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Project: MOE – FLS Reference Designs for Standard Classroom Upgrade

Formula Block

Structural Calculations: Auckland

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Document prepared by:

Aurecon NZ Pty Ltd
Old Bank Chambers, Level 1
102 Customhouse Quay
Wellington, 6140

T +64 4 472 9589
F +64 4 472 9922
E wellington@aurecongroup.com
W aurecongroup.com

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Name	Phil Don	**Name**	John Finnegan
CPEng Number	-	**CPEng Number**	145791
Title	Structural Engineer	**Title**	Technical Director

FLS Reference Designs for Standard Classroom Upgrade– Formula Block

Structural Calculations: Auckland

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Summary

The Ministry of Education (MOE) wishes to provide schools with standard options to modify their classrooms to allow for a Flexible Learning Space (FLS).

This document provides detailed calculations for the standard options to create a FLS for a Formula Block. These details are intended to be generic and have been developed with the following site characteristics:

Importance Level	2
Site Subsoil Class	D
Location	Auckland

This document provides structural calculations for three FLS options set out by the MOE. These will be labelled Option 1, Option 2 and Option 3 as per the preliminary structural schemes. The main variances between the three options lie in the footprint the smaller breakout areas have relative to the general flexible learning areas. The FLS options are based around 4 classroom blocks that will be expanded into a larger learning environment that can be altered as required to suit the class type.

For these strengthening schemes, the lateral bracing elements are to have a minimum capacity of 67%NBS.

1. Building Overview

1.1 Original Structure

Formula Blocks were constructed between 1962 and 1970. They are of lightweight timber construction and are characterised by consecutive pairs of classrooms with common cloakroom and brick clad toilet facilities. The image below shows a typical two-classroom Formula Block.

Front View of Typical Formula Block

In the transverse direction either glulam timber portal frames or steel portal frames are present. The floor plan of a 4-classroom unit is shown in the figure below:

