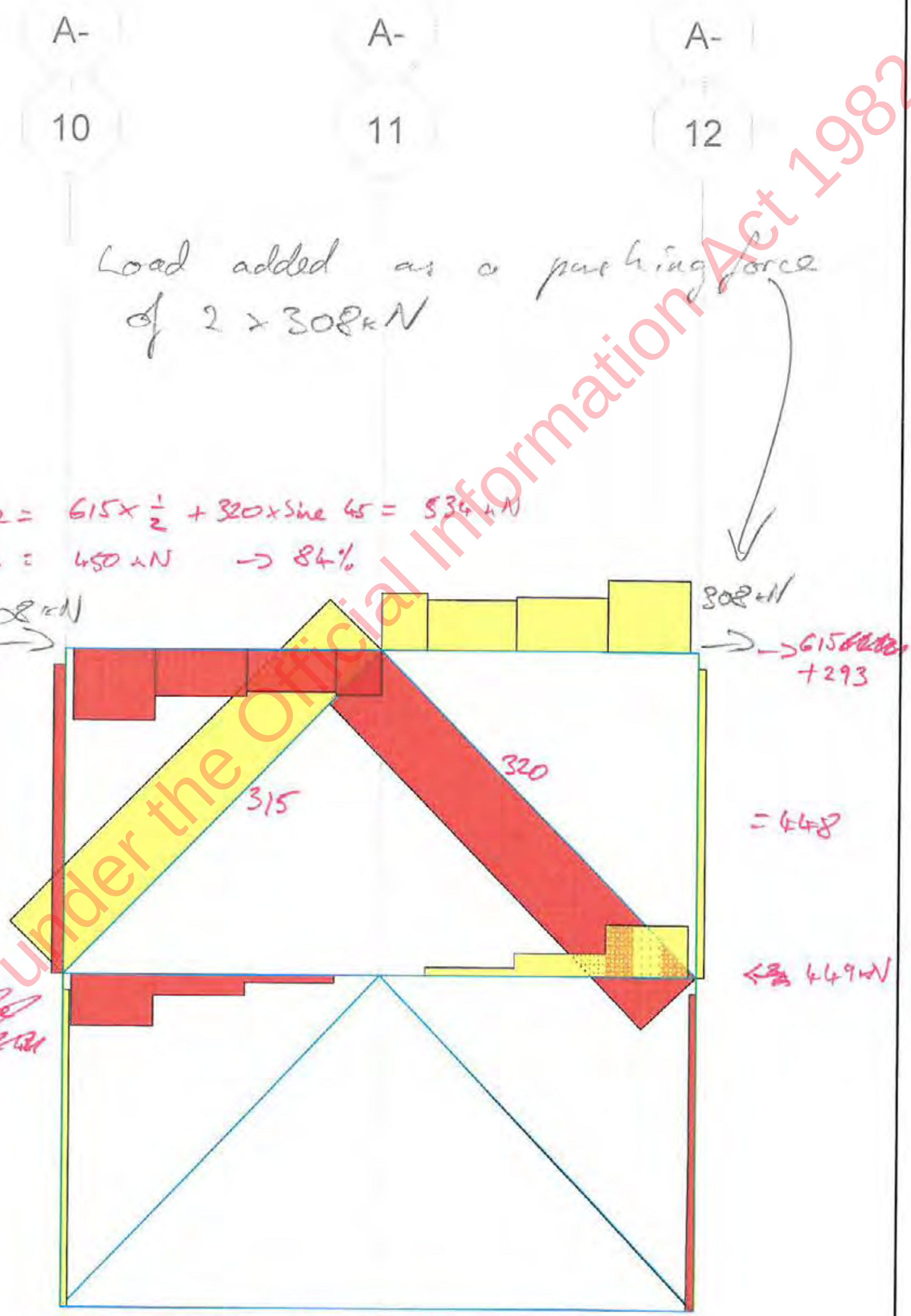


East Block v2.7e.EDB Elevation View - A- Axial Force Diagram (18.DSL+EQX--0.3EQY-) [kN]

V2.7e with 615kN Applied at L2.



@ L2 = 615+293 -448 = 460 kN → 55%
@ L1 = 449 kN → 56%
∴ > 50% NBS

East Elevation V2.7e.EDB A- Axial Force Diagram (18b.DSL+EQX--0.3EQY-+Penthouse push) [kN]

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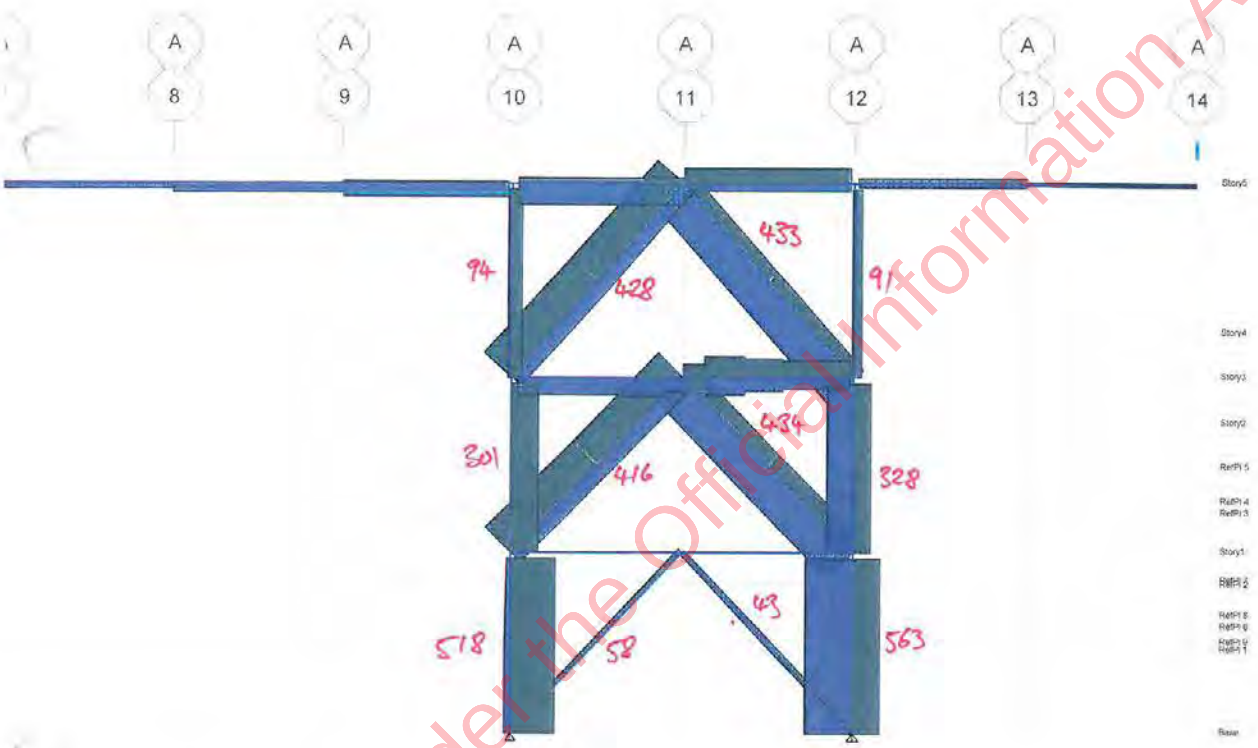
CHECKS ON BRACING MEMBERS

AXIAL FORCES IN BRACE MEMBERS



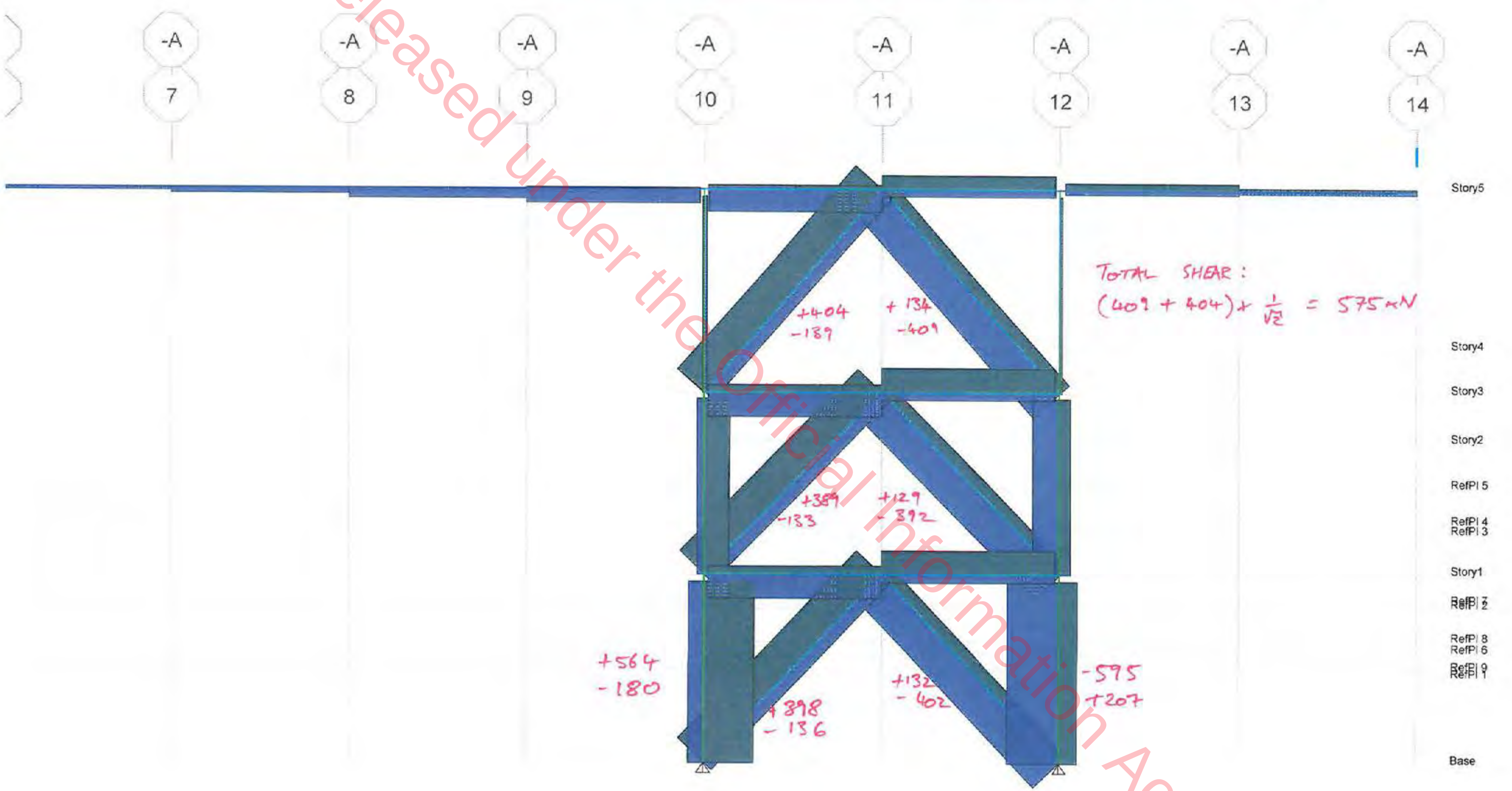
DIAG. BRACE CAPACITY : $\phi N_c = 429\,kN$

COLUMN CAPACITY : $\phi N_c = 1200\,kN$

∴ O.K. – Diagonal braces may buckle ~ $\mu = 1.25$ demands
–acceptable – Category 3 demands.

∴ O.K.

MODEL V2.2 - CBF DISCONNECTED AT L1 & L2



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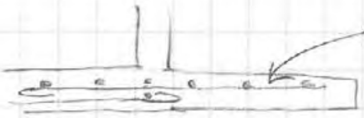
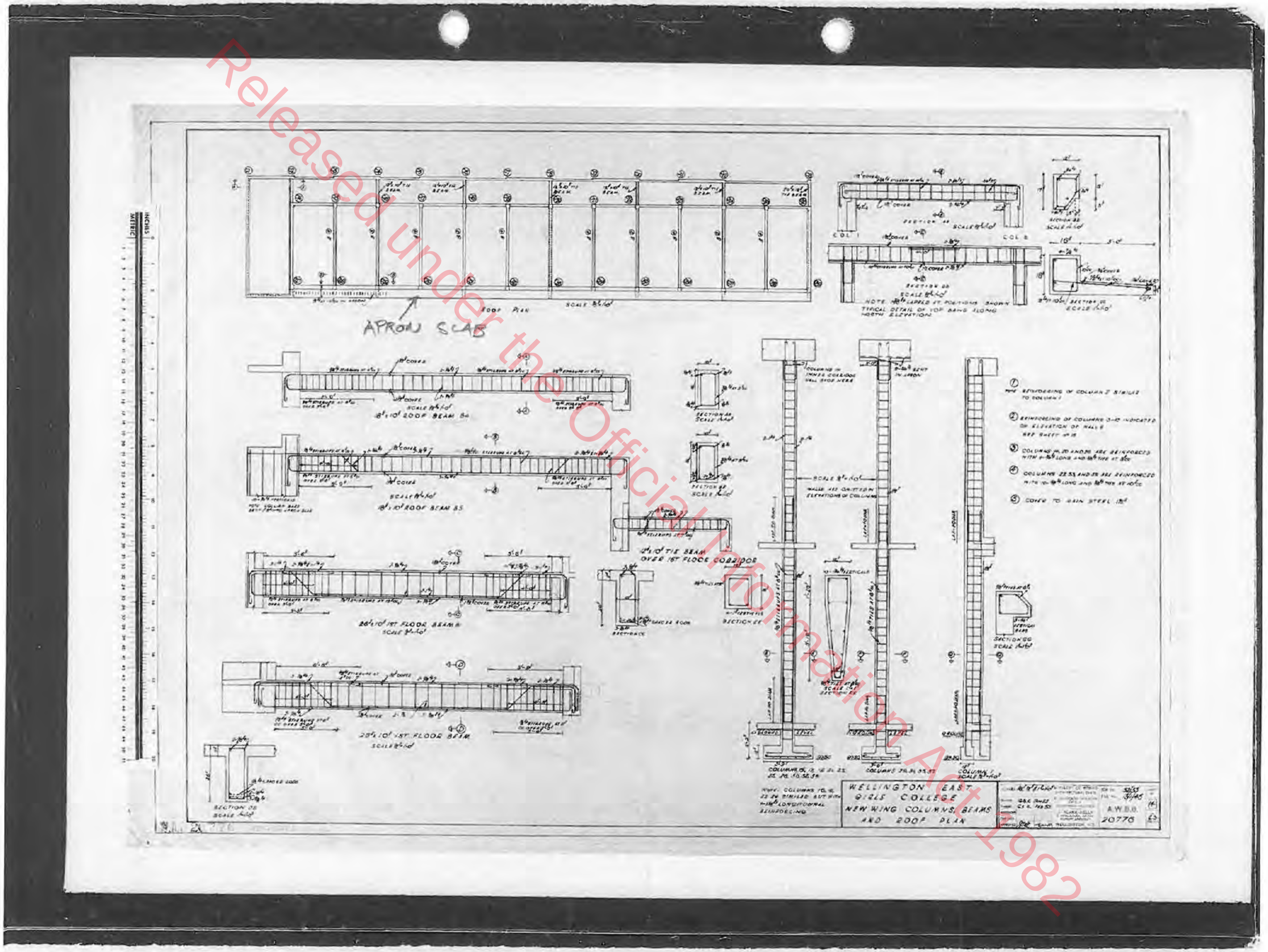
STRENGTH OF EXISTING CONCRETE APRON - A7 2nd FLOOR



Shear Capacity along its length = 145 kN/m

Length = 48.463 m

∴ Total Capacity = 7027 kN - O.K.


WELLINGTON EAST
GIRLS COLLEGE
NEW WING COLUMNS, BEAMS
AND ROOF PLAN
ROOF PLAN
APRON SLAB
20770

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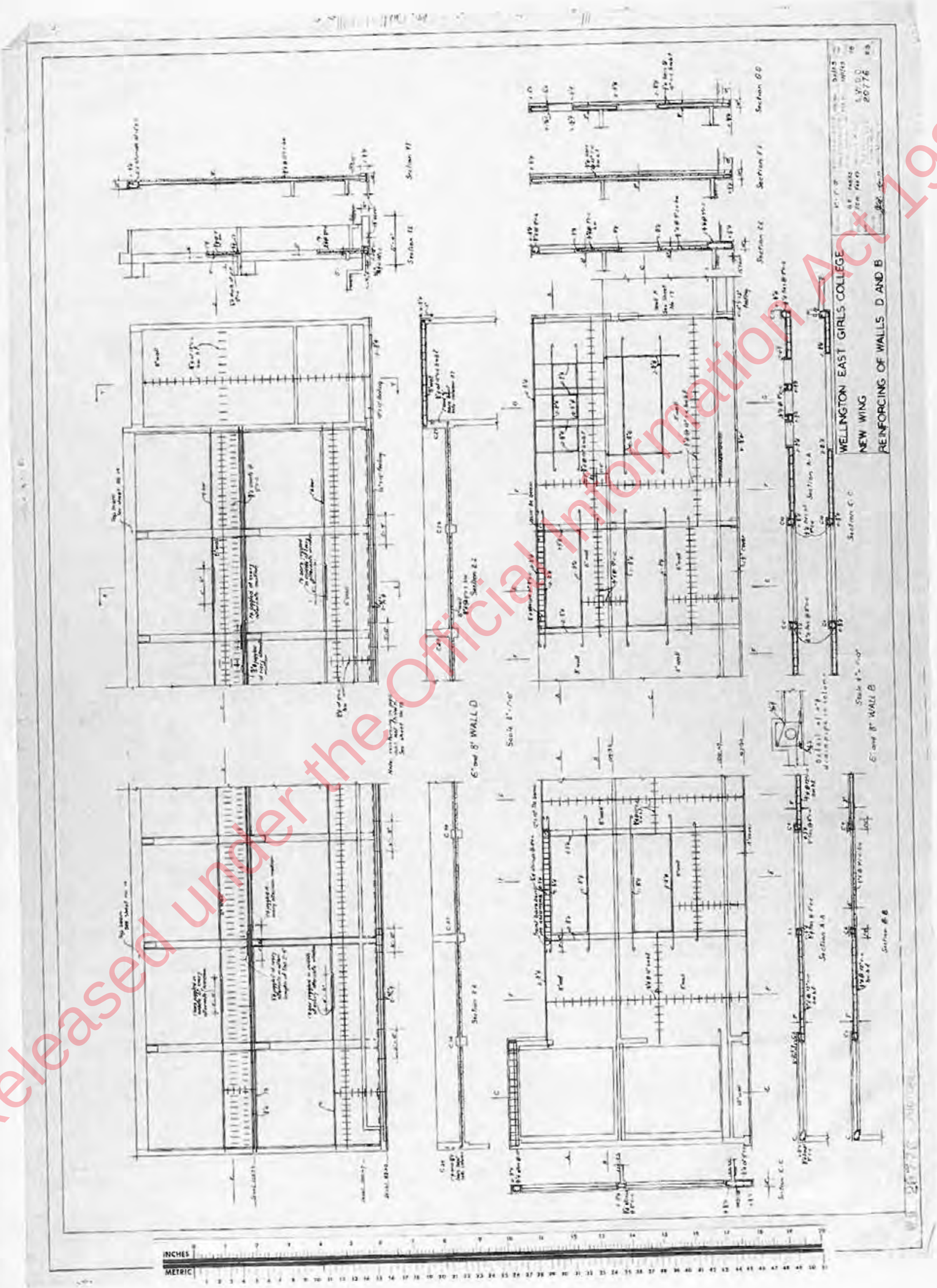
WELLINGTON EAST GIRLS COLLEGE
NEW WING
REINFORCING OF WALLS D AND B
6" and 8" WALL D
6" and 8" WALL B
INCHES
METRIC

CALCULATION SHEET

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NORTH ELEVATION
FRAME CAPACITY FOR DEFORMATION

By inspection, a column-sway mechanism more likely if frame can be pushed enough.

