BC180994

190 High Street
Review of Seismic Strengthening
December 2018
22815

SILVESTER/CLARK
CONSULTING ENGINEERS

BUILDING CONSENT

GRANTED
16/08/2019

HUTT CITY COUNCIL

HUTT CITY
TE AWA KAIRANGI

STRUCTURAL CALCULATIONS

FOR

SEISMIC STRENGTHENING AT UPPER FLOOR LEVEL

AT

190 HIGH STREET, LOWER HUTT

December 2018

Silvester Clark Ltd
Consulting Engineers
153 Thorndon Quay
Wellington

BC180994

ENGINEERING NEW ZEALAND OFFICIAL MEMBER | NEW ZEALAND INSTITUTE OF ARCHITECTS INCORPORATED | ACENZ

SILVESTER/CLARK CONSULTING ENGINEERS

Building Code Clause(s) B1

PRODUCER STATEMENT – PS1 – DESIGN

(Guidance notes on the use of this form are printed on page 2)

BUILDING CONSENT GRANTED 16/08/2019 HUTT CITY COUNCIL

ISSUED BY: SILVESTER CLARK LIMITED (Design Firm)

TO: Y Huang, X Zhao & JN Trustees Limited (Owner/Developer)

TO BE SUPPLIED TO: Hutt City Council (Building Consent Authority)

IN RESPECT OF: Seismic Strengthening at upper floor level (Description of Building Work)

AT: 190 High Street, Lower Hutt (Address)

LOT DP SO

We have been engaged by the owner/developer referred to above to provide Review of Seismic Strenghtening (Extent of Engagement) services in respect of the requirements of clause(s) B1 of the Building Code for All ☐ or Part only ☒ (as specified in the attachment to this statement), of the proposed building work.

The design carried out by us has been prepared in accordance with:

☒ Compliance Documents issued by the Ministry of Business, Innovation & Employment B1/VM1 (verification method / acceptable solution) or

☐ Alternative solution as per the attached schedule

The proposed building work covered by this producer statement is described on the drawings titled 22815 and numbered SK1, SK2, SK3, SK4; together with the specification, and other documents set out in the schedule attached to this statement.

On behalf of the Design Firm, and subject to:

(i) Site verification of the following design assumptions: N/A,

(ii) All proprietary products meeting their performance specification requirements;

I believe on reasonable grounds that a) the building, if constructed in accordance with the drawings, specifications, and other documents provided or listed in the attached schedule, will comply with the relevant provisions of the Building Code and that b), the persons who have undertaken the design have the necessary competency to do so. I also recommend the following level of construction monitoring/observation:

☐CM1 ☒CM2 ☐CM3 ☐CM4 ☐CM5 (Engineering Categories)

I, Ignatius Black (Name of Design Professional) am: ☒CPEng 259219 #

I am a Member of: IPENZ and hold the following qualifications: MIPENZ (Structural and Civil), CPEng.

The Design Firm issuing this statement holds a current policy of Professional Indemnity Insurance no less than $200,000*. The Design Firm is a member of ACENZ.

SIGNED BY Ignatius Black ON BEHALF OF SILVESTER CLARK LIMITED (Design Firm)

Date 14/12/2018 (signature) I.P.M

Note: This statement shall only be relied upon by the Building Consent Authority named above. Liability under this statement accrues to the Design Firm only. The total maximum amount of damages payable arising from this statement and all other statements provided to the Building Consent Authority in relation to this building work, whether in contract, tort or otherwise (including negligence), is limited to the sum of $200,000.*

This form is to accompany **Form 2 of the Building (Forms) Regulations 2004** for the application of a Building Consent.

CONTENTS

PREAMBLE

These calculations are for seismic strengthening to 93%NBS.

This building was previously strengthened in 2009 using the NZSEE 2006 Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. We have reviewed this assessment using the July 2017 MBIE document "The Seismic Assessment of Existing Building" Technical Guidelines. This review identified that the ductility assumed for the strengthening works were not inline with the MBIE Technical Guideline.

The weakest link in the 2009 strengthening work was the capacity of the cantilevered concrete columns, that were strengthened, at the outside wall lines.