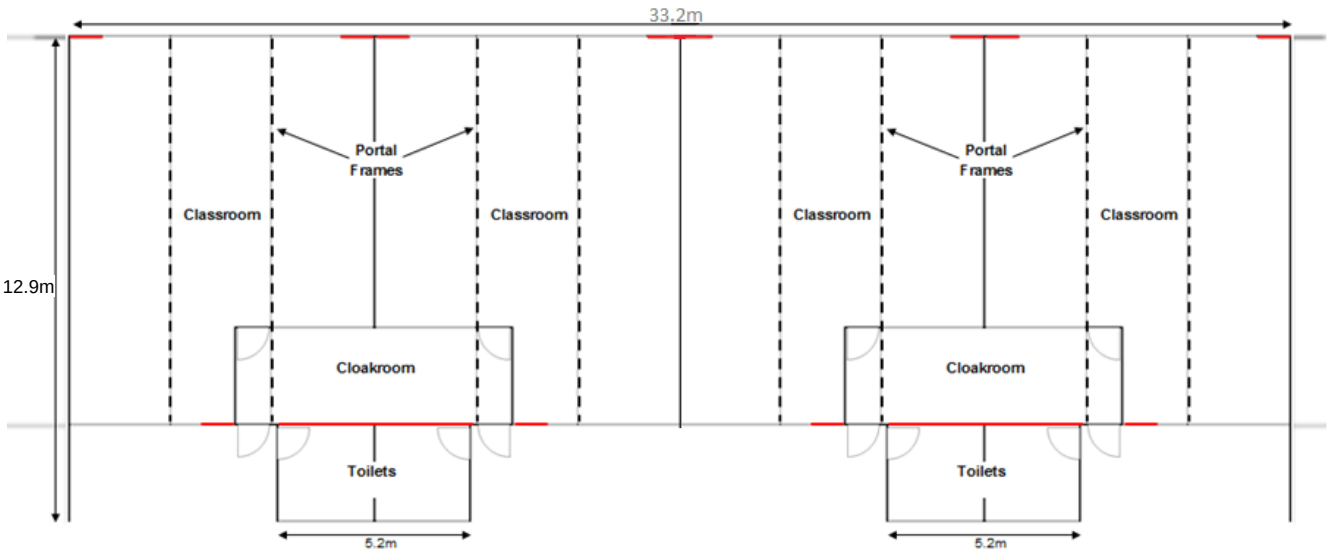
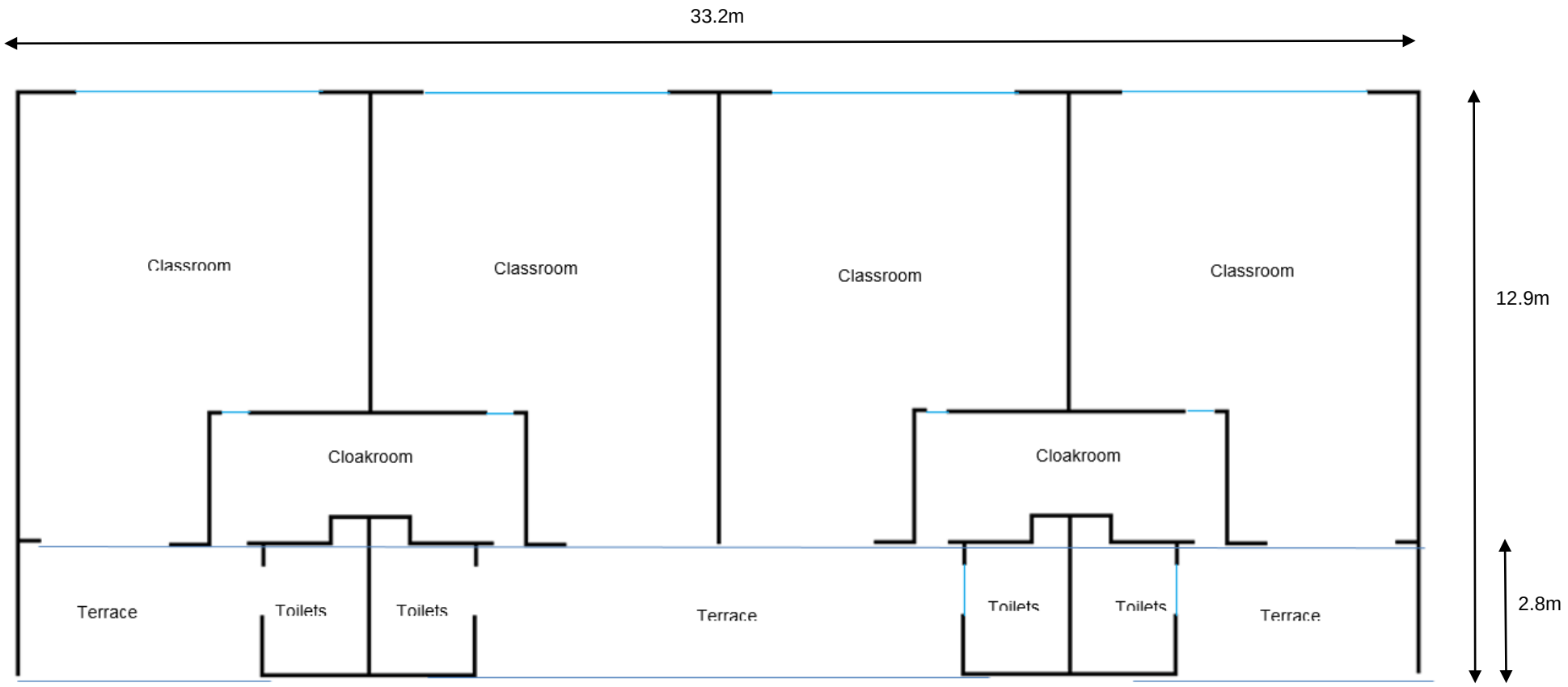