CHECK ON COLUMNS

Ground - 1st Floor

AXIAL LOAD

Roof

$\overbrace{0.35 \times 4.032 \times \tfrac{1}{2} \times 17.475}^{\approx 35.3\,m^2} = 1\ kN$

2nd Floor

$2\ kPa \times 4.032 \times \tfrac{1}{2} \times 17.475 = 71$

$0.3 \times 3 \times 4.032 \times \tfrac{1}{2} \times 17.475 = 32$

1st Floor

Beams + Slab $= 0.175 \times 25 \times 4.032 \times \tfrac{1}{2} \times 17.475 = 155$

$0.3 \times 3 \times 4.032 \times \tfrac{1}{2} \times 17.475 = 32$

$291\ kN$

Say 300 kN / column in E.Q. case.

Clear height : 2.924 m

COLUMN TYPE 1 =

4 - 1" Long
+ 3/8" Ties @ 8" c/c

Bending about minor axis :

$M_n = 112\ kNm$ with 300 kN axial force.

Taking overstrength of 1.25 :

$V^*_{o/s} = \dfrac{1.25 \times 2 \times 112}{2.924} = 96\ kN$

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COLUMN TYPE 1

WELCOME TO CONPROP(V 1.7) ** AN EXCEL SPREADSHEET FOR CALCULATING GEOMETRIC PROPERTIES & MOMENTS FOR CONCRETE SECTIONS UNDER UNCRACKED, CRACKED & ULTIMATE CONDITIONS.

[Flanges (top and/or bottom) and axial load are optional. For walls/columns see note 2 or the MANUAL.]

Checked : ____ /____ /____

Project:		
	Date: 11-Sep-15	Time: 12:21

STEP 1 ..... Describe the Uncracked Section...
(use consistent units.. e.g. N and mm through out the spreadsheet)

Total Section depth (d) =	305		mm
Web width (w) =	406		mm
Top flange width **excluding** web (b1) =	0	<----	mm
Top flange thickness (t) =	0	THESE	mm
Bottom flange width **excluding** web (b2) =	0	6 values	mm
Bottom flange thickness (b) =	0	may	mm
Axial compressive load (P) and,	300,000	be	N
Depth from top surface of this load (di)	152.5	zero	mm
Assumed tensile cracking stress (f't)	0	<-----	N/mm2
Steel Elastic Modulus (Es)	200,000		N/mm2

STEP 2 ..... Describe steel sizes and locations..................
describe location of the centroid of up to 10 bar bundles from either the top or the bottom surface. Describe Location of each bundle from only one surface.

Modular ratio (n=Es*(1+Ct)/Ec) =	8	$= E_s/E_c$

TOP BARS		
No. Bars	Bar Diam	Distance From Top Surface
2	25.40	50.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00
0	0.00	0.00

BOTTOM BARS			
No of Bars	Bar Diam	Distance From Bottom Surface	
2	25.40	50.00	mm
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	
0	0.00	0.00	

Checked : ________ /____/____

STEP 4 ..... Describe section properties for ULTIMATE conditions.........

(values entered in steps 1&2 may be varied for this part of the analysis.)

Concrete ultimate strain (e)	**0.03**
Ratio of (stress block)/(N.A.) depths........................	**0.85**
Axial compressive load (P) and, ...	**300,000**
depth from top surface of this load (di)	**152.5**
Crack root tensile stress (say 0.5f't)	**0.0**
Concrete Elastic Modulus (Ec)	**25,084**
Concrete compressive strength (f'c)........................	**30**
Steel Elastic Modulus (Es)........................	**200,000**
Steel Yield Stress (Fy)........................	**245**

=3320(f_c)

Analysis results shown below correspond to the conditions that exist when the peak compression strain equals (e) given above. A rectangular stress block with average stress=0.85f'c is assumed.

RESULTS FOR ULTIMATE MOMENT SECTION ANALYSIS:

(a) CRACK PROPAGATING FROM BOTTOM

Depth to N.A.(zero stress) from top (c)........................	9.41E+01		
Steel Stress (Maximum Tension)........................	2.45E+02		
Crack Depth	2.11E+02		
Total Tension Force (including P)........................	5.48E+05	Ratio T/C =	
Total Compression Force -incl. comp steel........................	1.05E+06	0.522	Mn
Ideal Flex strength (Mi)---SEE NOTE 2........................	1.12E+08	(=1.0 for iteration	**112**
Section Curvature (from curv = e/c)........................	3.19E-04	convergence)	θMn
			95
			KNm

(b) CRACK PROPAGATING FROM TOP

Depth to N.A.(zero stress) from bottom (c)........................	9.41E+01		
Steel Stress (Maximum Tension)........................	2.45E+02		
Crack Depth	2.11E+02		
Total Tension Force (including P)........................	5.48E+05	Ratio T/C =	
Total Compression Force -incl. comp steel........................	1.05E+06	0.522	Mn
Ideal Flex strength (Mi)---SEE NOTE 2........................	1.12E+08	(=1.0 for iteration	**112**
Section Curvature (from curv = e/c)........................	3.19E-04	convergence)	θMn
			95
			KNm

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Column Type 1: GF-1st

Clear height			2.924 m	
D	12 in	=	305 mm	
b	16 in	=	406.4 mm	
Cover	1.5 in	=	38 mm	
Ties	0.38 in	=	9.525 mm	
Number	2	=	2 Bars	
Spacing	8 in	=	203 mm	
Main Bars	1 in	=	25 mm	
Effective depth			282.575 mm	
Concrete f'c			30 MPa	
Steel yield fy			245 MPa	
Mn top			112 kNm	(From Conprop)
Mn Bottom			112 kNm	(From Conprop)
Overstrength factor			1.25	
V^*_o = OS x (Mn_{top}+Mn_{bottom})/h_{cl}			**96 kN**	
Axial load N			300 kN	(Compression only)

Shear Capacity Assessment

NZS3101 Method

10-11	Vb = 0.2x (sqrtf'c) x bd	126 kN	
10-14	kn	1.24	(Equation works for compression only)
	Vc	156 kN	
10-7	Vs = Avfytd/s	49 kN	
2.3.2.2	Safety factor	0.75	
	Vp	**154 kN**	**OK**

NZSEE 2006 Method

	k	0.29	
7(7)	Vc = K x (sqrtf'c) x bd	182 kN	
	d"	219 mm	
7(9)	Vs = Avfytd"cot30/s	65 kN	
	Alpha	0.130	
7(10)	Vn = N* x tan(alpha)	39 kN	
7(6)	Safety factor	0.72	
	Vp	**207 kN**	**OK**

	k	0.2	
7(7)	Vc = K x (sqrtf'c) x bd	126 kN	
	d"	219 mm	
7(9)	Vs = Avfytd"cot30/s	65 kN	
	Alpha	0.130	
7(10)	Vn = N* x tan(alpha)	39 kN	
7(6)	Safety factor	0.72	
	Vp	**166 kN**	**OK**

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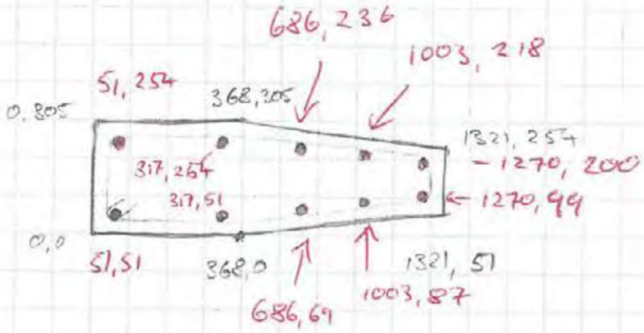
FIN COLUMNS



● = 3/4" Vertical

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WELLINGTON EAST GIRLS' COLLEGE – EAST BLOCK

T-Section

Key:
- Inputs
- Reinf Area
- x-coordinate
- y-coordinate

Table 3. Coordinates for Center of mass (from spColumn)

Xo=	613.47
Yo=	152.5

Total no. of points
Coordinates

Table 1. Section Coordinates

7	
0	0
368	0
1321	51
1321	254
368	305
0	305
0	0
0	0
0	0
0	

Number of openings

Table 4. Transformed Section Coordinates

7	
-613.47	-152.5
-245.47	-152.5
707.53	-101.5
707.53	101.5
-245.47	152.5
-613.47	152.5
-613.47	-152.5
-613.47	-152.5
-613.47	-152.5
0	

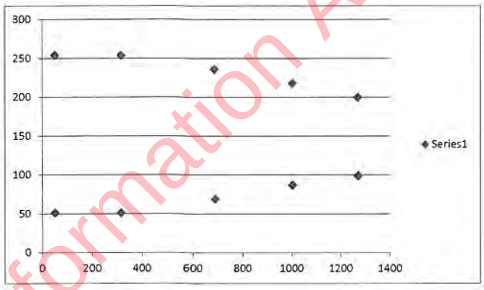
No. of Bars
*Add more cells at the bottom if required

Table 2. Reo Area and Coordinates

area	x-coord	y-coord
10		
285	51	51
285	317	51
285	689	69
285	1003	87
285	1270	99
285	1270	200
285	1003	218
285	686	236
285	317	254
285	51	254

Table 5. Reo Area and Transformed Coordinates

10		
285	-562.47	-101.5
285	-296.47	-101.5
285	75.53	-83.5
285	389.53	-65.5
285	656.53	-53.5
285	656.53	47.5
285	389.53	65.5
285	72.53	83.5
285	-296.47	101.5
285	-562.47	101.5



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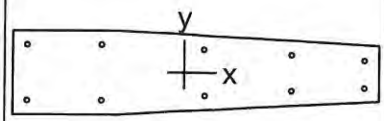


1321 x 305 mm

Code: ACI 318-11

Units: Metric

Run axis: About X-axis

Run option: Investigation

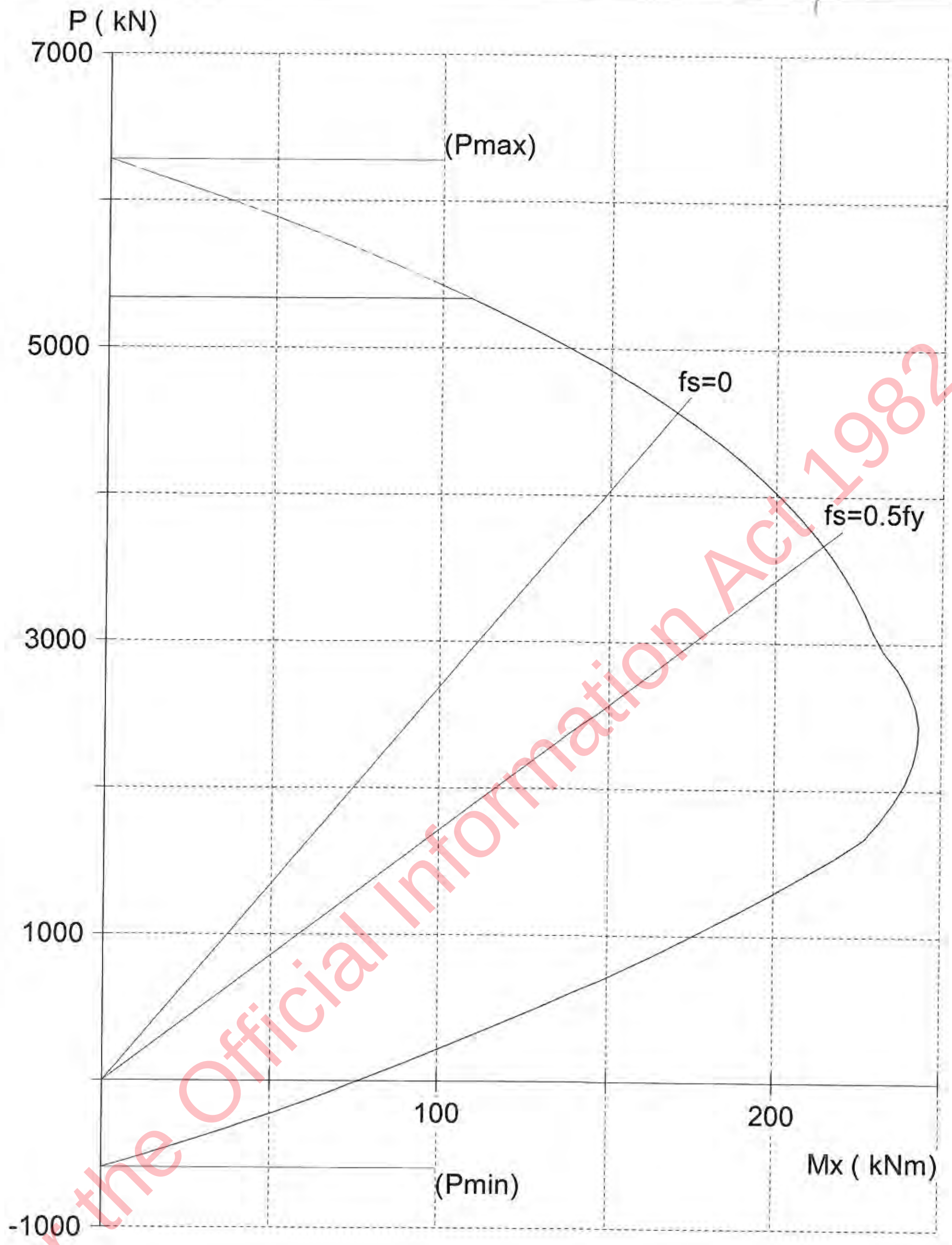
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 09/14/15

Time: 14:34:55



spColumn v4.81. 15 day trial license. Locking Code: 4-2D135. User: Opus, Opus International Consultants

File: P:\projects\5-PA010.00 MOE Seismic Panel Agreement to Assess Public Buildings\10 Structura...\fin column.col

Project:

Column:

Engineer:

f'c = 30 MPa	fy = 245 MPa	Ag = 354302 mm^2	10 bars
Ec = 25084 MPa	Es = 200000 MPa	As = 2850 mm^2	rho = 0.80%
fc = 25.5 MPa		Xo = 0 mm	Ix = 2.22e+009 mm^4
e_u = 0.003 mm/mm		Yo = -0 mm	Iy = 4.96e+010 mm^4
Beta1 = 0.85		Min clear spacing = 82 mm	Clear cover = N/A

