SEISMIC STRENGTHENING

At

11 JOHNSONVILLE ROAD

For

JUDE BEST

12/12/19

Issued For	Revision	Date	Calcs by	Approved by
Building Consent	A	12/12/19	HN/MS	IB

SILVESTER/CLARK CONSULTING ENGINEERS

Wellington, 153 Thorndon Quay
PO Box 12618, 6144
New Zealand tel: 64 4 473 8361
wgtnoffice@silvesterclark.co.nz

CONTENTS

PREAMBLE

These calculations are to provide a brief description of the project located at 11 Johnsonville Road, Johnsonville. A Detailed Seismic Assessment (DSA) was completed on the property early in 2019. The DSA deemed the building to be Earthquake Prone (EQP); EQP elements included the building not having a roof diaphragm and the concrete frame at the front of the fish'n chip shop. The DSA also noted one additional element below 67% NBS; this was the internal concrete frame to the 1st floor buildings. Outside of these elements the building was rated at 80% New Building Standard (NBS), these calculations provide the design to address the critical elements to increase there strength to 80% NBS.

Once the works are completed the building can be deemed 80% NBS.

CODES USED

The following design codes have been used:

- Design loads to NZS1170.0, .1, .2, .5
- Reinforced concrete design to NZS3101
- Steelwork design to NZS3404

PERMANENT ACTIONS / DEAD LOADS

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

The loads are described on calculation pages 11,12.

IMPOSED ACTIONS / LIVE LOADS

The loads are described on calculation pages 11,12.

SEISMIC DESIGN PARAMETERS

The seismic design coefficients have been calculated using NZS1170.5 with the following parameters used:

- Soil cat = C
- Sp= 0.7
- T1< 0.4
- Z= 0.4
- R= 1.0 for permanent case
- μ= 1.25

BRACING DESIGN

New bracing includes;

- Structural Steel Portal Frame
- Structural Steel X-bracing to wall and roof.

FOUNDATIONS DESIGN

There is a shallow foundation pad designed to support the structural steel portal frame.

CALCULATIONS

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No 01 JOB No 28028

BY HN DATE 05/09/2019

JOB 11 Johnsonville Rd, sdr

TABLE OF CONTENTS

01. Notes from DSA

- Overall % NBS < 34 – (No roof diaphragm in the Bun Bakery)
  – (Concrete frame in the fish & chips shop)

- Concrete frame between two buildings
  < 67% NBS

- Built in 1972 and extended in 1984
- Maintenance works carried out in 1997
- First floor refurbishment in 2000
- IL 2
- Soil class C
- T = 0.4 sec
- μ = 1.25 (Concrete structure)
- μ = 2 – 2.5 (Timber ")
- Z = 0.4

- Material properties

  Concrete = 37.5 MPa

  Steel D = 270 MPa

  HD = 380 MPa.

Assessed Seismic Rating
The results of the DSA indicate the building's seismic rating to be **<34% NBS** assessed in accordance with the guideline document *The Seismic Assessment of Existing Buildings – Technical Guidelines for Engineering Assessments*, dated July 2017.
The seismic rating assumes that Importance Level 2, (IL2),in accordance with the Joint Australian/New Zealand Standard – Structural Design Actions Part 0, AS/NZS1170.0:2002, is appropriate.

Therefore, this is a Grade D Building, following the NZSEE grading scheme.
Grade D buildings represent a risk to occupants *10-25 times greater* that expected for a new building, indicating a *High risk* exposure.

A building with a seismic rating less than 34% NBS is considered to be an Earthquake-Prone Building (EPB) in terms of the Building (Earthquake-prone Buildings) Amendment Act 2016 and a building rating less than 67% NBS as an Earthquake Risk Building (ERB) by the New Zealand Society for Earthquake Engineering.

The existing structure at *11 johnsonville Road*
is *therefore categorised as an earthquake prone building*

DEADLINE FOR COMPLETING SEISMIC WORK

As the building is in an area of High Seismic risk and is not a priority building, it therefore requires that seismic work is completed within 15 years.

STRUCTURAL WEAKNESSES

The assessment identified the following structural weaknesses in the building:
(Refer to the building layout in Appendix C for gridline references)

	Element	Percentage NBS
1	In the transverse direction_ (Southern Building/Bun Bakery)	
	Ground floor beams	81% NBS flexural 100% NBS shear
	Ground floor concrete columns	
	Exterior	100% NBS both flexure and shear
	Interior	100% NBS both flexure and shear
	Centre	97% NBS flexure 100% NBS shear
	First floor cantilevered concrete columns	
	Exterior	81% NBS flexure 100% NBS shear
	Interior	100% NBS flexure 100% NBS shear
	Concrete frame in the front	100% NBS both flexure and shear
	In the transverse direction_ (Northern Building/Fish and Chips)	
	Timber framed walls	100% NBS shear
	Concrete frame in the front	
	Concrete beam	<34% NBS shear (open stirrups)
	Column	53%NBS flexure

	Foundation pad (525mmx525mmx250mm)	71%NBS axial
2	In the longitudinal direction (Southern Building/Bun Bakery) Along gridline A Reinforced masonry walls Reinforced masonry walls Along gridline E Concrete roof beams First floor columns (300x300) First floor columns (400x400) In the longitudinal direction (Northern Building/Fish and Chips) Along gridline G Timber framed walls	 100% NBS in-plane shear 100% NBS out-of-plane shear 83% NBS flexure 62% NBS shear 60% NBS flexure 100% NBS shear 100% NBS flexure 100% NBS shear 100% NBS shear
3	Roof diaphragm Southern building Northern building	 <34% NBS tension 100%NBS
4	Floor diaphragm Southern building Northern building	 100% NBS (localised investigation is required to confirm floor supports) >67%NBS (localised investigation is required to confirm floorboard)
5	Connections between two buildings At roof At floor	 100% NBS shear and pull out 100% NBS shear and pull out

Localised investigation is required for the highlighted elements before the final report.

The assessment has not identified severe structural weaknesses (SSWs) in the building.
No secondary structural and non-structural aspects were considered in the assessment of the seismic rating:

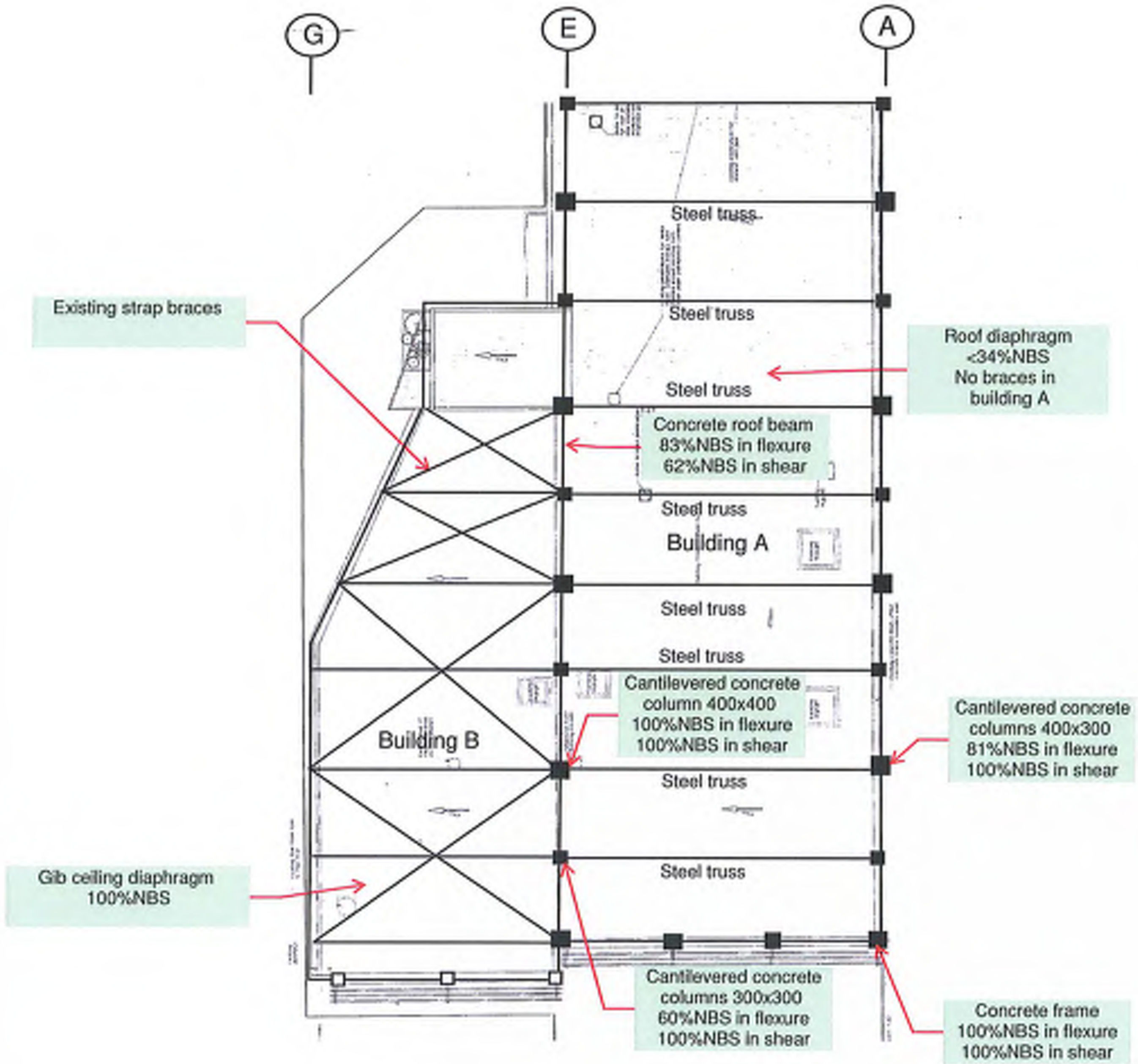
BASIS FOR THE ASSESSMENT

This assessment has been based on the following information:

- Building drawings from City Engineers dated at 1955
- Building drawings by Roberts and Mercer Registered Architect dated at October 1971 and September 1977
- Building drawings from Valley Construction Ltd dated 1983
- Building drawings by Kathrine Gebbie Registered Architect dated at September 1997
- Building drawings from TSE Group Ltd dated at July 2001
- Building inspection and photographs on 18th March 2019
- Building photographs

05

%NBS ACHIEVED

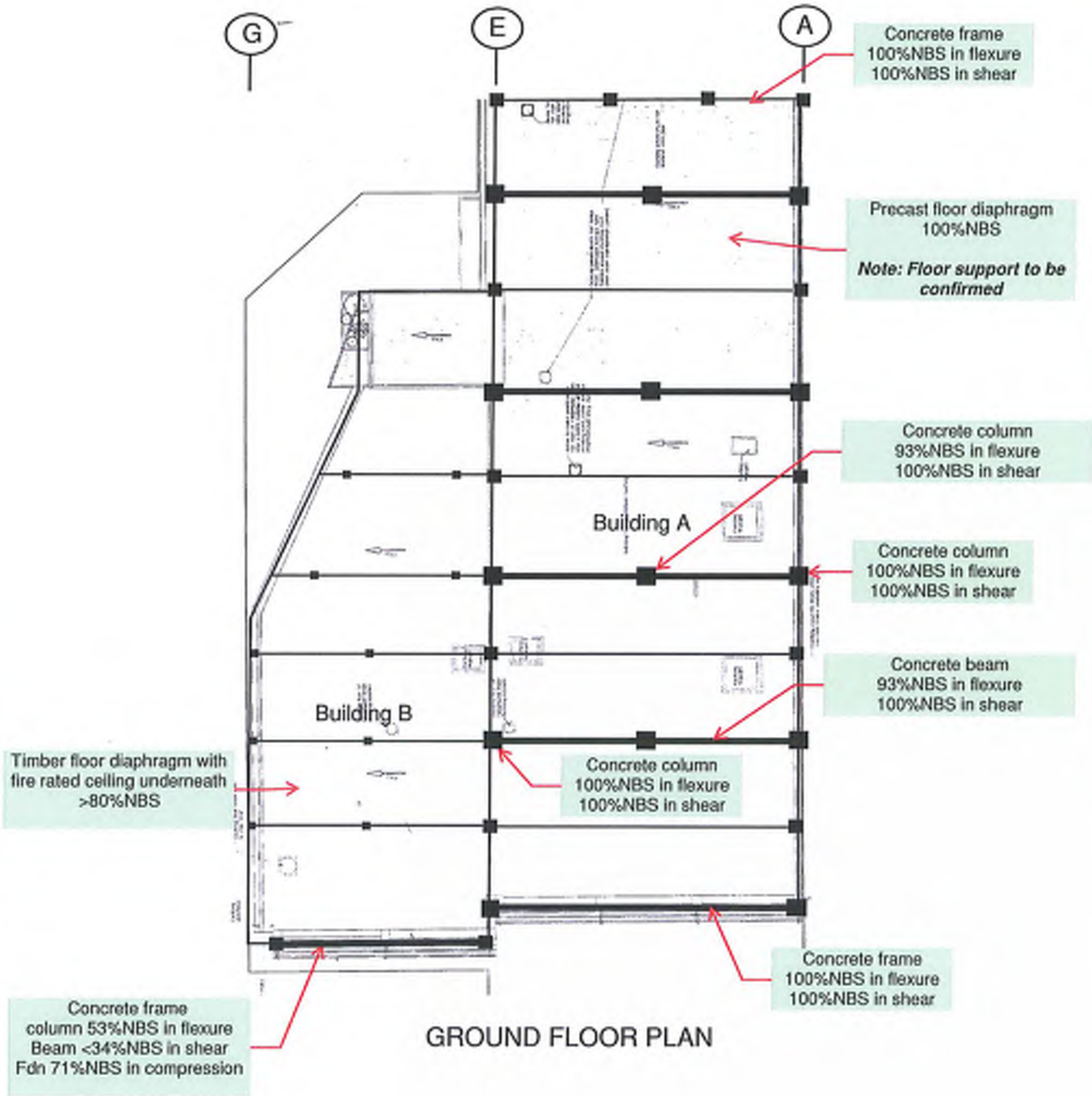


FIRST FLOOR PLAN

NOT TO SCALE

SILVESTER/CLARK
CONSULTING ENGINEERS

Date: 14-06-2019	11 Johnsonville Road Wellington	Sketch #: SK 01	Project #: 23028
By: HN			

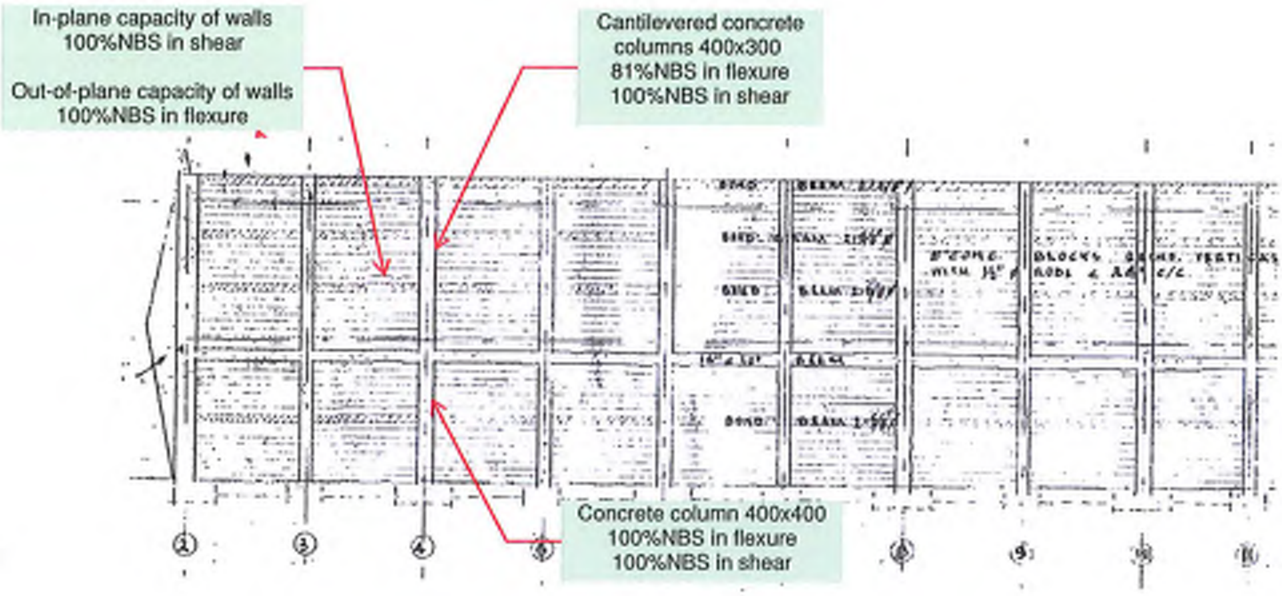


GROUND FLOOR PLAN

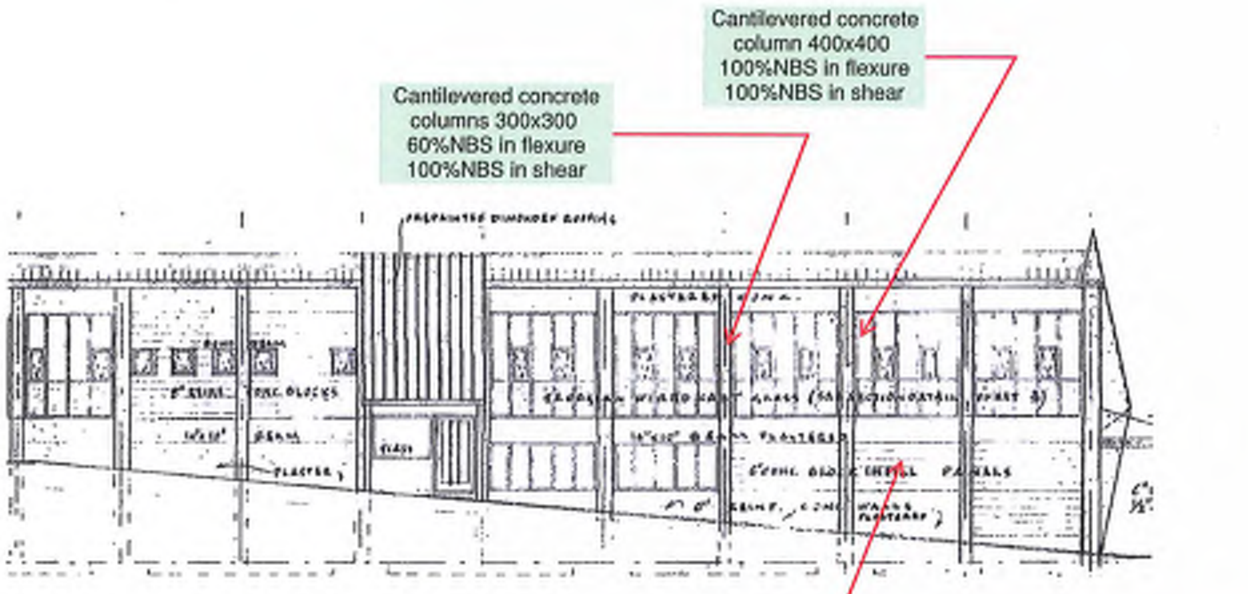
NOT TO SCALE

SILVESTER/CLARK
CONSULTING ENGINEERS

Date: 14-06-2019	11 Johnsonville Road Wellington	Sketch #: SK 02	Project #: 23028
By: HN			



SOUTH ELEVATION_ Grid line A

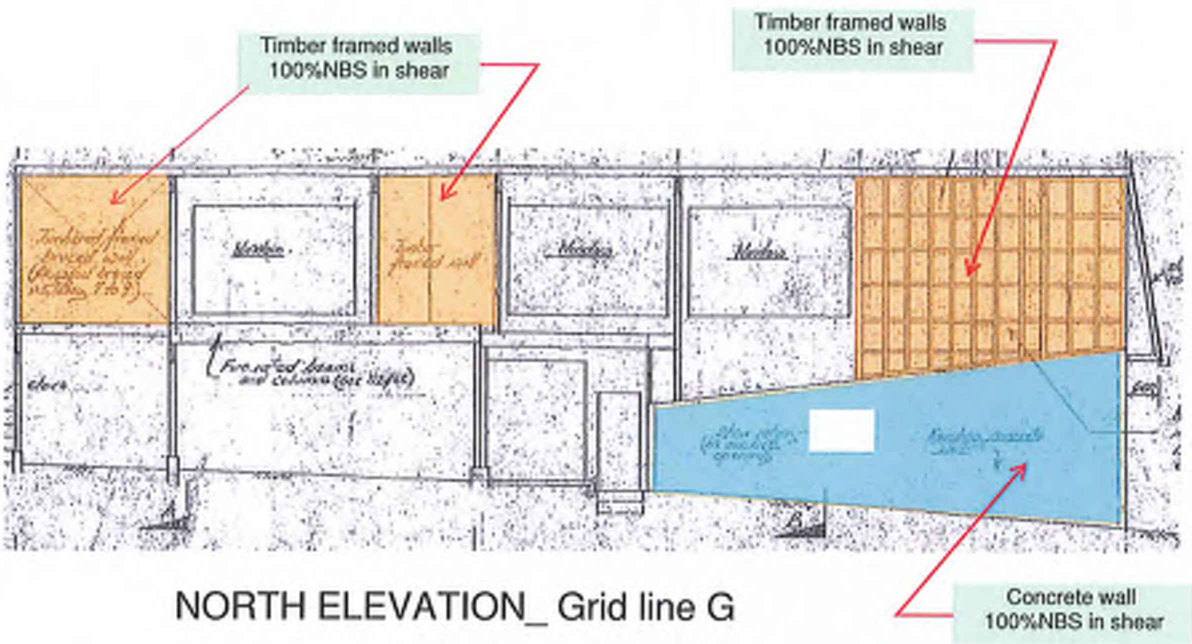


GRIDLINE_ E

NOT TO SCALE

SILVESTER/CLARK
CONSULTING ENGINEERS

Date: 14-06-2019	11 Johnsonville Road Wellington	Sketch #: SK 03	Project #: 23028
By: HN			