Figure 1 – Plan View of a Typical Formula Block



Floor plan of a 4-classroom unit showing wall layout

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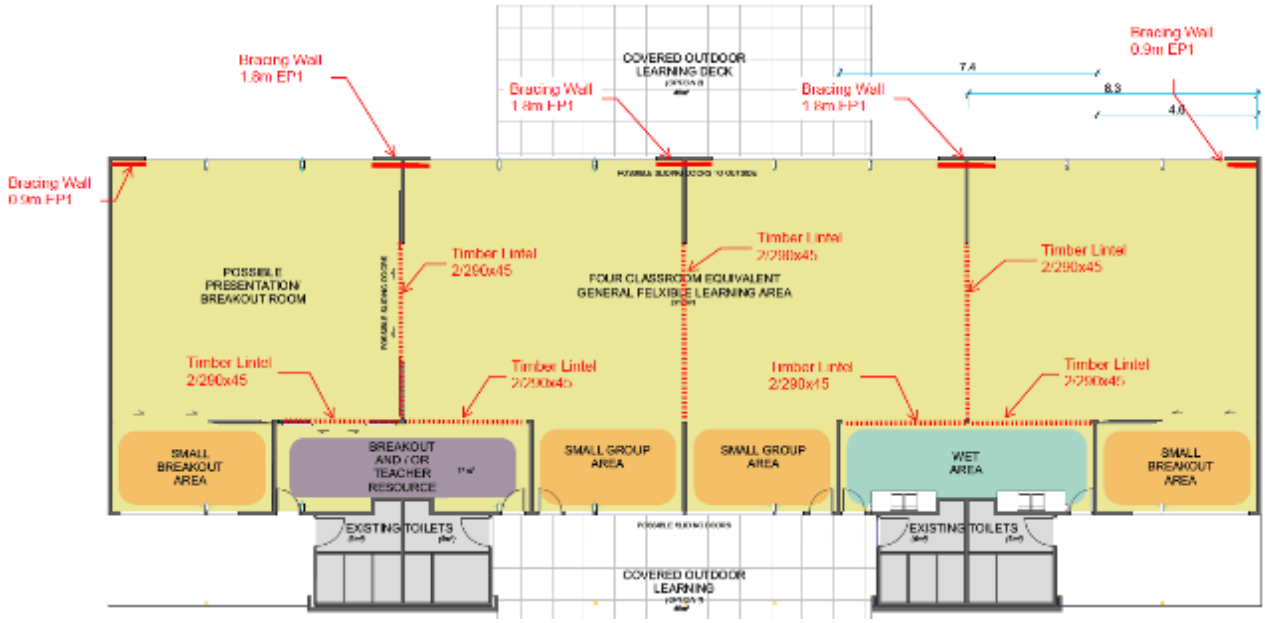
1.2 FLS Upgrade – Option 1

Option 1 for the FLS upgrade consists of opening up the four classrooms to become one large flexible learning space. At the rear of the classrooms, alterations are made with the existing layout, creating small breakout spaces and work areas.

To allow for the FLS upgrade, as well as to strengthen the lateral bracing capacity of the structure to meet a minimum capacity of 67%NBS, timber lintels are to be installed across the openings in the classroom walls and new Ecoply and Gib bracing walls are to be installed throughout the structure.

The figure below shows a plan layout with the preliminary structural scheme for the new FLS layout.