Codes Used

The following design codes have been used:

- Design loads to NZS 1170.0, .1, .2 and .5
- Concrete Design to NZS 3101
- Steel Design to NZS 3404

Dead Loads

Gravity dead loads have been calculated using generally accepted material weights.

Seismic Design Parameters

The seismic design coefficients have been calculated using NZS 1170.5 with the following parameters used:

• Importance Level (IL)	IL2
• Zone Factor (Z)	0.4
• Site Subsoil Class	D
• Part Category	P3
• Seismic Coefficient (C)	**1.11**

Strengthening works

These strengthening works involve further increasing the capacity at upper level in the transverse direction. This involves augmenting the 2009 strengthening with the further addition of steel members that have a moment connection to the steel roof beam. Thus, creating a frame at floor level to brace the roof structure. The new length of steel members is welded to both the existing steel beam and the 2009 steel vertical members that was connected to the concrete columns. There is also some strengthening required to the existing steel roof beam where the new steel members connects to create a moment knee joint.

After these works are completed, we consider that this building will achieve in excess of 93%NBS. The weakest part of the system is the connection at of the 2009 steel member at the base. However, if this connection was to fail the frame that has been created across the building by these works will create a higher level of resilience meaning collapse type failures **will not occur** at 93%NBS.

CALCULATIONS

BC180994

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PAGE No 1.1 JOB No 22 815

BY PDN DATE 11/12/2018

JOB 190 High St – Review of Seismic Strengthening

WEIGHT AT ROOF LEVEL

– ROOF	12.2 m × 0.4 kPa	= 4.9
– PARAPETS	2 × (1.2 m × 0.15 m) × 24.5 kN/m³	= 8.8
– BAND BEAM	2 × (0.3 m × 0.3 m) × 24.5 kN/m³	= 4.4
– 1/2 BRICK WALLS	2 × ½ × 2 m × 0.105 m × 2 × 18 kN/m³	= 7.6

(2 walls; wall height; 2 layers)