Confinement: Other

phi(a) = 0.85, phi(b) = 0.85, phi(c) = 0.65

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```
                  oooooo          o
                 oo    oo         oo
 ooooo   oooooo  oo        ooooo  oo     oo   oo  o oooooooooo   o ooooo
oo    o  oo   oo oo       oo   oo oo     oo   oo   oo   oo   oo   oo   oo
oo       oo   oo oo       oo   oo oo     oo   oo   oo   oo   oo   oo   oo
 ooooo   oo   oo oo       oo   oo oo     oo   oo   oo   oo   oo   oo   oo
     oo  oooooo  oo       oo   oo oo     oo   oo   oo   oo   oo   oo   oo
o    oo  oo      oo    oo oo   oo oo  o  oo   oo   oo   oo   oo   oo   oo
 ooooo   oo       oooooo   ooooo   ooo    ooooo o  oo   oo   oo   oo   oo (TM)
```


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General Information:
====================
File Name: P:\projects\5-PA010.00 MOE Seismic Panel Agreement to Assess Public Bui...\fin column.col
Project:
Column: Engineer:
Code: ACI 318-11 Units: Metric

Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:
====================
f'c = 30 MPa fy = 245 MPa
Ec = 25084 MPa Es = 200000 MPa
Ultimate strain = 0.003 mm/mm
Beta1 = 0.85

Section:
========
Exterior Points

No.	X (mm)	Y (mm)	No.	X (mm)	Y (mm)	No.	X (mm)	Y (mm)
1	-613	-153	2	-245	-153	3	708	-102
4	708	102	5	-245	153	6	-613	153

Gross section area, Ag = 354302 mm^2
Ix = 2.22397e+009 mm^4 Iy = 4.9642e+010 mm^4
rx = 79.2278 mm ry = 374.315 mm
Xo = 0.000298458 mm Yo = -1.00158e-006 mm

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (mm)	Area (mm^2)	Size	Diam (mm)	Area (mm^2)	Size	Diam (mm)	Area (mm^2)
# 3	10	71	# 4	13	129	# 5	16	200
# 6	19	284	# 7	22	387	# 8	25	510
# 9	29	645	# 10	32	819	# 11	36	1006
# 14	43	1452	# 18	57	2581			

Confinement: Other; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.85, phi(b) = 0.85, phi(c) = 0.65

Pattern: Irregular
Total steel area: As = 2850 mm^2 at rho = 0.80% (Note: rho < 1.0%)
Minimum clear spacing = 82 mm

Area mm^2	X (mm)	Y (mm)	Area mm^2	X (mm)	Y (mm)	Area mm^2	X (mm)	Y (mm)
285	-562	-102	285	-296	-102	285	76	-84
285	390	-66	285	657	-54	285	657	48
285	390	66	285	73	84	285	-296	102
285	-562	102						

Axial Load and Corresponding Moment Capacities:
===

Load No.	PhiPn kN	PhiMnx kNm	NA depth mm	Dt depth mm	eps_t	Phi
1	-0.0	76.28	42	254	0.01523	0.850
		-77.00	42	254	0.01523	0.850
2	25.0	79.09	43	254	0.01484	0.850
		-79.80	43	254	0.01484	0.850
3	50.0	81.88	44	254	0.01447	0.850
		-82.59	44	254	0.01447	0.850
4	75.0	84.66	45	254	0.01411	0.850
		-85.37	45	254	0.01411	0.850
5	100.0	87.43	45	254	0.01377	0.850
		-88.14	45	254	0.01377	0.850
6	125.0	90.19	46	254	0.01344	0.850
		-90.90	46	254	0.01344	0.850
7	150.0	92.94	47	254	0.01312	0.850
		-93.65	47	254	0.01312	0.850
8	175.0	95.67	48	254	0.01281	0.850
		-96.38	48	254	0.01281	0.850
9	200.0	98.38	49	254	0.01252	0.850
		-99.10	49	254	0.01252	0.850
10	225.0	101.02	50	254	0.01227	0.850

		-101.73	50	254	0.01227	0.850
11	250.0	103.64	51	254	0.01203	0.850
		-104.35	51	254	0.01203	0.850
12	275.0	106.25	51	254	0.01180	0.850
		-106.96	51	254	0.01180	0.850
13	300.0	108.86	52	254	0.01157	0.850
		-109.57	52	254	0.01157	0.850

*** End of output ***

@ 300 kN AXIAL LOAD : $\phi M_n = 110$ kNm

With $\phi = 1$ $\therefore$ $\frac{110}{0.85} = 129$ kNm

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Column Type 2: GF-1st

Clear height			2.924 m	
D	12 in	=	250 mm	
b	16 in	=	1321 mm	
Cover	1.5 in	=	38 mm	
Ties	0.38 in	=	9.525 mm	
Number	2	=	2 Bars	
Spacing	8 in	=	203 mm	
Main Bars	1 in	=	25 mm	
Effective depth			227.775 mm	
Concrete f'c			30 MPa	
Steel yield fy			245 MPa	
Mn top			129 kNm	(From Conprop)
Mn Bottom			129 kNm	(From Conprop)
Overstrength factor			1.25	
V^*_o = OS x (Mn_{top}+Mn_{bottom})/h_{cl}			**110 kN**	
Axial load N			300 kN	(Compression only)

Shear Capacity Assessment

NZS3101 Method

10-11	Vb = 0.2x (sqrtf'c) x bd	330 kN	
10-14	kn	1.09	(Equation works for compression only)
	Vc	360 kN	
10-7	Vs = Avfytd/s	39 kN	
2.3.2.2	Safety factor	0.75	
	Vp	**299 kN**	**OK**

NZSEE 2006 Method

	k	0.29	
7(7)	Vc = K x (sqrtf'c) x bd	478 kN	
	d"	164 mm	
7(9)	Vs = Avfytd"cot30/s	49 kN	
	Alpha	0.130	
7(10)	Vn = N* x tan(alpha)	39 kN	
7(6)	Safety factor	0.72	
	Vp	**408 kN**	**OK**

	k	0.2	
7(7)	Vc = K x (sqrtf'c) x bd	330 kN	
	d"	164 mm	
7(9)	Vs = Avfytd"cot30/s	49 kN	
	Alpha	0.130	
7(10)	Vn = N* x tan(alpha)	39 kN	
7(6)	Safety factor	0.72	
	Vp	**301 kN**	**OK**

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CHECK COLUMN-TIE SPACING

FOR 16" x 12" COLUMNS

· Bent about minor axis :

$d = 255mm$

$s = 8" = 203mm$

Bar = 1" ϕ $\rightarrow$ $s = 8d_b$

NZSEE 2006 : $\mu = 2$ can be assumed.

FOR FIN COLUMNS

$d \sim 200$ – Average.

Sim spacing etc.

∴ Allowable assumed ductility capacity $\mu = 2$

∴ $\boxed{\mu = 2}$

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FRAME ON GL B (NORTH ELEVATION) DISPLACEMENTS
- See next pages for analysis.

AT 1st FLOOR

Model output indicates very small displacements along this line for both the x and y directions:

Max = ~1mm in each direction.

↳ By inspection - not critical.

AT 2nd FLOOR

Displacements along x direction = 10mm max

y = 17mm max

Storey height = 4.147m, 17mm deflection

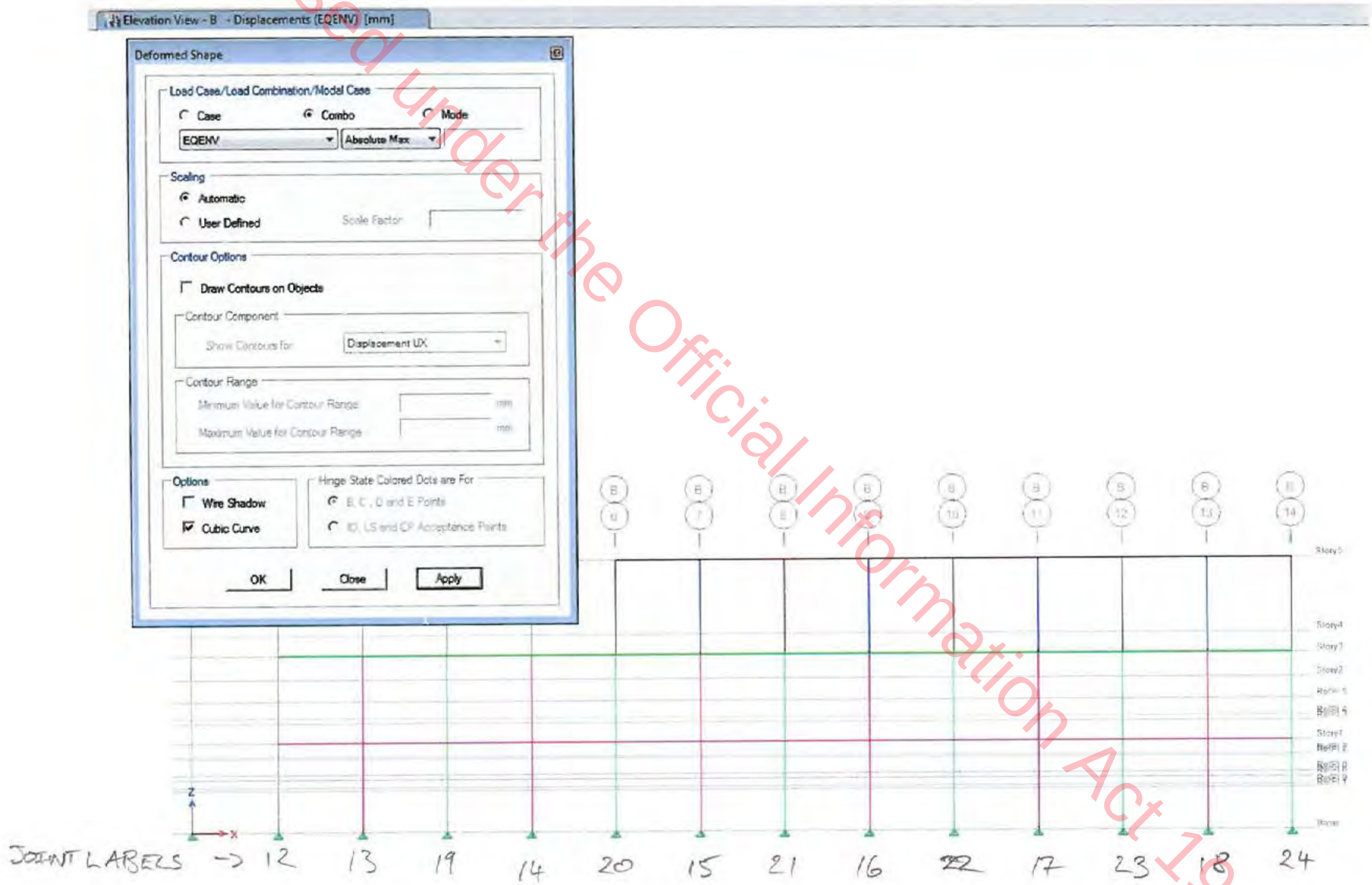
↳ Drift = 0.4%

Scaling for ductility → × 1.25 = 0.5%

- Well within limits.

By inspection, columns are able to accommodate expected drifts without much, if any, damage.

Building Displacements on GLB (North Elevation Frame)



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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	13	14	EQENV Max	8.2	9.2	-0.3	0.0	0.0	0.0
Story3	13	14	EQENV Min	-3.6	-7.5	-0.4	0.0	0.0	0.0
Story1	13	37	EQENV Max	0.6	0.8	-0.3	0.0	0.0	0.0
Story1	13	37	EQENV Min	-0.1	-0.3	-0.3	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	19	18	EQENV Max	8.5	13.1	0.4	0.0	0.0	0.0
Story3	19	18	EQENV Min	-3.7	-5.2	-1.9	0.0	0.0	0.0
Story1	19	43	EQENV Max	0.6	0.8	-0.1	0.0	0.0	0.0
Story1	19	43	EQENV Min	-0.1	-0.3	-1.0	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	14	113	EQENV Max	8.8	17.9	0.0	0.0	0.0	0.0
Story3	14	113	EQENV Min	-3.7	-4.5	-0.2	0.0	0.0	0.0
Story1	14	38	EQENV Max	0.6	0.8	0.0	0.0	0.0	0.0
Story1	14	38	EQENV Min	-0.1	-0.3	-0.2	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	20	26	EQENV Max	9.1	18.7	0.1	0.0	0.0	0.0
Story3	20	26	EQENV Min	-3.7	-3.8	-2.5	0.0	0.0	0.0
Story1	20	44	EQENV Max	0.6	0.8	-0.2	0.0	0.0	0.0
Story1	20	44	EQENV Min	-0.1	-0.3	-1.3	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	15	31	EQENV Max	9.4	9.8	-0.4	0.0	0.0	0.0
Story3	15	31	EQENV Min	-3.7	-2.0	-0.4	0.0	0.0	0.0
Story1	15	39	EQENV Max	0.6	0.8	-0.3	0.0	0.0	0.0
Story1	15	39	EQENV Min	-0.1	-0.2	-0.3	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	21	281	EQENV Max	9.7	2.1	0.3	0.0	0.0	0.0
Story3	21	281	EQENV Min	-3.9	-0.5	-0.2	0.0	0.0	0.0
Story1	21	45	EQENV Max	0.6	0.9	0.2	0.0	0.0	0.0
Story1	21	45	EQENV Min	-0.1	-0.2	-0.2	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	16	36	EQENV Max	9.9	10.8	-0.4	0.0	0.0	0.0
Story3	16	36	EQENV Min	-4.1	-2.8	-0.4	0.0	0.0	0.0
Story1	16	40	EQENV Max	0.6	1.0	-0.3	0.0	0.0	0.0