FLS Layout and Structural Scheme Option 1

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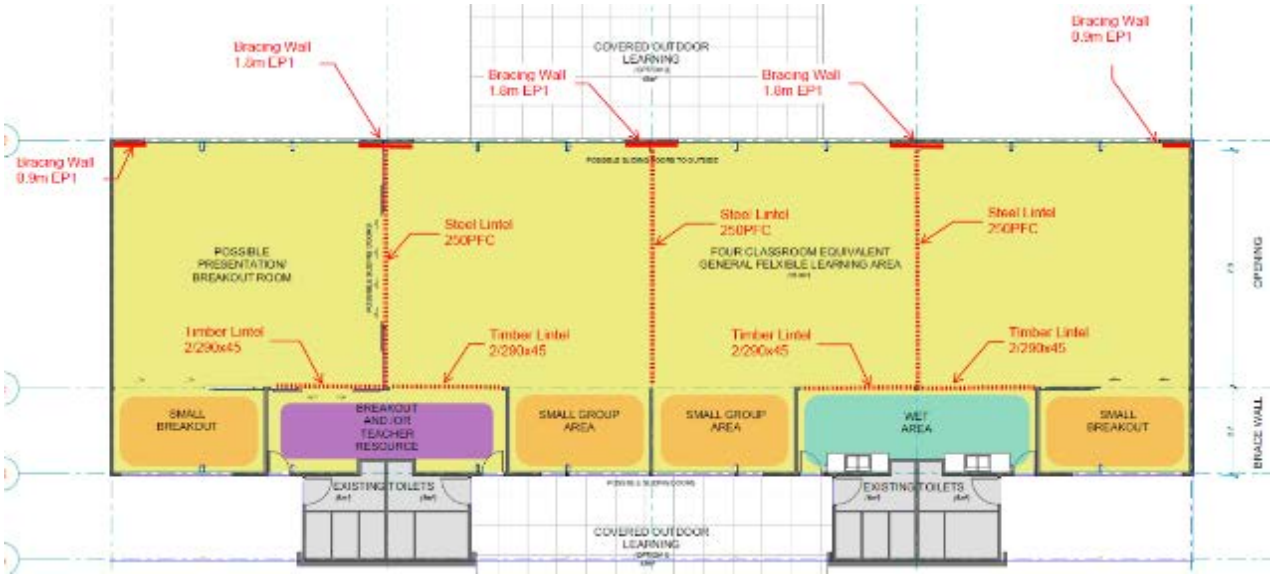
1.3 FLS Upgrade – Option 2

Option 2 for the FLS upgrade varies from Option 1 through creating larger openings between the main classroom areas. The structural system for this option will require the installation of steel lintels in areas rather than timber lintels (due to the larger spans), as well as the installation of Ecoply and Gib bracing walls similar to Option 1.

The figure below shows a plan layout with the preliminary structural scheme for the new FLS layout.