~ 26 kN/m

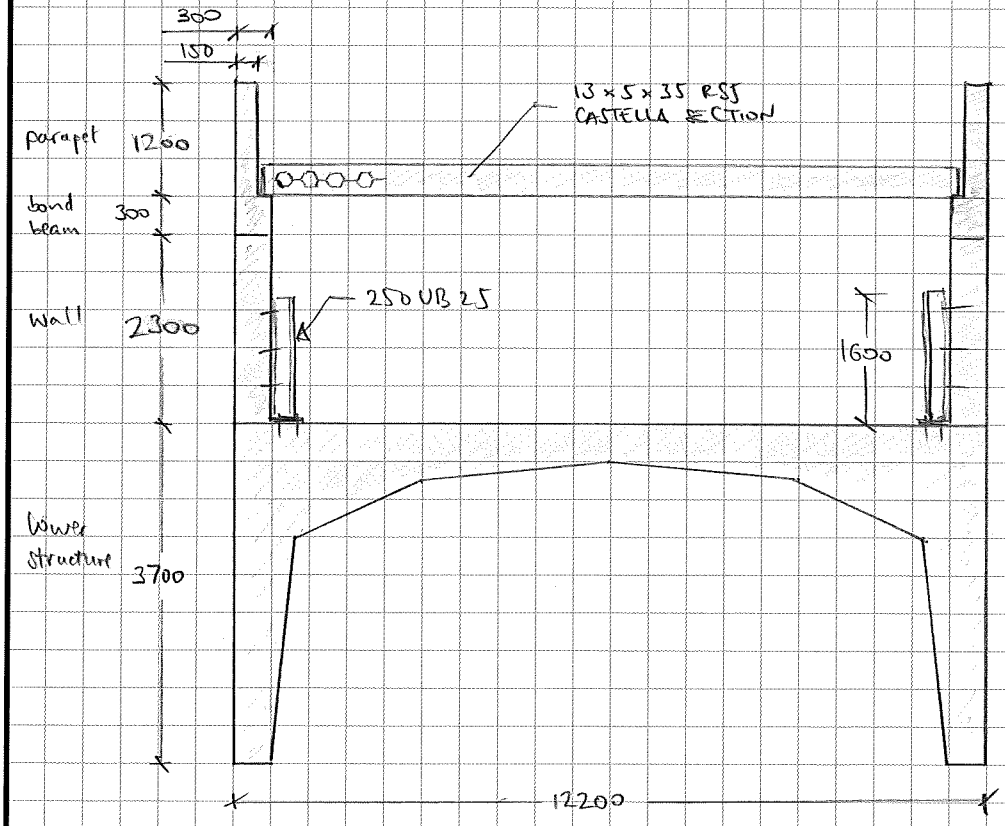
PART COEFFICIENT

$C_p(T_p) = 1.11$ (Sec 1.2)

LATERAL LOADS (Seismic)

$F_{ph} = 1.11 \times 26\ kN/m \times 5.5\ m = 157\ kN$ (5.5 m = bay width)

→ each column will take $F_{ph}/2 = 94\ kN$



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PAGE No.	1.2	JOB No	22815
BY	PDN	DATE	5/12/2018
JOB	190 High st Lower Hutt		

Design Response Coefficient for Parts to NZS1170.5 Section 8

This spreadsheet calculates the design response coefficient for parts ($C_p(T_p)$) as per NZS 1170.5 section 8.2.

Site hazard coeficient

Site subsoil class	D	
$C_h(T)$=	1.12	g, from NZS1170.5:2004, table 3.1
Z=	0.4	from NZS1170.5, table 3.3
Importance Level, IL=	IL2	
R_u=	1.00	refer to table 3.5 in NZS1170.5
N(T,D)=	1.0	for T less than 1.5 seconds
C(0)=	0.45	

8.3 Floor Height Coefficient

h_i=	3.7	m, height of level part is on
h_n=	7	m, height of building
C_{hi}=	1.62	

8.4 Part Spectra Shape Coefficient

T_p=	0.37	s
$C_i(T_p)$=	2	

8.2 Design Response Coefficient for Parts

$C_p(T_p)= C(0)C_{Hi}C_i(T_p)$ .....eq 8.2(1)

$C_p(T_p)$=	1.45

8.5 Design Actions in Parts

μ_p=	1.25	For P5 & P6 use μ_p=1.0
C_{ph}=	0.85	
Part Category=	P3	
R_p=	0.90	from table 8.1
W_p=	0	kN or kPa- optional

$F_{ph}= C_p(T_p)C_{ph}R_pW_p \leq 3.6W_p$ ....eq8.5(1)

F_{ph}=	1.11	x W_p
or F_{ph}=	0	kN or kPa

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Page # 1.3
Job # 22815

TABLE NO. 3.9 PROPERTIES OF BEAMS TO BRITISH STANDARD 4. 1932

IMPERIAL UNITS See separate page for notes

Ref No.	Size D x B	Approximate Mass/ft	Metric Equivalent D x B	Metric Equivalent Mass/m	Obtain Properties from table no:-
	ins	lbs	mm	kg	
BSB 101	3 x1.50	4	76x38	6	3.7 or 3.8
BSB 102	3 x3	8.5	76x76	13	3.7
BSB 103	4 x1.75	5	102x44	7	3.7 or 3.8
BSB 104	4 x3	10	102x76	15	3.8
BSB 105	4.75x1.75	6.5	121x44	10	3.7
BSB 106	5 x3	11	127x76	16	3.7
BSB 107	5 x4.50	20	127x114	30	3.8
BSB 108	6 x3	12	152x76	18	3.7 or 3.8
BSB 109	6 x4.50	20	152x114	30	3.7
BSB 110	6 x5	25	152x127	37	3.7 or 3.8
BSB 111	7 x4	16	178x102	24	3.7
BSB 112	8 x4	18	203x102	27	3.7 or 3.8
BSB 113	8 x5	28	203x127	42	3.7
BSB 114	8 x6	35	203x152	52	3.7 or 3.8
BSB 115	9 x4	21	229x102	31	3.7 or 3.8
BSB 116	9 x7	50	229x178	74	3.8
BSB 117	10 x4.50	25	254x114	37	3.8
BSB 118	10 x5	30	254x127	45	3.7
BSB 119	10 x6	40	254x152	60	3.8
BSB 120	10 x8	55	254x203	82	3.8
BSB 121	12 x5	32	305x127	48	3.7
BSB 122	12 x6	44	305x152	66	3.7
BSB 123	12 x6	54	305x152	69	3.7
BSB 124	12 x8	65	305x203	97	3.8
BSB 125	13 x5	35	330x127	52	3.8
BSB 126	14 x6	46	356x152	69	3.7
BSB 127	14 x6	57	356x152	85	3.7
BSB 128	14 x8	70	356x203	104	3.8
BSB 129	15 x5	42	381x127	63	3.7
BSB 130	15 x6	45	381x152	67	3.8
BSB 131	16 x6	50	406x152	74	3.8
BSB 132	16 x6	62	406x152	92	3.7
BSB 133	16 x8	75	406x203	112	3.8
BSB 134	18 x6	55	457x152	82	3.8
BSB 135	18 x7	75	457x178	112	3.7
BSB 136	18 x8	80	457x203	119	3.8
BSB 137	20 x6.50	65	508x165	97	3.8
BSB 138	20 x7.50	89	508x191	132	3.7
BSB 139	22 x7	75	559x178	112	3.8
BSB 140	24 x7.50	95	610x191	141	3.10