Story1	16	40	EQENV Min	-0.1	-0.2	-0.3	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	22	107	EQENV Max	10.1	19.5	0.3	0.0	0.0	0.0
Story3	22	107	EQENV Min	-4.1	-4.8	-2.7	0.0	0.0	0.0
Story1	22	46	EQENV Max	0.6	1.0	-0.2	0.0	0.0	0.0
Story1	22	46	EQENV Min	-0.1	-0.2	-1.5	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	17	109	EQENV Max	10.2	21.7	-0.3	0.0	0.0	0.0
Story3	17	109	EQENV Min	-4.0	-5.9	-0.4	0.0	0.0	0.0
Story1	17	41	EQENV Max	0.6	1.1	-0.3	0.0	0.0	0.0
Story1	17	41	EQENV Min	-0.1	-0.2	-0.3	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	23	117	EQENV Max	10.3	17.1	0.4	0.0	0.0	0.0
Story3	23	117	EQENV Min	-4.0	-4.7	-2.0	0.0	0.0	0.0
Story1	23	47	EQENV Max	0.6	1.1	0.0	0.0	0.0	0.0
Story1	23	47	EQENV Min	-0.1	-0.2	-0.1	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	18	111	EQENV Max	10.3	11.1	-0.3	0.0	0.0	0.0
Story3	18	111	EQENV Min	-4.0	-3.0	-0.4	0.0	0.0	0.0
Story1	18	42	EQENV Max	0.6	1.2	-0.2	0.0	0.0	0.0
Story1	18	42	EQENV Min	-0.1	-0.2	-0.3	0.0	0.0	0.0

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Story	Label	Unique Name	Load Case/Com	UX	UY	UZ	RX	RY	RZ
Story3	24	279	EQENV Max	10.4	2.3	0.4	0.0	0.0	0.0
Story3	24	279	EQENV Min	-4.0	-0.2	-0.1	0.0	0.0	0.0
Story1	24	48	EQENV Max	0.6	1.3	0.3	0.0	0.0	0.0
Story1	24	48	EQENV Min	-0.1	-0.2	-0.1	0.0	0.0	0.0

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NORTH ELEVATION CONCRETE FRAME

SUMMARY

The columns are likely to be able to hinge top and bottom without causing shear failure in the concrete.

Maximum allowable ductility = $\mu = 2$ due to limited ties.

Since building displacements will be low, frames are able to accommodate expected ductility demand.

↳ Max 1st Floor displacement = 1mm - V. small

∴ O.K.

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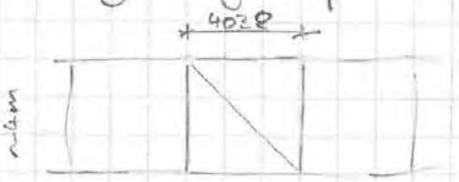
RB 25 CROSS-BRACING CAPACITY

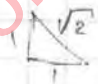
- Used to brace south elevation in longitudinal direction at 2nd Floor.

Min yield strength = 245.5 kN

Including angle factor :



Angle ~ 45°



∴ Lateral force required to reach 245.5 kN $= \frac{1}{\sqrt{2}} \times 245.5$

$= 174$ kN

2 Braces effective on this line

∴ $2 \times 174 = 348$ kN

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MODEL V2.2 - CBF DISCONNECTED

ELC - BRACES AT TOP STOREY

S/82

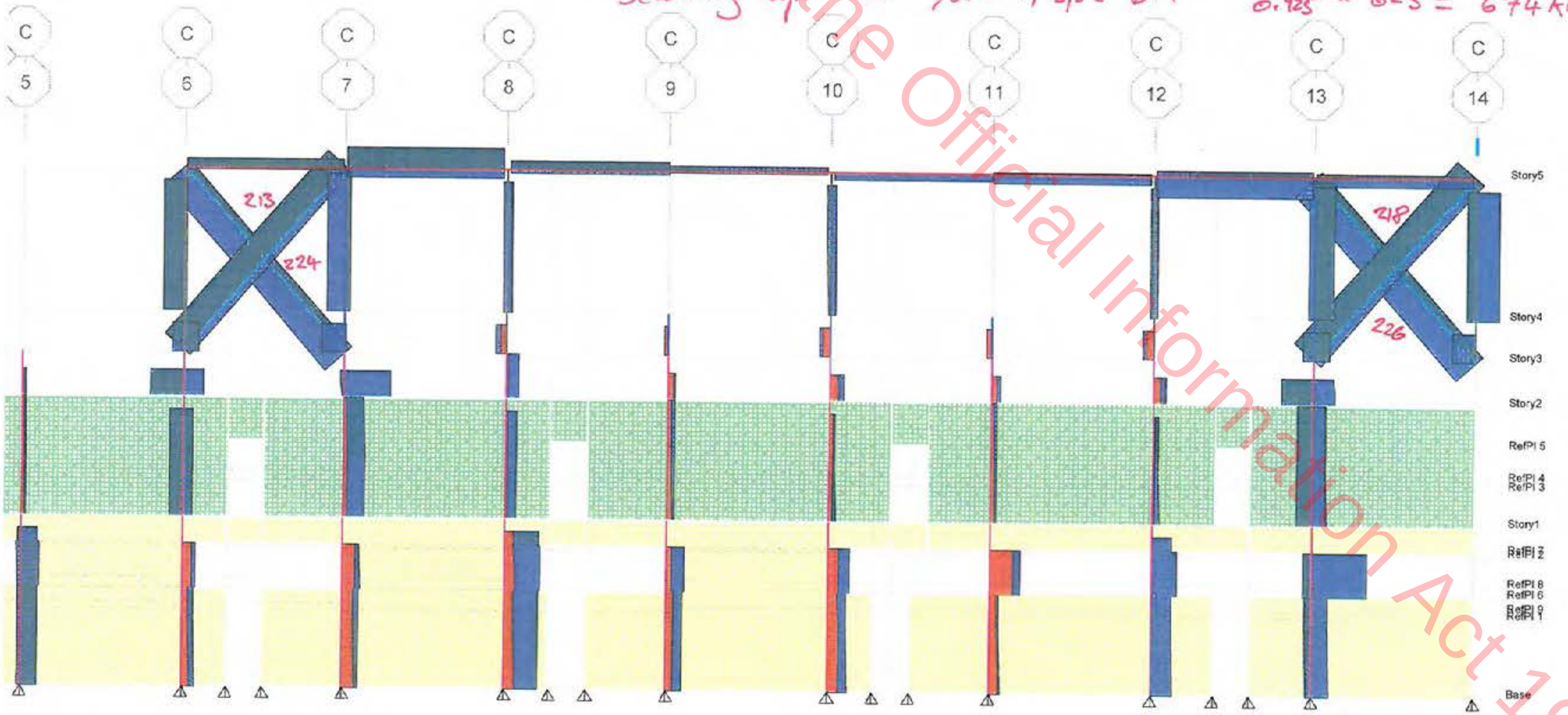
RB25 Braces: X-sectional area modified to 0.5x Actual to account for model acting in tension & compression.

$(213+224) \times \frac{1}{\sqrt{2}} = 309$

$(218+226) \times \frac{1}{\sqrt{2}} = 314$ kN

$\therefore$ TOTAL HORIZONTAL FORCE @ ROOF $= 309+314 = 623$ @ $\mu = 1.25$, $S_p = 0.9$

Scaling up to $\mu = 1$, $S_p = 0.9$: $\frac{1}{0.925} \times 623 = 674$ kN



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TOP STOREY : BRACES ON GL C

Total = 309 + 314 = 623 kN – Force from model – $\mu = 1$, $S_p = 0.9$ forces

2 Braces :

Tension in braces = $623 \times \sqrt{2} \times \frac{1}{2}$ = 441 kN/Brace

Capacity of RB 25 = 500MPa × 491 mm² = 246 kN

↳ 55% NBS .

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BRACING CHECKS AT TOP STOREY

LONGITUDINAL DIRECTION

Storey shear force from ETABS model = 911 kN

- Based on T = 0.4s.

3 Braced bays: $= \frac{911}{3} = 304$ kN each braced bay.

Scaling up for $\mu = 1$, $S_p = 0.9$:

$304 \times \frac{1}{0.925} = 329$ kN

Force in Brace $= 329 \times \sqrt{2} = 465$ kN / Brace.

∴ Similar to results directly taken from model braces.

TREATING TOP STOREY AS A PART

Weight $\simeq \frac{(201 + 376)}{2} = 289 \rightarrow 290$ kN

$h_n = 13$ m

$h_i = 8.4$ m

$C(0) = 0.52$ — $R_u = 1.3$

Force $= 724$ kN @ ULS — $\mu = 1$

Allow $S_p = 0.9 \rightarrow 724 \times 0.9 = 652$ kN

Per Braced Bay $= \frac{652}{3} = 217$ kN

∴ Force in Brace $= 217 \times \sqrt{2} = 307$ kN

Brace = RB25

Capacity = 245.5 kN

$\frac{245.5}{307} = 0.8 \rightarrow 80\%$

JUSTIFICATION FOR TREATING AS PART

The top storey is relatively light weight in comparison to the concrete structure below. Its structural form is also different - steel frames / steel braces as opposed to concrete walls. ∴ Acceptable to treat as a part.

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PARTS FORCE FROM ROOF = 652 kN – μ=1, S_p= 0.9, T = 0.4

T likely to be lower if rear braces not present

↳ Model v2.7b no + braces GLC – T for top storey = 0.44 s
↳ Sim to 0.4.

Allowing μ = 1.25 + some ductility in system:

h_i = 8.4 m
h_n = 13 m

↳ Horizontal force = 615 kN

↳ Applying simply as a horizontal force in the model v2.7e for load case DSL + EQx – 0.3EQy :

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Job Title: *WEGC East Block DSA*
Job Number: **Member Reference:**
Calcs By: *PMO* **Date:** *22/09/2015 12:11:42 p.m.*

Report created using Seismic Shear forces to NZS1170.5 Design Tool Version 1.1

HORIZONTAL SEISMIC SHEAR (V) TO NZS 1170.5 - Section 8

Input Data
Site Hazard Coefficient for T = 0 is 0.52
Period of the part = 0.4 Sec
Height of the attachment of the part = 8.4 m
Height from the base of the structure to the uppermost seismic weight or mass = 13 m
The part risk factor from table 8.1 = 1
Weight of the part = 290 kN
Ductility of the part = 1.0

Design Response Coefficient for parts Cl 8.2
Floor height coefficient Cl 8.3 = 2.400
Part spectral shape coefficient Cl 8.4 = 2.000
Design Response Coefficient for parts Cp(Tp) = 2.496

Horizontal design action Cl 8.5.1
The part horizontal response factor Cl 8.6 = 1.00
The part risk factor from table 8.1 = 1.0
Horizontal design action = **724** kN

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Calculation Sheet	
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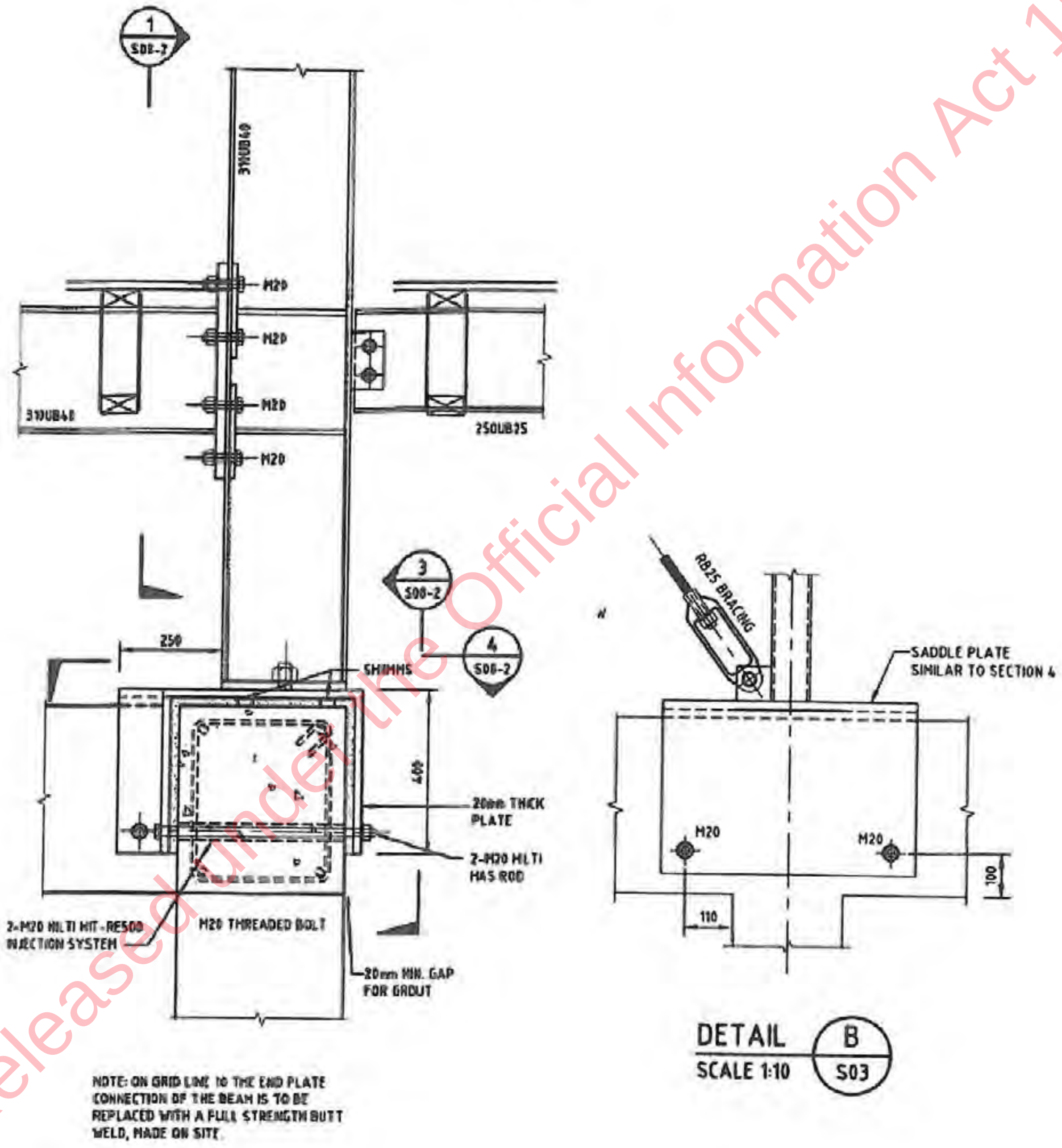
Connection on GL C – Connell Wagner Detail



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CONNECTION DETAIL - SITE INVESTIGATION

Connection different to Connell Wagner detail on drawings - indicative of no tie-beam being present.

- Still assume 2 - M20 HILTI HAS Rod

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CONNECTION OF BRACE TO CONCRETE COLUMNS

2 - M20 Hilti Anchors (HAS Rod) - Good quality anchors

Assuming Grade 5.8 : $V_{Rd,s} = 44.8$ kN

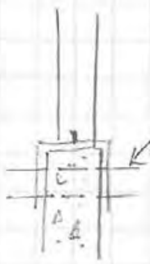
$2 \times 44.8 = 89.6$ kN

Bolts in double-shear $\rightarrow$ $2 \times 89.6 = 179$ kN

$\mu = 1, S_p = 0.9$

Demand $= \sim 210 \times \sqrt{2} = 297 \rightarrow \sim 300$ kN

$$\frac{179}{300} = 60\% \text{ NBS}$$



Taking safety factor off :

Assuming $\phi = 0.8 \rightarrow \frac{179}{0.8} = 224$ kN

$$\frac{224}{300} = 75\% \text{ NBS}$$

$\therefore$ $\boxed{\text{CONNECTION} \sim 75\% \text{ NBS}}$

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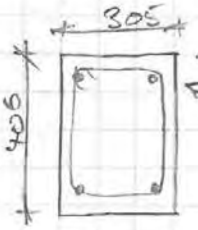
CANTILEVERING COLUMNS ON GL C - OUT-OF-PLANE

Columns at C:6 – C:14 support the new top storey.
From model,
Load (Lateral) = (kN Total – μ : 1.25, S_p = 0.9

~ $\frac{2}{3}$ total force on this line (line C)

COLUMNS :



4 - 1" ϕ — Cover to main bars = $1\frac{1}{2}$"

Ties : $^3/_8$" @ 8" c/c

Conservatively assuming no axial load :