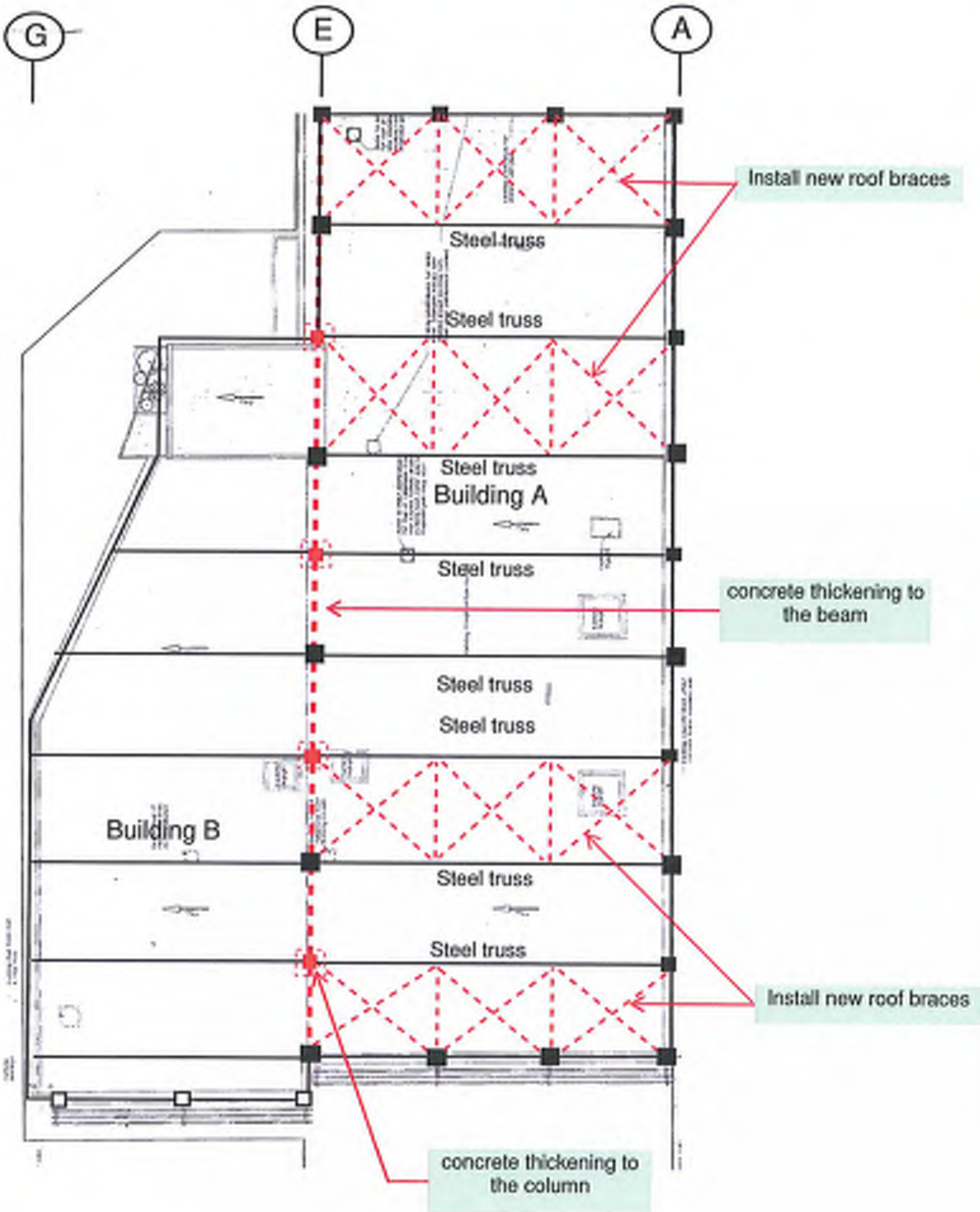
NORTH ELEVATION_ Grid line G

NOT TO SCALE

SILVESTER/CLARK CONSULTING ENGINEERS

Date: 14-06-2019	11 Johnsonville Road Wellington	Sketch #: SK 04	Project #: 23028
By: HN			

STRENGTHENING SCHEME



ROOF PLAN

NOT TO SCALE

SILVESTER/CLARK
CONSULTING ENGINEERS

Date: 14-06-2019	11 Johnsonville Road Wellington	Sketch #: SK 01	Project #: 23028
By: HN			

STRENGTHENING SCHEME

NORTHERN BUILDING
FRONT ELEVATION

NOT TO SCALE

SILVESTER/CLARK
CONSULTING ENGINEERS

Date: 14-06-2019	11 Johnsonville Road Wellington	Sketch #: SK 02	Project #: 23028
By: HN			

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No 05 JOB No 23028

BY HN DATE 25/03/2019

JOB 11 Johnsonville Rd

02 - DESIGN WEIGHTS

Roof Building (A) - On South

- Roof cladding/sheets .... 0.08 kPa
- steel truss and timber purlins 0.2
- Ceiling 0.12
- Insulation 0.03
- Services 0.05

0.48 kPa

Roof Building (B) - On North

- Roof cladding 0.08
- Timber purlins and rafters 0.1
- Ceiling 0.12
- Insulation 0.03
- Services 0.05

0.38 kPa

Floors

- Timber flooring (g) 0.4, SDL 0.2 — 0.6 kPa; (q) 1.5 (Residential)
- Precast floor "Conspan" 3.0 kPa, SDL 0.2 kPa — 3.2 kPa; 1.5 "

3" screed 665 mesh
Similar to Rib Infill
10m long floor

Claddings

- Timber framed walls 0.4 kPa
- 190 mm thick block walls $\left(21 \frac{kN}{m^3}\right)$ 4 kPa

Door & windows 0.3kPa

Average value for windows, doors and claddings 0.35 kPa

Roof live load (9) 0.25 kPa.
(8) 0 kPa (for non-accessible roofs)

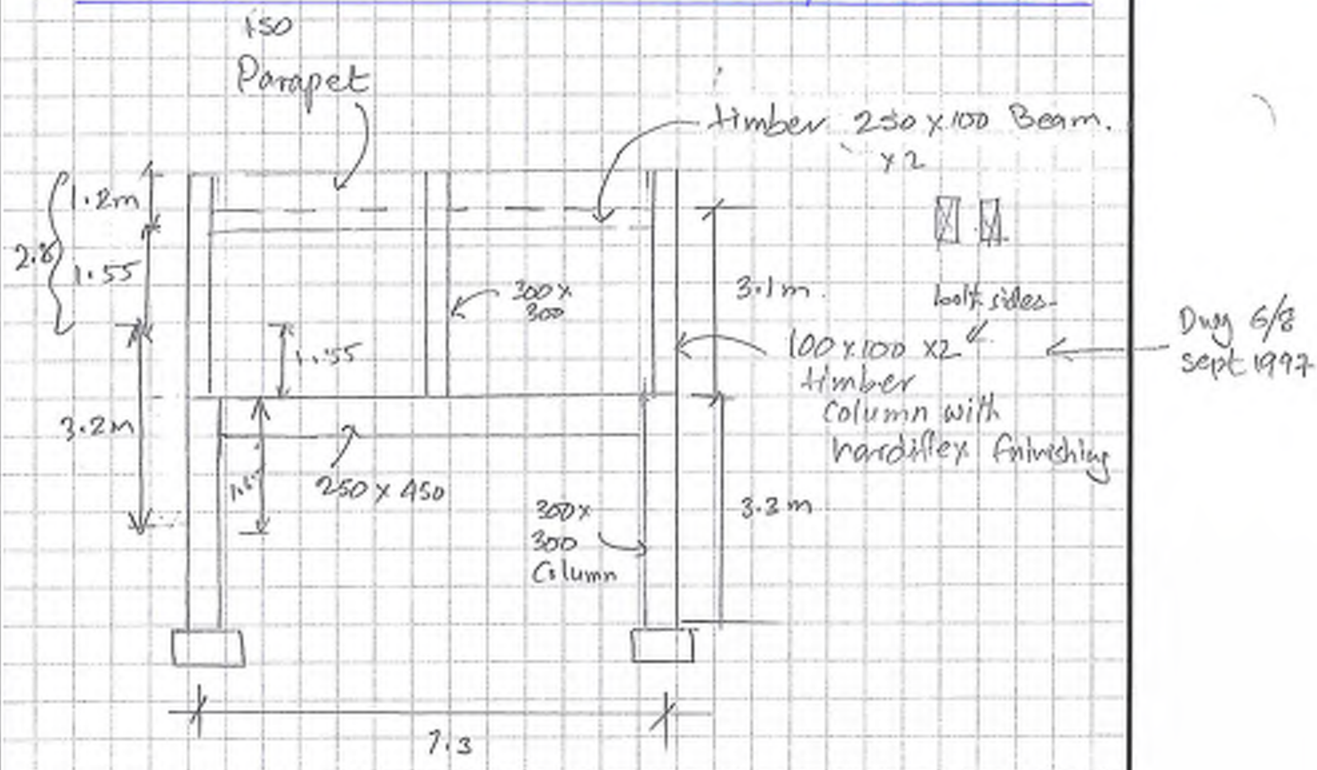
SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No 37 ~~12~~ JOB No 23078

BY HN DATE 25/04/2019

JOB 11 Johnsonville Rd, DSA

03. - Gridline 1 - Frame (Fish & Chips shop)



At Roof

Density of timber

Roof	7.3m x (1m / 2) x 0.38 kPa (wall inside 1m)	=	1.4 kN
Columns	0.1 m² x (1.6m + 1.2m) x 3nos x 5 kN/m3	=	4.2 kN
beam	0.25m x 0.1m x 2 nos x 7.3m = 0.37 x 5 kN/m3	=	1.8 kN
Cladding at parapet	1.2m x 7.3m x 0.5 kN/m2	=	4.4 kN
Side walls	(3.2/2 m + 1.2m) x (1m / 2) x 0.35 kPa	=	0.6 kN
Internal wall	(3.2m / 2) x 7.3m x 0.35 kPa	=	5 kN
			17 kN.

$A = 0.3m \times 0.3m - 0.1 \times 0.1m$
$= 0.08 m^2$

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No 37.23 JOB No 23028

BY HN DATE 25/04/2019

JOB 11 Johnsonville Rd, DSA

At floor

Floor diaphragm , 4m (Half of the building length) x 7.3m x 0.4 kPa = 11.7 kN

Columns above ≈ 4.2 kN P 37.22

Columns below 0.3m x 0.3 x 1.65m x 24 $\frac{kN}{m^3}$ x 2 nos = 7 kN

Beam 0.25 x 0.45 x 7.3m x 24 $\frac{kN}{m^3}$ = 20 kN

Side walls 1.5m x 4m x 0.4 kPa x 1 nos 2.5

Concrete in the ground floor 1.65m x 4m x 4 kPa (190 block wall) x 1 nos = 26.5

Note: Wall between the buildings already considered in the bakery building which is more stiff frame.

Front - upper floor open
- ground floor 1.65 m x 7.3 m x 0.35 kPa = 4 kN

Internal walls - Not in this half of the building =

Σ 80

Live load 4m x 7.3m x 0.3 x $\left(0.3 + \frac{3}{\sqrt{58m^2}}\right)$

= 6 kN

Σ 86 kN

SILVESTER/CLARK
CONSULTING ENGINEERS

Equivalent Static Distribution of Seismic Forces (NZS 1170.5)

Inputs	
Outputs	

S_p	0.7	
μ	1.25	
Site soil category	C	
Estimate for T_1	0.4	seconds
$C_h(T_1)$	2.36	
R	1.0	
N(T,D)	1.0	
Z	0.4	
k_μ	1.1428571	
$C_d(T_1)=S_p*C_h(T_1)*R*N(T,D)*Z/k_\mu$	0.578	

Total number of floors	2	
0.92V	60	kN
0.08V	5	kN
V	66	kN
sum F_i	66	kN
V and sum F_i same?	YES	

Designed by	HN
Date	6-Sep-19
Job no.	22464

Force Distributions and Calculations

	Level	W_i	Interstorey	Height	$W_x.H_x$	$W_x.H_x$/sum($W_x.H_x$)	F_i	V_x	M_{ot}
		(kN)	Height (m)	(m)	(kN.m)		(kN)	(kN)	(kN.m)
0	Roof	18.7	3.6	7.30	137	0.281	22	22	0
1	Floor level	94.6	3.7	3.70	350	0.719	43	66	80
2	Ground	0	0	0.00					322
	Sums	113	-	-	487	1.00	-	-	-

Sum equal to 1?	YES

10% added to weight to factor in eccentricity

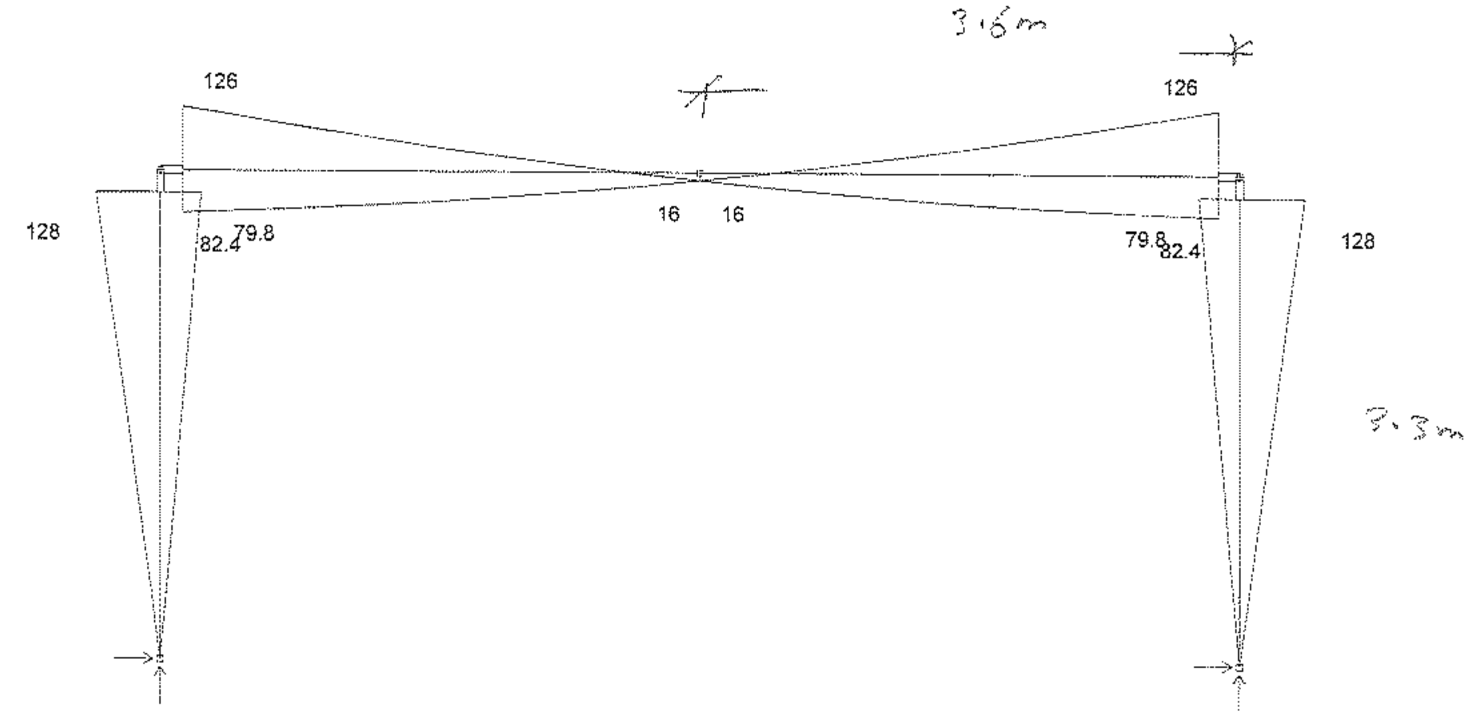
15

15

Envelope for Moment Mz
Maximum
Minimum
Enveloped Cases:
11 C G+Q+Eu
12 C G+Q-Eu

3.1 Demands - Option 1

$\mu = 1.25$
$S_p = 0.7$ (Advised after review)



theta: 270 phi: 0

Bending Moment, Mz

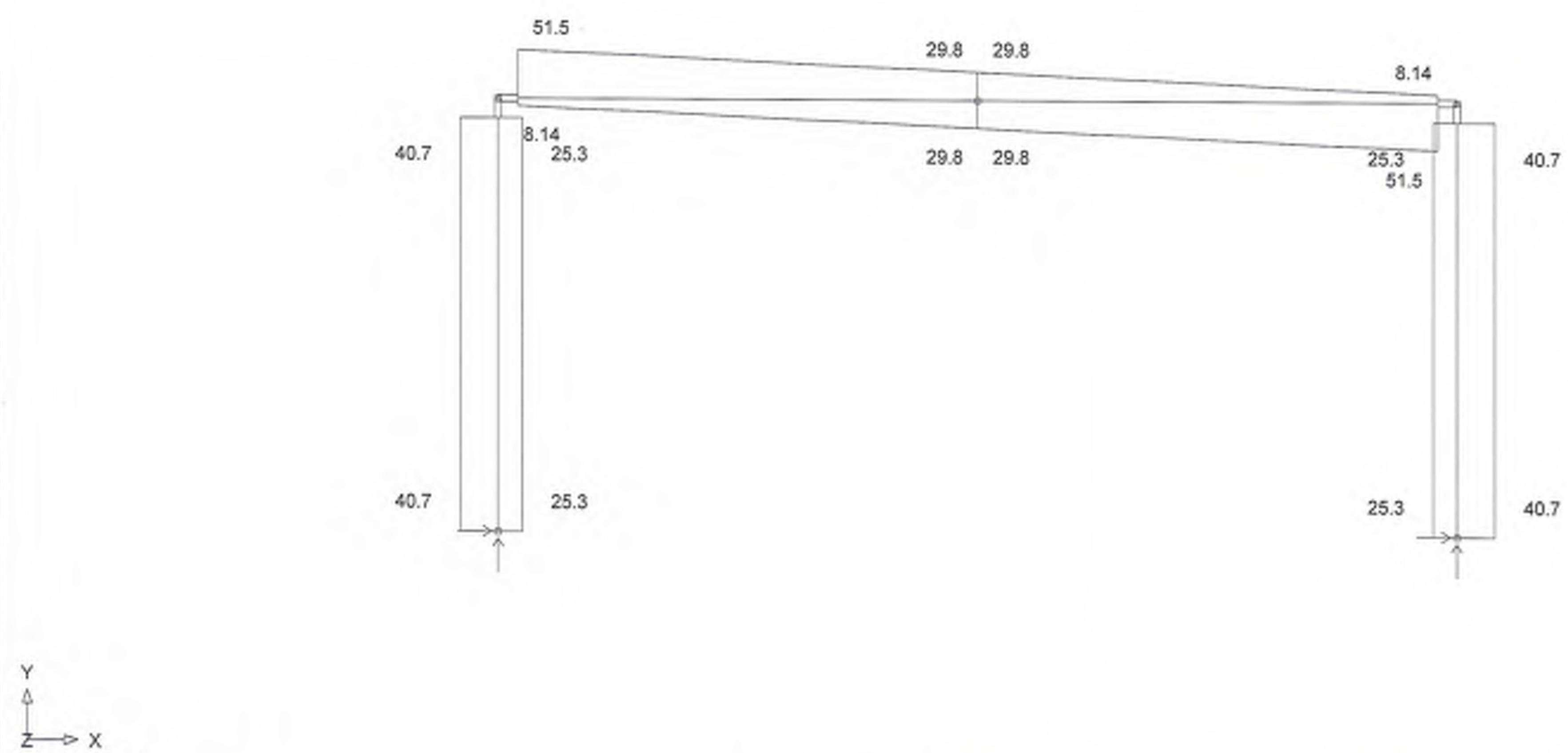
16

Microstran V9

Hassan
Job: Frame_Gridline 1_Frame outside
Transverse Internal Frame

06/09/2019
09:43:58 a.

Envelope for Shear Fy
Maximum
Minimum
Enveloped Cases:
11 C G+Q+Eu
12 C G+Q-Eu



theta: 270 phi: 0

13

Shear Force, Fy

7

Microstran.Advanced V9.20.1.24

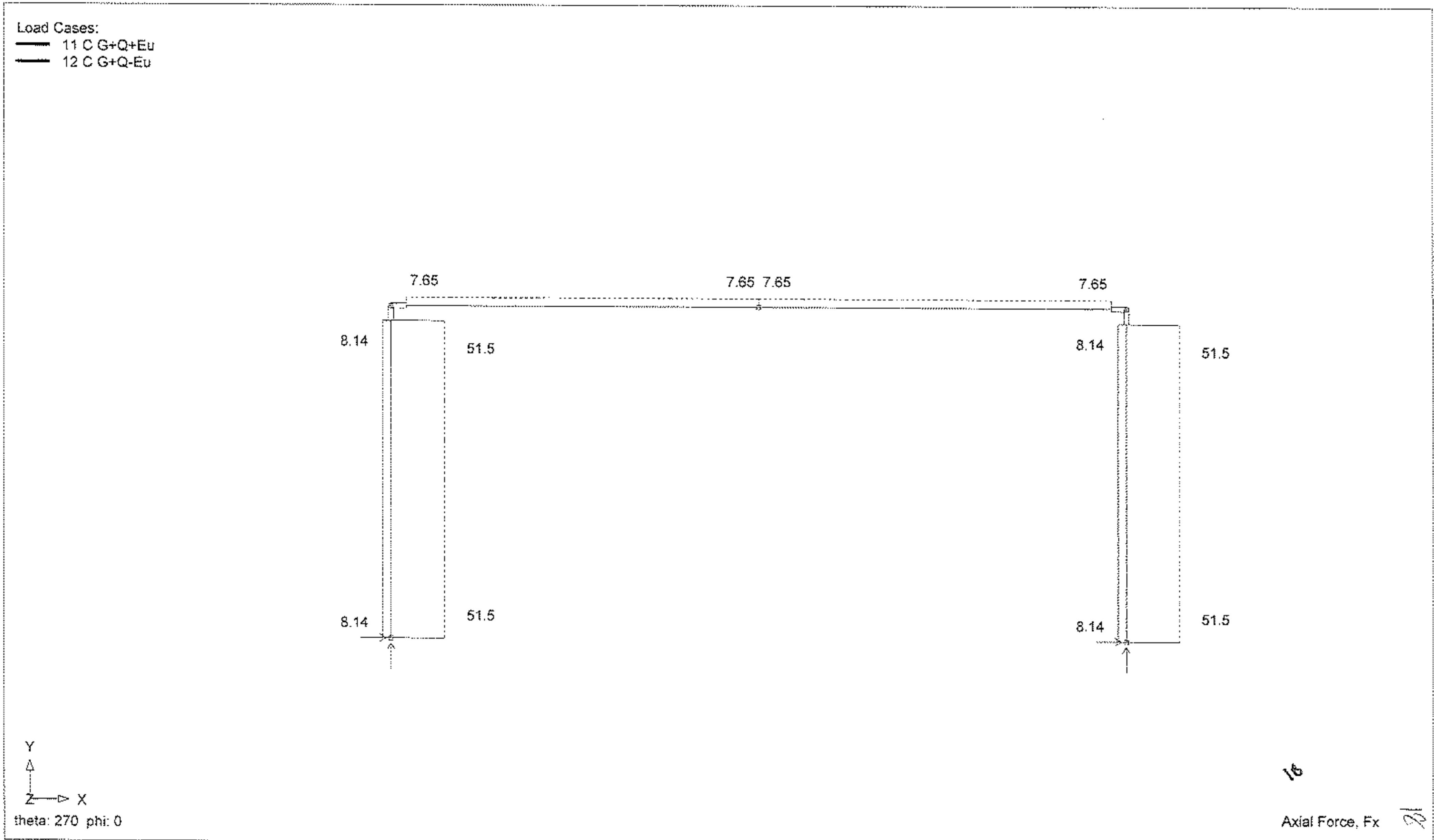
J:\23028 11 Johnsonville Road DSA and Strengthening Scheme\Engineering\Strengthening\Frame_Gridline 1_Frame outside.msw

Microstran V9

Hassan
Job: Frame_Gridline 1_Frame outside

06/09/2019
09:45:02 a.