Notes relative to Tables 3.7 to 3.10 inclusive

These tables list most of the properties of British Standard Beams over the period 1903 to 1959 when these sections were replaced by Universal Beam and Column sections.

Three separate issues of British Standard 4 are relevant viz:-

BS4 1903

British Standard Beams with reference BSB (later classed as *old* British Standard Beams) shown in Table 3.7.

BS4 1921

British Standard Beams with reference NBSB and British Standard Heavy Beams and Pillars with reference NBSHB (commonly classed as *new* British Standard Beams) shown in Table 3.8.

BS4 1932

British Standard Beams with reference BSB101 to BSB140. This consisted of a list of sizes taken from both the 1903 and 1921 ranges with *one* new section only, namely BSB140 – 24 inches x 7½ inches x 95lbs/ft.

These are shown in Table 3.9 and rather than repeat section properties reference is made to the previous tables. However, since the section reference BSB140 is new the properties are given in Table 3.10.

British Standard 4, 1903, 1921 and 1932 were of course issued before the adoption of the metric system in the United Kingdom and all properties were provided in Imperial units.

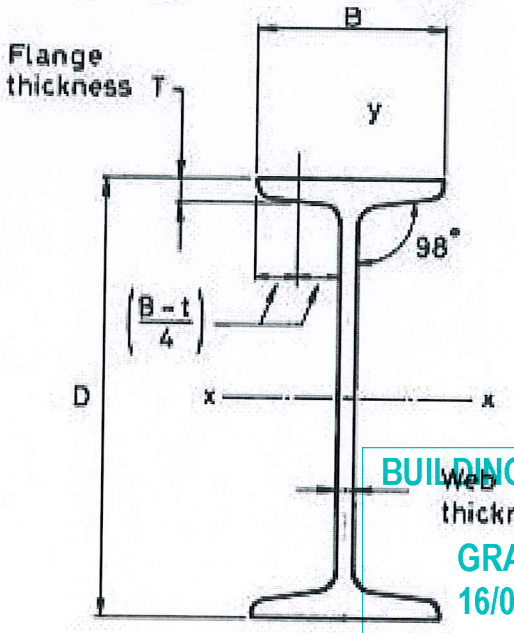

Figure 3.10 British Standard Beam.