M_n = 60 kNm

Force F to yield in flexure = $\frac{60}{1.1}$ = 55

= 55 kN

SHEAR CAPACITY

Basing only on stirrups ϕV_s :

2 × $\frac{3}{8}$" @ 8" : $0.75 \times 2 \times 245 \times \frac{71}{203} \times 254$ = 33 kN

$\phi V_c = 0.75 \times 0.2 \times \sqrt{30} \times 406 \times 254$ = 85 kN

TOTAL = 118 kN

∴ Flexural failure more likely first. But then V_c will diminish towards ~ $0.1\sqrt{f'c}$: – NZSEE 2006

$\phi V_c = 0.75 \times 0.1 \times \sqrt{30} \times 305 \times 254$ = 32 kN

∴ $\phi(V_s + V_c)$ = 65 kN — Still higher than 'yield demand'. But with overstrength, possibly shear governed.

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IF X-BRACES ARE EFFECTIVE & SUPPORTING FLOOR

N2.7 model :

Shear in model = 188 + 165 + 4 + 52 + 66 + 52 + 64 + 162 + 162
= 915 kN total

If a tie-beam is present → equal demands :

Capacity : 55 kN each → x 8 = 440 kN

Allowing μ = 2 ductility ~ 90% NBS.

∴ o.k.

Without tie - yield quite quickly and demands taken by CBF.

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RELYING ON CANTILEVERING COLUMNS ON GL A TO SUPPORT TOP STOREY

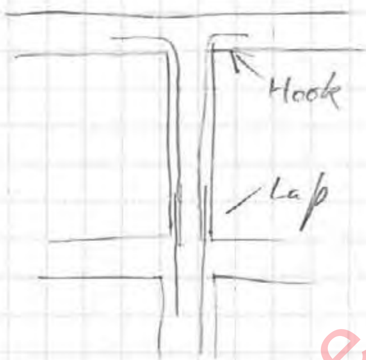
TYPE 1 columns : M_n = 112 kNm   S/55

2 : M_n = 129 kNm   S/61

Laps are at bottom of 1st floor, however:

Lap = 40 × Diameter : ~ 1000mm

But, bars are hooked into tie beam at top.



↳ If columns cantilever from top:

Number of columns = 6× Type 1 = 6× 112 = 672 kNm
6× Type 2 = 6× 129 = 774 kNm

Σ 1446 kNm

Resistance F = $\frac{\Sigma M}{H}$ = $\frac{1446}{4.147}$ = 349 kN
To FFL

Model V2.7e total demand on front elevation:
Shear = 14+26 + 14+26+15+28+16+27+16+29+17+28 = 258 kN

↳ Doesn't cause yield.

↳ > 100%

OPUS

COLUMNS' FLEXURAL CAPACITY

Singly Reinforced Beams

Characteristic Material Values					
1st Yield (MPa)		Young's Modulus (MPa)		Strains at 1st yield	
f_y	245	E_s	200000	ε_s	0.00123
f'_c	30			ε_c	0.003

Depth D (mm)	Breadth b_w (mm)	Cover (mm)	Tension Reinforcement Diameter	Number of Bars	Effective depth d (mm)	As (mm^2)	Maximum aggregate size (mm)
305	406	38	25.4	2	254.3	1013	20

Flexure

7.4.2.7 Equivalent concrete rectangular stress distribution

α 0.85

$\beta 1$ 0.85

7.4.2.8 Theoretical Balanced Condition

$a=\beta_1 c$

$c_{balanced}$ 180.568 mm

a 153.483 mm

Lever arm=d-(a/2) = 177.559 mm

Concrete Force Fc 1589.01 kN

$M_{balanced}$ 282.142 kNm

9.3.8.1 Maximum reinforcement so that $c<0.75c_{balanced}$.

c_{max}= 135.426 mm

$a=\beta_1 c$ 115.112 mm

Lever arm=d-(a/2) = 196.744 mm

Concrete Force F_c 1191.76 kN

As_{max} = 4864.31 mm^2

9.3.8.2.1 Min As = 589.976 mm^2

Actual As = 1013 mm^2

Steel Force = 248 kN

c 28 mm

Lever arm=d-(a/2) = 240 mm

M_n 60 kNm

2.3.2.2 (c) ϕ 1

7.4.1 **$M^* = \phi M_n$ = 60 kNm**

S/91

CALCULATION SHEET

Project/Task/File No: WEGC MAIN BLOCK

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Computed: AMO 22/09/2015

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Since no tie beam shown on the original construction drawings between the tops of the columns, all lateral force from above will go to two columns:

Force =

Capacity = 55 x 2 = 110 kN

Columns capable of sustaining ~ μ = 2 demand

OPUS

CALCULATION SHEET

S/92

Project/Task/File No: WEGC EAST BLOCK DSA

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If overstrength factor = 1.25 ; 1.25 × 55 = 69 kN

∴ Shear probably O.K.

Spacing of ties : 8" = 203

↳ Between $\frac{1}{2}$d & d → Use 2 conservative.

If Per column, force required to yield columns = 55 kN

Number of columns = 9

∴ 9 × 55 = 495 kN

CALCULATION SHEET S/93

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Project Description:

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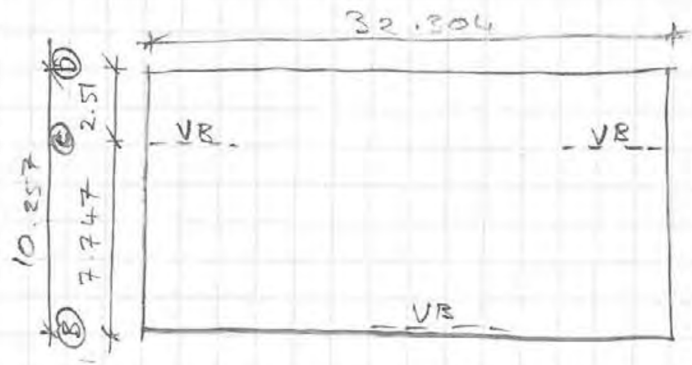
TOP STOREY BRACING – LONGITUDINAL



TOTAL DEMAND = 652 kN – $\mu = 1$, $S_p = 0.9$

On GLC :

Loaded area = $2.51 \times 32.304 + \frac{1}{2} \times 7.747 \times 32.304 = 206\ m^2$

$\frac{206}{(10.257 \times 32.304)} = 62\%$

$\therefore$ Load = $62\% \times 652 = 405$ kN

If $\mu = 2$, $S_p = 0.7$:

$k_\mu = 1.57$
$S_p = 0.7$

$\mu = 2 \rightarrow \frac{S_p}{k_\mu} = 0.44$

$\mu = 1$, $S_p = 0.9$:

$k_\mu = 1$, $S_p = 0.9 \rightarrow \frac{S_p}{k_\mu} = 0.9$

$\therefore$ Ratio = 2

$\therefore$ If $\mu = 2 \rightarrow$ Factor = 50% $\rightarrow 405 \times \frac{1}{2} = 202$ kN

Capacity ~ 110 kN $\rightarrow$ 54 %

BUT, capacity of big CBF on GLB probably fine if torsion taken out by transverse frames. Check

OPUS

CALCULATION SHEET

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Computed: PMO 29/09/2015

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RELYING ONLY ON FRONT CBF

- Model re-run without vertical braces on GLC.
- Loads in braces go up ~ 500 kN. This is ~ 1.67 × cap.

↳ 60% NBS — LOWER BOUND

- Period of top storey will increase as cantilevering columns yield! This will have the effect of lowering demand.
- Cantilevering columns will still provides some resistance.

- Overall ~ 70% seems fair.

↳ ETABS model indicates period without X braces ~ 0.45s

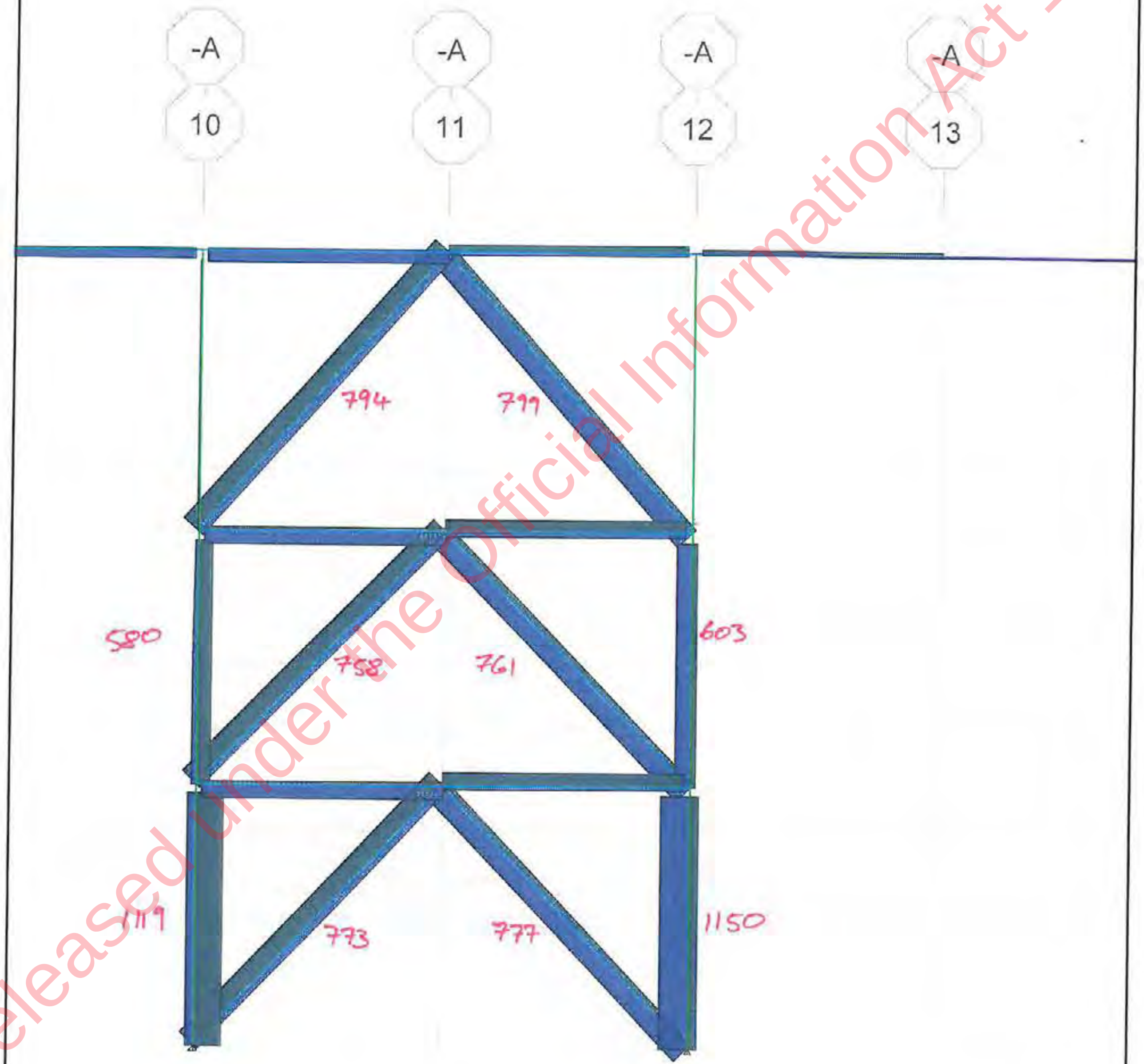
$C_d(T) = 0.9$ vs 0.98

$\therefore$ 0.92 factor → $\frac{60}{0.92} = 65\%$

+ Some contribution from X braces at other side -

70% seems fair.

MODEL V 2.4 :
NO VBs ON GLC – ALL TOP STOREY LONG.
BRACING DONE BY CBF ON FRONT OF
BUILDING :



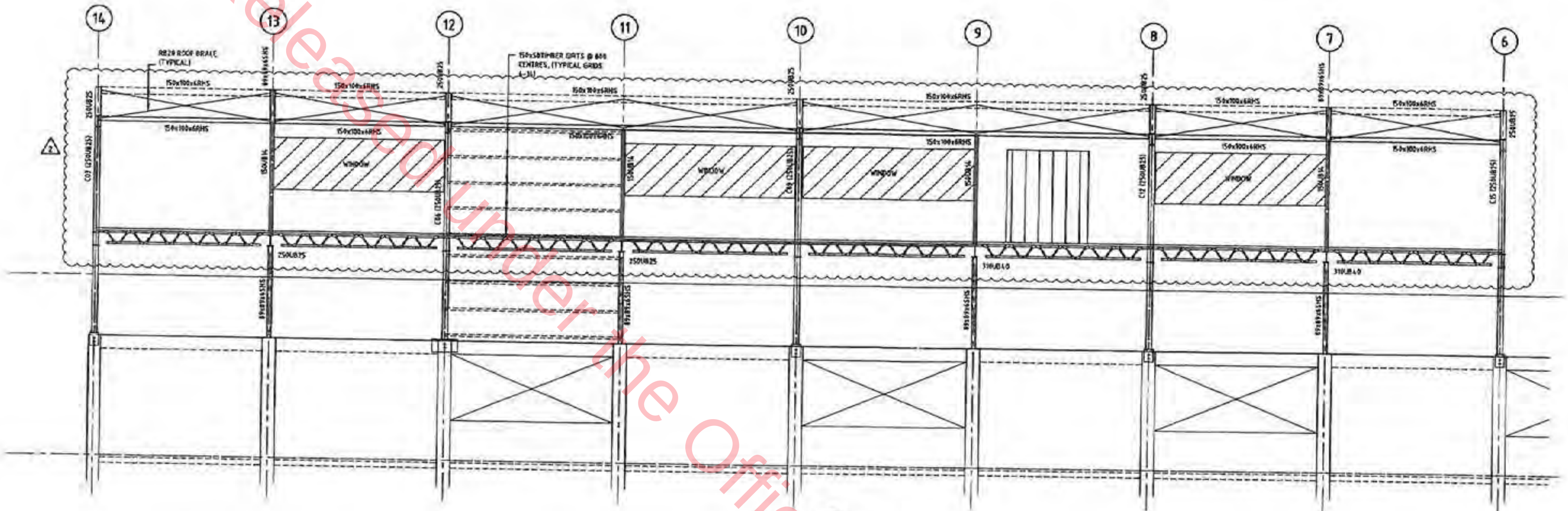
DIAG. BRACE CAPACITY – $\phi N_c = 429$ kN
System good for $\mu = 1.25$ only – sections will be LO
-Cat. 3. $\therefore \frac{N_c}{N^*} = \frac{429}{799} \times \frac{1}{0.9} = 60\%$

COLUMNS = O.K. – $\phi N_c = 1200$ kN

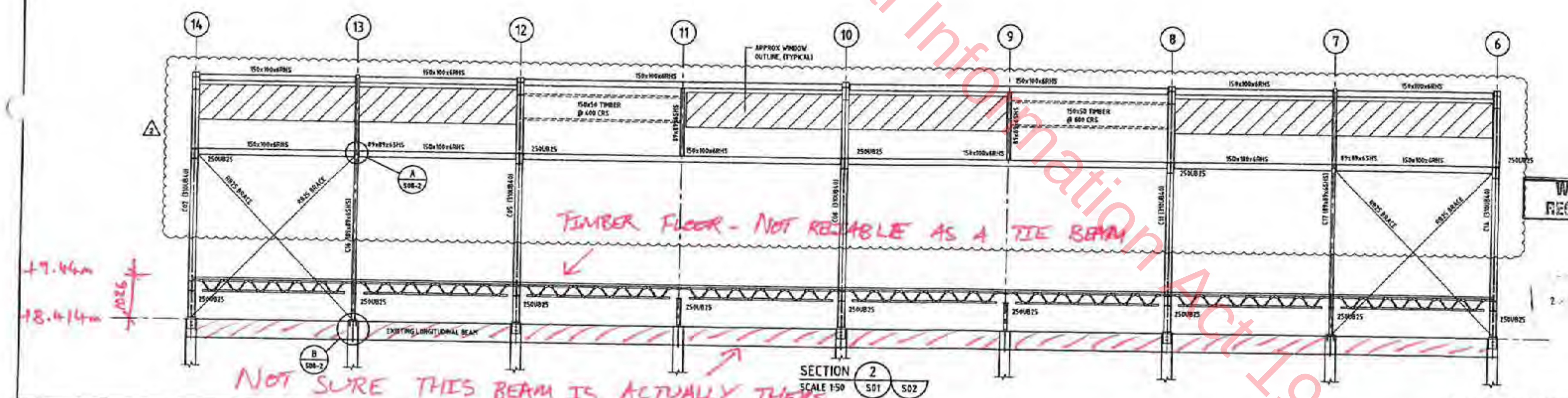
East Block v2.4.EDB Elevation View - -A Axial Force Diagram (EQENV) [kN]

S/96

A1



SECTION 1 S01 S02
SCALE 1:50



SECTION 2 S01 S02
SCALE 1:50

W.C.C. RECEIVED

CONSENT/TENDER

Rev.	Date	Revision Details	By	Ver.	App.
2	10.08.05	FOR CONSENT/TENDER	CM		
1	05.08.05	PRELIMINARY	ABM		

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Connell Wagner Limited, Lumley House, 3 Hunter Street (PO Box 1591), Wellington New Zealand. Telephone: +64 4 472 8988. Facsimile: +64 4 472 8822. Email: cwwel@conwag.com

Client:	Project:	Drawn	Signed	Date	Drawing Title:	CW Project No.
WELLINGTON EAST GIRLS COLLEGE	EXTENSION TO EAST WING BLOCK 7	ABM		00-00-00	SOUTH ELEVATION	7970 12
		Designed TW		00-00-00		Scale 1:50 (@ A1) 1:100 (@ A3)
		Verified CJM	Signed	Date		Drawing No S 03
		Approved MM	Signed	Date		Rev. 2

S/97

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK

Sheet No of

Project Description:

Office:

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UTILISING TIMBER FLOOR AS TIE BEAM:

Floor system = 400 / 300 Series Posistrut - Timber truss - joists.

- Connection to the columns not likely to be that reliable

↳ Discount.

SUMMARY OF ASSESSMENT OF BRACES

- ~~RB~~ Reidbraces themselves ~ 80% NBS
- Connections (Anchors) ~ 75% NBS
- Rely on tie-beam being present at bottom of braces.
- Original construction drawings indicate that the tie beam assumed in the additions is not actually there.