Transverse Internal Frame



16

22

Microstran V9

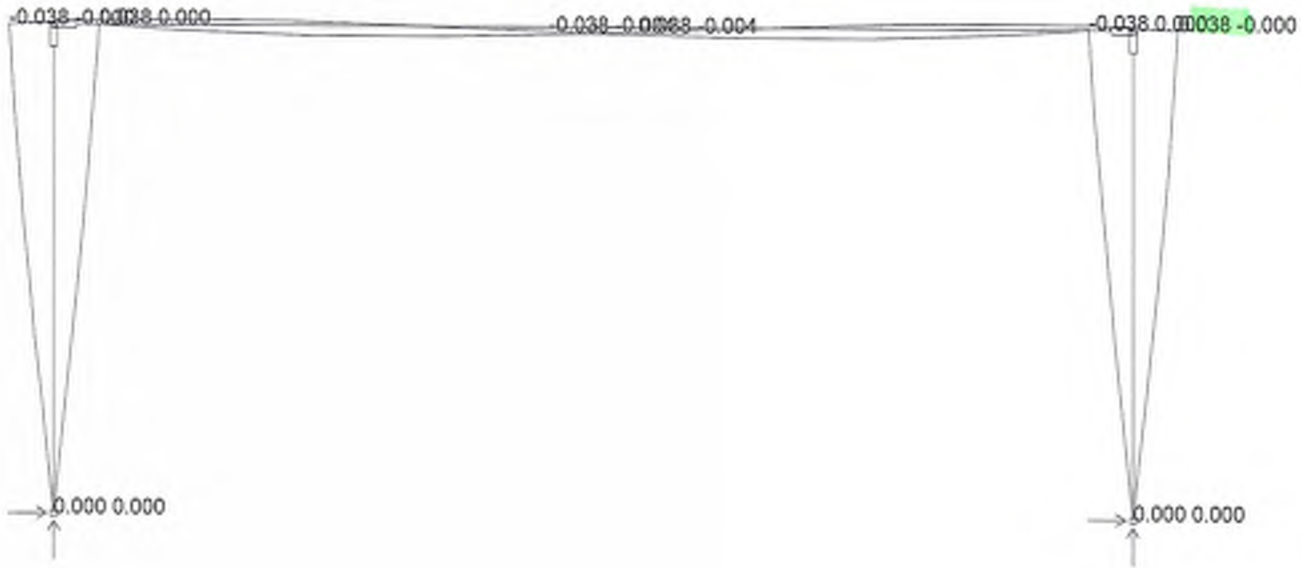
Hassan
Job: Frame_Gridline 1_Frame outside
Transverse Internal Frame

06/09/2019
09:44:57 a.

Load Cases:
11 C G+Q+Eu
12 C G+Q-Eu

$\Delta_{lim} = 3300 \times 2.5\%$
$= 82.5\ mm$

$\Delta_{actual} = 38 \times 1.25 \times 1.2 \times 1$
$= \underline{57\ mm}$



theta: 270 phi: 0

Displaced Shape

Microstran.Advanced V9.20.1.24

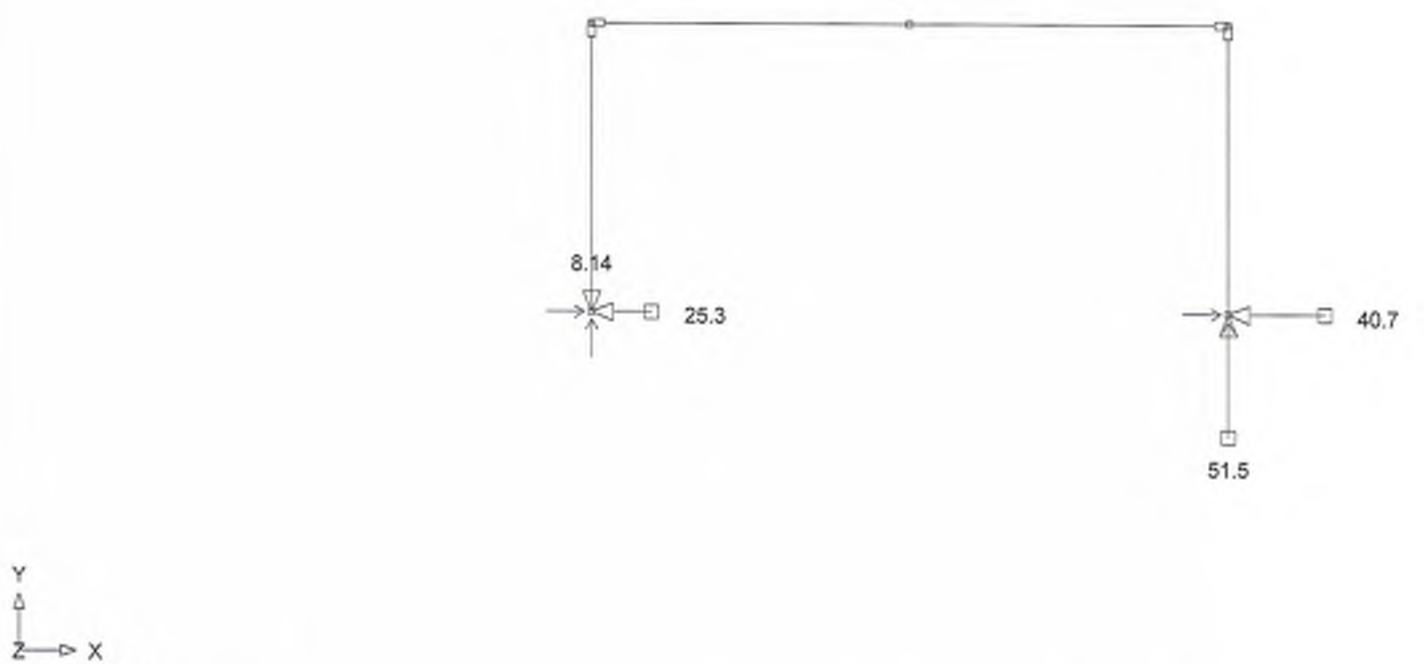
J:\23028 11 Johnsonville Road DSA and Strengthening Scheme\Engineering\Strengthening\Frame_Gridline 1_Frame outside.msw

Microstran V9

Hassan
Job: Frame_Gridline 1_Frame outside
Transverse Internal Frame

06/09/2019
10:01:06 a.

Load Cases:
11 C G+Q+Eu



theta: 270 phi: 0

Support Reactions

3.2 Member Design

Beam

target

$M^* = 126\ kN{\cdot}m \times (80\%) = 101\ kN{\cdot}m$ — P15

$\alpha_m = 1.75$

Try 250 PFC

$L_e = 3.6\ m.$

$\phi M_{bx} = 65.5\ kN{\cdot}m \times 1.75 = 115\ kN{\cdot}m > 101\ kN{\cdot}m$ OK

$\phi M_{bx}\ (115\ kN{\cdot}m) < \phi M_{sy}\ (152\ kN{\cdot}m)$

$\phi M_{bx} = \dfrac{115\ kN{\cdot}m}{0.9} = 127\ kN{\cdot}m$

Column Design

$\phi M_{sx} = 127\ kN{\cdot}m \times \overset{\phi_{os}}{1.15} = 146\ kN{\cdot}m$

$\phi = 1.0$

$V^*_{col} = \dfrac{146\ kN{\cdot}m}{3.3m} = 44.2\ kN$

$V^*_{beam} = \frac{2 \times 146 \text{ kN}\cdot\text{m}}{7.3\text{m}}$

Lateral $\longrightarrow$ $= 40$ kN

$V^*_{beam} = G + \psi_E Q = 86\text{ kN} + 17\text{ kN}$ — P13, P14

↓ (gravity) $= \frac{103 \text{ kN}}{2 \text{ column}}$

$= 52$ kN

$V^*_{beam} = 40 + 52 = 92$ kN

$N^*_c = \underline{92 \text{ kN}}$.

∴ Column demands

$M^* = 146$ kN·m

$V^* = 44$ kN

$N^*_c = 92$ kN

$N^*_T = 8.0$ kN (uplift) from microstran. (P19a)

18

Try 300 PFC

$l_e = 3.3m$

$\phi M_{by} = \frac{88.4 \text{ kN}\cdot m}{0.9} \times 1.75 = 172 \text{ kN}\cdot m > 146 \text{ kN}$ ok

$\phi M_{by} > \phi M_{sx}$

$\therefore$ Use ϕM_{sx} (152 kN·m) > 146 kN·m

$\therefore$ Adopt PFC 250 Beam
PFC 300 Column

Web capacity check

PFC 250 $\Rightarrow \phi V_v = 348$ kN > 44 kN ok
PFC 300 $\Rightarrow \phi V_v = 415$ kN > 92 kN ok

Capacity table T 5.2-5

Column combined check

$M^* \leq \phi M_{bx} \left(1 - \frac{N^*}{\phi N_c}\right)$

$\phi M_{bx} = \phi M_{sx} = 152$ kN·m

$N^* = 92$

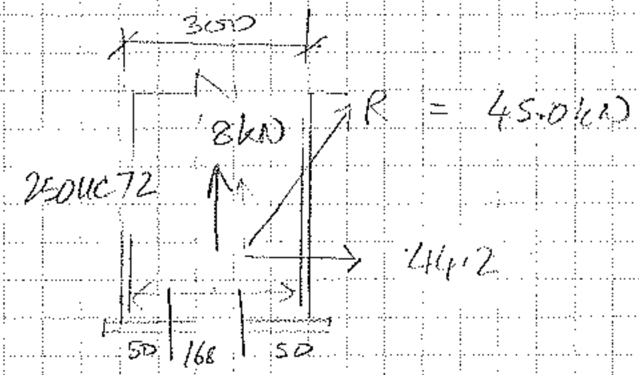
$\phi N_c = 1220$ kN $l_e = 6.0$ m

$\therefore \phi M_{bx}\left(1 - \frac{N^*}{\phi N_c}\right) = 152\left(1 - \frac{92}{1220}\right) = \frac{140}{0.9}$ kN·m = 156 kN·m > 146 kN·m ok

(Adopt PFC 250 Beam
PFC 300 Column)

3.3 Connection design

Base connection



— P21

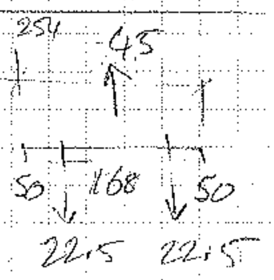
$R^* = 45\,kN$

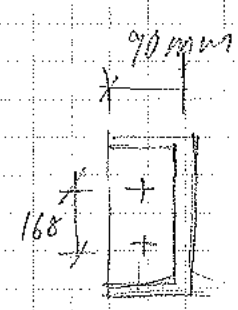
Try M16 bolts $\phi N_t = 104 \times 2 = 208\,kN > 45\,kN$ ok

$\phi V_f = 59.3kN \times 2 = 119\,kN >$ " ok

∴ Adopt 2 - M16 bolts (8.8/S)

Base plate in tension





$M^* = 22.5kN \times \left(\frac{0.168\,m}{2}\right) = 1.89\ kN \cdot m$

Try with 16 mm base plate

$\phi M_s = \frac{0.9 \times 250 \times 270 \times 16^2}{6} = 2.8\,kN \cdot m > 1.89\,kN \cdot m$ ok

∴ Adopt 280 X 275 mm x 16 mm MSP

4 - Along gridline E

Roof level

Roof Building (A) $5m \times 28m \times 0.48\,kPa$ = 67 kN
" (B) $3.65m \times 21m \times 0.38\,kPa$ = 29 kN

Weight of the roof beam $0.25m \times 0.25m \times 28m \times 24\,kN/m^3$ = 58.8 kN

Conc. columns $0.4m \times 0.4m \times 1.8m \times 10\,nos \times 24\,kN/m^3$ = 69 kN
End beams $0.4m \times 0.3m \times 8.7m \times 2 \times 24$ " = 50 kN
End walls front $1.8m \times 8.7m \times 0.35\,kPa$ = 5.5 kN
" rear $0.19m \times 1.8m \times 8.7m \times 21\,kN/m^3$ = 62 kN

Front Col $7\,kN \times 2$ = 14 kN (P-39)
Parapet $0.15m \times 0.9m \times 8.5m \times 24\,kN/m^3$ = 27.5 kN
Internal walls (Gib) $20\,kN + 10\,kN$ = 30 kN (P-39)

Internal wall (stair well) $3.65m \times 1.8m \times 0.19m \times$
190 mm block $21\,kN/m^3$ = 26 kN

411.3 kN

Front wall Conc.

$(0.4m \times 0.25m + 0.15m \times 0.9m) \times 5m \times 24\,kN/m^3$ = 28 kN

439 kN

Floor level

Floor Building (A)	g	28m x 5m x 3.2kPa	= 448	kN
	q	28m x 5m x 1.5kPa x 0.3 x 0.5	= 32	kN
" (B)	q	21m x 3.65m x 1.5kPa x 0.3 x 0.5	= 17	kN
	g	21m x 3.65m x 0.6kPa	= 46	kN
Floor beam Wt		0.35m x 0.25m x 28m x 24 kN/m3	= 59	kN
Conc. Columns		0.4m x 0.4m x 3.15m x 10 x 24 kN/m3	= 121	kN
End beams front		0.4m x 0.85m x 8.7m x 24 kN/m3	= 71	kN
End walls front		3.15m x 8.7m x 0.35 kPa	= 10	kN
rear		3.15m x 8.7m x 0.19m x 21 kN/m3	= 109	kN
Front column			= 12	kN (P 39)
Internal walls Gib		(A) 30 kN + (B) 15 kN	= 45	kN (P 39)
" Block		3.65m x 3.15m x 0.19m x 21 kN/m3	= 46	kN

Σ 1016 kN

Front wall conc. 0.15m x 1.85m x 1m x 24 kN/m3 7 kN

Σ 1023 kN

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No.		JOB No	23028
BY	HN	DATE	##########
JOB	11 Johnsonville Road- DSA		

Notes: **WALL ALONG GRID E**

10% added to weights to factor in eccentricity

Equivalent Static Distribution of Seismic Forces (NZS 1170.5)

This spreadsheet follows NZS 1170.5:2004 - Part 5: Earthquake actions - New Zealand

Ductility, μ	1.25	
Period, T_1	0.40	sec
$C_d(T_1)$	0.74	

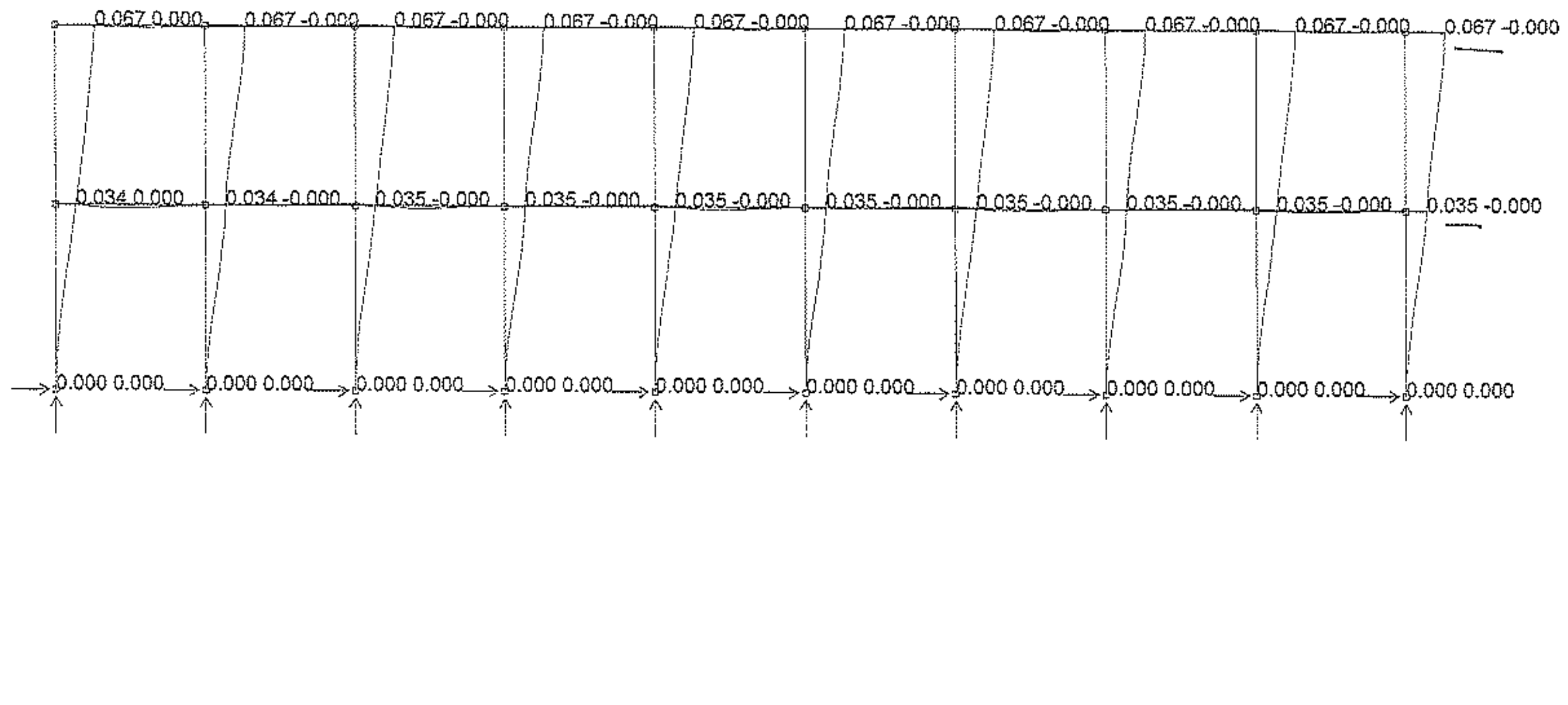
Total no. of floors above	2	
Total Base Shear, V	**1196**	kN
0.92V	1100	kN
0.08V	96	kN
V and sum F_i same?	YES	

Inputs	
Outputs	

		Level	W_i (kN)	Interstorey Height (m)	Height (m)	$W_x.H_x$ (kN.m)	$W_x.H_x/\Sigma(W_x.H_x)$	F_i (kN)	V_x (kN)	M_{ot} (kN.m)
Rf	1	2	482.9	3.6	7.30	3525	0.46	600	600	0
1Flr	1	1	1125.3	3.7	3.70	4164	0.54	596	1196	2160
GFlr	1	0	~~0~~	0.00	0.00					6583
		Sums	1608	7.30		7689	1.00	**1196**		
							Sum equal to 1?	YES		

weight from
p 46 and 47 + (10%)

Load Cases:
11 C G+Q+Eu



theta: 270 phi: 0

Displaced Shape

= input cell
= output

Calculate Period Using Rayleigh Method

	W_i (kN)	F_i (kN)	d_i (m)	d_i^2 (m2)	Fd_i kNm	Wd_i^2 (kNm2)
Roof	483.0	600.0	0.067	0.0045	40.20	2.17
3rd			0	0.0000	0.00	0.00
2nd			0	0.0000	0.00	0.00
1st	1125.0	596.0	0.035	0.0012	20.86	1.38
				Σ=	61.06	3.55

F_i those used in Microstran of other analysis

D_i displacement at level I from Micror similar analysis

1st Translational period: T_1= 2*PI()*SQRT(H15/(9.81*G15))

T_1= 0.48 s

ω

29

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No.		JOB No	23028
BY	HN	DATE	#########
JOB	11 Johnsonville Road- DSA		

Equivalent Static Distribution of Seismic Forces (NZS 1170.5)

Notes:	WALL ALONG GRID E *10% added to weights to factor in eccentricity*

This spreadsheet follows NZS 1170.5:2004 - Part 5: Earthquake actions - New Zealand

Ductility, μ	1.25	
Period, T_1	0.48	sec
$C_d(T_1)$	0.63	

Total no. of floors above	2	
Total Base Shear, V	1019	kN
0.92V	938	kN
0.08V	82	kN
V and sum F_i same?	YES	

Inputs	
Outputs	

		Level	W_i (kN)	Interstorey Height (m)	Height (m)	$W_x.H_x$ (kN.m)	$W_x.H_x/\Sigma(W_x.H_x)$	F_i (kN)	V_x (kN)	M_{ot} (kN.m)
Rf	1	2	482.9	3.6	7.30	3525	0.46	511	511	0
1Flr	1	1	1125.3	3.7	3.70	4164	0.54	508	1019	1841
GFlr	1	0	0	0.00	0.00					5612
		Sums	1608	7.30		7689	1.00	1019		
							Sum equal to 1?	YES		

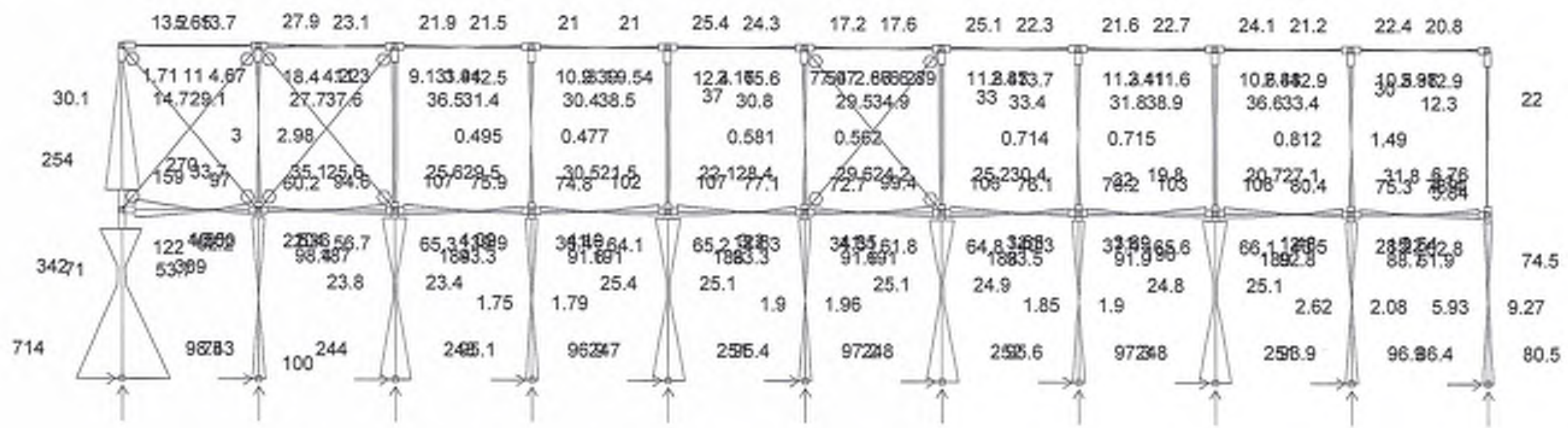
10

Microstran V9

Hassan
Job: Gridline E_with brace
23028

05/09/2019
02:33:25 p.