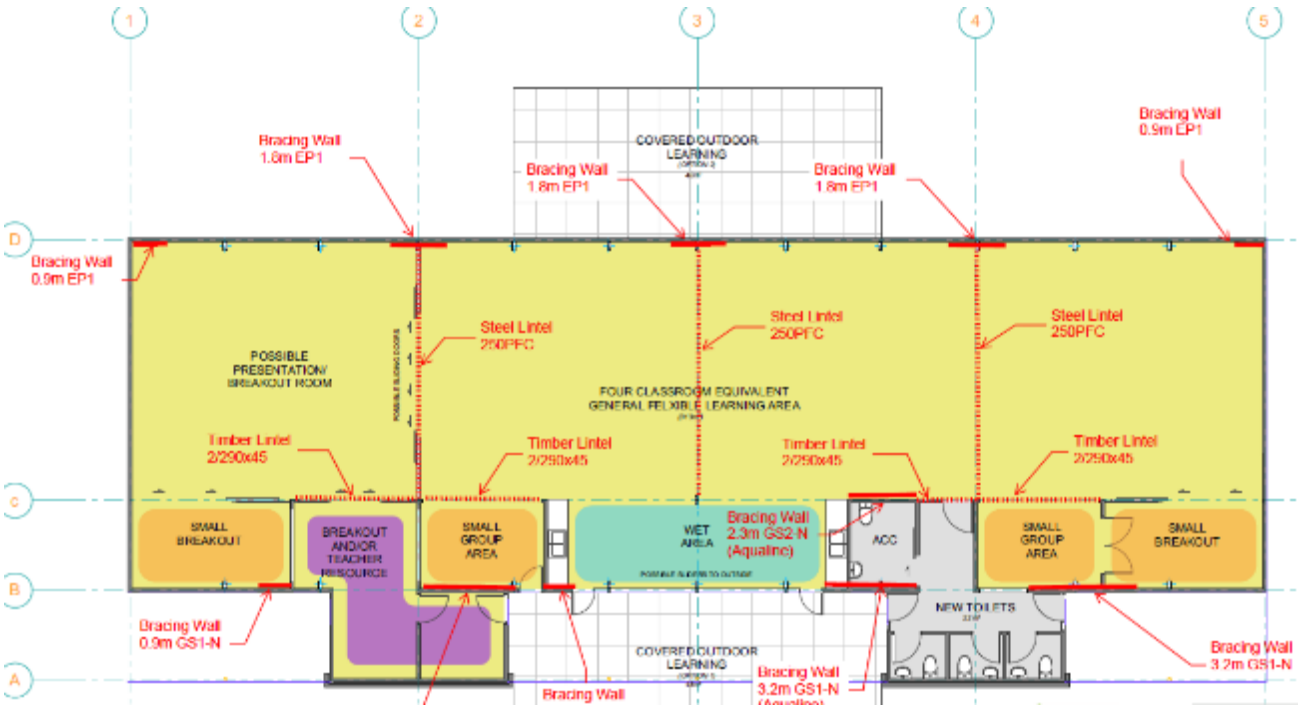


FLS Layout and Structural Scheme Option 2

1.4 FLS Upgrade – Option 3

Option 3 for the FLS upgrade is similar to Option 2, where large openings require the installation of steel lintels. This option varies from Option 2 at the rear of the structure where walls are removed and new walls added around the existing toilet areas. This requires additional Gib bracing walls to be installed in these areas.

The figure below shows a plan layout with the preliminary structural scheme for the new FLS layout.



FLS Layout and Structural Scheme Option 3

2. Lateral Loading Demands

2.1 Seismic Loading

2.1.1 General Parameters

The lateral forces that must be resisted by the elements in a building are a function of the seismic weight and the horizontal acceleration as estimated by the horizontal design action coefficient.

Earthquake actions are calculated as per the equivalent static method, which applies the force due to the weight of the roof and upper half of the walls at roof level and the force due to the total weight of the building at foundation level.

From NZS1170.0, section 4.2.1, the ultimate limit state load combination that relates to earthquake actions is as follows:

$$E_{d,dst} = G + E_u + \Psi_c Q$$

$$G = Permanent\ Actions$$

$$E_u = Earthquake\ Actions$$

$$Q = Imposed\ Actions$$

$$\Psi_c = Earthquake\ Combination\ Factor$$

Permanent Actions:

Permanent actions include the self-weight of all building elements. These are estimated based on the materials present in each element.

Roofs – Roofs will be assumed to have a weight of 0.4 kPa.

Timber Walls – All timber walls and windows will be assumed to have a weight of 0.3 kPa

Brick Veneer – Brick veneer will be assumed to have a weight of 19 kN/m^3.

Imposed Actions:

Formula Blocks have a concrete floor slab on grade which transfer live loads directly to the ground and are expected to perform satisfactorily in a seismic event. Therefore, live loads need not be considered.