- Cannot confirm (easily) whether tie beam is there or not.
- CBF can take alot of load of braces.

↳ Prob. good for 70% NBS ~ reasonably conservative.

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK

Sheet No of

Project Description:

Office:

Computed: AMO 22/09/2015

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ROOF BRACING

– FORCES FROM ETABS

RB20

Worse-case force $= (55+71)\text{kN} \times \frac{1}{0.925} = 136\text{kN}$

Capacity $= 500 \times 314 = 157\text{kN}$

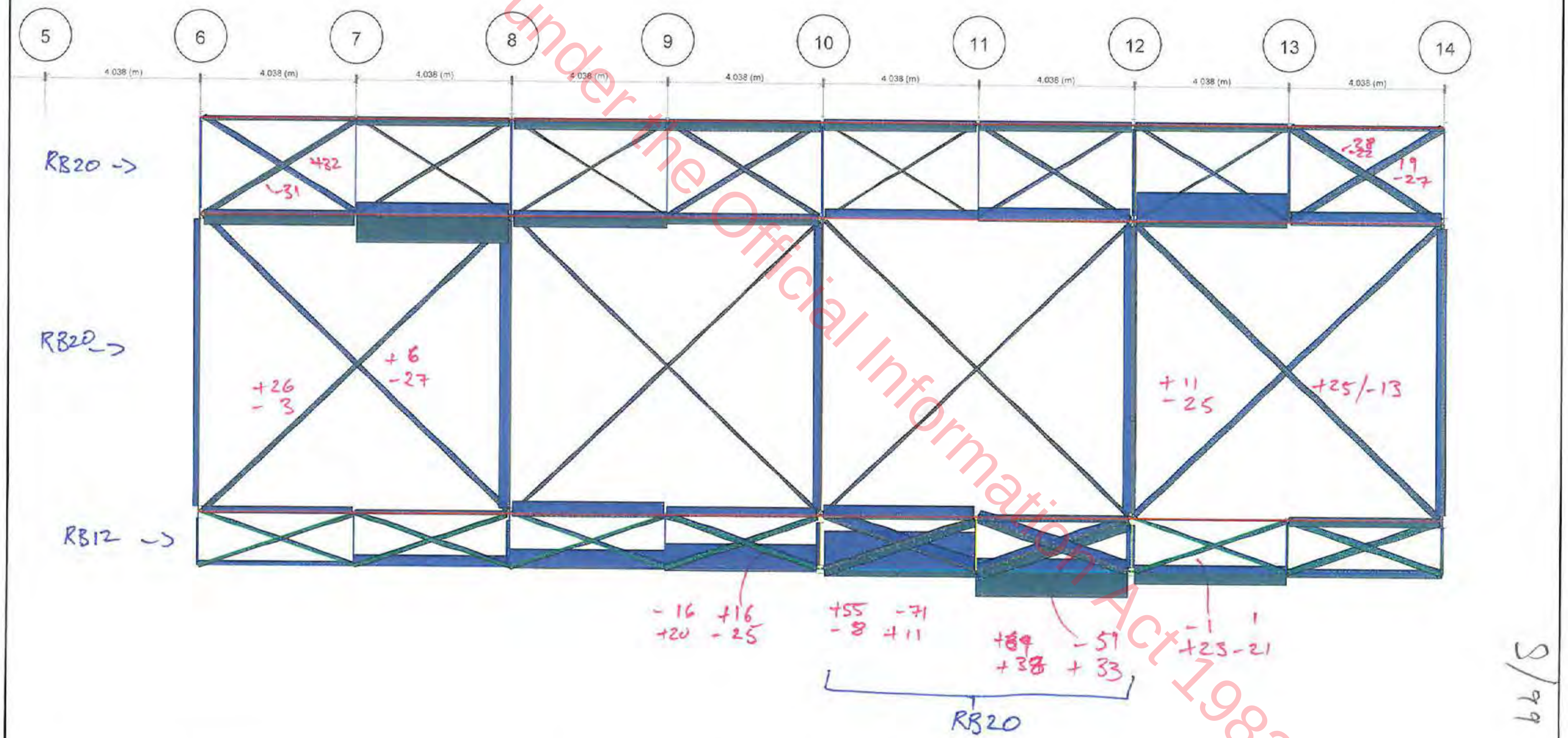
∴ O.K.

RB12

Worse case $= 45\text{kN} \times \frac{1}{0.925} = 49\text{kN}$

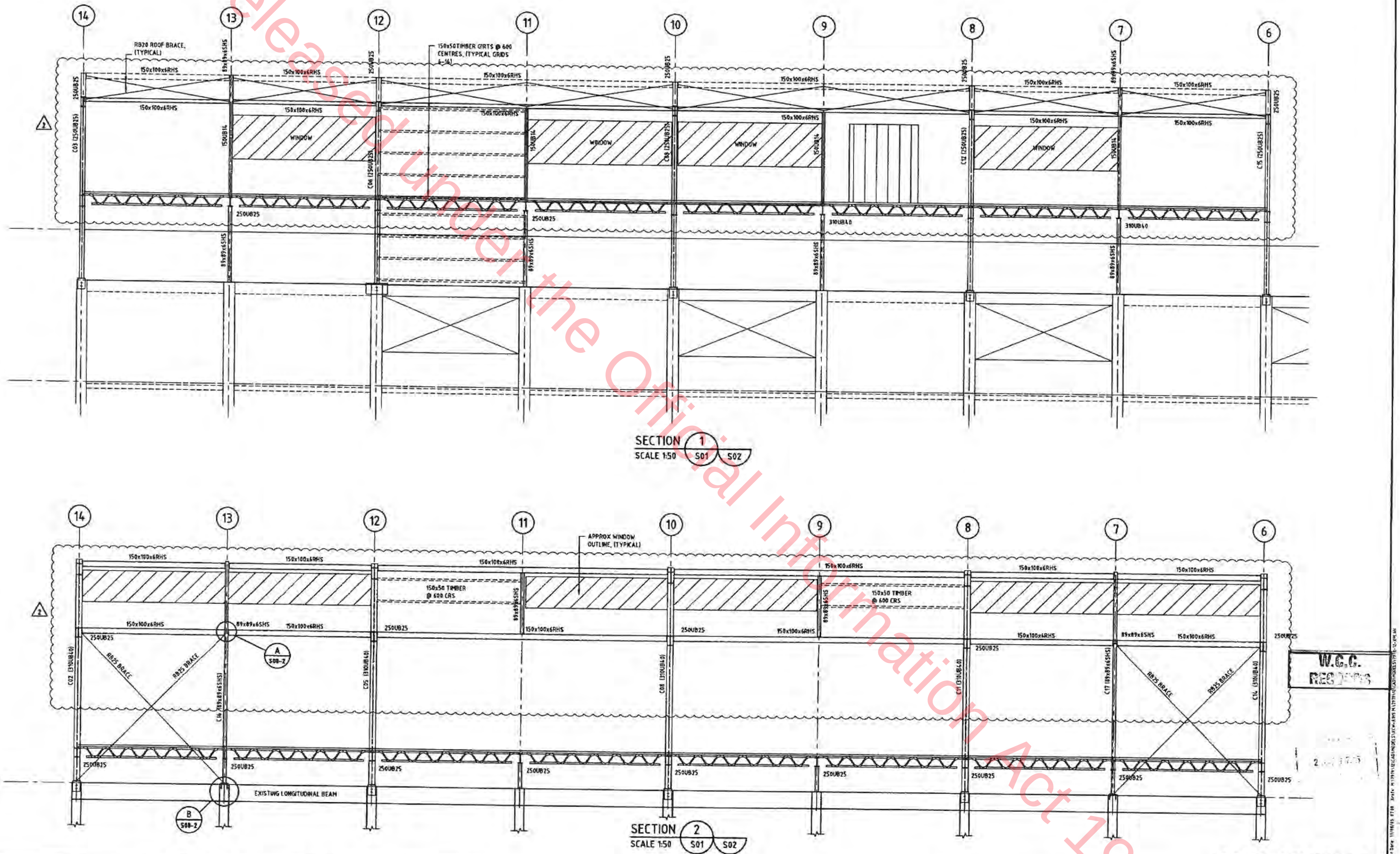
Capacity $= 500 \times 113 = 56.5\text{kN}$ – O.K.

∴ $\boxed{\text{ALL ROOF BRACING O.K.}}$



Plan View - Story5 - Z = 13 (m) Axial Force Diagram (EQENV) [kN]

A1



SECTION 1 S01 S02
SCALE 1:50

SECTION 2 S01 S02
SCALE 1:50

W.G.C.
RECORDS

S/100

CONSENT/TENDER

Rev.	Date	Revision Details	By	Ver.	App.
2	10.08.05	FOR CONSENT/TENDER	CM	7. ω	
1	05.08.05	PRELIMINARY	ABM	7. ω	

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Connell Wagner Limited
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Telephone: +64 4 472 9589
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Email: cwwel@conwag.com

Client: WELLINGTON EAST GIRLS COLLEGE

Project: EXTENSION TO EAST WING BLOCK 7

	Signed	Date
Drawn ABM		00-00-00
Designed TW	7. ω	00-00-00
Verified CJM		
Approved MM		

Drawing Title: SOUTH ELEVATION

CW Project No. 7970 12

Scale 1:50 (@ A1) 1:100 (@ A3)

Drawing No. S 03 Rev. 2

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK

Project Description:

Sheet No S/101 of

Office:

Computed: PMO1 09/2015

Check: / /

PORTAL FRAMES

For 310UB40 beams

$\phi M_{sx} = 182$ kNm

$M_{sx} = 203$ kNm

Demand - Generally ~ 250 - 300 kNm demand in floor beam
(μ = 1.25, Sp = 0.9 Demands).

∴ Ductility demand ~ 1.5 - 2

- Also, conservative period assumed - actually likely longer than 0.4 seconds. ∴ Demands conservative.

Drawings show stiffeners opposite flanges. Looks quite a good detail - may cause some yielding in shear of the web but should be fine for $\mu = 2$.

LATERAL - TORSIONAL BUCKLING

Tie beams at corners.
Fly brace in centre of rafter.

- LTB may not be fully supressed by studies have shown that $\mu = 2$ cap. is still likely for assessment purposes.

∴ O.K.

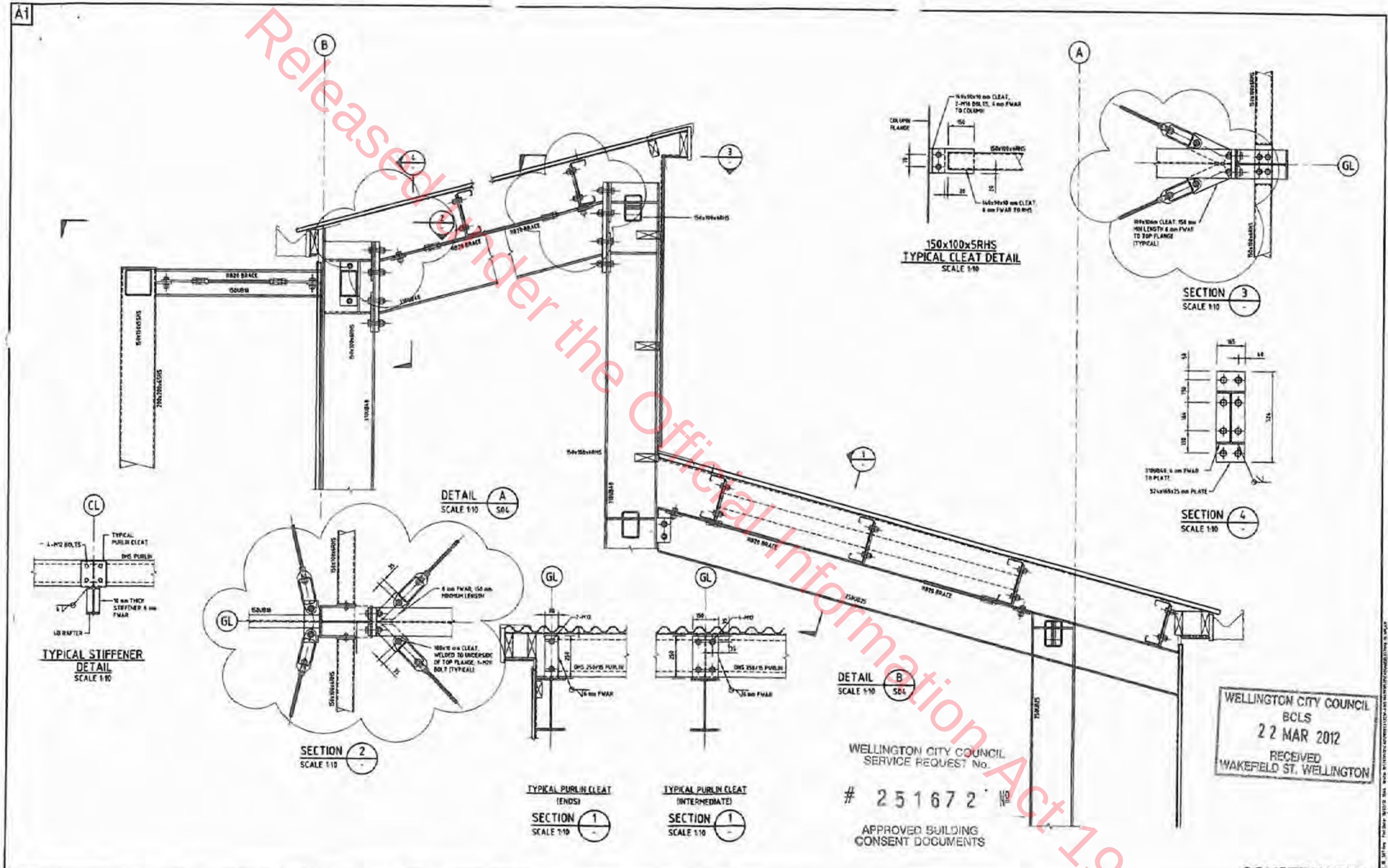
∴ $\boxed{\text{PORTALS} > 100\%}$

OPUS

S/102

Beam	
Section type	UB
Section	310 UB 40.4
fy	320 MPa
Zx	633000 mm3
M_{sx}	202.56 kNm
Φ	0.9
ΦM_{sx}	182 kNm
Overstrength Φ_{oms}	1.25
$\Phi_{oms} M_{sx}$	253 kNm

S/103



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BCLS
22 MAR 2012
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WELLINGTON CITY COUNCIL
SERVICE REQUEST No.
2 5 1 6 7 2

APPROVED BUILDING
CONSENT DOCUMENTS

CONSTRUCTION

Rev.	Date	Revision Details	By	Ver.	App.
B	??/??/??	????????????????????			
A	29.11.05	ISSUE FOR CONSTRUCTION	TW		
3	05.09.05	RE-ISSUE FOR TENDER	TW		
2	10.08.05	FOR CONSENT / TENDER	TW		
1	05.08.05	PRELIMINARY	ABN		

AMENDMENT B NOT ISSUED YET 16/02/12

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Client: WELLINGTON EAST GIRLS COLLEGE

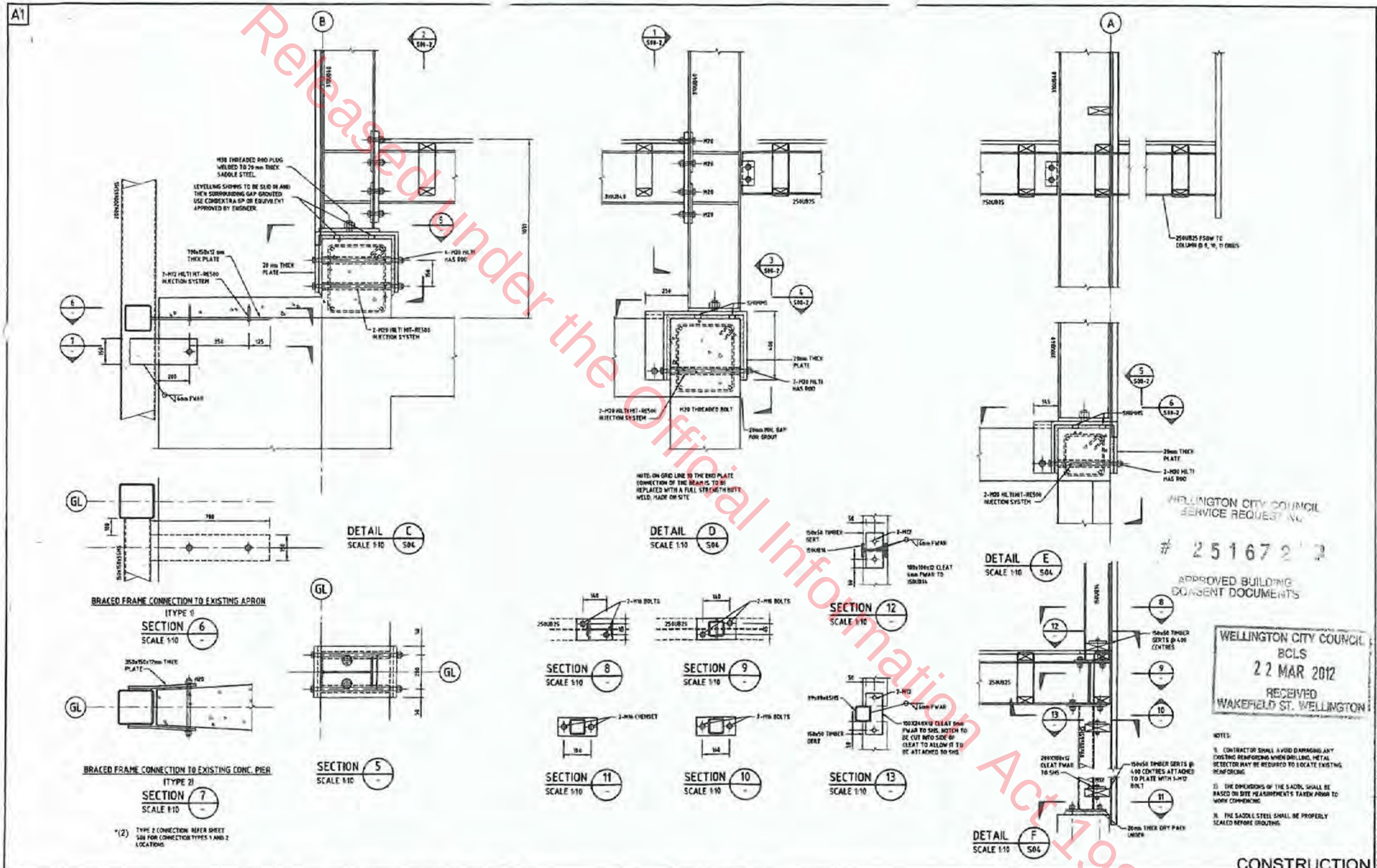
Project: EXTENSION TO EAST WING BLOCK 7

Drawn	Signed	Date
ABN		05.08.05
Designed TW		05.08.05
Verified CJM		10.08.05
Approved MH		10.08.05

Drawing Title: DETAILS SHEET ONE

CW Project No. 7970 12
Scale 1:10 (@ A1) 1:20 (@ A3)
Drawing No. S 07
Rev B

3/104



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251672

APPROVED BUILDING CONSENT DOCUMENTS

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NOTES:

1. CONTRACTOR SHALL AVOID DAMAGING ANY EXISTING REINFORCING WHEN DRILLING. METAL DETECTOR MAY BE REQUIRED TO LOCATE EXISTING REINFORCING.
2. THE DIMENSIONS OF THE SADDL SHALL BE BASED ON SITE MEASUREMENTS TAKEN PRIOR TO WORK COMMENCING.
3. THE SADDL STEEL SHALL BE PROPERLY SEALED BEFORE GROUTING.

CONSTRUCTION

Rev	Date	Revision Details	By	Ver.	App.
A	29.11.05	ISSUE FOR CONSTRUCTION	TW		
3	05.09.05	RE-ISSUE FOR CONSENT	TW		
2	10.08.05	FOR CONSENT/TENDER	TW		
1	05.08.05	PRELIMINARY	ABM		

AMENDMENT A NOT ISSUED 16/02/12

Connell Mott MacDonald
...building the future

Client: WELLINGTON EAST GIRLS COLLEGE

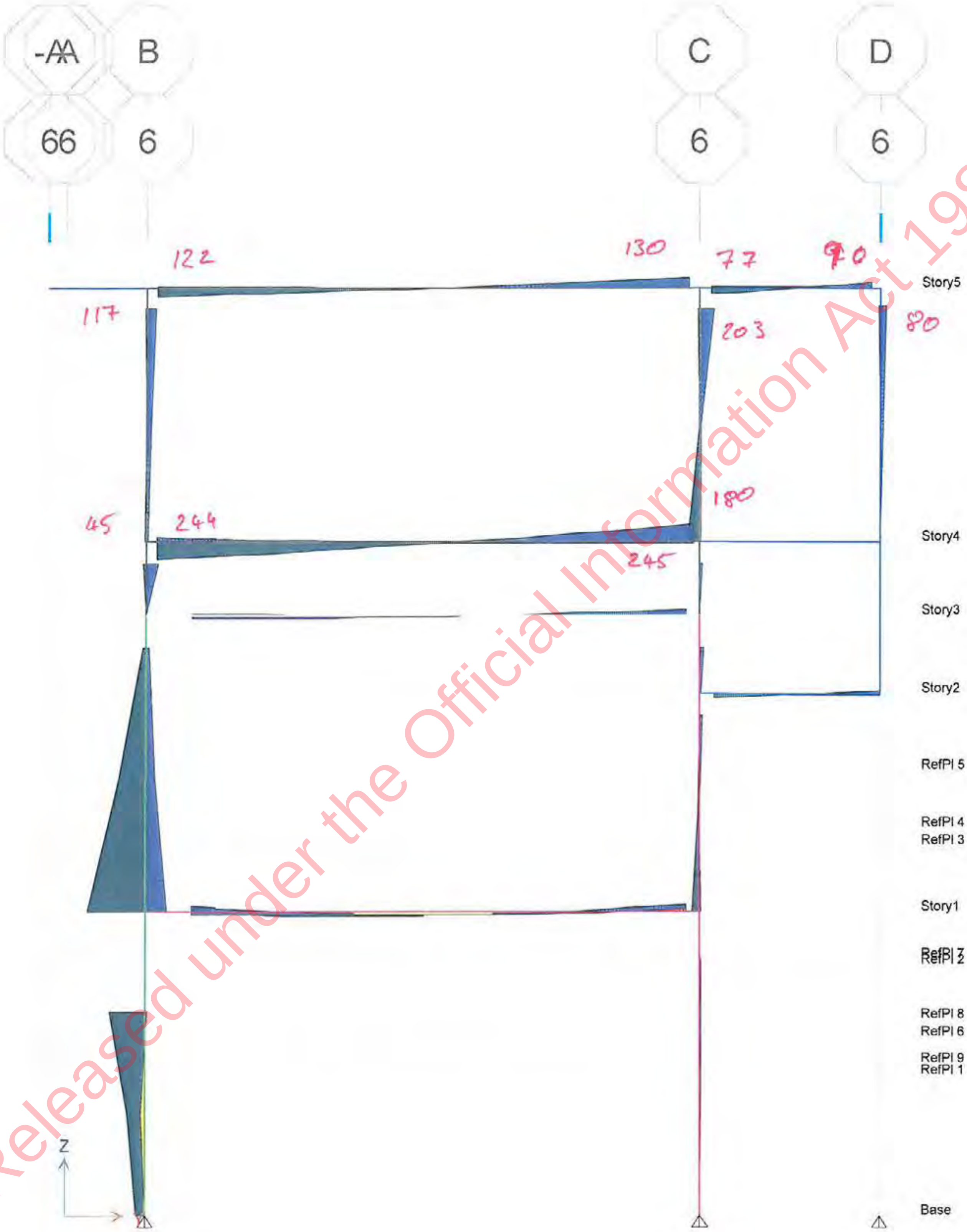
Project: EXTENSION TO EAST WING BLOCK 7

		Date
Drawn	ABM	05.08.05
Designed	TW	05.08.05
Verified	CJM	10.08.05
Approved	MM	10.08.05

Drawing Title: DETAILS SHEET TWO

CM Project No. 7970 12
Scale 1:10 (@ A1) 1:20 (@ A3)
Drawing No. S 08-1 Rev. A

S/105



-AA
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D
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130
77
90
117
203
80
45
244
180
245
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
Base
Z

S/106


-AA
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88
8
8
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143
106
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309
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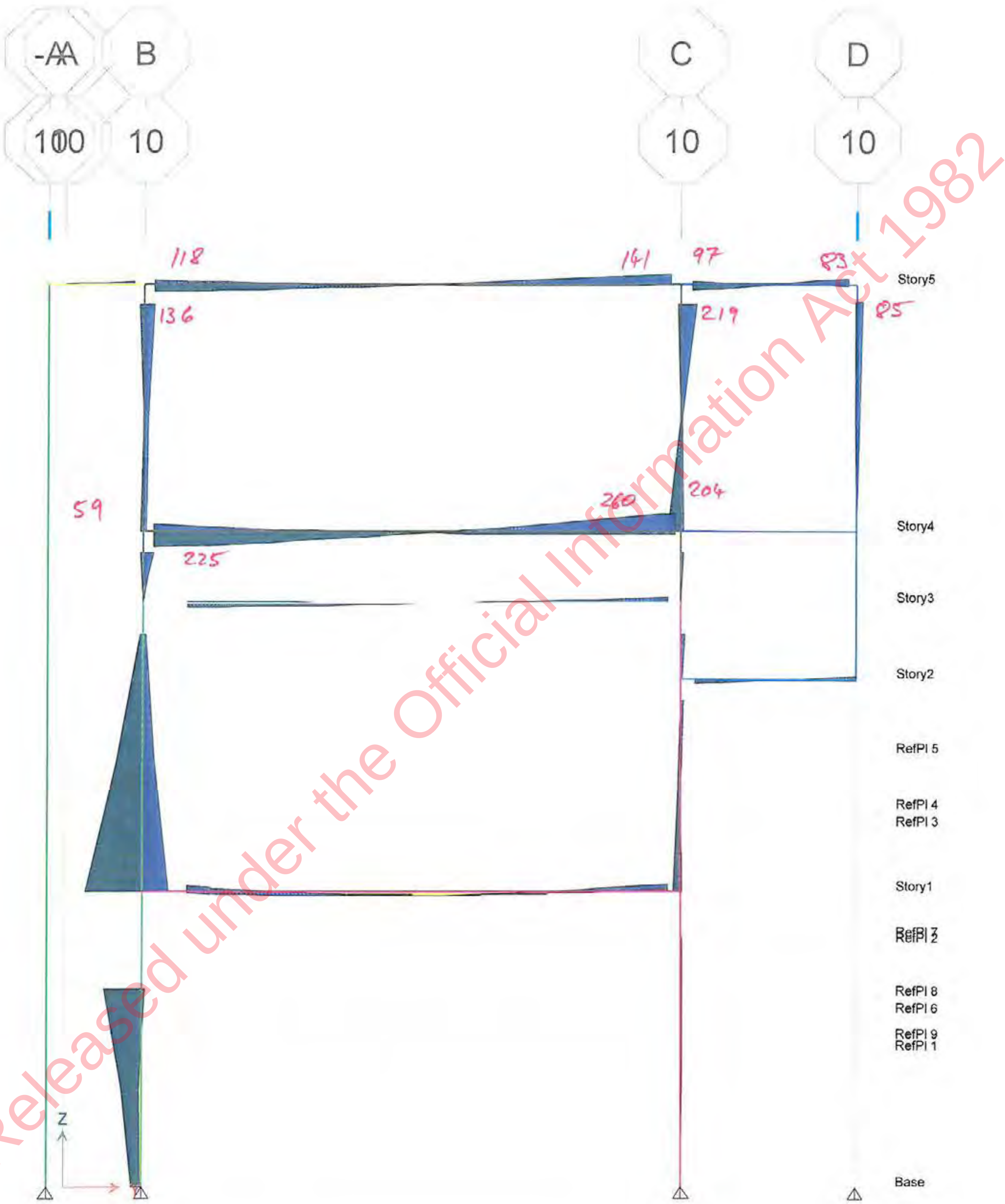
21
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
Base
Z

S/107



A
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141
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83
136
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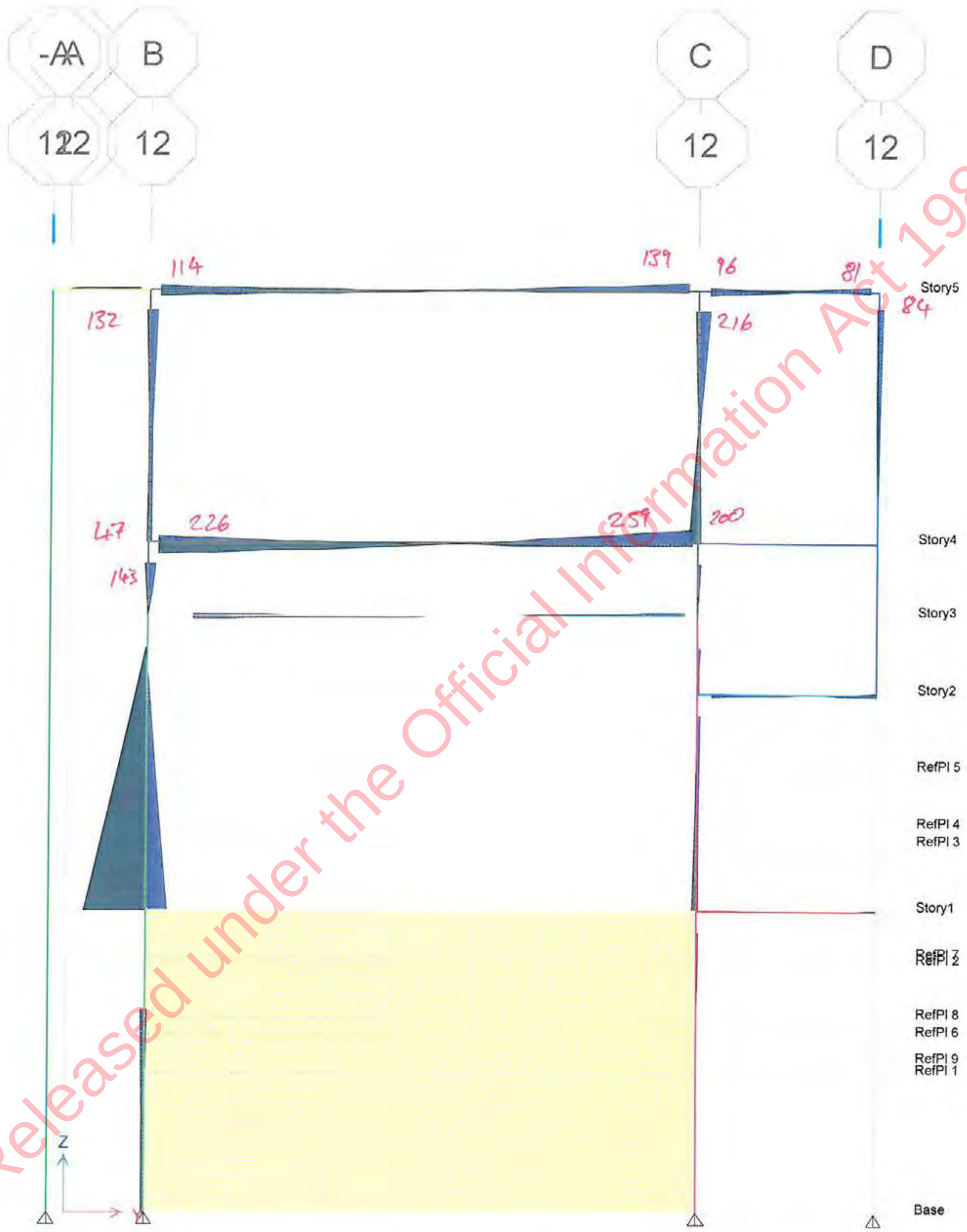
85
59
260
204
225
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
Base
Z

S/108



-AA
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216
47
226
259
200
143
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
Base
Z

S/109

-A
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129
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81
117
209
84
270
262
37
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214
Story5
Story4
Story3
Story2
RefPl 5
RefPl 4
RefPl 3
Story1
RefPl 7
RefPl 2
RefPl 8
RefPl 6
RefPl 9
RefPl 1
Base
Z

S/110

CALCULATION SHEET

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Project Description: ______ | Office: ______

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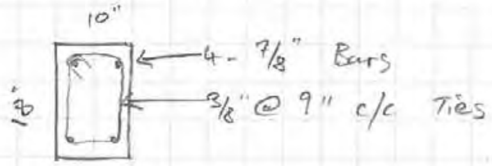
ORIGINAL ROOF BEAMS - OUT-OF-PLANE

Typical beam :



$d = 254 - 38 - 9.5 - \frac{22}{2} = 195\,mm$

$M_n = 35\,kNm$

DEMAND

DEAD LOAD - UDL

Roof - = 0.25

Ceiling & Services = 0.20

Σ 0.55 kPa × 4.039 m = 2.22 kN/m

S/W = 0.457 × 0.254 × 25 = 2.9 kN/m

5.12 kN/m

Treating as a part with $\mu = 2$ capacity :

$h_i = 8.4\,m$

$h_n = 14.5\,m$

Total weight = 5.12 × 8 = 41 kN

For $\mu = 2$ → Force F_h = 56 kN

$\therefore M^* = 56 \times \frac{8}{8} = 56\,kNm$

Capacity = 35 kNm

$\frac{35}{56} = 63\%$

If $\mu = 3$ → $F_h = 46\,kN$

$M^* = 46\,kNm$

$\frac{35}{46} = 76\%$

Check ductility capacity - Ties @ 9" = 229 mm = ½ depth.

↳ $\mu = 2$ prob. about right.

OPUS

S/111

Singly Reinforced Beams

Characteristic Material Values					
1st Yield (MPa)		Young's Modulus (MPa)		Strains at 1st yield	
f_y	245	E_s	200000	ε_s	0.00123
f'_c	30			ε_c	0.003

Depth D (mm)	Breadth b_w (mm)	Cover (mm)	Tension Reinforcement Diameter	Number of Bars	Effective depth d (mm)	As (mm^2)	Maximum aggregate size (mm)