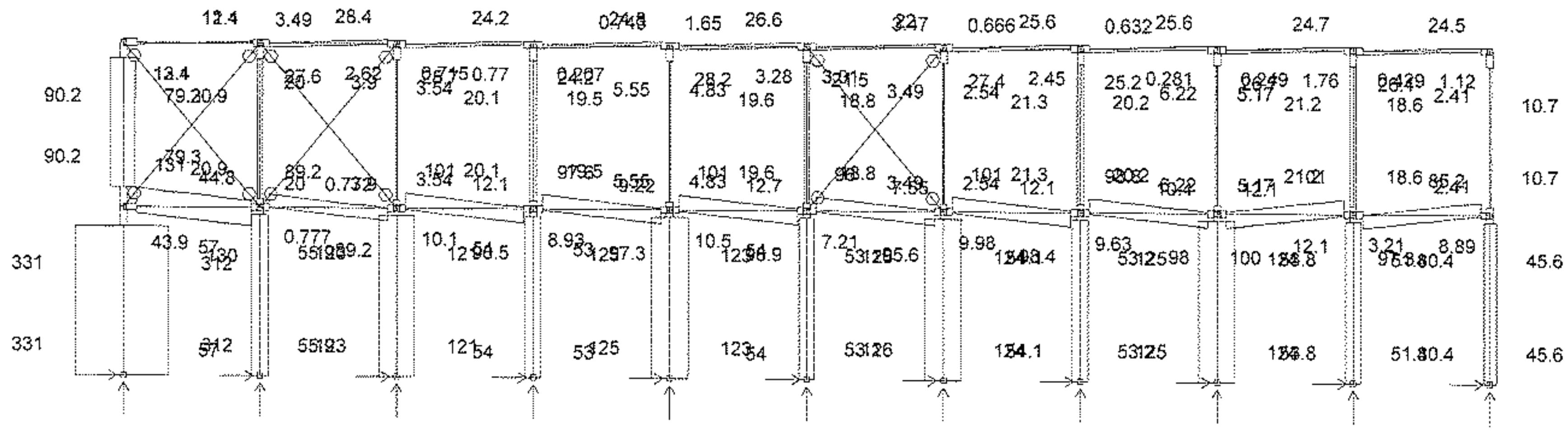
Envelope for Moment Mz
Maximum
Minimum
Enveloped Cases:
11 C G+Q+Eu
12 C G+Q-Eu



theta: 270 phi: 0

Bending Moment, Mz

Envelope for Shear Fy
Maximum
Minimum
Enveloped Cases:
11 C G+Q+Eu
12 C G+Q-Eu



Y
Z X
theta: 270 phi: 0

Shear Force, Fy

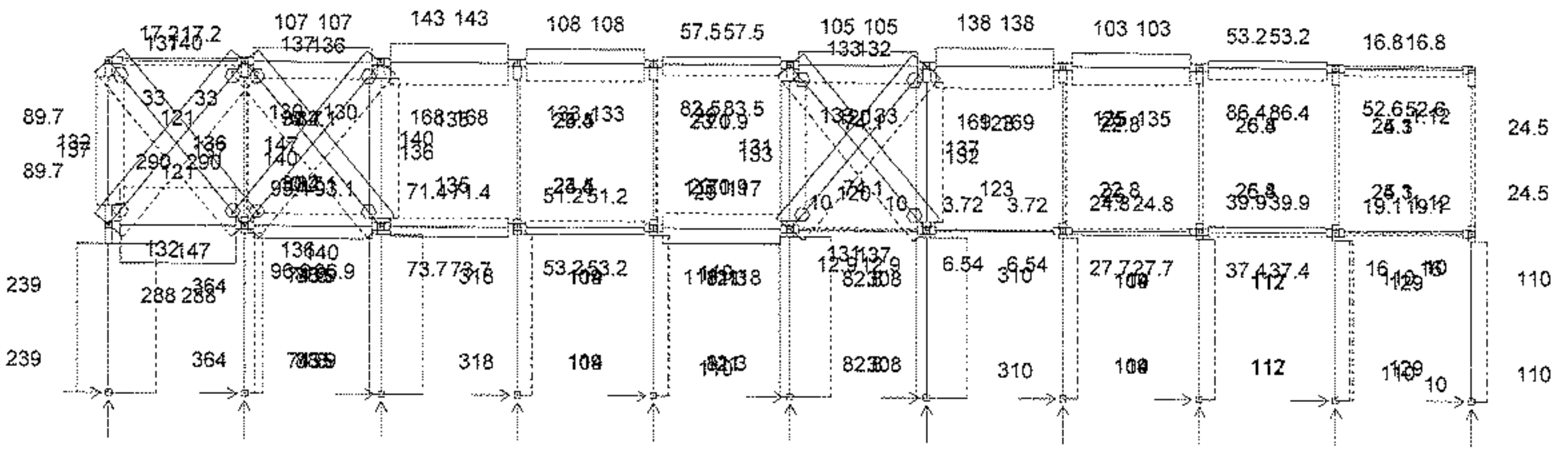
31

Microstran V9

Hassan
Job: Gridline E_with brace
23028

05/09/2019
02:34:19 p.

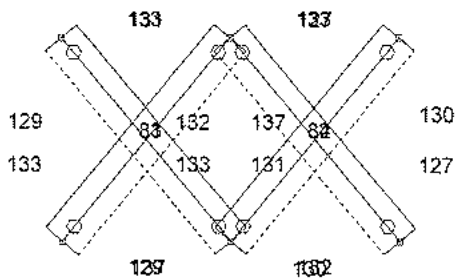
Load Cases:
11 C G+Q+Eu
12 C G+Q-Eu



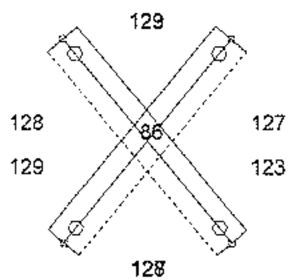
theta: 270 phi: 0

Axial Force, Fx

Load Cases:
11 C G+Q+Eu
12 C G+Q-Eu



$N_E^* = 147$ kN (max)



Y
Z X
theta: 270 phi: 0

33

Axial Force, Fx

4.1 Member check

Try with 100 x 8 MSF

$N_t^* = 147$ kN max (80%) — P34

$= 122$ kN $\times 1.1$ (C_s, Cat 3) $= 134$ kN

$\phi N_t = \phi f_y A_g$
$0.85 f_u k_{te} A_n$ } lesser of

$= 0.9 \times 250 \times 800 = 180$ kN

$= 0.85 \times 410 \times 1 \times (800 - 18 \times 6)$

$= 241$ kN

ϕN_t (180 kN) $> N_t^*$ (134 kN) OK

Connections

2-M16 bolts $\phi V_f = 59.3 \times 2 = 119$ kN < 134 kN

Try 2-M20 bolts $\phi V_f = 92.6 \times 2 = 185$ kN > 134 kN OK

Adopt 100 x 8 MSF
2-M20 bolts

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No 35 JOB No 23028

BY HN DATE 5/09/2019

JOB 11 Johnsonville Rd, Jr

05. Roof diaphragm (Bun bakery)

In the transverse direction

Roof 28m x 10m x 0.48 kPa = 134.4 kN

End wall rear 10m x 1.8m x 0.19m x 21 kN/m³ = 72 kN

End wall front 10 m x 1.8m x 0.35kPa = 6.3 kN

Parapet 0.15m x 0.9m x 10m x 24 kN/m³ = 32 kN

Perimeter columns 0.4m x 0.4m x 1.8m x 20 nos x 24 kN/m³ = 138 kN

Front col 0.4m x 0.4m x 1.8 m x 24 kN/m³ = 7 kN

Internal walls 28m x 10m x (20/) x 0.35 kPa = 20 kN

410 kN

Refer to the attached P38, 39 for the roof dimensions and seismic demands.

Seismic demand = 410 kN x 2.19 (g) (P 37)
= 898 kN

(μ = 1.0)

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No.		JOB No	23028
BY	HN	DATE	28/03/2019
JOB	11 Johnsonville Road		

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 **C**

$C_h(T)$=	1.33	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)=	0.53	

8.3 Floor Height Coefficient

h_i=	**7.7**	m, height of level part is on
h_n=	**7.7**	m, height of building
C_{hi}=	2.28	

8.4 Part Spectra Shape Coefficient

T_p=	**0.40**	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)

$C_p(T_p)$=	2.43

8.5 Design Actions in Parts

μ_p=	**1.00**	For P5 & P6 use μ_p=1.0
C_{ph}=	1.00	
Part Category=	**P3**	
R_p=	**0.90**	from table 8.1
W_p=	0	kN or kPa- optional

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

	F_{ph}=	2.19	x W_p
or	F_{ph}=	0	kN or kPa

Silvester Clark
Consulting Engineers

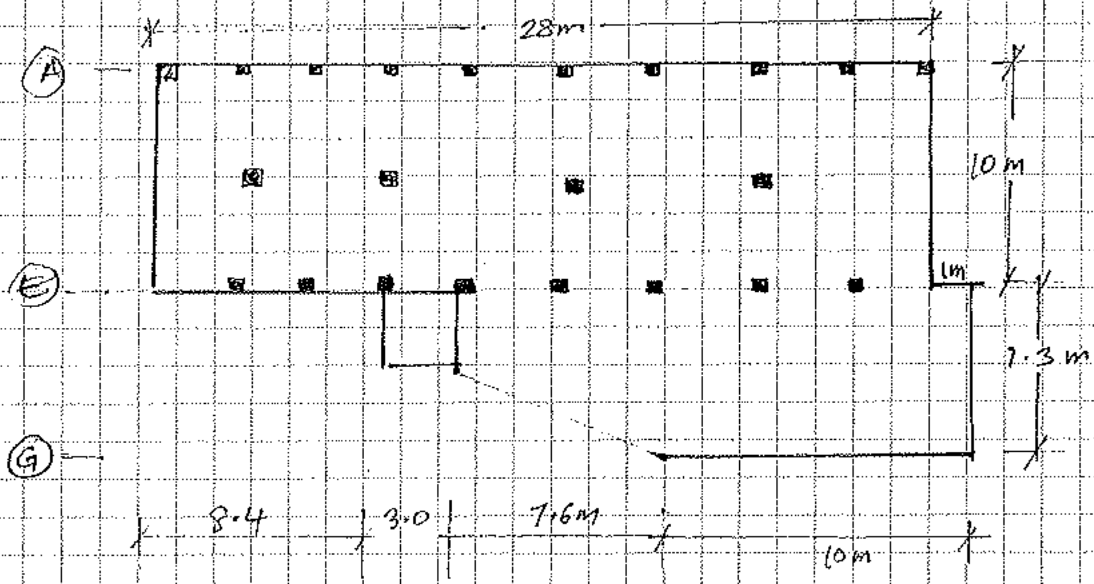
- SEISMIC WEIGHTS & DEMANDS - LONG. DIRECTION

In the longitudinal direction frames and walls on gridline A, E and G have been considered.
Refer to page 07 and 08 for gridline and floor plan layout.

5.1 Along gridline A - Building (A)

Seismic weight.
TW = 2.5 m.

Roof level



Roof level on gridline A

Roof 28m x 5m x 0.48 kPa = 67 kN

Perimeter wall 28m x 2.2m (Including 400mm above truss) x 0.19m x 21 kN/m3 = 246 kN

Perimeter Col. 0.4m x 0.4m x 1.8m x 10nos x 24 kN/m3 = 69 kN

(no. of columns vary between original dwgs and modified/refurbished floor plans)

Endwalls, rear	$5m \times 1.8m \times 0.19m \times 21\,kN/m^3$	= 36 kN
Col	$0.4m \times 0.4m \times 1.8m \times 24\,kN/m^3$	= 7 kN
" front	$5m \times 1.8m \times 0.35\,kPa$	= 3 kN
Parapet	$0.15 \times 0.9m \times 5m \times 24\,kN/m$	= 16 kN
Front Col	$0.4m \times 0.4m \times 1.8m \times 24\,kN/m^3$	= 7 kN
roof beam front and rear	$0.4 \times 0.3m \times 5m \times 2\,nos \times 24\,kN/m^3$	= 29 kN
Internal walls	$28m \times 10m \times 0.2 \times 0.35\,kPa$	= 20 kN

Note : There is no concrete roof beam on the perimeter

Floor level

Floor (g)	$28m \times 5m \times 3.2\,kPa$	= 448 kN
Floor (q)	$28m \times 5m \times 1.5\,kPa \times 0.3 \times 0.5$	= 32 kN
Perimeter wall	$28m \times 3.15m \times 0.19m \times 21\,kN/m^3$	= 352 kN
Perimeter col	$0.4m \times 0.4m \times 3.15m \times 10\,nos \times 24\,kN/m^3$	= 121 kN
Rear end wall	$8m \times 3.15m \times 0.19m \times 21\,kN/m^3$	= 63 kN
Rear end col	$0.4m \times 0.4m \times 3.15m \times 24\,kN/m^3$	= 12 kN
Front end wall	$5m \times 3.15m \times 0.35\,kPa$	= 6 kN
Front end col	$0.4m \times 0.4m \times 3.15m \times 24\,kN/m^3$	= 12 kN
Perimeter beam	$0.35m \times 0.25m \times 28m \times 24\,kN/m^3$	= 59 kN
Internal walls	= 20 kN (upper)	= 20 kN
	= ½ 10 kN (ground)	= 10 kN
Front ~~and end~~ wall beam	$0.4m \times 850mm \times 5m \times 24\,kN/m^3$	= 41 kN

A

Option 2

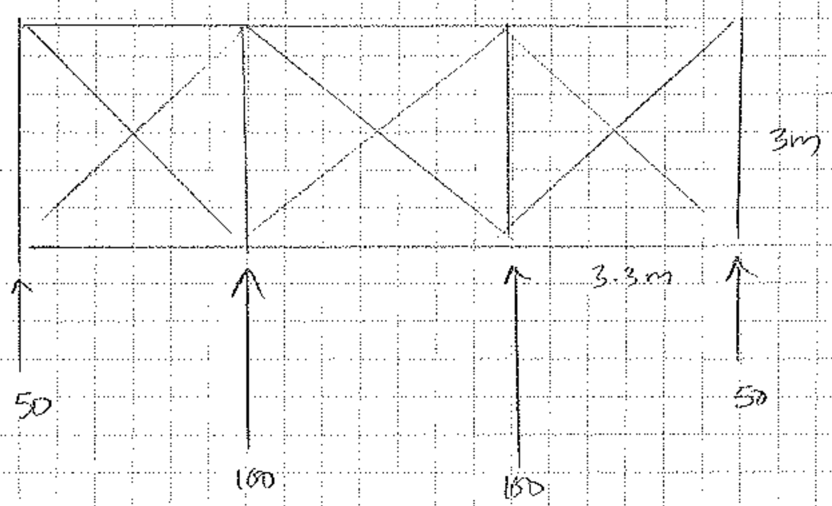
Three sets of bracing

$V^* = 898 \text{ kN}$

Each set $= \frac{898}{3} = 299 \text{ kN}$

Each node (Internal) $= \frac{299}{3} = \underline{100 \text{ kN}}$

(End ") $= 50 \text{ kN}$

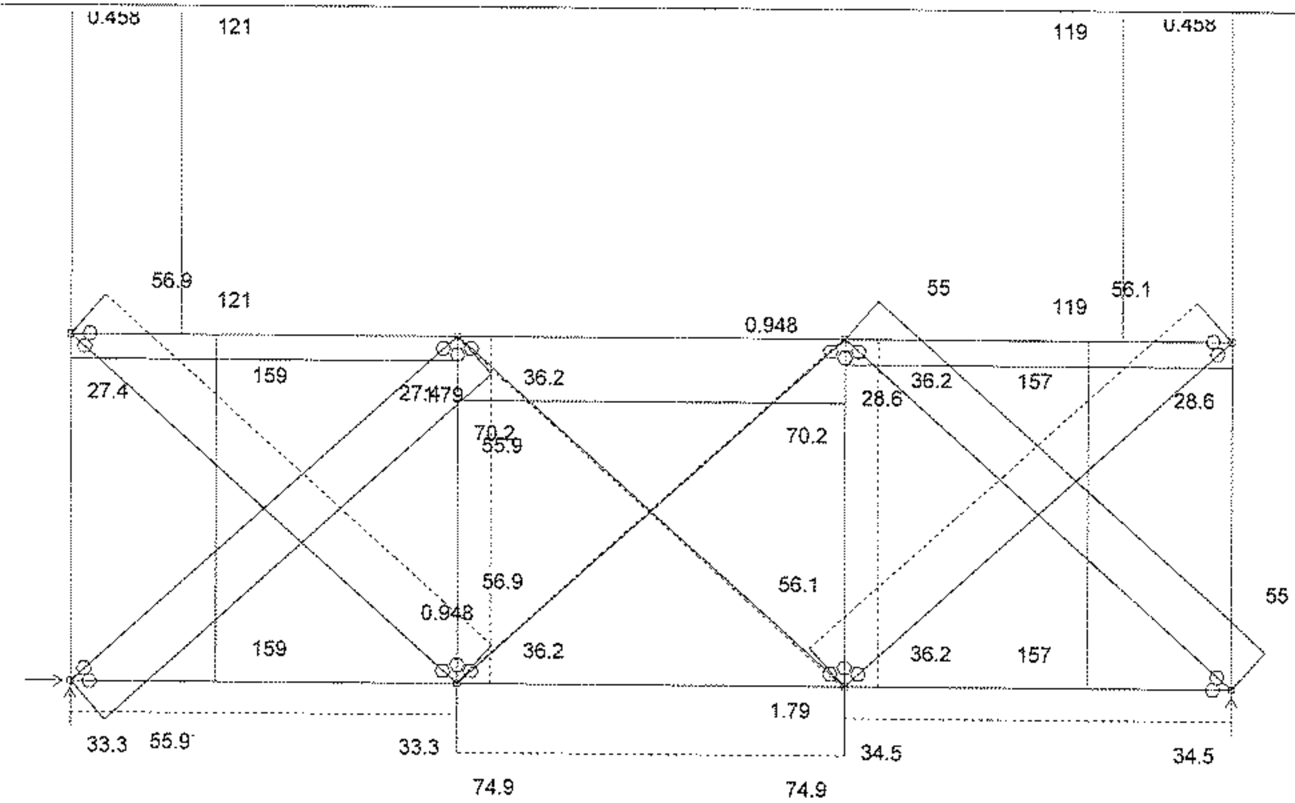


Demands

$N_t^* = 76 \text{ kN (max)}$

$N_c^* = 50 \text{ kN (max)}$

Load Cases:
1 P Vy



theta: 270 phi: 0

Axial Force, Fx

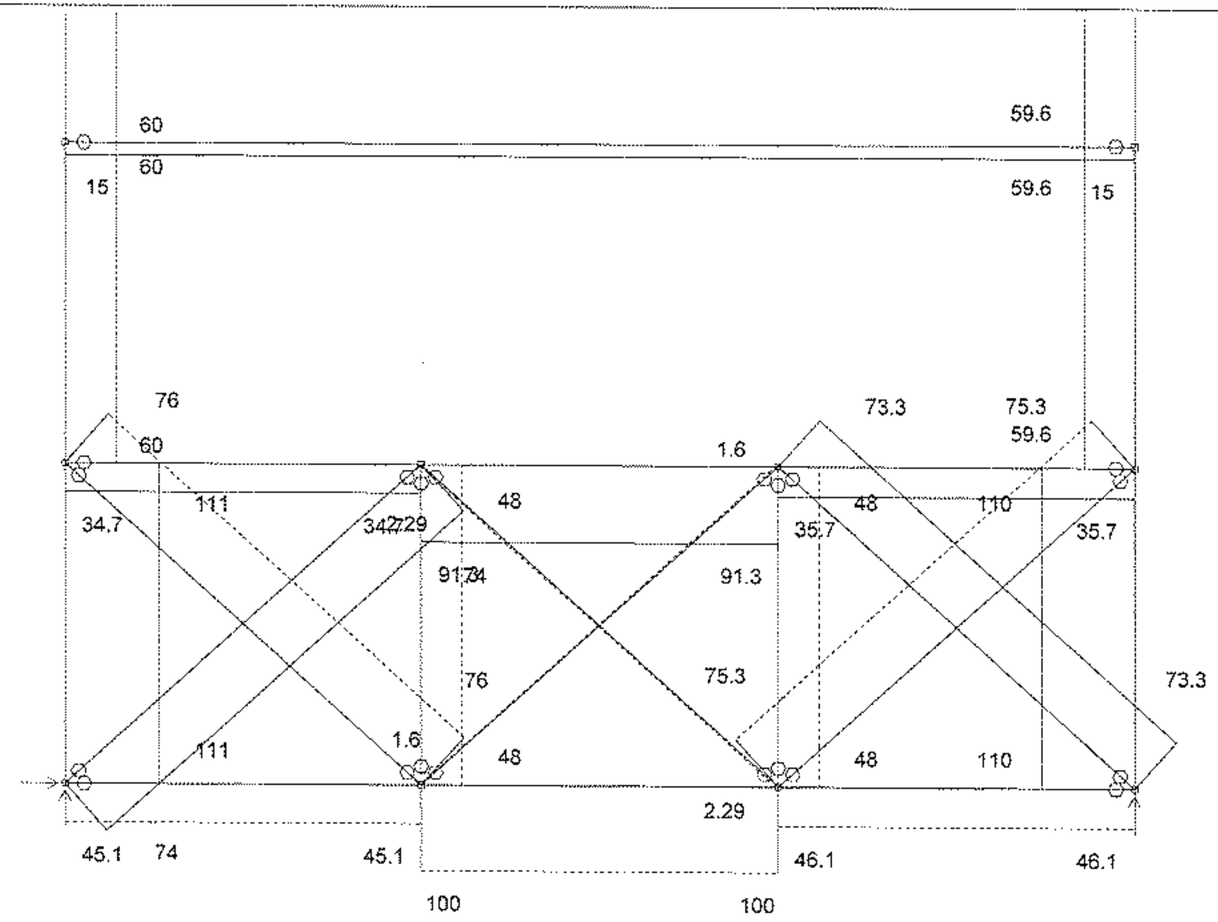
40

Microstran V9

Hassan
Job: Roof diapgragm
23028

05/09/2019
03:10:20 p.