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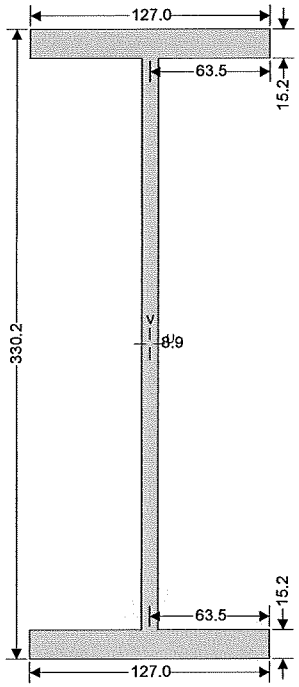
	Calcs for				Start page no./Revision 1.4	
	Calcs by P	Calcs date 11/12/2018	Checked by	Checked date	Approved by	Approved date



13 x 5 x 35 RSJ

CALCULATION OF SECTION PROPERTIES

Tedds calculation version 2.0.04

Area

A = **6535.47** mm^2

2nd moment of area

I_u = **116.×10⁶** mm^4 I_v = **5.22×10⁶** mm^4 I_x = **116.×10⁶** mm^4 I_y = **5.22×10⁶** mm^4

Radius of gyration

r_u = **133.2** mm r_v = **28.3** mm r_x = **133.2** mm r_y = **28.3** mm

Plastic section modulus (only shapes with all rectangles at 90 degs)

S_x = **809.×10³** mm^3 S_y = **129.×10³** mm^3

Distance to combined centroid

X_e = **0.0** mm Y_e = **0.0** mm

Distance to equal axis area (only shapes with all rectangles at 90 degs)

X_p = **0.0** mm Y_p = **0.0** mm

Elastic section modulus

Z_x = **703.×10³** mm^3 Z_y = **82.2×10³** mm^3

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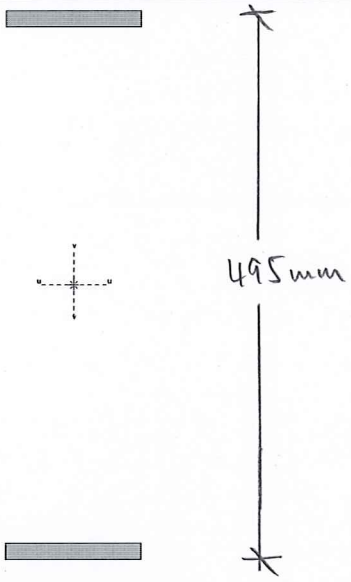
	Calcs for				Start page no./Revision 1.5	
	Calcs by P	Calcs date 11/12/2018	Checked by	Checked date	Approved by	Approved date



CALCULATION OF SECTION PROPERTIES

Tedds calculation version 2.0.04

Area

$A = 3860.80\ mm^2$

2nd moment of area

$I_u = 237.\times10^6\ mm^4$ $I_v = 5.19\times10^6\ mm^4$ $I_x = 237.\times10^6\ mm^4$ $I_y = 5.19\times10^6\ mm^4$

Radius of gyration

$r_u = 247.5\ mm$ $r_v = 36.7\ mm$ $r_x = 247.5\ mm$ $r_y = 36.7\ mm$

Plastic section modulus (only shapes with all rectangles at 90 degs)

$S_x = 956.\times10^3\ mm^3$ $S_y = 123.\times10^3\ mm^3$

Distance to combined centroid

$X_e = 0.0\ mm$ $Y_e = 0.0\ mm$

Distance to equal axis area (only shapes with all rectangles at 90 degs)

$X_p = 0.0\ mm$ $Y_p = 0.0\ mm$

Elastic section modulus

$Z_x = 927.\times10^3\ mm^3$ $Z_y = 81.7\times10^3\ mm^3$

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Characteristic/Nominal Strengths

C6B.1 United Kingdom

The characteristic (lower)/nominal material properties of historical UK steelwork are given in Tables C6B.1 to C6B.4. Geometric properties of UK sections can be obtained from publications such as that by Bates (1991).

Table C6B.1: Characteristic/nominal properties of mild structural steels from the UK (Bates, 1991 and Bussell, 1997)

Period	Plate thickness (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
<1906	All	-*	432	-
1906-48*	All	-*	432	0.2
1948-68	≤19	247	432	0.16
	t >19	232	432	0.16

Note:
*A nominal yield strength of 210 MPa may be used for steel manufactured before 1948 in the UK.