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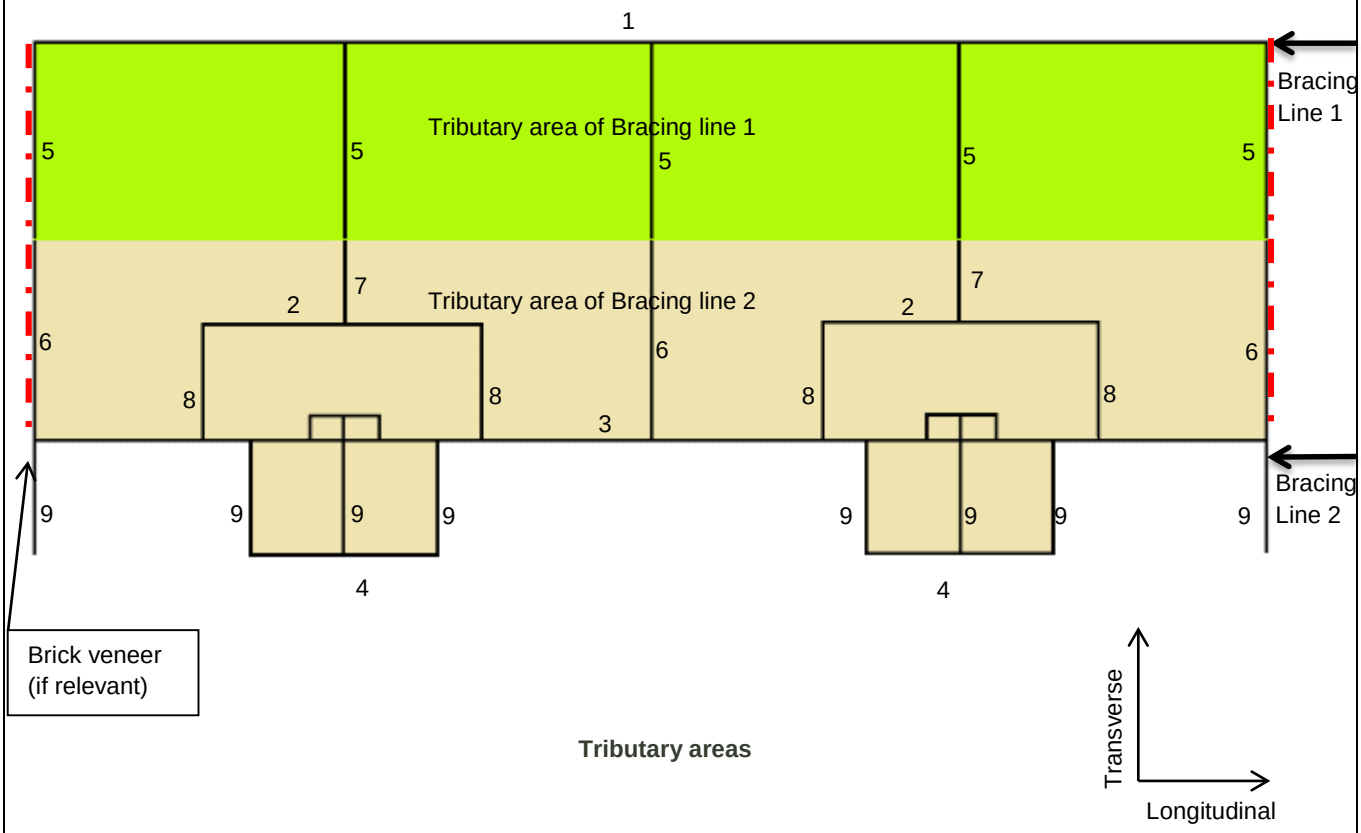
2.1.2 Seismic Weight of Classroom Section

Seismic weight will be calculated on the basis of tributary area of each bracing line. Roof weight calculations will ignore the terrace areas as these contribute negligible weight.



Tributary areas

Typical Unit Weights:

Wall weight	WW = 0.3kPa	Typical for timber framed buildings
Roof weight	RW = 0.4kPa	Typical for timber framed buildings
Combination factor	ψ_E = 0.3	NZS 1170.5, section 4.2

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Tributary Weight - Bracing Line 1

Wall Weight

Wall	Number of Members	Length	Height	
1	1	33.2m	2.6m	W1 = **25.90 kN**
5	5	5.05m	3m	W5 = **22.73 kN**

Total W_{BL1} = **48.62 kN**

Roof Weight

Plan Area

$Area_{BL1}$ = 33.2 m × 5.05 m
$Area_{BL1}$ = **167.66** m^2

Total Roof Weight

$Roof_{Bl1}$ = 0.4 × $Area_{BL1}$
$Roof_{BL1}$ = 67.06 kN

Tributary Weight – Bracing Line 2

Wall Weight

Wall	Number of Members	Length	Height	Weight
2	2	7.7m	3m	W2 = **13.86 kN**
3	1	33.2m	2.6m	W3 = **25.90 kN**
4	2	5.2m	2.3m	W4 = **7.18 kN**
6	3	5.05m	3m	W6 = **13.64 kN**
7	2	2.25m	3m	W7 = **4.05 kN**
8	4	2.8m	2.8m	W8 = **9.41 kN**
9	8	2.8m	2.45m	W9 = **16.46 kN**