Load Cases:
1 P Vy



theta: 270 phi: 0

Axial Force, Fx

41

4.1 Member design

$N_t^* = 76$ kN (max) (83/..) = 63 kN

$\mu = 1.0$
$C_s = 1.0$

Try with 65×5 EA

$\phi N_t = \phi f_y A_g$
$0.85 f_u k_t A_n$ } lesser

$= 0.9 \times 250 \times 581 = 131$ kN > 63 ... ok

$= 0.85 \times 410 \times 0.9 \times (581 - 18 \times 6)$
$= 148$ kN

INCREASE SIZE TO 80×8 EA FOR EASIER CONNECTING TO EXISTING STRUCTURE.

Connections

2-M16 bolts 59.3×2 = 118.6 kN > 63 kN ok

Adopt 80×8 EA
2-M16 bolts

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No 93 JOB No 23028

BY HN DATE 5/09/2019

JOB 11 Johnsonville Rd

$N^*_c = 50\ kN$

$l_e = 3\ m$

$\phi N_c = (75 \times 75 \times 4\ SHS) = \underline{149 kN} > \underline{50 kN}$

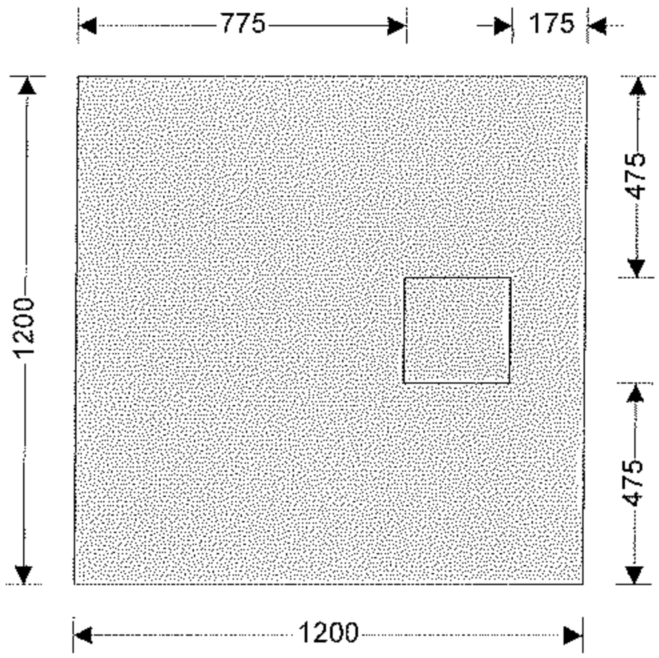
ok

UPSIZE TO 89x89SHS6.0

PAD FOOTING ANALYSIS AND DESIGN

In accordance with AS3600-2009 incorporating Amendment No. 1

Tedds calculation version 2.0.05.01



Pad footing details

Length of pad footing	L = **1200** mm
Width of pad footing	B = **1200** mm
Area of pad footing	$A = L \times B$ = **1.440** m^2
Depth of pad footing	h = **500** mm
Depth of soil over pad footing	h_{soil} = **0** mm
Density of concrete	ρ_{conc} = **24.0** kN/m^3

Column details

Column base length	l_A = **250** mm
Column base width	b_A = **250** mm
Column eccentricity in x	e_{PxA} = **300** mm
Column eccentricity in y	e_{PyA} = **0** mm

Soil details

Density of soil	ρ_{soil} = **0.0** kN/m^3
Design shear strength	ϕ' = **30.0** deg
Design base friction	δ = **0.0** deg
Ultimate design bearing capacity	$P_{bearing}$ = **250** kN/m^2

Load factors for stability

Dead load factor - stabilizing	γ_{sG} = **1.00**
Dead load factor - destabilizing	γ_{dG} = **1.00**
Imposed load factor - destabilizing	γ_{dQ} = **1.00**
Wind load factor - destabilizing	γ_{dW} = **1.00**

TEKLA Tedds

Project				Job no.	
Calcs for				Start page no./Revision 44	
Calcs by HN	Calcs date 6/09/2019	Checked by	Checked date	Approved by	Approved date

Axial loading on column

Dead axial load on column — P_{GA} = **126.0** kN

Imposed axial load on column — P_{QA} = **0.0** kN

Wind axial load on column — P_{WA} = **0.0** kN

Total axial load on column — $P_A = \gamma_{dG} \times P_{GA} + \gamma_{dQ} \times P_{QA} + \gamma_{dW} \times P_{WA}$ = **126.0** kN

Foundation loads

Dead surcharge load — F_{Gsur} = **0.000** kN/m^2

Imposed surcharge load — F_{Qsur} = **0.000** kN/m^2

Pad footing self weight — $F_{swt} = h \times \rho_{conc}$ = **12.000** kN/m^2

Soil self weight — $F_{soil} = h_{soil} \times \rho_{soil}$ = **0.000** kN/m^2

Total foundation load — $F = [\gamma_{dG} \times (F_{Gsur} + F_{swt} + F_{soil}) + \gamma_{dQ} \times F_{Qsur}] \times A$ = **17.3** kN

Calculate pad base reaction

Total base reaction — $T = F + P_A$ = **143.3** kN

Eccentricity of base reaction in x — $e_{Tx} = (P_A \times e_{PxA} + M_{xA} + H_{xA} \times h) / T$ = **264** mm

Eccentricity of base reaction in y — $e_{Ty} = (P_A \times e_{PyA} + M_{yA} + H_{yA} \times h) / T$ = **0** mm

Check pad base reaction eccentricity

$abs(e_{Tx}) / L + abs(e_{Ty}) / B$ = **0.220**

Base reaction acts outside of middle third of base

Calculate pad base pressures

q_1 = **0.000** kN/m^2

q_2 = **0.000** kN/m^2

$q_3 = 2 \times T / [3 \times B \times (L / 2 - abs(e_{Tx}))]$ = **236.777** kN/m^2

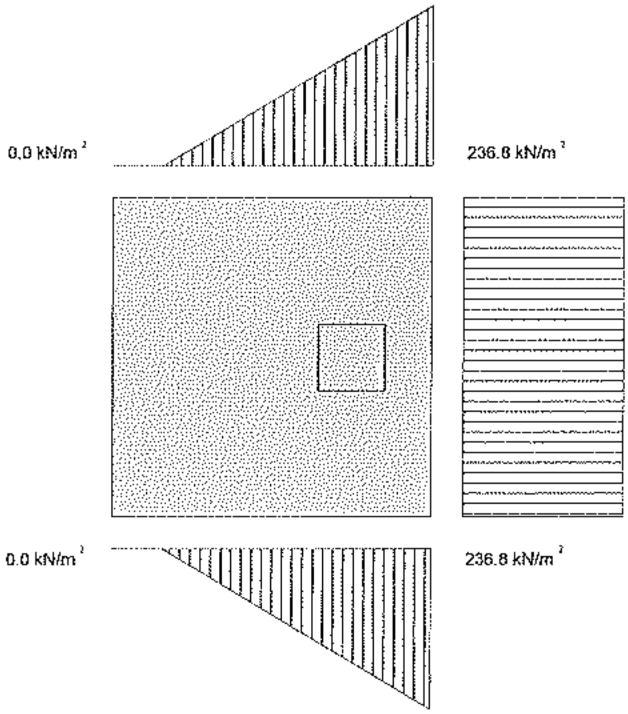
$q_4 = 2 \times T / [3 \times B \times (L / 2 - abs(e_{Tx}))]$ = **236.777** kN/m^2

Minimum base pressure — $q_{min} = min(q_1, q_2, q_3, q_4)$ = **0.000** kN/m^2

Maximum base pressure — $q_{max} = max(q_1, q_2, q_3, q_4)$ = **236.777** kN/m^2

Factor of safety for base pressure — $F_{sb} = P_{bearing} / q_{max}$ = **1.056**

PASS - Maximum base pressure is less than ultimate design bearing capacity



9

Load factors for design

Dead load factor $\gamma_{fG} = \mathbf{1.0}$

Imposed load factor $\gamma_{fQ} = \mathbf{1.0}$

Wind load factor $\gamma_{fW} = \mathbf{0.0}$

Ultimate axial loading on column

Ultimate axial load on column $P_{uA} = \gamma_{fG} \times P_{GA} + \gamma_{fQ} \times P_{QA} + \gamma_{fW} \times P_{WA} = \mathbf{126.0}$ kN

Ultimate foundation loads

Ultimate foundation load $F_u = [\gamma_{fG} \times (F_{Gsur} + F_{swt} + F_{soil}) + \gamma_{fQ} \times F_{Qsur}] \times A = \mathbf{17.3}$ kN

Ultimate horizontal loading on column

Ultimate horizontal load in x direction $H_{xuA} = \gamma_{fG} \times H_{GxA} + \gamma_{fQ} \times H_{QxA} + \gamma_{fW} \times H_{WxA} = \mathbf{0.0}$ kN

Ultimate horizontal load in y direction $H_{yuA} = \gamma_{fG} \times H_{GyA} + \gamma_{fQ} \times H_{QyA} + \gamma_{fW} \times H_{WyA} = \mathbf{0.0}$ kN

Ultimate moment on column

Ultimate moment on column in x direction $M_{xuA} = \gamma_{fG} \times M_{GxA} + \gamma_{fQ} \times M_{QxA} + \gamma_{fW} \times M_{WxA} = \mathbf{0.000}$ kNm

Ultimate moment on column in y direction $M_{yuA} = \gamma_{fG} \times M_{GyA} + \gamma_{fQ} \times M_{QyA} + \gamma_{fW} \times M_{WyA} = \mathbf{0.000}$ kNm

Calculate ultimate pad base reaction

Ultimate base reaction $T_u = F_u + P_{uA} = \mathbf{143.3}$ kN

Eccentricity of ultimate base reaction in x $e_{Txu} = (P_{uA} \times e_{PxA} + M_{xuA} + H_{xuA} \times h) / T_u = \mathbf{264}$ mm

Eccentricity of ultimate base reaction in y $e_{Tyu} = (P_{uA} \times e_{PyA} + M_{yuA} + H_{yuA} \times h) / T_u = \mathbf{0}$ mm

Calculate ultimate pad base pressures

$q_{1u} = \mathbf{0.000}$ kN/m²

$q_{2u} = \mathbf{0.000}$ kN/m²

$q_{3u} = 2 \times T_u / [3 \times B \times (L / 2 - abs(e_{Txu}))] = \mathbf{236.777}$ kN/m²

$q_{4u} = 2 \times T_u / [3 \times B \times (L / 2 - abs(e_{Txu}))] = \mathbf{236.777}$ kN/m²

Minimum ultimate base pressure $q_{minu} = min(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = \mathbf{0.000}$ kN/m²

Maximum ultimate base pressure $q_{maxu} = max(q_{1u}, q_{2u}, q_{3u}, q_{4u}) = \mathbf{236.777}$ kN/m²

Calculate rate of change of base pressure in x direction

Left hand base reaction $f_{uL} = (q_{1u} + q_{2u}) \times B / 2 = \mathbf{0.000}$ kN/m

Right hand base reaction $f_{uR} = (q_{3u} + q_{4u}) \times B / 2 = \mathbf{284.133}$ kN/m

Length of base reaction $L_x = 3 \times (L / 2 - e_{Txu}) = \mathbf{1009}$ mm

Rate of change of base pressure $C_x = (f_{uR} - f_{uL}) / L_x = \mathbf{281.726}$ kN/m/m

Calculate pad lengths in x direction

Left hand length $L_L = L / 2 + e_{PxA} = \mathbf{900}$ mm

Right hand length $L_R = L / 2 - e_{PxA} = \mathbf{300}$ mm

Calculate ultimate moments in x direction

Ultimate moment in x direction $M_x = C_x \times (L_L - L + L_x)^3 / 6 - F_u \times L_L^2 / (2 \times L) = \mathbf{10.870}$ kNm

Calculate rate of change of base pressure in y direction

Top edge base reaction $f_{uT} = (q_{2u} + q_{4u}) \times L / 2 = \mathbf{142.066}$ kN/m

Bottom edge base reaction $f_{uB} = (q_{1u} + q_{3u}) \times L / 2 = \mathbf{142.066}$ kN/m

Length of base reaction $L_y = B = \mathbf{1200}$ mm

Rate of change of base pressure $C_y = (f_{uB} - f_{uT}) / L_y = \mathbf{0.000}$ kN/m/m

Calculate pad lengths in y direction

Top length $L_T = B / 2 - e_{PyA} = \mathbf{600}$ mm

Bottom length $L_B = B / 2 + e_{PyA}$ = **600** mm

Calculate ultimate moments in y direction

Ultimate moment in y direction $M_y = f_{uT} \times L_T^2 / 2 + C_y \times L_T^3 / 6 - F_u \times L_T^2 / (2 \times B)$ = **22.980** kNm

Material details

Characteristic strength of concrete f'_c = **25** MPa

Characteristic strength of reinforcement f_{sy} = **300** MPa

Characteristic strength of shear reinforcement f_{vy} = **250** MPa

Nominal cover to reinforcement c_{nom} = **50** mm

Moment design in x direction

Diameter of tension reinforcement ϕ_{xB} = **16** mm

Number of tension bars N_{xB} = **7**

Tension reinforcement provided **7 No. 16 dia. bars bottom (175 centres)**

Area of tension reinforcement provided $A_{s\_xB\_prov} = N_{xB} \times \pi \times \phi_{xB}^2 / 4$ = **1407** mm^2

Depth of tension reinforcement $d_x = h - c_{nom} - \phi_{yB} - \phi_{xB} / 2$ = **426** mm

Min area of tension reinforcement (cl. 16.3.1) $A_{s\_x\_min} = 0.114 \text{ MPa} \times B \times h^2 \times \sqrt{(f'_c / 1 \text{ MPa})} / (f_{sy} \times d_x)$ = **1338** mm^2

PASS - Tension reinforcement provided exceeds minimum tension reinforcement required

Compressive stress factor $\alpha_2 = \max(0.67, \min(0.85, 1.0 - 0.003 \times f'_c/1 \text{ MPa}))$ = **0.850**

Compression block depth factor $\gamma = \max(0.67, \min(0.85, 1.05 - 0.007 \times f'_c/1 \text{ MPa}))$ = **0.850**

Neutral axis perimeter $k_{ux} = A_{s\_xB\_prov} \times f_{sy} / (\alpha_2 \times \gamma \times f'_c \times B \times d_x)$ = **0.046**

Neutral axis perimeter is less than 0.36

Capacity reduction factor $\phi_f = \max(0.6, \min(0.8, 1.19 - 13 \times k_{ux} / 12))$ = **0.80**

Design bending moment $M^*_x = M_x$ = **10.870** kNm

Moment capacity of base $M_{capx} = \phi_f \times A_{s\_xB\_prov} \times f_{sy} \times [d_x - (A_{s\_xB\_prov} \times f_{sy} / (2 \times \alpha_2 \times f'_c \times B))]$

M_{capx} = **141.099** kNm

PASS - Moment capacity of base exceeds nominal moment strength required

Moment design in y direction

Diameter of tension reinforcement ϕ_{yB} = **16** mm

Number of tension bars N_{yB} = **7**

Tension reinforcement provided **7 No. 16 dia. bars bottom (175 centres)**

Area of tension reinforcement provided $A_{s\_yB\_prov} = N_{yB} \times \pi \times \phi_{yB}^2 / 4$ = **1407** mm^2

Depth of tension reinforcement $d_y = h - c_{nom} - \phi_{yB} / 2$ = **442** mm

Min area of tension reinforcement (cl. 16.3.1) $A_{s\_y\_min} = 0.114 \text{ MPa} \times L \times h^2 \times \sqrt{(f'_c / 1 \text{ MPa})} / (f_{sy} \times d_y)$ = **1290** mm^2

PASS - Tension reinforcement provided exceeds minimum tension reinforcement required

Compressive stress factor $\alpha_2 = \max(0.67, \min(0.85, 1.0 - 0.003 \times f'_c/1 \text{ MPa}))$ = **0.850**

Compression block depth factor $\gamma = \max(0.67, \min(0.85, 1.05 - 0.007 \times f'_c/1 \text{ MPa}))$ = **0.850**

Neutral axis perimeter $k_{uy} = A_{s\_yB\_prov} \times f_{sy} / (\alpha_2 \times \gamma \times f'_c \times L \times d_y)$ = **0.044**

Neutral axis perimeter is less than 0.36

Capacity reduction factor $\phi_f = \max(0.6, \min(0.8, 1.19 - 13 \times k_{uy} / 12))$ = **0.80**

Design bending moment $M^*_y = M_y$ = **22.980** kNm

Moment capacity of base $M_{capy} = \phi_f \times A_{s\_yB\_prov} \times f_{sy} \times [d_y - (A_{s\_yB\_prov} \times f_{sy} / (2 \times \alpha_2 \times f'_c \times L))]$

M_{capy} = **146.504** kNm

PASS - Moment capacity of base exceeds nominal moment strength required

Calculate ultimate shear force at d from top face of column

Ultimate pressure for shear $q_{su} = (q_{2u} + C_y \times (B / 2 - e_{PyA} - b_A / 2 - d_y) / L + q_{4u}) / 2$

q_{su} = **118.389** kN/m^2

Area loaded for shear $A_s = 3 \times (L / 2 - e_{Tx}) \times (B / 2 - e_{PyA} - b_A / 2 - d_y) = \mathbf{0.033}$ m²

Ultimate shear force $V_{su} = A_s \times (q_{su} - F_u / A) = \mathbf{3.541}$ kN

Shear design at d from top face of column

Strength reduction factor in shear $\phi_s = \mathbf{0.70}$

Design shear strength $V^*_{su} = V_{su} = \mathbf{3.541}$ kN

Concrete shear factors $\beta_1 = \max(1.1 \times (1.6 - d_y / 1000 \text{ mm}), 0.8) = \mathbf{1.274}$

$\beta_2 = 1$

$\beta_3 = 1$

Concrete shear strength $V_{uc} = \phi_s \times \beta_1 \times \beta_2 \times \beta_3 \times L \times d_y \times \min(f'_c{}^{1/3} \times (1 \text{ MPa})^{2/3}, 4 \text{ MPa}) \times [A_{s\_yB\_prov} / (L \times d_y)]^{1/3}$

$V_{uc} = \mathbf{191.451}$ kN

PASS - Design shear strength is less than concrete shear strength

Calculate ultimate punching shear force at perimeter of d / 2 from face of column

Ultimate pressure for punching shear $q_{puA} = q_{4u}\text{-}[(L/2-e_{PxA}+l_A/2+d/2)/2]\times C_x/B+[(B/2-e_{PyA}-b_A/2-d/2)+(b_A+2\times d/2)/2]\times C_y/L$

$q_{puA} = \mathbf{161.416}$ kN/m²

Average effective depth of reinforcement $d = (d_x + d_y) / 2 = \mathbf{434}$ mm

Area loaded for punching shear at column $A_{pA} = (L/2-e_{PxA}+l_A/2+d/2)\times(b_A+2\times d/2) = \mathbf{0.439}$ m²

Length of punching shear perimeter $u_{pA} = 2\times(L/2-e_{PxA}+l_A/2+d/2)+(b_A+2\times d/2) = \mathbf{1968}$ mm

Ultimate shear force at shear perimeter $V_{puA} = P_{uA} + (F_u / A - q_{puA}) \times A_{pA} = \mathbf{60.387}$ kN

Punching shear stresses at perimeter of d / 2 from face of column

Strength reduction factor in shear $\phi_s = \mathbf{0.70}$

Design shear strength $V^*_{puA} = V_{puA} = \mathbf{60.387}$ kN

Ratio of column long side to short side $\beta_{hA} = \max(l_A, b_A) / \min(l_A, b_A) = \mathbf{1.000}$

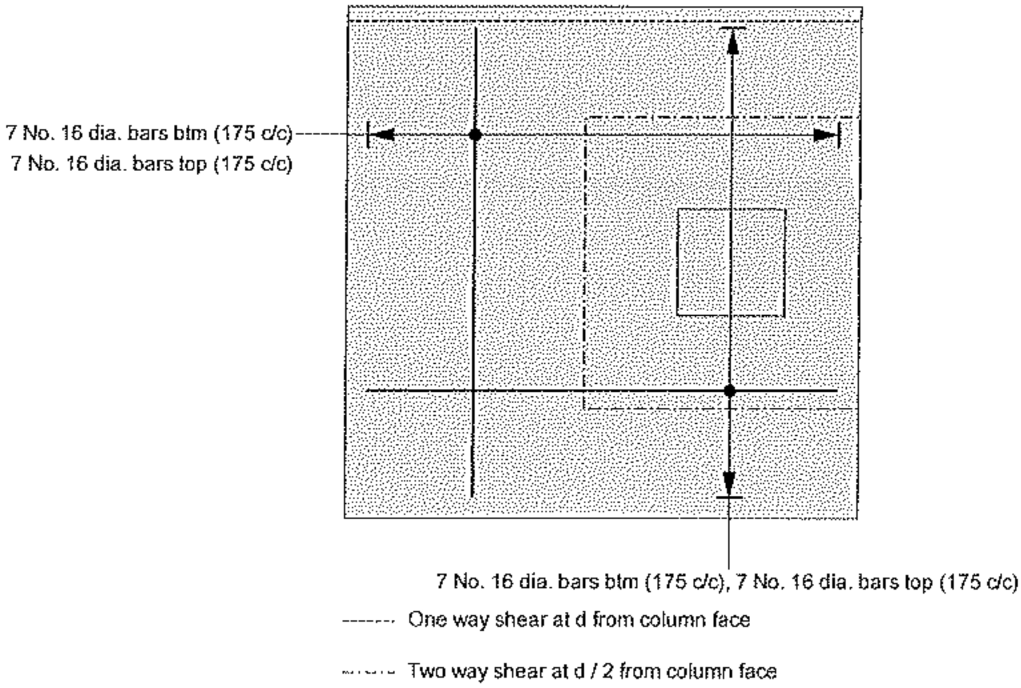
$f_{cv} = \min([1 + 2 / \beta_{hA}], 2) \times 0.17 \text{ MPa} \times \sqrt{(f'_c / 1 \text{ MPa})} = \mathbf{1.700}$ MPa

Concrete shear strength with no moment $V_{uo} = \phi_s \times u_{pA} \times d \times f_{cv} = \mathbf{1016.393}$ kN

Torsion strip width $a = b_A + 2 \times (d/2) = \mathbf{684}$ mm

Concrete shear strength $V_u = V_{uo} / [1 + (u_{pA} \times M^*_y / (8 \times V^*_{puA} \times a \times d))] = \mathbf{772.717}$ kN

PASS - Design shear strength is less than concrete shear strength



7 No. 16 dia. bars btm (175 c/c)
7 No. 16 dia. bars top (175 c/c)
7 No. 16 dia. bars btm (175 c/c), 7 No. 16 dia. bars top (175 c/c)
One way shear at d from column face
Two way shear at d / 2 from column face

AMENDED DESIGN

INFO FROM CLIENT

+ CONFIRMED LOCATION OF GRD FLOOR PORTAL FRAME
+ APPROVAL OF 1ST FLOOR / ROOF STRENGTHENING.