Table C6B.2: Characteristic/nominal properties of mild structural steels from the UK manufactured to BS 4360:1968 (1968-86)

Grade	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
40 A, B & C	$t \leq 16$	232	402	0.22
	$16 < t \leq 38$	224	402	0.22
40 D & E	$t \leq 16$	263	402	0.22
	$16 < t \leq 38$	247	402	0.22
43 A, B & C	$t \leq 16$	247	432	0.20
	$16 < t \leq 38$	239	432	0.20
43 D & E	$t \leq 16$	278	432	0.20
	$16 < t \leq 38$	270	432	0.20

Table C6B.3: Characteristic/nominal properties of high tensile structural steels from the UK (Bussell, 1997 and Bates, 1991)

Period	Plate thickness, t (mm)	Yield strength (MPa)	Tensile strength (MPa)	Ultimate strain (mm/mm)
1934-65 (BS 548)	$t \leq 32$	355	571	0.14
1943-62	$t \leq 19$	324	541	0.14
(BS 968)	$t > 19$	293	510	0.14
1962-68	$t \leq 16$	355	494	0.15
(BS 968)	$16 < t \leq 32$	340	494	0.15

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JOB

Capacity of castellated 13 x 5 x 35 RSJ

The beam is laterally restrained at 1200 mm crs by the roof purlins.

f_y = 247 MPa (see 1.6) $\phi = 1$

$S_x = 956 \cdot 10^3$ mm^3

$\phi M = 1.0 \times f_y \times S_x =$

$= 1.0 \times 247$ N/mm^2 $\times 956 \cdot 10^3$ mm^3 =

$= 236$ kNm

Capacity of 250UB25 (column)

This member is restrained to the concrete column by anchors @ 300 mm crs

f_y = 320 MPa $\phi = 1$

$S_x = 319 \cdot 10^3$ mm^3

$\phi M = 1.0 \times f_y \times S_x =$

$= 102$ kNm

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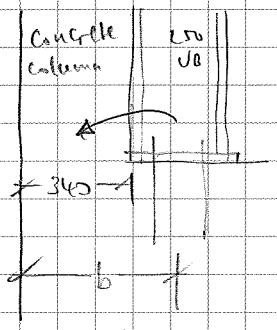
JOB

Capacity of column to floor connection



Lever arm

$b = 340 + 250/2 = 465$ mm

$T = 4 \times 60$ kN $= 240$ kN

$M = b \times T = 112$ kNm

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BY PDN DATE 11/12/2018

JOB

Create a frame model assuming the existing 250UB and 13x5 RSJ are connected thus making a continuous frame.

Model details presented at 1.10 to 1.12

Model results are presented below, against calculated capacity

	acting load	actual capacity	% NBS
FLOOR FIXING	120 kNm	112 kNm	93%
COLUMN (high level)	92 kNm	102 kNm	>100%
RAFTER	92 kNm	236 kN	>100%

Therefore it is sufficient to provide structural continuity amongst existing members to

ACHIEVE 93% NBS CAPACITY

Refer to SK1 & SK2.

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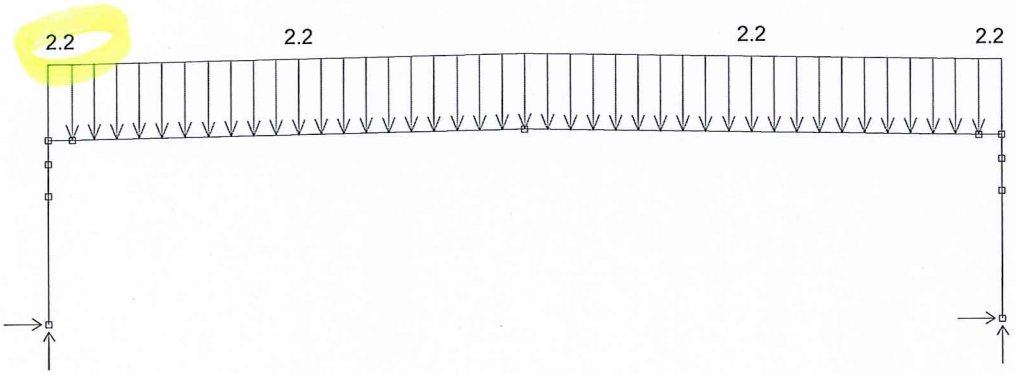
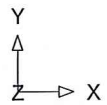
10:55:54 a.