Total **W_{BL2} = 90.49 kN**

Roof Weight

Plan Area

$Area_{BL2}$ = 33.2 m × 5.05 m + (5.2 m × 2.8 m) × 2

$Area_{BL2}$ = 196.78 m^2

Total Roof Weight

$Roof_{BL2}$ = 0.4 × $Area_{BL2}$

$Roof_{BL2}$ = 78.71 kN

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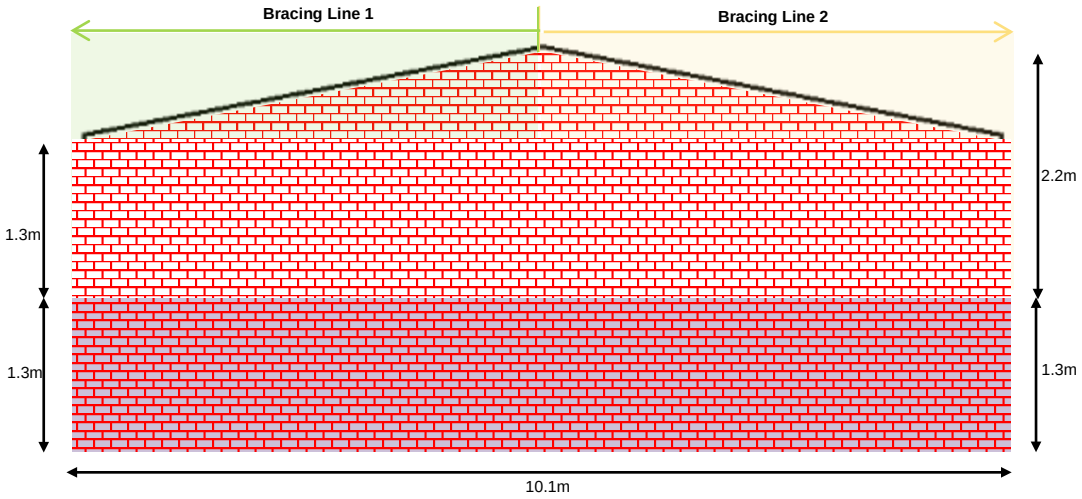
2.1.3 Weight of Brick Veneer to End Walls

Formula Blocks may or may not feature brick end walls. The added weight of these walls can have a significant effect on the lateral loading demand under NZS1170.5.

The lower half of the brick wall will not form a part of the seismic weight calculation.



Brick End Wall

Brick Wall Weight (per bracing line)

Tributary Area	$A = (1.3m+(0.9m \times 0.5)) \times (10.1m \times 0.5)$
	$A = \mathbf{8.84m^2}$
Thickness	$t = 3$ in $= \mathbf{76.20}$ mm
Volume	$Vol_{Brick} = A \times t$
	$Vol_{Brick} = \mathbf{0.67}\ m^3$
Density	$\rho_{Brick} = 19\ kN/m^3$
Weight	$W_{Brick} = \rho_{Brick} \times Vol_{Brick}$
	$\mathbf{W_{Brick} = 12.79\ kN}$

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2.1.4 Horizontal Design Action Coefficients

Seismic Coefficient Variables

Importance Level:	**Importance Level 2 – Normal Structure**
Design Working Life:	**50 Years**
Fundamental Period:	**T_n = 0.4s**
Site Subsoil Class (Section 3.1.3):	**C – Shallow Soil**
Site Location (Table 3.3):	**Site Location 7 - Auckland**

NZS1170.5 – Section 3.1: Elastic Site Spectra for Horizontal Loading:

C(T) = Ch(T) Z R N(T,D)

Where:

Ch(T) = spectral shape factor from Clause 3.1.2

Z = hazard factor from Clause 3.1.4

R = the return period factor from Clause 3.1.5

N(T,D) = near fault factor from Clause 3.1.6

This gives the following seismic coefficients for various levels of ductility.