ACTION FOR BUILDING CONSENT

1. DESIGN PORTAL FOR NEW ARRANGEMENT.
2. CONNECTION DESIGN FOR 1ST F & ROOF X-BRACE

SILVESTER/CLARK
CONSULTING ENGINEERS

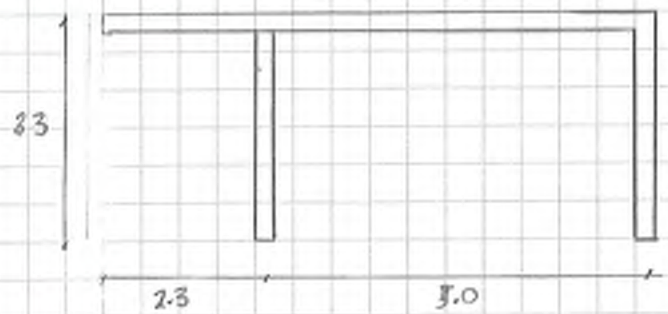
PAGE No AC 2 JOB No 23028

BY MS. DATE 11/12/19

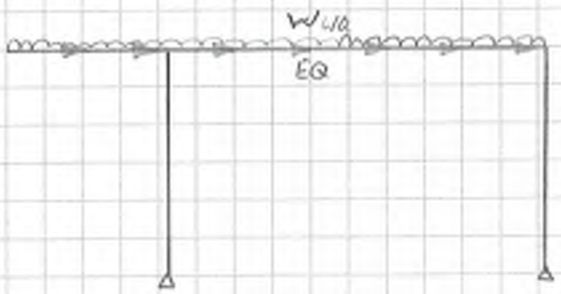
JOB 11 JOHNSONVILLE RD

PORTAL FRAME DESIGN

PORTAL GEOMETRY



PORTAL LOADING



Nom. G = 0.6kPa TW = 2.8m

∴ W_G = 1.70 kN/m

Q: 1.5kPa TW: 2.8m

W_Q = 4.20 kN/m

EQ: 66kN

APPLY AS UDL ALONG BEAM

∴ W_{EQ} = 9.04kN

DESIGN COMBOS/LIMIT

ULS LOAD CASES ~ $1.2G + 0.3Q + EQ^{+/-}$

AND STIFFNESS REQ 2.5% DEFLECTION

PORTAL FRAME DESIGN.

MEMBER DESIGN.

TRY 300PFC FOR BEAM/COLUMN.

STIFFNESS CHECK, OK.
SEE AC4.

NOW BEAM DESIGN

$M^* = 129$ KNm SEE AC5
$V^* = 62.3$ KN SEE AC6
$N^* = 20.8$ KN SEE AC7.

DESIGN TO MEMDES SEE AC8/AC9

NOW 300PFC OK @ 96%

NOW COLUMN DESIGN.

SEISMIC PROVISION, TO ENSURE DUCTILE BEHAVIOUR APPLY OVERSTRENGTH FACTORS TO COLUMN.
THIS WILL ENSURE WEAK BEAM/STRONG COLUMN MECHANISM

CONSIDER. $\mu = 1.25$ ~ CATEGORY 3 STRUCTURE

COLUMNS ~ CATEGORY 2 MEMBERS.

$\therefore$ $\phi_{os} = 1.05$.
$\phi_{om} = 1.30$.
$\phi_{os} = 1.25$.
SO $\phi = 1.71$

$\therefore$ $M^*_{\phi} = 192$ KNm SEE AC5.
$V^* = 34$ KN SEE AC6
$N^* = 76.6$ KN. SEE AC7/[illegible]

NOW. 300PFC NG.

380PFC OK @ 81%
SEE AC10/11

AC4

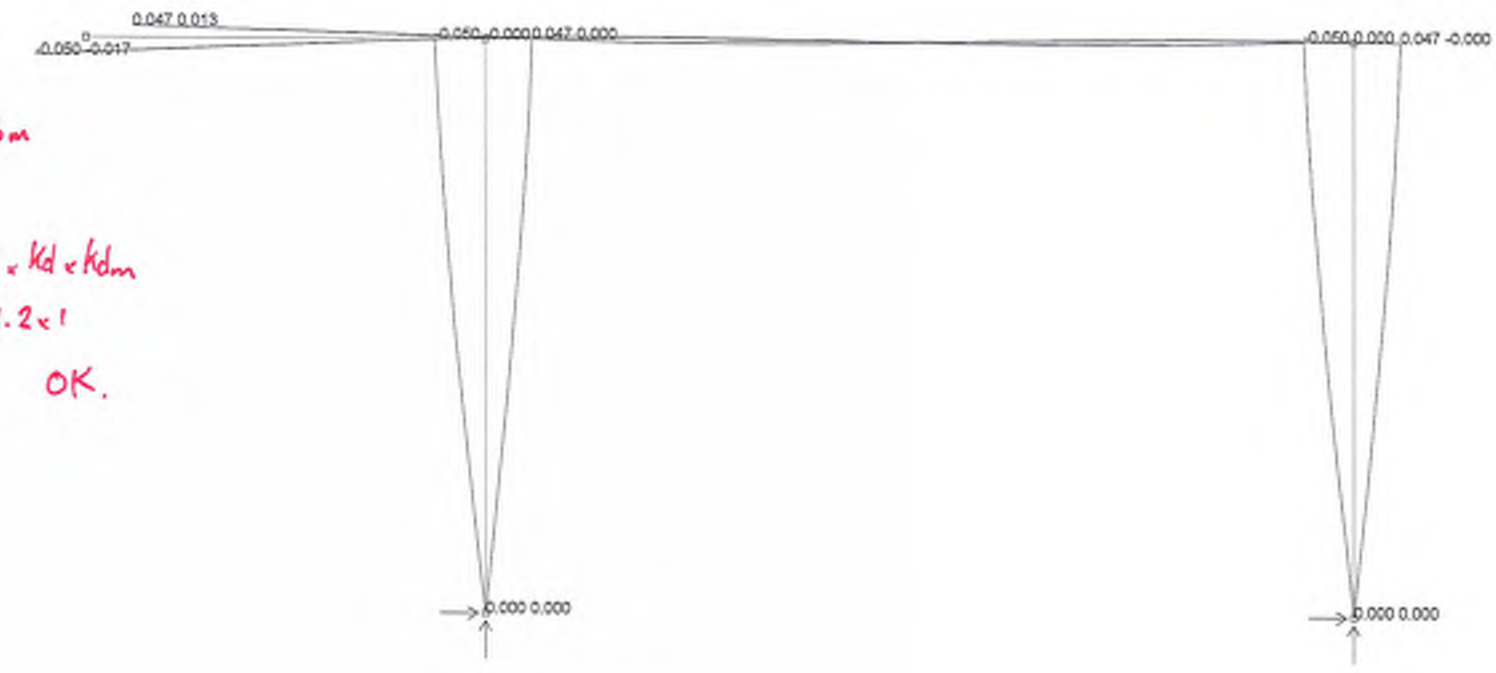
Microstran V9

Murphy
Job: Frame Gridline 1 - For Client Preferred Arrangement

Transverse Internal Frame

11-Dec-19
10:27:16 AM

Load Cases:
- 11 C G+Q+Eu
- 12 C G+Q-Eu



Δ_{lim} = 2.5% of 3.3m
= 82.5mm

Δ_{DCS} = 50mm × μ × Kd × Kdm
= 50 × 1.25 × 1.2 × 1
= 75mm OK.

Y
X
theta: 270 phi: -2

ULS Deformed Shape

Displaced Shape

AC5

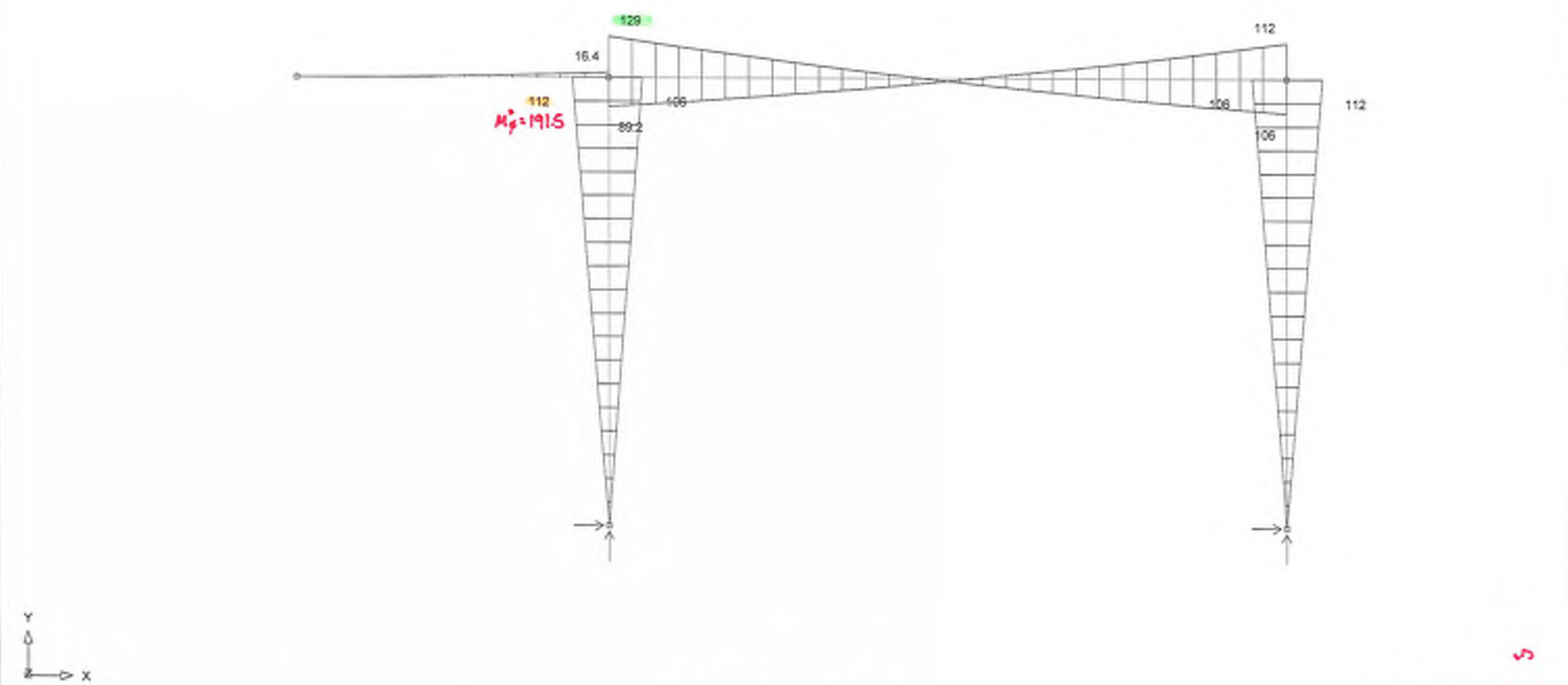
Microstran V9

Murphy
Job: Frame_Gridline 1 - For Client Preferred Arrangement

Transverse Internal Frame

11-Dec-19
10:32:33 AM

Load Cases:
11 C G+Q+Eu
12 C G+Q-Eu



ULS Bending Moment Diagram

Bending Moment, Mz

Microstran.Advanced V9.20.1.24

J:\23028 11 Johnsonville Road DSA and Strengthening Scheme\Engineering\Calculations\Frame_Gridline 1 - For Client Preferred Arrangement.msw

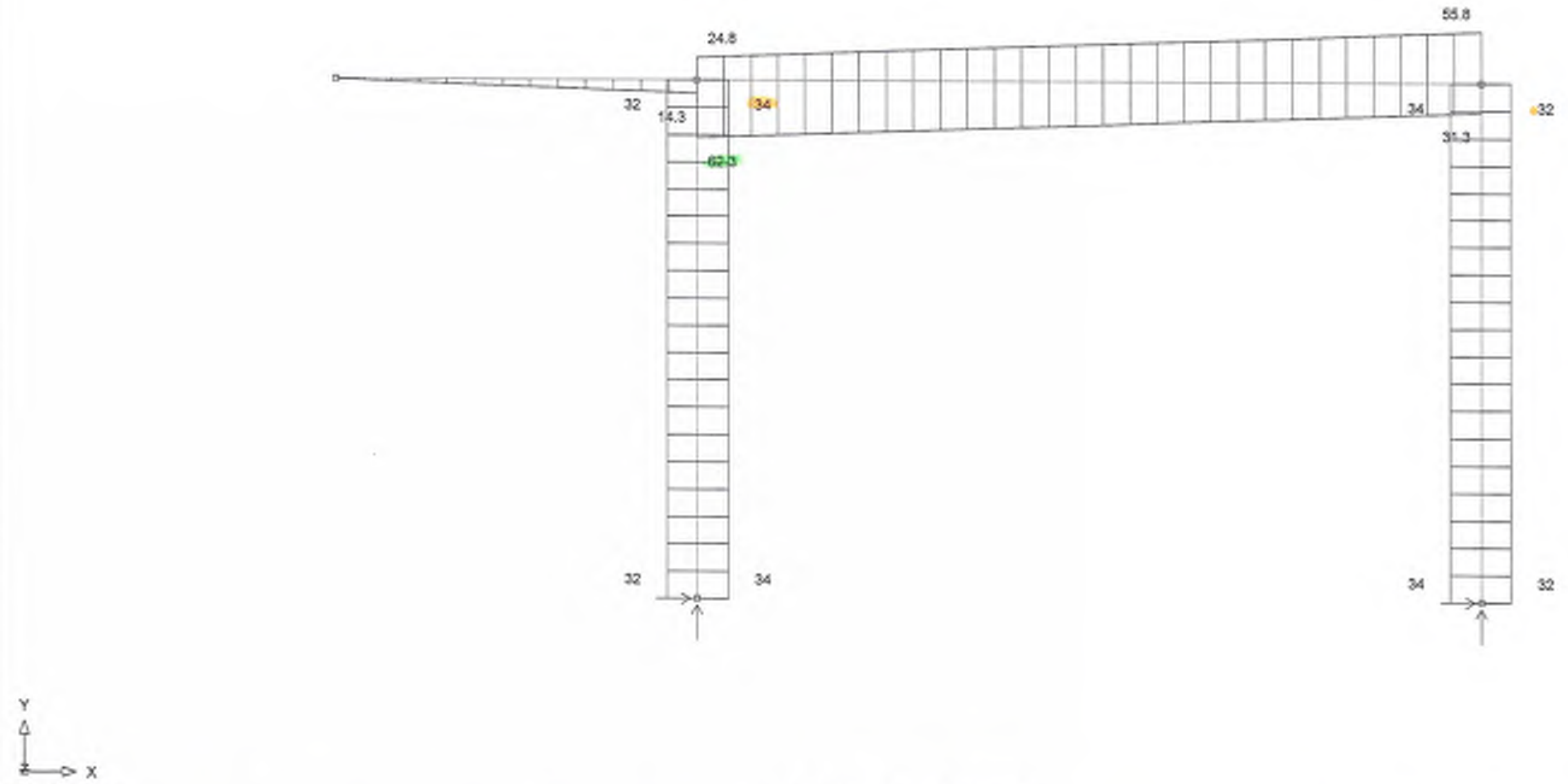
ACG

Microstran V9

Murphy
Job: Frame_Gridline 1 - For Client Preferred Arrangement
21-27 Dudley Street
Transverse Internal Frame

11-Dec-19
10:33:47 AM

Load Cases:
11 C G+Q+Eu
12 C G+Q-Eu



ULS Shear Force Diagram

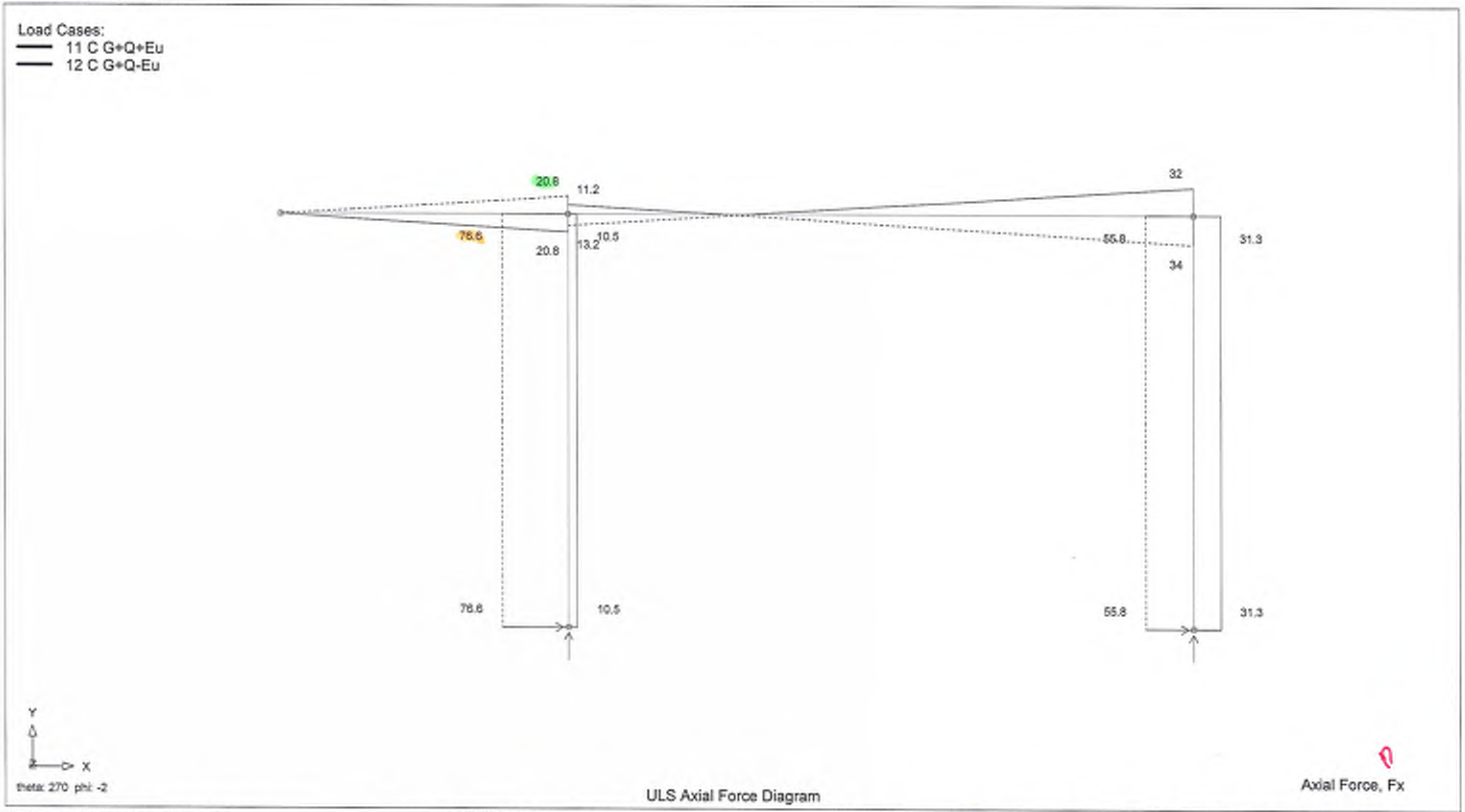
Shear Force, Fy

AC7

Microstran V9

Murphy
Job: Frame_Gridline 1 - For Client Preferred Arrangement
21-27 Dudley Street
Transverse Internal Frame

11-Dec-19
10:33:59 AM



ULS Axial Force Diagram

Microstran.Advanced V9.20.1.24

J:\23028 11 Johnsonville Road DSA and Strengthening Scheme\Engineering\Calculations\Frame_Gridline 1 - For Client Preferred Arrangement.msw

AC8

MemDes Calculations @ 10:37:03 11-12-2019 by MS

Project : 11 Johnsonville R - Seismic Strengthenin
Description : Portal Frame - Beam

Section : 300PFC Grade 300+
$d = 300$ mm $b = 90$ mm $t_f = 16.0$ mm $t_w = 8.0$ mm
Area = 5110 mm^2 $f_{yf} = 300$ MPa $f_{yw} = 320$ MPa $f_u = 440$ MPa