Load Cases:
1 P G



Roof load
0.4 kPa x 5.5 m = 2.2 kN/m
↑
bay width



theta: 270 phi: 0

Microstran V9

pietro
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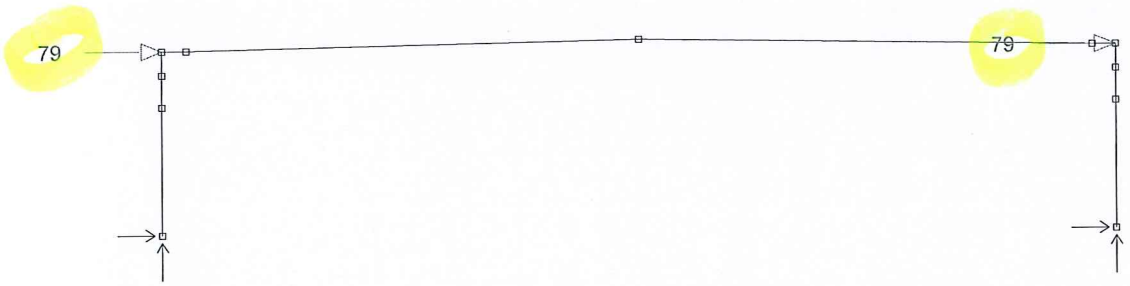
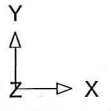
Load Cases:
2 P Eq



EQ load

Fh/2 = 157 kN / 2 = 79 kN



theta: 270 phi: 0

Microstran V9

pietro
Job: frame2
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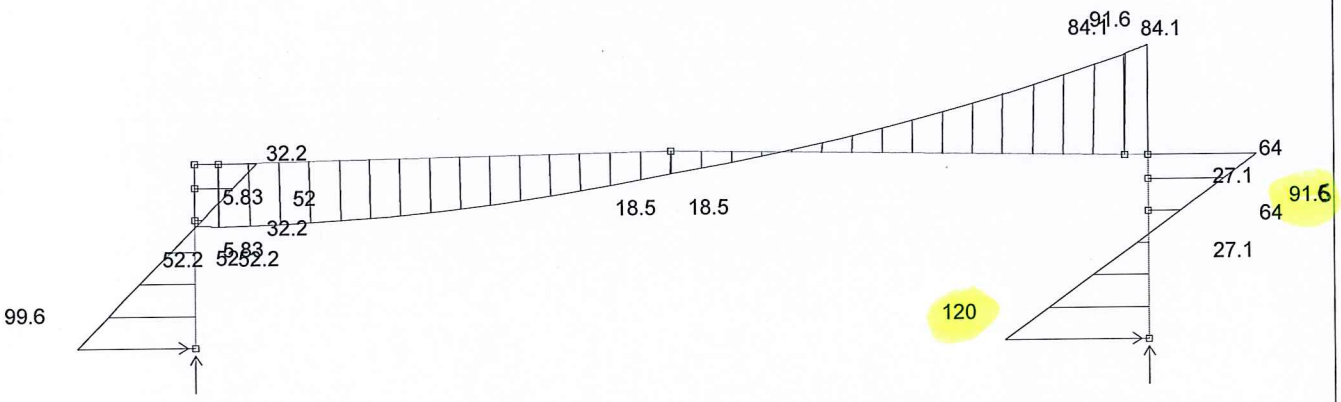
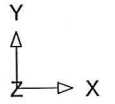
Load Cases:
3 C G+Eq





theta: 270 phi: 0

Bending Moment, Mz

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PAGE No

JOB No 2285

BY JCG

DATE 23 APRIL 2019

JOB 190 HIGH St.