254	457	48	22.225	2	194.888	[illegible]	20

Flexure

7.4.2.7 Equivalent concrete rectangular stress distribution

α 0.85

$\beta 1$ 0.85

7.4.2.8 Theoretical Balanced Condition

$a=\beta_1 c$

$c_{balanced}$ 138.382 mm

a 117.624 mm

Lever arm=d-(a/2) = 136.075 mm

Concrete Force Fc 1370.74 kN

$M_{balanced}$ 186.523 kNm

9.3.8.1 Maximum reinforcement so that $c<0.75c_{balanced}$

$c_{max}=$ 103.786 mm

$a=\beta_1 c$ 88.2183 mm

Lever arm=d-(a/2) = 150.778 mm

Concrete Force F_c 1028.05 kN

As_{max} = 4196.13 mm^2

9.3.8.2.1 Min As = 508.935 mm^2

Actual As = 776 mm^2

Steel Force = 190 kN

c 19 mm

Lever arm=d-(a/2) = 185 mm

M_n 35 kNm

2.3.2.2 (c) ϕ 1

7.4.1 **$M^* = \phi M_n$ = 35 kNm**

CALCULATION SHEET

Project/Task/File No: WEGC EAST BLOCK DSA

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Taking some fixity at connections:

$$M^* = \frac{WL}{12} \rightarrow \frac{56 \times 8}{12} = 37 \text{ kNm}$$

$$\frac{35}{37} = 95\%$$

$\therefore$ O.K.

OPUS

CALCULATION SHEET S/113

Project/Task/File No: WEGC EAST BLOCK Sheet No of

Project Description: Office:

Computed: PMO / 6/10 / 2015

Check: / /

2nd FLOOR DIAPHRAGM

- Timber diaphragm - Horizontal.
- Lateral forces transferred to transverse steel beams at each bay.

Connection of steel beam = 8 - M20 in shear :

Assuming Grade 4.6/S bolts : $\phi V_{f} = 44.6$ kN / Bolt

$8 \times 44.6 = 357$ kN

Number of connections along GL B = 9

$\therefore$ $9 \times 357 = 3211$ kN

$\therefore$ O.K.

By inspection, timber floor will be adequate as a diaphragm.

$\therefore$ > 100%

CALCULATION SHEET

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FOUNDATIONS

The Foundations for the building are shallow strip and pad footings, with a ground-bearing slab.

The East Block is founded on rock. Its stability system comprises relatively long, squat shear walls. The new CBF that was installed on the north elevation has pad foundations.

CHECK ON CBF PADS

Load = 850 kN — Model V. 2.2
560 kN V. 2.1

Size of pad : 1650 x 1650 x 600

Weight : 25 x volume = 41 kN

∴ Max force = 850 + 41 = 890 kN

Pressure = $\frac{890}{1.65^2}$ = 327 kPa

Allowable bearing pressure likely to be in order of 300 - 400 kPa minimum.

∴ $\boxed{\text{PAD FOUNDATIONS O.K.}}$

WALL STRIP FOOTINGS

Wall strip footings = 14" or 18" wide, with widenings at ends.
356mm 457mm

TYPICAL WALL - ON GL 5 : - 356mm wide

P = 414 kN
M = 2929 kNm
Length = 7.747m

∴ Eccentricity = 7.075m

- May get some rocking with minor damage to ground / footing, but building is robust, so should be able to tolerate some rocking without compromising its stability.

OPUS

CBF FRAME REACTIONS S/115
V 2.1

Story	Joint Label	Unique Name	Load Case/Com	FX	FY	FZ	MX	MY	MZ
Base	185	329	EQENV Max	-4	2	63	0	0	0
Base	185	329	EQENV Min	-40	0	-558	0	0	0
Base	186	368	EQENV Max	19	1	595	0	0	0
Base	186	368	EQENV Min	-29	-2	-333	0	0	0

CBF REACTIONS
V.2.2

S/116

Story	Joint Label	Unique Name	Load Case/Com	FX	FY	FZ	MX	MY	MZ
Base	192	370	EQENV Max	93	1	279	0	0	0
Base	192	370	EQENV Min	-273	-3	-848	0	0	0
Base	193	371	EQENV Max	91	1	887	0	0	0
Base	193	371	EQENV Min	-275	-3	-300	0	0	0

CALCULATION SHEET

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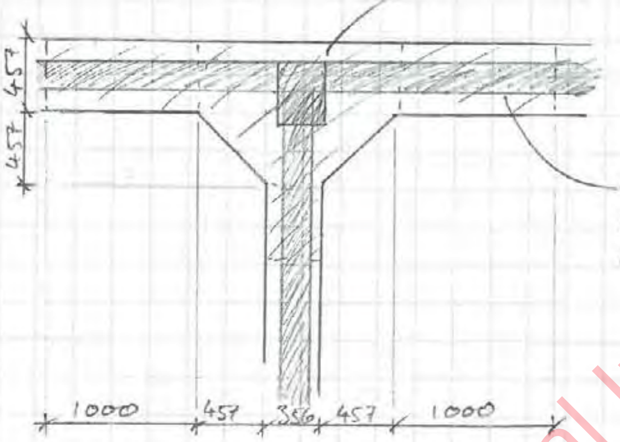
WALL ON GL12

FOUNDATION AT END OF WALL

- Check bearing

Area : $0.952\,m^2 + 2 \times 0.457 + 0.178 = 2.044\,m^2$



Wall will ensure 'T' of footing is engaged - 1m either side assumed effective.

$\mu = 2, S_p = 0.7$ Demands

If load in rocking case : $851 + 280 = 1131\,kN$

↳ Say 1200 kN

∴ Pressure $= \frac{1200}{2.044} = 587\,kPa$

If allowable = 500 kPa → 85%

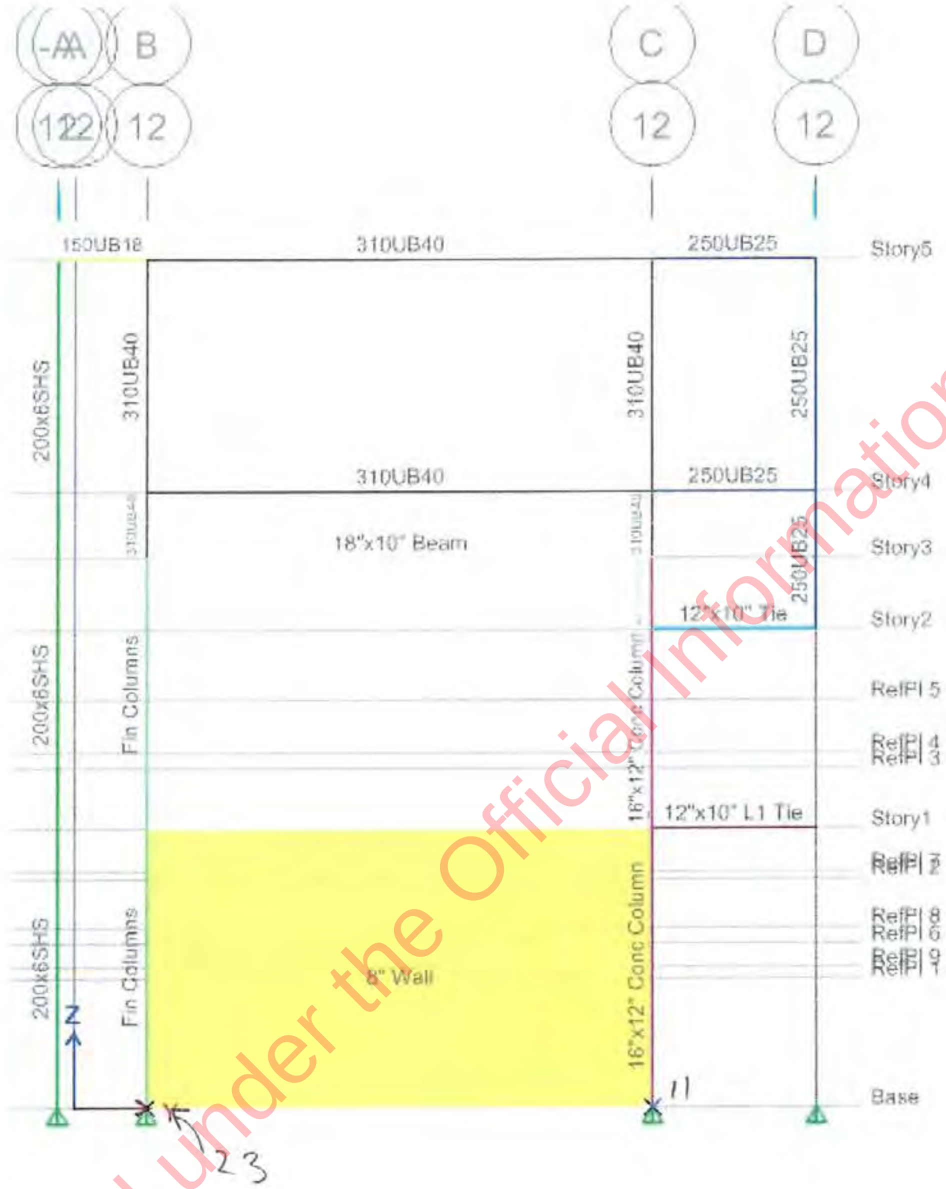
400 → 68%

↳ 80% looks reasonable.

OPUS

S/118

GL12 Wall Reactions



$\mu = 1.25$, $S_p = 0.9$ DEMANDS

Story	Joint Label	Unique Name	Load Case/Com	FX	FY	FZ	MX	MY	MZ
Base	11	101	21.DSL+EQY+0.3EQX-	-21	-926	1192	0	0	0
Base	11	101	EQENV Max	-8	113	1192	0	0	0
Base	11	101	EQENV Min	-43	-926	166	0	0	0
Base	23	59	21.DSL+EQY+0.3EQX-	0	-640	-781	0	0	0
Base	23	59	EQENV Max	0	97	605	0	0	0
Base	23	59	EQENV Min	0	-640	-781	0	0	0

GL 12 WALL ENDS

$\mu = 2$, $S_p = 0.7$ DEMANDS

S/120a

Story	Joint Label	Unique Name	Load Case/Com	FX	FY	FZ	MX	MY	MZ
Base	11	101	21.DSL+EQY+0.3EQX-	-20	-540	851	0	0	0
Base	11	101	EQENV Max	-13	48	851	0	0	0
Base	11	101	EQENV Min	-32	-540	270	0	0	0
Base	23	59	21.DSL+EQY+0.3EQX-	0	-347	-280	0	0	0
Base	23	59	EQENV Max	0	70	503	0	0	0
Base	23	59	EQENV Min	0	-347	-280	0	0	0

$\mu = 2$ used for rocking case.

As can be seen, some uplift even at this case = 280kN

Simply adding to the 851 will conservatively give an upper bound value for downward force on the end in compression: 280 + 851 = 1131 kN

S/1206

CALCULATION SHEET

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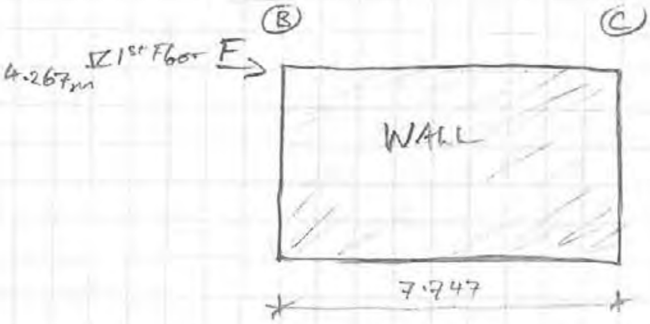
PUSHOVER OF WALL ON GL 12



Using ETABS model, $\mu=4$, $S_p=0.7$ demands give approx: uplift point Force = 510 kN - see next pages.

Displacement at uplift point will be small, say 5mm.
Seismic weight attributable to the wall = 2034 kN - see next page.

Using Opus 7-CED 272 (N2) Building %NBS calculator for non-linear pushover method:
For damping factor of 10%:
Spectral Displacement Demand = 80mm = 1.9% Storey drift.

Frame elements are able to tolerate some displacement, and the columns have a ductility capability of $\mu=2$ (see S/54 onwards) and are flexurally governed.

Conservatively limiting acceptable drift at 1st floor to 1.5%:

1.5% = 64 mm

$\frac{64}{80}$ = 80 %

OPUS

WALL AT GL 12 S/120 bi

- PUSHOVER POINT OF UPLIFT

Story	Joint Label	Unique Name	Load Case/Com	FX	FY	FZ	MX	MY	MZ
Base	11	101	27.DSL+E QY++0.3E QX-	-20	-328	664	0	0	0
Base	11	101	EQENV Max	-16	12	664	0	0	0
Base	11	101	EQENV Min	-27	-328	328	0	0	0
Base	23	59	27.DSL+E QY++0.3E QX-	0	-187	-6	0	0	0
Base	23	59	EQENV Max	0	54	448	0	0	0
Base	23	59	EQENV Min	0	-187	-6	0	0	0

Uplift in model when FZ turns negative.
Running the model (V2.5) with $\mu = 4$, $S_p = 0.7$
Shows wall uplifting as end '23' goes into tension
(-6 kN $\triangleq$ 0 kN)

Lateral Force at this point = $F_Y(11) + F_Y(23) = 328 + 187 = 515$ kN
∴ Wall starts to lift up at ~ 510 - 515 kN
↳ Use 510 kN

∴ Shear force in wall at this point = 515 kN

Total building shear force = Base shear = 2152 kN ($\mu = 4$, $S_p = 0.7$)

Total building weight = 8500 kN

∴ 'Weight' resisted by this wall = $8500 \times \frac{515}{2152}$ =
= 2034 kN

MODEL V2.6 μ=4, Sp=0.7 S/120bii
STOREY SHEARS

Story	Load Case/Com	Location	P	VX	VY	T	MX	MY
Story5	EQENV Max	Bottom	257	61	90	3365	2481	-8526
Story5	EQENV Min	Bottom	229	-19	-300	-11040	1032	-9348
Story4	EQENV Max	Bottom	791	54	168	7880	4283	-23557
Story4	EQENV Min	Bottom	648	-180	-553	-20704	4011	-29322
Story3	EQENV Max	Bottom	827	102	-84	349	6599	-12351
Story3	EQENV Min	Bottom	592	-296	-233	-5761	4462	-21154
Story2	EQENV Max	Bottom	2621	187	-29	4270	24581	-49317
Story2	EQENV Min	Bottom	2072	-599	-443	-10251	18310	-66324
Story1	EQENV Max	Bottom	10111	646	646	32340	78153	-264275
Story1	EQENV Min	Bottom	10111	-2152	-2152	-65005	58358	-284070