Member Effective Length Calcs
Eff. Len L_e = 5.00 * 1.12 * 1.00 * 1.00 = 5.60m

Major Axis Bending
Design Action M^*_x = 129.0 kNm
Section Bending Capacity $M_{sx} = f_{yf} Z_{ex}$ = 169.20 kNm
$\alpha_m = 1.7 * 129.0 / [(70.0)^2 + (12.0)^2 + (-47.0)^2]^{0.5} = 2.500$
Reference Buckling Moment Calculation : M_o
M_o = 83.8 kNm (Eqtn 5.6.1.1(4))
$\alpha_s = 0.38$
α_m α_s < 1.0, => Segment NOT Fully Restrained
M_{bx} = 2.50 * 0.38 * 169.2 = 162.7
Major axis capacity Ratio = M^*_x / ϕ M_{bx}
= 0.88, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V^*_x = 62.0 kN
Nominal Shear Yield capacity V_w = 460.8 kN
$\alpha_v = 4.17 >= 1.0$ => full web shear capacity
$V_u = V_w$ = 460.8 kN
Mom-Shear Interaction Check : Moment ratio > 0.75 => reduced shear capacity available
Shear capacity ratio = V^*_x / ϕ V_{vm}
= 0.18, ---- OK ----

Axial Calculations
Design Action N_d = 20.8 kN [Comp], LeAxx = 5.00 m, LeAxy = 5.00 m
Sect. Compression Capacity $N_s = k_f A_n f_{ycomb}$
= 1577.6 kN
Major axis buckling : $\alpha_c = 0.8272$
$\alpha_{cx} < 1.0$ => $N_{cx} = \alpha_{cx} N_s = 1305.0$
Minor axis buckling : $\alpha_c = 0.1717$
$\alpha_{cy} < 1.0$ => $N_{cy} = \alpha_{cy} N_s = 270.9$
Minimum Capac. $N_c$min = 270.9
Axial buckling capac. Ratio = N_d / ϕ $N_c$min
= 0.085, ---- OK ----

Combined Actions Checks
Significant Axial Load test (Cl. 8.1.4(a)) NOT Satisfied **** NOK ****
Significant Axial Load test (Cl. 8.1.4(b) NOT Satisfied **** NOK ****

10

AC9

+ Check Alternative Provisions, (Clause 8.1.5)
Alt. Prov. Cl 8.1.5 (a)-(c) NOT OK for Minor axis, NOT OK for Major axis actions
β_{mox} = 0.82

Clause 8.3.2/4 :
$M_{rx} = M_{sx}$ $(1 - N^* / \phi N_s)$ =< M_{sx}, [Alt. Prov. NOK]
= 166.7

Clause 8.3.3/4 :
$M_{ry} = M_{sy}$ $(1 - (N^* / \phi N_s))$ =< M_{sy} [Alt. Prov. NOK]
= 24.3
Load / Capacity Ratio = $M^*_x / (0.9 M^r_x)$
= 0.86, ---- OK ----

Clause 8.4.2.2 : Major : $M_{ix} = M_{sx}$ $(1 - N^* / \phi N_{cx})$ =< M_{rx}
= 166.2
Load / Capacity Ratio = $M^*_m / \phi M_i$
= 0.862 ---- OK ----

Clause 8.4.4.1 :
$M_{ox} = M_{bx}$ $(1 - N^* / \phi Ncy)$ =< M_{rx}
= 148.9
Load / Capacity Ratio = $M^*_x / \phi M_{ox}$,
= 0.963, ---- OK ----

============================ SUMMARY ======================

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.96 ---- OK ----

==

SESOC MemDes v 3.8.1 : Calculations by MS
Project : 11 Johnsonville R - Seismic Strengthenin at 10:37:03 AM on 11-Dec-19
Description : Portal Frame - Beam

10

AC10

MemDes Calculations @ 10:48:47 11-12-2019 by MS

==

Project : 11 Johnsonville R - Seismic Strengthenin
Description : Portal Frame - Column

Section : 380PFC Grade 300+
$d = 380$ mm $b = 100$ mm $t_f = 17.5$ mm $t_w = 10.0$ mm
Area = 7030 mm^2 $f_{yf} = 280$ MPa $f_{yw} = 320$ MPa $f_u = 440$ MPa

Member Effective Length Calcs
Eff. Len L_e = 3.30 * 1.15 * 1.00 * 1.00 = 3.81m

Major Axis Bending
Design Action M^*_x = 192.0 kNm
Section Bending Capacity $M_{sx} = f_{yf} Z_{ex}$ = 264.88 kNm
$\alpha_m = 1.7 * 192.0 / [(144.0)^2 + (96.0)^2 + (48.0)^2]^{0.5} = 1.817$
Reference Buckling Moment Calculation : M_o
M_o = 230.5 kNm (Eqtn 5.6.1.1(4))
$\alpha_s = 0.56$
α_m α_s >= 1.0, => Segment Fully Restrained
$M_{bx} = M_{sx}$ = 264.88 kNm
Major axis capacity Ratio = M^*_x / ϕ M_{bx}
= 0.81, ---- OK ----

Shear Calculations (Unstiffened Web)
Design Action V^*_x = 34.0 kN
Nominal Shear Yield capacity V_w = 729.6 kN
α_v = 4.00 >= 1.0 => full web shear capacity
$V_u = V_w$ = 729.6 kN
Mom-Shear Interaction Check : Moment ratio > 0.75 => reduced shear capacity available
Shear capacity ratio = V^*_x / ϕ V_{vm}
= 0.06, ---- OK ----

Axial Calculations
Design Action N_d = 76.6 kN [Comp], LeAxx = 3.30 m, LeAxy = 3.30 m
Sect. Compression Capacity $N_s = k_f A_n f_{ycomb}$
= 2109.6 kN
Major axis buckling : $\alpha_c = 0.9444$
$\alpha_{cx} < 1.0 \Rightarrow N_{cx} = \alpha_{cx} N_s = 1992.3$
Minor axis buckling : $\alpha_c = 0.3873$
$\alpha_{cy} < 1.0 \Rightarrow N_{cy} = \alpha_{cy} N_s = 817.1$
Minimum Capac. $N_c$min = 817.1
Axial buckling capac. Ratio = N_d / ϕ $N_c$min
= 0.104, ---- OK ----

Combined Actions Checks
Significant Axial Load test (Cl. 8.1.4(a)) IS Satisfied ---- OK ----
Loading PASSES Cl 8.1.4, => Combined Actions Checks are not required
==================== SUMMARY ====================

13

ACll

**** U.L.S. Capacity Check Passed, Load Cap. Ratio = 0.81 ---- OK ----

==

SESOC MemDes v 3.8.1 : Calculations by MS
Project : 11 Johnsonville R - Seismic Strengthenin at 10:48:47 AM on 11-Dec-19
Description : Portal Frame - Column

PORTAL FRAME DESIGN

CONNECTION DESIGN

FOR KNEE JOINT HAVE FULLY WELDED STIFFNERS TO LINE UP FLANGES. STIFFENERS TO BE SAME THICKNESS AS CORRESPONDING BEAM / COLUMN.

BASE CONNECTION - SEE AC 15.

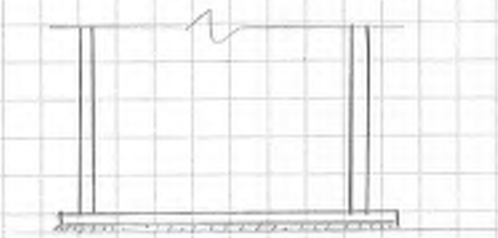
DOWNWARD FORCE ; $N^* = 76.6$ KN.

UPLIFT FORCE ; $N^* = 31.3$ KN.

BASE SHEAR FORCE ; $V^* = 34$ KN

DUE TO NEW ARRANGEMENT LOADS HAVE CHANGE AND PREVIOUS DESIGN NOT APPLICABLE.

NOW CONSIDER



SAY TWO BOLTS.

$\therefore \quad N_t^* = 15.7$ KN $\quad V^* = 17$ KN.

TRY M16 ; $\phi N_t = \phi A_s f_{uf}$ GRADE 4.6S

$= 0.8?? \times 16^2 \times 0.25 \times 400$

$= 64$ KN

$\phi V = \phi\, 0.62 \times f_{uf} (A_c \times n)$

$= 0.8 \times 0.62 \times 400 (\pi \times 16^2 \times 0.25 \times 1)$

$= 40$ KN

CHECK COMBINED $\left(\frac{V^*}{\phi V}\right)^2 + \left(\frac{N^*}{\phi N}\right)^2 = 0.24 < 1$ OK

PORTAL FRAME DESIGN

CONNECTION DESIGN

PLATE DESIGN.

$N^* = \sqrt{V^{*2} + N^{*2}}$

$w_f = 17.5\,mm$
gauge $= 1.5d + 10$
$= 35\,mm$

$l = 380 - 17.5 \times 2 - 35 \times 2$
$= 275\,mm.$

$M^* = \frac{\sqrt{15.7^2 + 17^2}}{2} \times 275/2$

$= 1.59\,KNm$

TRY 20mm PLATE MS

$\phi M_p = 0.9 \times 250MPa \times 110mm \times \frac{20^2 mm}{6}$

$= 1.63\,KNm$ ∴ OK.

FOOTING DESIGN.

PREVIOUS SET OF CALC'S OK, USED $N^* = 126KN$

∴ OK FOR 76.6KN. ↳ Pg 51-54

ENSURE HOLD DOWN CAN BE ACHIEVED

USE WEIGHT OF COLUMN/BEAM (1st/R)

∴ COLUMN ~ 400mm SQ.

BEAM ~ 675mm × 250mm

∴ $l_c = 6.6m.$ $l_b/2 = 3.5m.$

& FOOTING ~ 1200 × 1200 × 500

SILVESTER/CLARK
CONSULTING ENGINEERS

PAGE No AC14 JOB No 23028

BY MS DATE 11/12/19

JOB 11 JOHNSONVILLE RD

PORTAL FRAME DESIGN.

CONNECTION DESIGN

CHECK OVERALL HOLD DOWN.

$W_c = 0.4m^2 \times 6.6m \times 24KN/m^3$

$W_b = 0.675m \times 0.250m \times 3.5m \times 2 \times 24KN/m^3$.

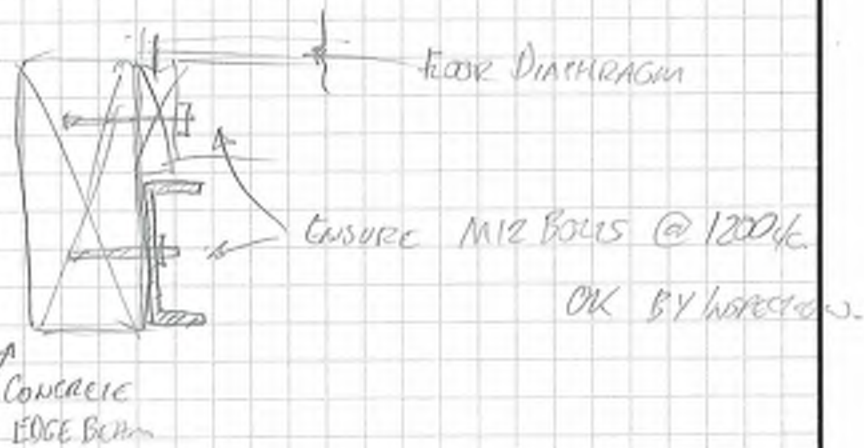
$W_f = 1.2m^2 \times 0.5m \times 24KN/m^3$

$\Sigma W = 68KN. > 31.3KN. \therefore OK.$

BEAM TO DIAPHRAGM.

BEAM LOADED 9.04KN/m

CONSIDER CROSS-SECTION.



NEED TO CHECK EDGE BEAM OF FLOORING HAS. BOLTS TO CONCRETE BEAM

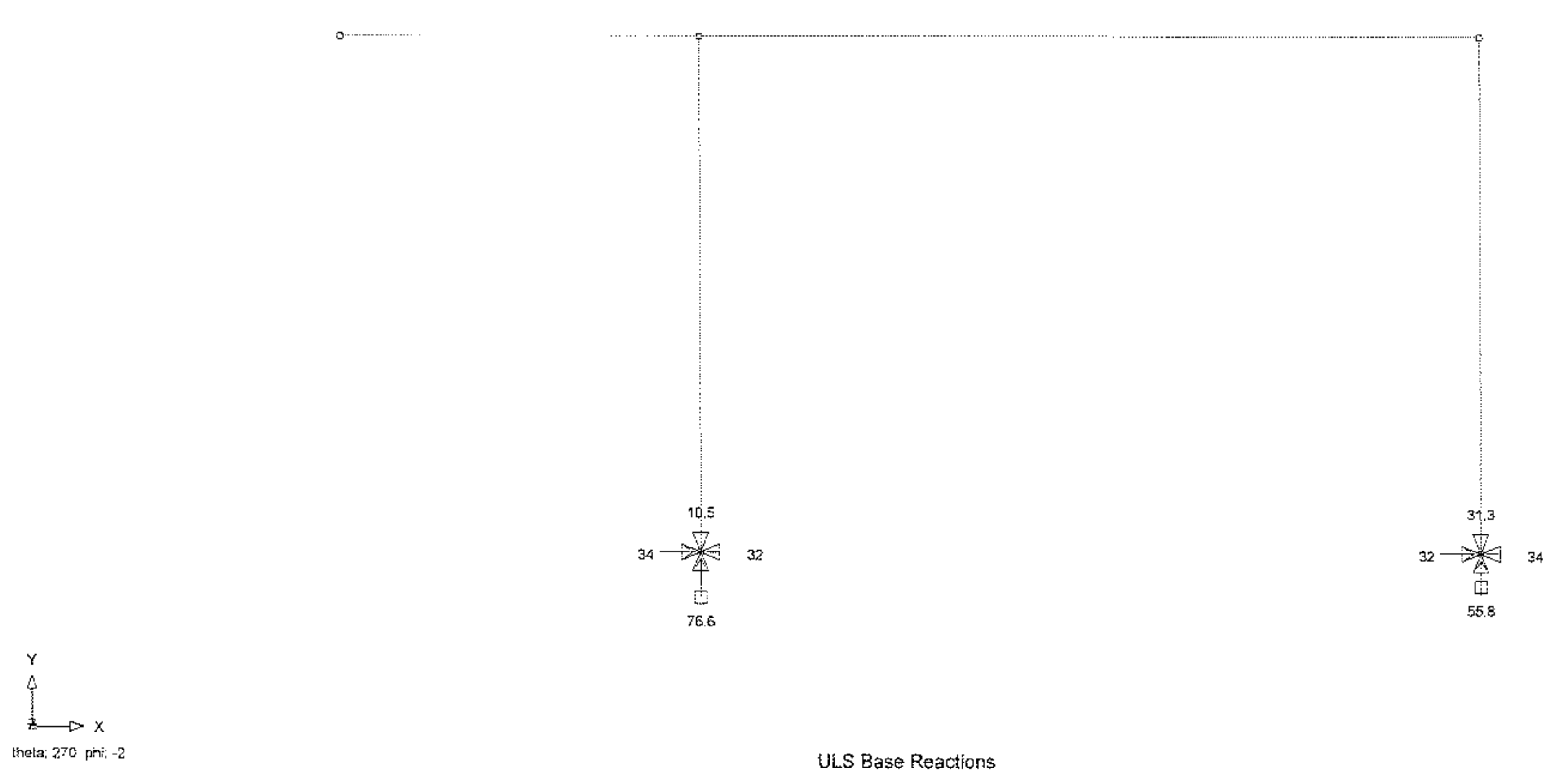
AC15

Microstran V9

Murphy
Job: Frame_Gridline 1 - For Client Preferred Arrangement
21-27 Dudley Street
Transverse Internal Frame

11-Dec-19
11:14:51 AM

Load Cases:
11 C G+Q+Eu
12 C G+Q-Eu



13

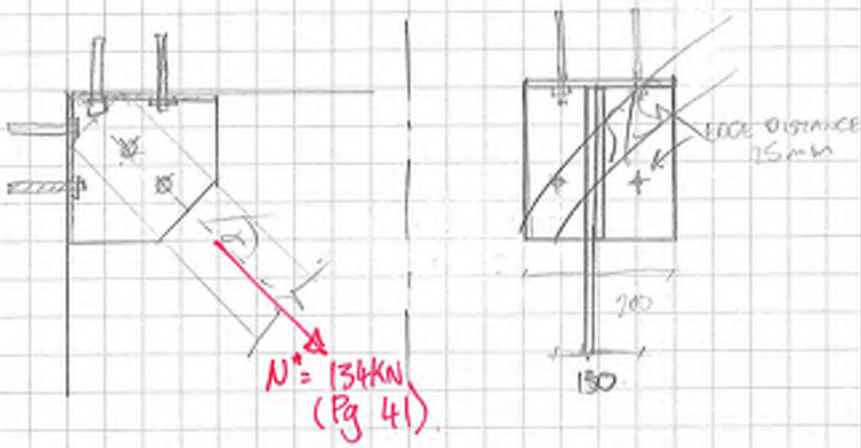
Support Reactions

ULS Base Reactions

CONNECTION DESIGN

1ST FLOOR X-BRACE

CONSIDER



NOW PLATE TO GUSSET, PREVIOUSLY DESIGNED 2/M20 BOLTS.

DESIGN GUSSET TO BASE PLATE,

CONSIDER PRINCIPLE AXIS $\alpha \approx 45°$

$\therefore \quad V_H = V_V = \sqrt{\frac{134^2}{2}}$

$= 94.7KN.$

SAY 200×200 PL

ADD. TRY 6mm FWAR. t_t?

$\therefore \quad V_w = 0.6 \times f_{uw} \times t_t \times k_r \times l_w$ $\qquad t_t = \sqrt{\frac{6^2}{2}} = 4.24mm.$

$l_w = 400mm.$

$\phi V_w = 0.6 \times 410 \times 4.24 \times 1 \times 400 \times 0.6^{\times\phi}$

$= 250KN. > 94.7KN$ ok $\qquad$ 6m FWAR OK.

CONNECTION DESIGN.

1ST FLOOR X-BRACE.

DESIGN BASE PLATE TO CONCRETE.

CONSIDER PRINCIPLE AXIS, & BOLT WORKING IN TENSION & SHEAR.

SAY 4/M16. $N_t^* = 134\,kN$.

$2\phi N_{tc} = \sqrt{4\phi N^2 + 4\phi V^2} = 150\,kN$ OK

$\phi N_{tf} = 64\,kN$
$\phi V_f = 40\,kN$.
(AS PER AC12)

NOW USE RAMSET CALCULATOR

∴ DEMAND OF SINGLE BOLT.

$$\sqrt{N^{*2} + V^{*2}} = \underset{= 150/4}{33.5\,kN} \Rightarrow V^* = N^* = 23\,kN.$$

NOW FROM RAMSET ; M16 8.8s. @ 250mm EMBED. EPCON C8.

$\phi V = 39.9\,kN$
$\phi N = 25.5\,kN$.

$$\therefore \sqrt{\phi V^2 + \phi N^2} = 47\,kN \quad \therefore OK.$$

NOTE AS PER RAMSET CALCULATOR THERE IS A COMBINED FAILURE. SEE AC18

INCREASE SIZE OF STUD. M20. SEE AC19

+ CAN REDUCE EMBEDMENT TO 200 mm

CHECK SIZE OF PLATE AND GAUGE WIDTHS.

$a_e = 30\,mm$ EDGE DISTANCE
$S = 60\,mm$. OK

2 BOLTS IN A ROW OK.

AC18

Cracked Concrete - ChemSet Anchor Stud Design Calculator

European Technical Assessment: ETA-10/0309 Design Methods: AS 5216:2018 (formerly TS101) or EN 1992-4 (formerly ETAG/TR029)

Ramset™

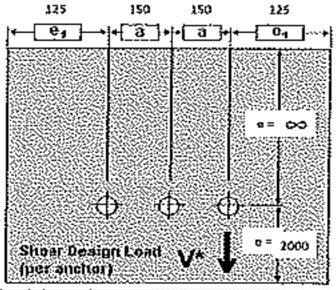
CRACKED Concrete - EPCON™ C8 Xtrem™

EPCON C8

Anchor Type: Threaded Rod - Gr 8.8 Anchor Size d_b = M 16

Input Description (Strength Limit State Design)	Input Data (per anchor)		
1. Number of anchors (n)	n =	2	-
2. Anchor Spacing (a)	a =	150	mm
3. Concrete Edge Distance (e)	e =	2000	mm
4. Concrete Cylinder Strength (f'c)	f'c =	40	MPa
5. Cracked Conc. (C) or Non-Cracked (N)	C or N	C	-
6. Effective Depth (h>6xdh)	h =	250	mm
7. Anchor Stud size (d_b) - M10 → M30	d_b =	16	mm
8. Concrete Edge Distance Corner (e1)	e_1 =	125	mm
9. Internal to a row (I) or end of row (E)	I or E	I	row
10. Dry Hole (D) or Flooded hole (F)	D or F	F	-
11. Min Concrete Sub'te Thickness (b_m)	b_m =	286	mm
12. Anchor Stud Grade (5.8, 8.8, 316 SS)	Grade =	8.8	Gr
13. Fixture Thickness (t)	t =	10	mm
14. Effective Length (L_e)	L_e =	260	mm
15. Design Tensile Load-per anchor (N*)	N* =	23	kN
16. Design Shear Load-per anchor (V*)	V* =	23	kN
17. Direction of Shear design load (α)	α =	0	°
18. Sustained Load (SUS or N/A)	SUS or N/A	N/A	-
19. Service Temperature (°C)	-40°C to	+40°C	-

Plan View - Generic
Dimensions in (mm)



Project Details

Project Name:-
Seismic Strengthening
Project Site Address:-
11 Johnsonville Road
Company Name:-
Silvester Clark Ltd.
Design Identification:-
MS
Date:-
11-12-19

ChemSet™ Anchor Stud "Specification"

Typ Gr 8.8 Thr'd Rod M16 x 285 mm long

EPCON™ C8 Xtrem™ "Specification"

Part No. C8-450

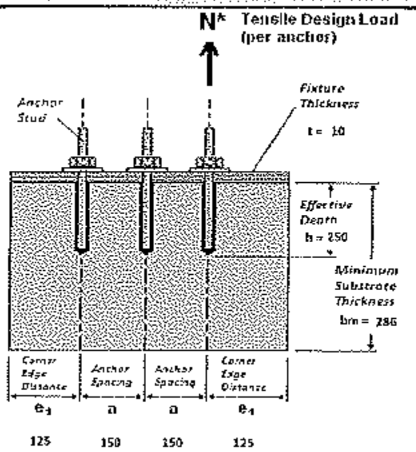
Hole Diameters (mm)	Capacity Reduction Factors	
Drill d_h = 18	Conc Tension	$1/1.8 = \phi_c = 0.56$
Fixture d_f = 18	Conc Shear	$1/1.5 = \phi_c = 0.67$
	Steel Tension	$1/1.5 = \phi_n = 0.67$
	Steel Shear	$1/1.5 = \phi_v = 0.67$