EXISTING INFILL WALLS ARE 11" CAVITY BRICK

WALLS APPEAR TO BE IN GOOD CONDITION

ASSESSMENT OF INFILL BRICK WALLS (OUT-OF-PLANE)

REVISED
10/06/2019
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SUPPLEMENTARY CALC'S

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PAGE No 1 JOB No 22815

BY JBG DATE 23 APRIL 2019

JOB 190 HIGH ST

DETERMINE OUT-OF-PLANE ULTIMATE STRENGTH OF INTACT WALL (11" CAVITY BRICK WITH TIMBER FRAME) FROM NZSEE GUIDELINES C7.6.1 TAKE LOAD WALL (CANTILEVER)

$$q_{PLOT} = 780\gamma\left(f'_m\right)^{0.75}\left(t^2\left(\frac{\alpha_{ARCH}}{L_{INF}^{2.5}} + \frac{\beta_{ARCH}}{h_{INF}^{2.5}}\right)\right)$$

$f'_m = 10.6$ MPa

$h_{INF} = 6'-6"$ OR 2m

$t_1 = 110$ — CONSERVATIVELY IGNORE INTERNAL LINING

$L_{INF} = 18'$ OR 5.5m

$$\alpha_{ARCH} = \frac{1}{3000}\left[\left(23500\right)\left(1985\times10^6\right)\left(2000\right)\right]^{0.25} = 18$$

$\alpha_{ARCH} = 250$ (356mm)

$E_C = 4700\sqrt{25} = 23500$

$$I_{bc} = \frac{356(406)^3}{12} = 1985\times10^6 \text{ mm}$$

$$I_{bb} = \frac{279(305)^3}{12} + \frac{305(457)^3}{12} = 279\text{mm}$$

1'-2" 1'-4" 406mm 18" OR 457mm 12" OR 305mm 11"

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$$I_{gb} = \frac{660 \times 10^6 + 2426 \times 10^6}{2}$$

$$= 1543 \times 10^6 \text{ mm}^4$$

$$\beta_{ANCH} = \frac{1}{5500}\left[\left(23500\right)\left(1543 \times 10^6\right)\left(5500\right)^2\right]^{0.25}$$

$$= 33$$

$$\gamma = 1.1\left[1 - \frac{2000}{55\,(110)}\right] = 0.74$$

$$f_{pnb} = 730\,(0.74)\,(10.6)^{0.75}\,(110)^2\left(\frac{50}{5500^{2.5}} + \frac{33}{2000^{2.5}}\right)$$

$$= 7.9 \text{ kPa}$$

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PAGE No 3 JOB No 22815

BY JBB DATE 23 APRIL 2019

JOB 190 HIGH ST.

DEMAND

FROM ATTACHED SPREADSHEET

$F_{ph} = 3 kPa < q_{prob}$

WITH BACK WALL 100% MPES

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PAGE No.		JOB No	22815
BY	JEG	DATE	23-Apr-19
JOB	190 High Street		

Design Response Coefficient for Parts to NZS1170.5 Section 8

This spreadsheet calculates the design response coefficient for parts ($C_p(T_p)$) as per NZS 1170.5 section 8.2.

Site hazard coeficient

Site subsoil class	D	
$C_h(T)$=	1.12	g, from NZS1170.5:2004, table 3.1
Z=	0.4	from NZS1170.5, table 3.3
Importance Level, IL=	IL2	
R_u=	1.00	refer to table 3.5 in NZS1170.5
N(T,D)=	1.0	for T less than 1.5 seconds
C(0)=	0.45	

8.3 Floor Height Coefficient

h_i=	3.7	m, height of level part is on
h_n=	5.9	m, height of building
C_{hi}=	1.62	

8.4 Part Spectra Shape Coefficient

T_p=	0.75	s
$C_i(T_p)$=	2	

8.2 Design Response Coefficient for Parts

$C_p(T_p)$= $C(0)C_{Hi}C_i(T_p)$ .....eq 8.2(1)

$C_p(T_p)$=	1.45

8.5 Design Actions in Parts

μ_p=	1.00	For P5 & P6 use μ_p=1.0
C_{ph}=	1.00	
Part Category=	P4	
R_p=	1.00	from table 8.1
W_p=	2	kN or kPa- optional

$F_{ph} = C_p(T_p)C_{ph}R_pW_p \leq 3.6W_p$ ....eq8.5(1)

or

F_{ph}=	1.45	x W_p
F_{ph}=	3	kN or kPa

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