Base shear = 2152 kN

S/120c

Project: WEGC East Block DSA
Element: Typical Wall (GL12)
Calculation: Push-over curve, modified for mode shape, plotted on the NZS1170.5 Acceleration-Displacement demand spectra

Project Number:
Sheet No: of
Input By: PMO
Date: 9/10/2015

Design Spectrum

Site Subsoil Class		B
Zone Factor	Z	0.4
Return Period Factor	R	1.3
Distance to Major Fault[1]	D (km)	5
Structural Performance Factor	Sp	0.7

(1) Equal to 21 if N/A or exceedance probability <1/250

Pushover Curve

Building weight	W (kN)	2035.0					
Mode 1 mass participation factor	PF_1	1.00					
Mode 1 mass coefficient	α_1	1.00					
Pushover Curve Point	i	1	2	3	4	5 (at Δ_{ULS})	6 (at Δ_{CLS})
Base shear	V_i (kN)	0.0	510.0	510.0	510.0	510.0	510.0
Roof level displacement	$\Delta_{i,roof}$ (mm)	0	5.0	25.0	35.0	43.0	500.0
Spectra acceleration	S_{ai} (g)	0.00	0.251	0.251	0.251	0.251	0.251
Spectral displacement	S_{di} (mm)	0.00	5.00	25.00	35.00	43.00	500.00
Period	T_i (s)		0.28	0.63	0.75	0.83	2.83

– Treating as a single degree of freedom – conservative.

Structural Behaviour

Yield base shear	V_y (kN)	705.0
Yield displacement	Δ_y (mm)	10.0
System displacement ductility	μ_s	4.30
Effective response period	T_{eff} (s)	0.83

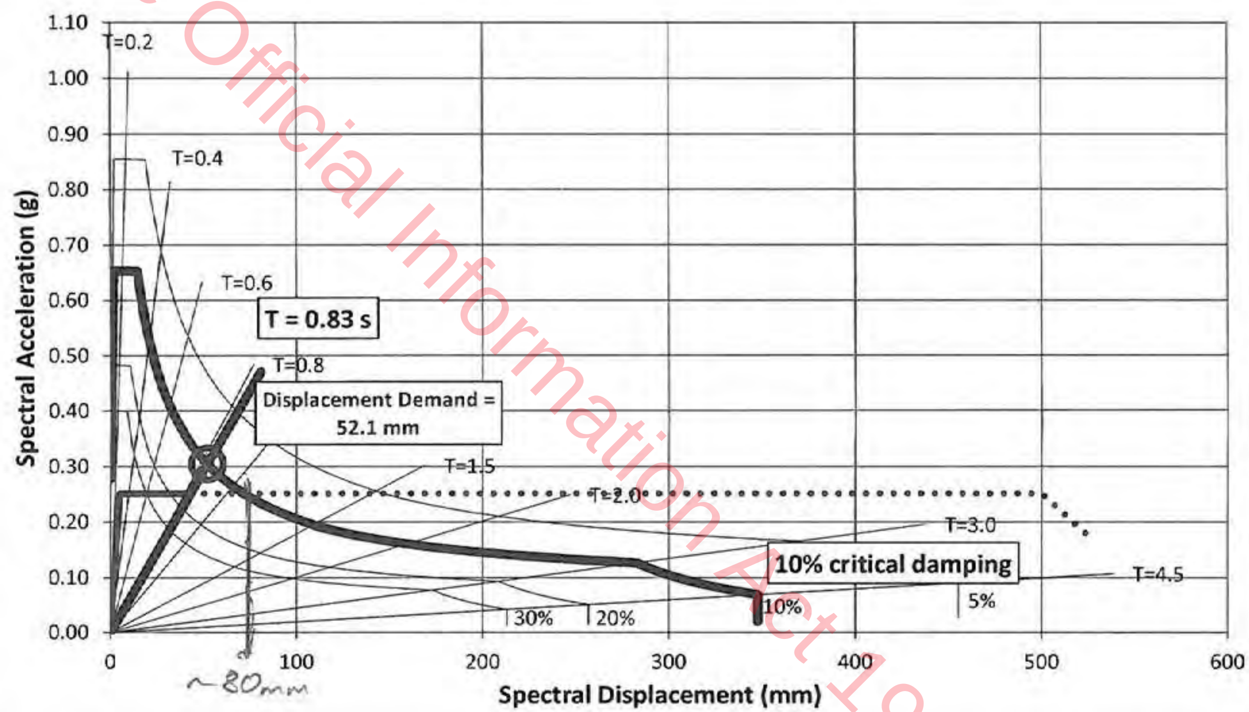
Effective Damping

Method: Custom

Effective system damping	ξ_{eff}	10.0%

Results

Spectral displacement capacity	S_{d_ult} (mm)	43.0
Spectral displacement demand	SD (mm)	52.1
Calculated capacity ratio with 10% critical damping	NBS at 0.83 s	83%



CALCULATION SHEET

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LIQUEFACTION

- No risk of liquefaction - rock.

SUMMARY

- Foundations should be adequate to support the structure above, and building is robust enough to tolerate some minor settlements or foundation damage (life-safety case).

∴ $\boxed{\text{FOUNDATIONS O.K.}}$

CALCULATION SHEET

S/122

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150 x 12 FLAT BRACES

Grade 300 MPa – $f_y = 310$ MPa

$A_g f_y = 150 \times 12 \times 310 = 558$ kN

With Holes : 2 lines of M16 anchors :

2 x 18mm ϕ holes = 36mm

7.2.1 $0.85\, k_{te} A_n f_u =$

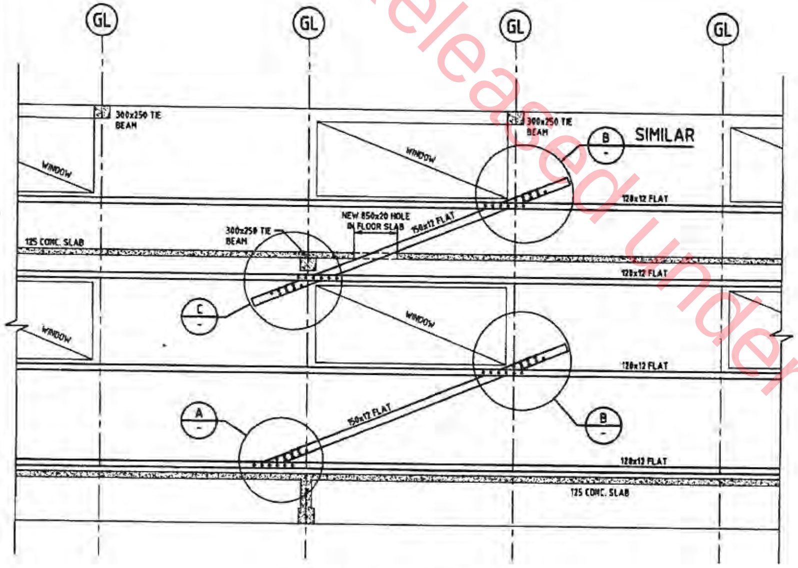
$k_{te} = 1$
$f_u = f_{ur} = 430$ MPa

$0.85 \times 1 \times (150 - 36) \times 12 \times 430 = 500$ kN – Governs.

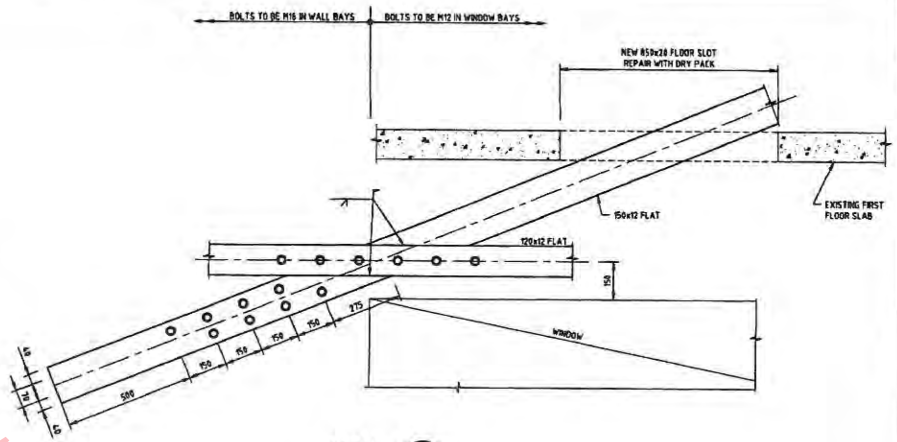
$\therefore$ $\boxed{\text{CAPACITY OF FLAT BRACES } (150 \times 12) = 500 \text{ kN}}$

- Demands on these will be very low, due to the difference in stiffness between these and the concrete walls that they're attached to.

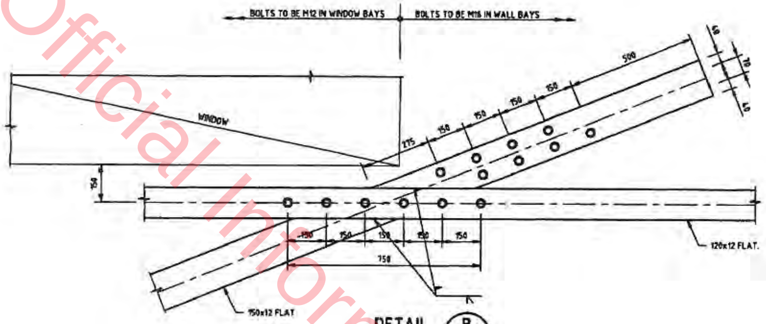
- By inspection – O.K.



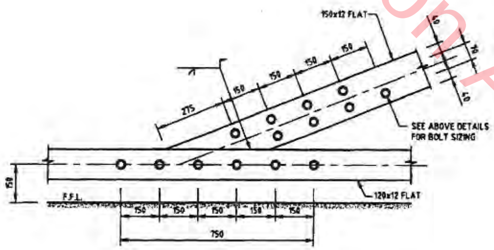
PART WALL B ELEVATION
SCALE 1:50



DETAIL C
SCALE 1:10



DETAIL B
SCALE 1:10



DETAIL A
SCALE 1:10

PRELIMINARY

ALL DIMENSIONS TO BE CONFIRMED ON SITE

NOTE:

BOLTS SHOWN IN THESE DETAILS ARE IN ADDITION TO THOSE IN SECTIONS 8 AND 9 ON DRAWING 5944-S05.

BOLTS TO BE IN APPROPRIATE 'HILTI' HIS-N THREADED SLEEVES. FIXED WITH 'HILTI' RE-500 INJECTION ADHESIVE.

SHEAR DESIGN VALUES FOR ANCHORS: M16 = 50 kN, M12 = 30 kN

ALL STRUCTURAL STEEL PLATE TO BE GRADE 300

Rev	Date	Revision Details	By	Ver.	App.
2	24/10/03	ISSUE FOR BUILDING CONSENT	GKS		
01	18.07.03	ISSUE FOR INFORMATION	GKS		

Connell Mott MacDonald ...building the future

Connell Wagner Limited
Maxwell House, 3 Hunter Street (PO Box 1591)
Wellington New Zealand

Client:

Project:
WELLINGTON EAST GIRLS COLLEGE
EAST WING BLOCK 7
STRUCTURAL STRENGTHENING

Drawing Title:
WALL STRENGTHENING
SHEET 2

WELLINGTON CITY COUNCIL
24 OCT 2003
WAKEFIELD ST

Drawn	Signed	Date	Verified	Signed	Date
JK		24/10/03			
Designed	Signed	Date	Approved	Signed	Date
GKS		24/10/03			

CW Project No: 7970
Scale: 1:10 (A1) 1:20(A3)
Drawing No: S06
Revision: 2

A1

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CALCULATION SHEET

S/124

Project/Task/File No: 5-PA010.37 | Sheet No of

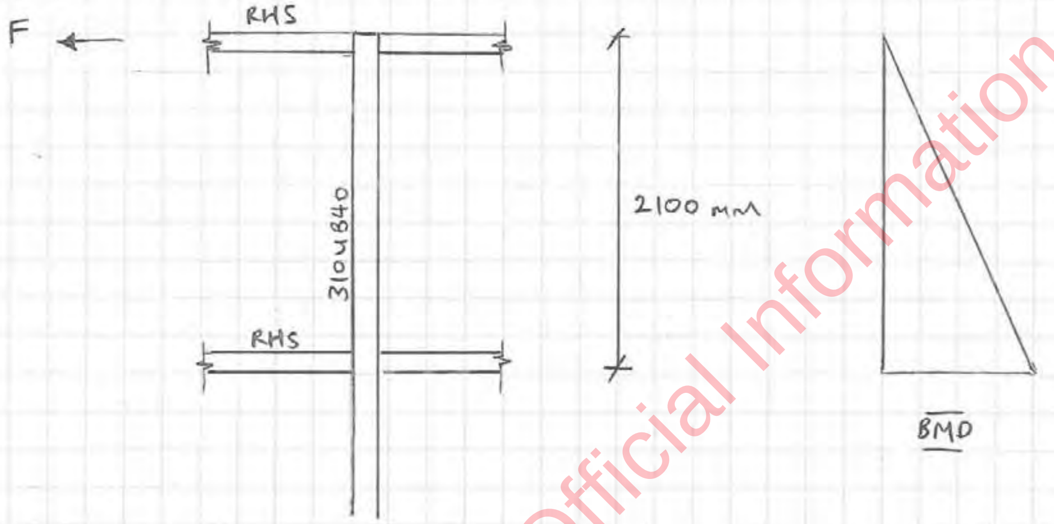
Project Description: WEGC East Wing DSA | Office: Wgtn

Computed: MJG / 11/ 11 /2015

Check: / /

2005 Addition – Check of columns in minor axis bending, transferring load from upper roof to lower roof

Step in Roof



Total 5 310UB40 columns, tied at upper and lower roof levels.

Minor axis bending capacity

$\phi = 1$ (Assessment)

$f_y = 320\,MPa$

$Z_{ey} = 139 \times 10^3\ mm^3$

$\phi M_{sy} = \phi f_y Z_{ey} = 44.5\,kNm$

Force reqd. to yield columns $= \frac{44.5}{2.1} \times 5 = 106\,kN$

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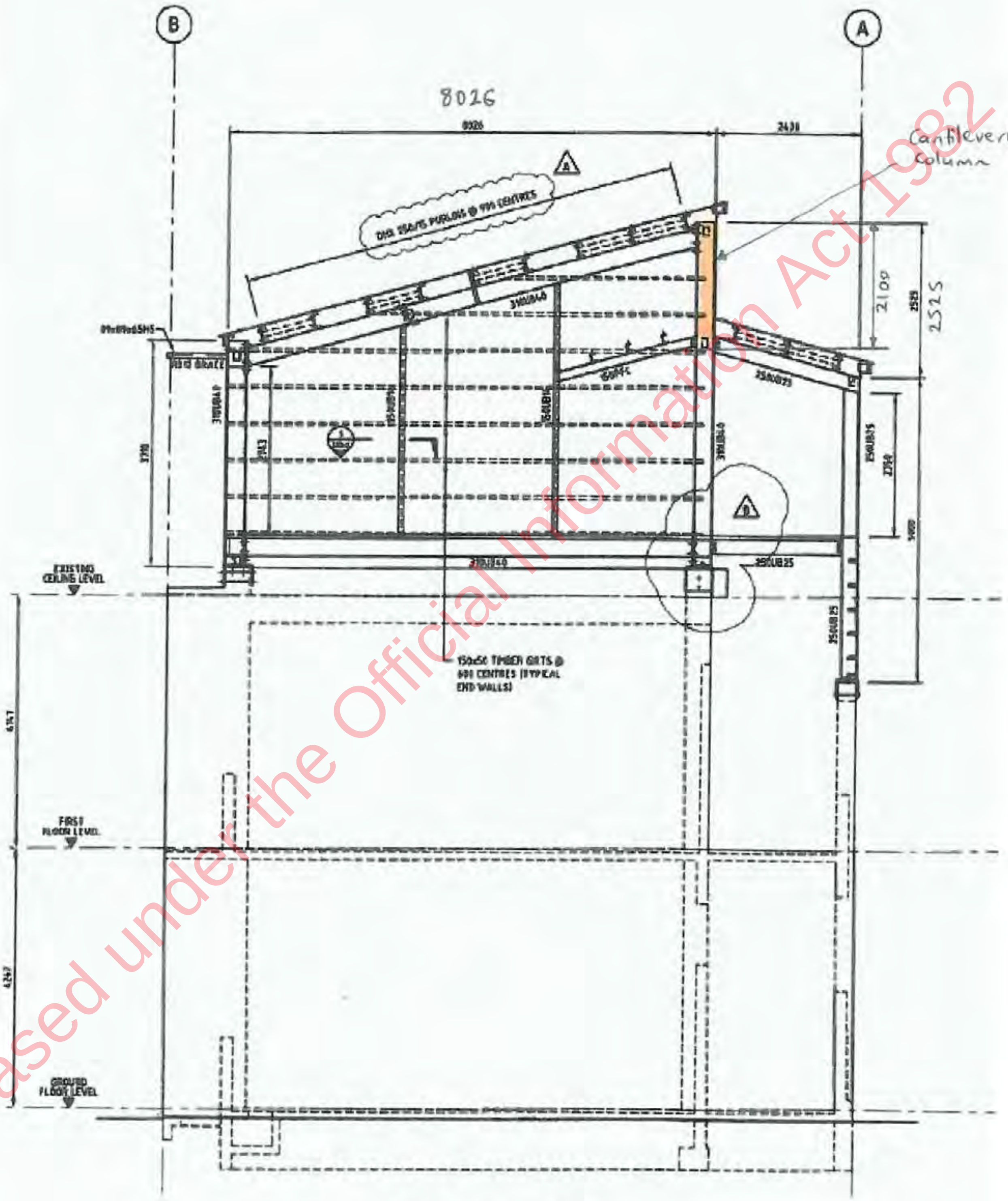
S-PA010.37
WEGC East Wing DSA

Sheet 5/125



END ELEVATION (WEST)

(VIEWED FROM OUTSIDE - PORTAL SETOUT)

SECTION 5
SCALE 1:50 S01 S02

CALCULATION SHEET S/126

Project/Task/File No: S-PA010.37 Sheet No of

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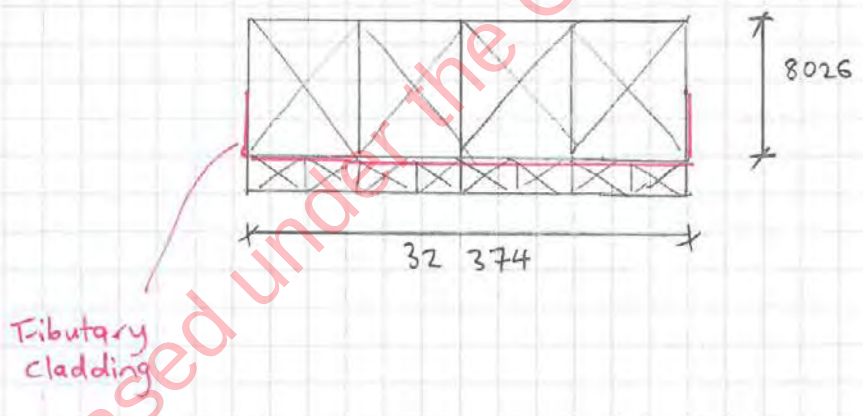
Demand

Assess to NZS1170.5, Section 8 - Steel framed on existing concrete structure.

Roof

Cladding	0.1 kPa
DHS 250/15 purlins @ 900 c/s	0.06 kPa
$\frac{0.056\ kN/m}{0.9m}$	
Steelwork	0.1 kPa
Ceiling + services	0.2 kPa
	0.45 kPa



Roof trib area $= 32.374 \times \frac{8.026}{2} = 130\,m^2$

Roof weight $= 0.45 \times 130 = 58.5\,kN$

Cladding

Allow 0.5kPa (Lightweight timber / glazing)

Partitions are self supporting.

Cladding trib area $= (8.026 + 32.374) \times \frac{2.1}{2} = 42\,m^2$

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CALCULATION SHEET

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Project Description: | Office:
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Cladding weight = 0.5 × 42 = 21 kPa

Demand by parts

$c(0) = 0.52$ (Sheet S/84) – $R_u = 1.3$, $Z = 0.4$, $C_h(0) = 1$

$C_{Hi} = 3$

$C_i(T_p) = 2$

$C_p(T_p) = 0.52 \times 3 \times 2 = 3.12$

$W_p = 58.5 + 21 = 79.5$ kN

Elastic demand $= 3.12 \times 79.5 = 248$ kN

Minor axis bending – allow $\mu = 2$

Demand $= 0.55 \; (C_{ph}) \times 248 = 136$ kN

$\%NBS = \frac{106}{136} = 78\%$ NBS

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CALCULATION SHEET S/128

Project/Task/File No: WEEC | Sheet No ... of ...

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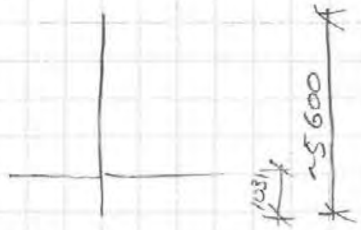
2^{nd} FLOOR DIAPHRAGM CONNECTION

- Relies on column bending to transfer force to structure below:



310 UB 40

$\phi M_{sy} = 138 \times 320 = 44.1$ kNm

$\phi M_{sx} = 629 \times 320 = 201$ kNm

$M^* = \dfrac{Pab}{l}$

$\therefore P_y = \dfrac{M^* l}{ab} = \dfrac{44.1 \times 5.6}{1.03 \times 4.57} = 52$ kN

$P_x = 239$ kN

Number of columns = 5

$\therefore F_y = 5 \times 52 = 260$ kN

$5 \times 239 = 1195$ kN

Weight of floor

$(0.9 + 0.8) \times 32.37 \times 11.379 = 265$ kN

Seismic coefficient = 2, say.

Half of load on this line

$\therefore$ should stay elastic. O.K.

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Appendix B

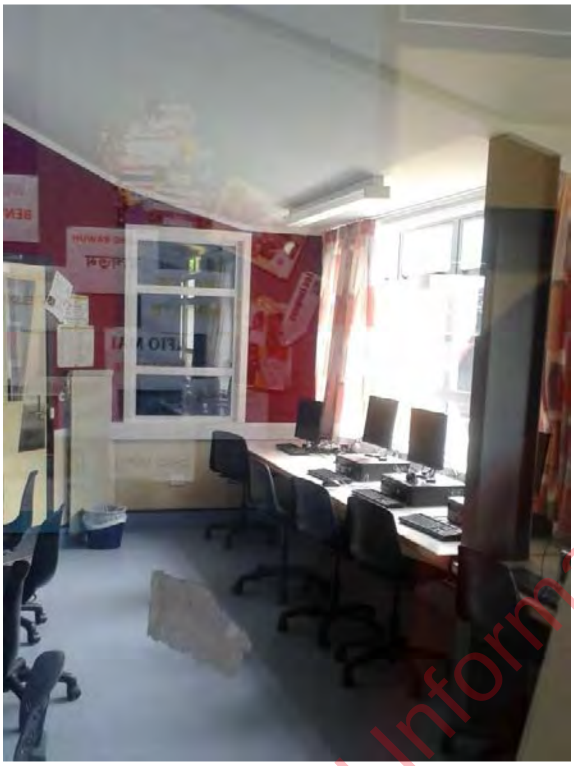
Photos of Building

North Elevation

West Elevation

South Elevation and Bridge

Front Elevation Braced Frame



2nd Floor Classroom

1st Floor Corridor

Appendix C

Plans of Building