Direction of Shear Design Load - α

α = 0° - towards edge
α = 180° - away from edge

Output Description (Strength Limit State Design)	Output Data (per anchor)		
NOT O.K. - NEED TO RE-DESIGN	MIN. CRITERIA for a,e & h - O.K.		
Des.Pullout & CONC.Tensile Capacity	ϕN_{uc} =	25.5	kN
Design STEEL Tensile Capacity	ϕN_{us} =	82.1	kN
Design CONC. Edge Shear Capacity	ϕV_{uc} =	538.2	kN
Design STEEL Shear Capacity	ϕV_{us} =	50.9	kN
Design Pryout Failure	ϕV_{ucp} =	39.9	kN
Drill hole diameter	d_h =	18	mm
TENSION O.K.			
Design Tensile Capacity	ϕN_{ur} =	25.5	kN
Tension Design Check	$N^*/\phi N_{ur}$ =	0.90	< 1
SHEAR O.K.			
Design Shear Capacity	ϕV_{ur} =	39.9	kN
Shear Design Check	$V^*/\phi V_{ur}$ =	0.58	< 1
COMBINED TENSION SHEAR NOT O.K. - REDESIGN			
Combined Check - $N^*/\phi N_{ur} + V^*/\phi V_{ur}$ =		1.48	> 1.2

Elevation View - Generic
Dimensions in (mm)



Anchor Loaded

"E" Anchor end of a row

"I" Anchor internal to a row

Anchor Size	d_b	Metric	8	10	12	16	20	24	30	36
Drill hole diameter	d_h	(mm)	N/A	12	14	18	22	26	35	N/A
Stressed Area	A_s	(mm^2)	N/A	58	84	157	245	353	561	N/A
Anchor Stud Yield Strength	f_y	(MPa)	N/A	640	640	640	640	640	640	N/A
Steel Tensile Capacity	ϕN_{us}	(kN)	N/A	28.2	41.9	82.1	123.9	179.5	299.2	N/A
Steel Shear Capacity	ϕV_{us}	(kN)	N/A	17.5	26.0	50.9	76.8	111.3	185.5	N/A
Edge distance for no conc.cone reduction	e_c	(mm)	N/A	1.5xh	1.5xh	1.5xh	1.5xh	1.5xh	1.5xh	N/A
Anchor spacing for no conc.cone reduction	a_c	(mm)	N/A	3xh	3xh	3xh	3xh	3xh	3xh	N/A
Absolute Minimum edge dist. & anc'r spac.	e_m & a_m	(mm)	N/A	50	60	80	100	120	150	N/A

Effective Depth - h		Design tensile capacity ϕN_{ur} (kN per anchor) Based on edge distance (e_c) and anchor spacing (a_c) for no conc.cone reduction							
65	(mm)	N/A							
70	(mm)	N/A	8.6						
80	(mm)	N/A	9.9						
90	(mm)	N/A	11.1	12.6					
100	(mm)	N/A	12.4	14.0					
110	(mm)	N/A	13.6	15.4	18.7				
120	(mm)	N/A	14.8	16.8	20.4				
125	(mm)	N/A	15.4	17.5	21.2				
150	(mm)	N/A	18.5	21.0	25.5	32.4			
160	(mm)	N/A	19.8	22.4	27.2	34.8	37.6		
170	(mm)	N/A	21.0	23.8	28.9	36.8	41.4		
190	(mm)	N/A	23.5	26.6	32.3	41.1	48.9		
210	(mm)	N/A	25.9	29.4	35.7	45.4	54.9		
240	(mm)	N/A	28.2	33.6	40.8	51.9	62.6	69.4	
280	(mm)	N/A	28.2	39.2	47.6	60.5	73.2	84.3	
350	(mm)	N/A	28.2	41.9	59.4	75.7	91.5	105.3	
450	(mm)	N/A	28.2	41.9	76.4	97.3	117.7	135.4	N/A
550	(mm)	N/A	28.2	41.9	82.1	118.9	143.8	165.5	N/A

The design engineer should ensure the structural element is capable of supporting these loads. Refer to Ramset™ Specifiers Anchoring Resource Book ANZ Ed2 for more information or explanation of Tech. Data.

AC19

Cracked Concrete - ChemSet Anchor Stud Design Calculator

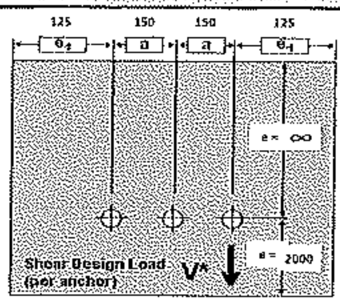
European Technical Assessment: ETA-10/0309 Design Methods: AS 5216:2018 (formerly TS101) or EN 1992-4 (formerly ETAG/TR029)

Ramset™ CRACKED Concrete - EPCON™ C8 Xtrem™

Anchor Type: Threaded Rod - Gr 8.8 Anchor Size d_b = M 20

Input Description (Strength Limit State Design)	Input Data (per anchor)		
1. Number of anchors (n)	n =	2	-
2. Anchor Spacing (a)	a =	150	mm
3. Concrete Edge Distance (e)	e =	2000	mm
4. Concrete Cylinder Strength (f'c)	f'c =	40	MPa
5. Cracked Conc. (C) or Non-Cracked (N)	C or N	C	-
6. Effective Depth (h>6xdh)	h =	200	mm
7. Anchor Stud size (d_b) - M10 → M30	d_b =	20	mm
8. Concrete Edge Distance Corner (e1)	e_1 =	125	mm
9. Internal to a row (I) or end of row (E)	I or E	I	row
10. Dry Hole (D) or Flooded hole (F)	D or F	F	-
11. Min Concrete Sub'te Thickness (b_m)	b_m =	244	mm
12. Anchor Stud Grade (5.8, 8.8, 316 SS)	Grade =	8.8	Gr
13. Fixture Thickness (t)	t =	10	mm
14. Effective Length (L_e)	L_e =	210	mm
15. Design Tensile Load-per anchor (N*)	N* =	23	kN
16. Design Shear Load-per anchor (V*)	V* =	23	kN
17. Direction of Shear design load (α)	α =	0	°
18. Sustained Load (SUS or N/A)	SUS or N/A	N/A	-
19. Service Temperature (°C)	-40°C to	+40°C	-

Plan View - Generic (Dimensions in (mm))



Project Details

Project Name:-
Seismic Strengthening
Project Site Address:-
11 Johnsonville Road
Company Name:-
Silvester Clark Ltd.
Design Identification:-
MS
Date:-
11-12-19

ChemSet™ Anchor Stud "Specification"

Typ Gr 8.8 Thr'd Rod M20 x 245 mm long

EPCON™ C8 Xtrem™ "Specification"

Part No. C8-450

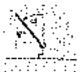
Hole Diameters (mm)

Drill d_h = 22
Fixture d_f = 22

Capacity Reduction Factors

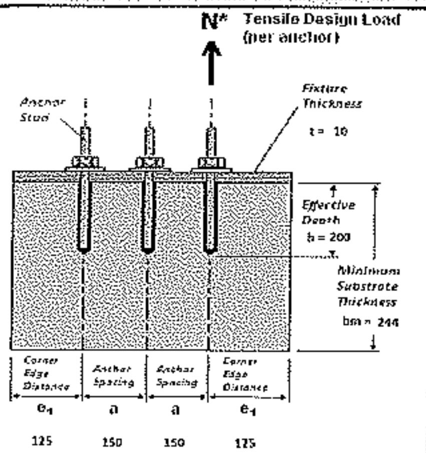
Conc Tension	1/1.8 = ϕ_c = 0.56
Conc Shear	1/1.5 = ϕ_c = 0.67
Steel Tension	1/1.5 = ϕ_n = 0.67
Steel Shear	1/1.5 = ϕ_v = 0.67

Direction of Shear Design Load - α

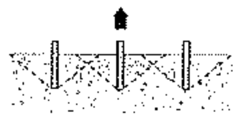


Output Description (Strength Limit State Design)	Output Data (per anchor)		
DESIGN O.K.	MIN. CRITERIA for a,e & h - O.K.		
Des.Pullout & CONC.Tensile Capacity	ϕN_{uc} =	27.0	kN
Design STEEL Tensile Capacity	ϕN_{us} =	123.9	kN
Design CONC. Edge Shear Capacity	ϕV_{uc} =	580.0	kN
Design STEEL Shear Capacity	ϕV_{us} =	76.8	kN
Design Pryout Failure	ϕV_{ucp} =	71.7	kN
Drill hole diameter	d_h =	22	mm
TENSION O.K.			
Design Tensile Capacity	ϕN_{ur} =	27.0	kN
Tension Design Check	$N^*/\phi N_{ur}$ =	0.85	<1
SHEAR O.K.			
Design Shear Capacity	ϕV_{ur} =	71.7	kN
Shear Design Check	$V^*/\phi V_{ur}$ =	0.32	<1
COMBINED TENSION SHEAR O.K.			
Combined Check - $N^*/\phi N_{ur} + V^*/\phi V_{ur}$ =		1.17	<1.2

Elevation View - Generic (Dimensions in (mm))



Anchor Loaded



Anchor Size	d_b	Metric	8	10	12	16	20	24	30	36
Drill hole diameter	d_h	(mm)	N/A	12	14	18	22	26	35	N/A
Stressed Area	A_s	(mm²)	N/A	58	84	157	245	353	561	N/A
Anchor Stud Yield Strength	f_y	(MPa)	N/A	640	640	640	640	640	640	N/A
Steel Tensile Capacity	ϕN_{us}	(kN)	N/A	28.2	41.9	82.1	123.9	179.5	299.2	N/A
Steel Shear Capacity	ϕV_{us}	(kN)	N/A	17.5	26.0	50.9	76.8	111.3	185.5	N/A
Edge distance for no conc.cone reduction	e_c	(mm)	N/A	1.5xh	1.5xh	1.5xh	1.5xh	1.5xh	1.5xh	N/A
Anchor spacing for no conc.cone reduction	a_c	(mm)	N/A	3xh	3xh	3xh	3xh	3xh	3xh	N/A
Absolute Minimum edge dist. & anc'r spac.	e_m & a_m	(mm)	N/A	50	60	80	100	120	150	N/A

Effective Depth - h		Design tensile capacity ϕN_{ur} (kN per anchor) Based on edge distance (e_c) and anchor spacing (a_c) for no conc.cone reduction							
65	(mm)	N/A							
70	(mm)	N/A	8.6						
80	(mm)	N/A	9.9						
90	(mm)	N/A	11.1	12.6					
100	(mm)	N/A	12.4	14.0					
110	(mm)	N/A	13.6	15.4	18.7				
120	(mm)	N/A	14.8	16.8	20.4				
125	(mm)	N/A	15.4	17.6	21.2				
150	(mm)	N/A	18.5	21.0	25.5	32.4			
160	(mm)	N/A	19.8	22.4	27.2	34.6	37.8		
170	(mm)	N/A	21.0	23.8	28.9	36.8	41.4		
190	(mm)	N/A	23.5	26.6	32.3	41.1	48.9		
210	(mm)	N/A	25.9	29.4	35.7	45.4	54.9		
240	(mm)	N/A	28.2	33.8	40.8	51.9	62.8	69.4	
280	(mm)	N/A	28.2	39.2	47.6	60.5	73.2	84.3	
350	(mm)	N/A	28.2	41.9	59.4	75.7	91.5	105.3	
450	(mm)	N/A	28.2	41.9	76.4	97.3	117.7	135.4	N/A
550	(mm)	N/A	28.2	41.9	82.1	118.9	143.8	165.5	N/A

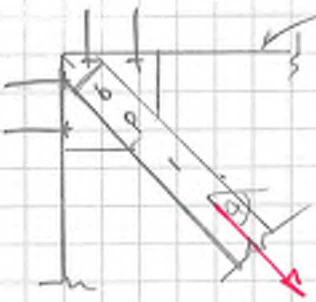
The design engineer should ensure the structural element is capable of supporting these loads. Refer to Ramset™ Specifiers Anchoring Resource Book ANZ Ed2 for more information or explanation of Tech. Data.

18

CONNECTION DESIGN

ROOF X-BRACE

CONSIDER



w 2/M16 (As Pg 50)

WHEN INTO CONCRETE NEED SIMILAR CONNECTION TO 1ST FLOOR X-BRACE

AND INTO STEEL HAVE PLATES WELDED DIRECTLY TO MEMBERS.

DESIGN GUSSET TO PLATE

CONSIDER PRINCIPLE AXIS $\alpha = 45°$

$\therefore \quad V_H = V_V = \sqrt{\frac{63^2}{2}}$

$= 44.5KN$

SAY 125 x 125mm PL

w 6mm FWAR.

$\phi V_w = \phi\, 0.6 f_{uw} \times t_t \times k_r \times l_w$

$= 0.6 \times 0.6 \times 410 \times 4.24 \times 1 \times 250$

$= 155KN > 44.5KN \therefore OK$

$\therefore$ OK FOR "TO STEEL" CONNECTION

CONNECTION DESIGN.

ROOF X-BRACE

DESIGN BASE PLATE TO CONCRETE

CONSIDER PRINCIPLE AXIS & BOLTS WORKING IN TENSION / & SHEAR

SAY 4/M12; $\phi N_t = \phi A_s f_{uf}$
$= 0.8 \times \pi \times 12^2 \times 0.25 \times 400$
$= 36.2 kN$

$\phi V = \phi\, 0.62 \times f_{uf} (A_c \times n)$
$= 0.8 \times 0.62 \times 400 \times (\pi \times 12^2 \times 0.25) \times 1$
$= 22.4 kN$

$\phi N_\alpha = \sqrt{4\phi N + 4\phi V}$
$= 85 kN > 68 kN \therefore OK$

NOW RAMSET CALCULATOR, M12 8.8s @ 200mm EMBED.

DEMAND ON SINGLE BOLT $N_\alpha^{*} = 15.8 kN$ / BOLT

$V^* = N^* = \sqrt{\frac{N_\alpha^{*2}}{2}}$
$= 11.7 kN.$

NOW OK AS PER RAMSET CALCULATOR

AC22

Cracked Concrete - ChemSet Anchor Stud Design Calculator

European Technical Assessment: ETA-10/0309 Design Methods: AS 5216:2018 (formerly TS101) or EN 1992-4 (formerly ETAG/TR029)

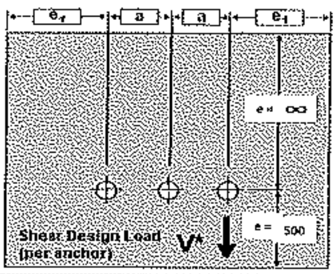
Ramset™

CRACKED Concrete - EPCON™ C8 Xtrem™

Anchor Type: Threaded Rod - Gr 8.8 Anchor Size d_b = M 12

Input Description (Strength Limit State Design)	Input Data (per anchor)		
1. Number of anchors (n)	n =	2	-
2. Anchor Spacing (a)	a =	150	mm
3. Concrete Edge Distance (e)	e =	500	mm
4. Concrete Cylinder Strength (f'c)	f'c =	40	MPa
5. Cracked Conc. (C) or Non-Cracked (N)	C or N	C	-
6. Effective Depth (h>6xdh)	h =	200	mm
7. Anchor Stud size (d_b) - M10 → M30	d_b =	12	mm
8. Concrete Edge Distance Corner (e1)	e_1 =	125	mm
9. Internal to a row (I) or end of row (E)	I or E	I	row
10. Dry Hole (D) or Flooded hole (F)	D or F	F	-
11. Min Concrete Sub'te Thickness (b_m)	b_m =	230	mm
12. Anchor Stud Grade (5.8, 8.8, 316 SS)	Grade =	8.8	Gr
13. Fixture Thickness (t)	t =	10	mm
14. Effective Length (L_e)	L_e =	210	mm
15. Design Tensile Load-per anchor (N*)	N* =	12	kN
16. Design Shear Load-per anchor (V*)	V* =	12	kN
17. Direction of Shear design load (α)	α =	0	°
18. Sustained Load (SUS or N/A)	SUS or N/A	N/A	-
19. Service Temperature (°C)	-40°C to	+40°C	-

Plan View - Generic (Dimensions in (mm))



Project Details

Project Name:-
Seismic Strengthening
Project Site Address:-
11 Johnsonville Road
Company Name:-
Silvester Clark Ltd.
Design Identification:-
MS
Date:-
11-12-19

ChemSet™ Anchor Stud "Specification"

Typ Gr 8.8 Thr'd Rod M12 x 230 mm long

EPCON™ C8 Xtrem™ "Specification"

Part No. C8-450

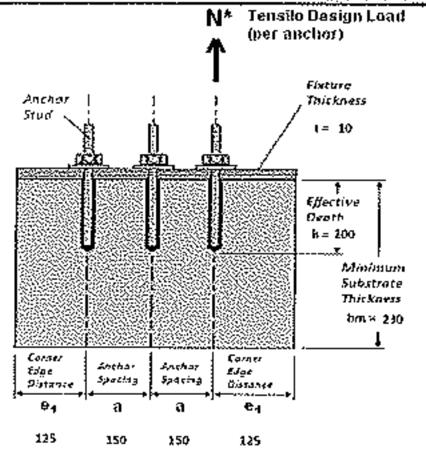
Hole Diameters (mm)	Capacity Reduction Factors	
Drill d_h = 14	Conc Tension	1/1.8 = ϕ_c = 0.56
Fixture d_f = 14	Conc Shear	1/1.5 = ϕ_c = 0.67
	Steel Tension	1/1.5 = ϕ_s = 0.67
	Steel Shear	1/1.5 = ϕ_s = 0.67

Direction of Shear Design Load - α

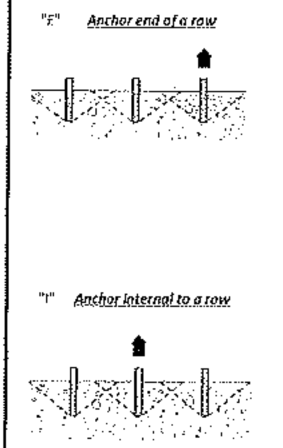
α = 0° - towards edge
α = 180° - away from edge

Output Description (Strength Limit State Design)	Output Data (per anchor)		
DESIGN O.K.	MIN. CRITERIA for a, e & h - O.K.		
Des.Pullout & CONC.Tensile Capacity	ϕN_{uc} =	17.5	kN
Design STEEL Tensile Capacity	ϕN_{us} =	41.9	kN
Design CONC. Edge Shear Capacity	ϕV_{uc} =	107.2	kN
Design STEEL Shear Capacity	ϕV_{us} =	26.0	kN
Design Pryout Failure	ϕV_{ucp} =	31.0	kN
Drill hole diameter	d_h =	14	mm
TENSION O.K.			
Design Tensile Capacity	ϕN_{ur} =	17.5	kN
Tension Design Check	$N^*/\phi N_{ur}$ =	0.69	<1
SHEAR O.K.			
Design Shear Capacity	ϕV_{ur} =	26.0	kN
Shear Design Check	$V^*/\phi V_{ur}$ =	0.46	<1
COMBINED TENSION SHEAR O.K.			
Combined Check - $N^*/\phi N_{ur} + V^*/\phi V_{ur}$ =		1.15	<1.2

Elevation View - Generic (Dimensions in (mm))



Anchor Loaded



Anchor Size	d_b	Metric	8	10	12	16	20	24	30	36
Drill hole diameter	d_h	(mm)	N/A	12	14	18	22	26	35	N/A
Stressed Area	A_s	(mm²)	N/A	58	84	157	245	353	561	N/A
Anchor Stud Yield Strength	f_y	(MPa)	N/A	640	640	640	640	640	640	N/A
Steel Tensile Capacity	ϕN_{us}	(kN)	N/A	28.2	41.9	82.1	123.9	179.5	289.2	N/A
Steel Shear Capacity	ϕV_{us}	(kN)	N/A	17.5	26.0	50.9	76.8	111.3	185.5	N/A
Edge distance for no conc.cone reduction	e_c	(mm)	N/A	1.5xh	1.5xh	1.5xh	1.5xh	1.5xh	1.5xh	N/A
Anchor spacing for no conc.cone reduction	a_c	(mm)	N/A	3xh	3xh	3xh	3xh	3xh	3xh	N/A
Absolute Minimum edge dist. & anc'r spac.	e_m & a_m	(mm)	N/A	50	60	80	100	120	150	N/A

Effective Depth - h		Design tensile capacity ϕN_{ur} (kN per anchor) Based on edge distance (e_c) and anchor spacing (a_c) for no conc.cone reduction							
65	(mm)	N/A							
70	(mm)	N/A	8.6						
80	(mm)	N/A	9.9						
90	(mm)	N/A	11.1	12.6					
100	(mm)	N/A	12.4	14.0					
110	(mm)	N/A	13.6	15.4	18.7				
120	(mm)	N/A	14.8	16.8	20.4				
125	(mm)	N/A	15.4	17.5	21.2				
150	(mm)	N/A	18.5	21.0	25.5	32.4			
160	(mm)	N/A	19.8	22.4	27.2	34.6	37.8		
170	(mm)	N/A	21.0	23.8	28.9	36.8	41.4		
190	(mm)	N/A	23.6	26.6	32.3	41.1	46.9		
210	(mm)	N/A	25.9	29.4	35.7	45.4	54.9		
240	(mm)	N/A	28.2	33.6	40.8	51.9	62.8	69.4	
280	(mm)	N/A	28.2	39.2	47.6	60.6	73.2	84.3	
350	(mm)	N/A	28.2	41.9	59.4	75.7	91.5	105.3	
450	(mm)	N/A	28.2	41.9	76.4	97.3	117.7	136.4	N/A
550	(mm)	N/A	28.2	41.9	82.1	118.9	143.8	165.6	N/A

The design engineer should ensure the structural element is capable of supporting these loads. Refer to Ramset™ Specifiers Anchoring Resource Book ANZ Ed2 for more information or explanation of Tech. Data.

DISCLOSURE STATEMENT

1. The following information has been supplied to Capital Commercial (2013) Limited ("Bayleys") to be made available for distribution on the vendor's behalf to potential purchasers to assist purchasers with their due diligence and to use at the purchaser's discretion.
2. Bayleys and the Vendor do not warrant the accuracy or completeness of the information and recommends that all recipients undertake their own due diligence, obtain their own reports to their satisfaction and seek independent advice prior to committing to purchaser.