Seismic Coefficient (ductility of 3.0) **$C_dT_{3.0}$ = 0.1274**

For the design of ductile strengthening elements (Ecoply and Gib Bracing Walls)

Seismic Coefficient (ductility of 2.5 Sp 0.5) **$C_dT_{2.5}$ = 0.1050**

For the assessment of existing timber framed elements

Seismic Coefficient (nominally ductile) **$C_dT_{1.25}$= 0.3157**

For the design and assessment of non-ductile elements

Seismic Coefficient (nominally ductile) **$C_dT_{1.00}$= 0.3900**

For the elastic design and assessment of elements where necessary

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Seismic Data - NZ1170:2002 Parts 0 and 5

Job Name:	MOE ILS	**Date:**	17/08/2015
Job No:	246313	**Designer:**	PGD
General Notes:	Formula Block		

Importance level (part 3, table 3.1)

2	Normal structures

Design working life

50

Required annual probablities of exceedance (part 3, table 3.3)

Limit state	ULS	SLS1	SLS2
1 / P	500	25	-

Limit state for this analysis

	ULS
1 / P	500

Natural periods

T_n^x =	0.4000	sec
T_n^y =	0.4000	sec

Ductility factors

Along X	3.00
Along Y	3.00

Site subsoil class (part 5, clause 3.1.3)

Deep or soft soil	Category	D

Site location (Tables 3.3 and 3.4)

7

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.13
Shortest distance D	Min.	N/A
	Max.	N/A

Return period factor (clause 3.1.5)

R	1.00

Horizontal design action coefficient (part 5, clause 5.2)

Direction	Analysis method	$C_h(T)$	ZR	N(T,D)	C(T)	S_p	k_μ	$C_d(T)$
X	ESM	3.0000	0.1	1.0	0.3900	0.700	2.143	**0.1274**
	MRSM	3.0000			0.3900			0.1274
Y	ESM	3.0000	0.1	1.0	0.3900	0.700	2.143	**0.1274**
	MRSM	3.0000			0.3900			0.1274

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Seismic Data - NZ1170:2002 Parts 0 and 5

Job Name:	MOE ILS	**Date:**	17/08/2015
Job No:	246313	**Designer:**	PGD
General Notes:	Formula Block		

Importance level (part 3, table 3.1)

2	Normal structures

Design working life

50

Required annual probablities of exceedance (part 3, table 3.3)

Limit state	ULS	SLS1	SLS2
1 / P	500	25	-

Limit state for this analysis

	ULS
1 / P	500

Natural periods

T_n^x =	0.4000	sec
T_n^y =	0.4000	sec

Ductility factors

Along X	2.50
Along Y	2.50

Site subsoil class (part 5, clause 3.1.3)

Deep or soft soil	Category	D

Site location (Tables 3.3 and 3.4)

7

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.13
Shortest distance D	Min.	N/A
	Max.	N/A

Return period factor (clause 3.1.5)

R	1.00

Horizontal design action coefficient (part 5, clause 5.2)

Direction	Analysis method	$C_h(T)$	ZR	N(T,D)	C(T)	S_p	k_μ	$C_d(T)$
X	ESM	3.0000	0.1	1.0	0.3900	0.500	1.857	**0.1050**
	MRSM	3.0000			0.3900			0.1050
Y	ESM	3.0000	0.1	1.0	0.3900	0.500	1.857	**0.1050**
	MRSM	3.0000			0.3900			0.1050

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Seismic Data - NZ1170:2002 Parts 0 and 5

Job Name:	MOE ILS	**Date:**	17/08/2015
Job No:	246313	**Designer:**	PGD
General Notes:	Formula Block		

Importance level (part 3, table 3.1)

2	Normal structures

Design working life

50

Required annual probablities of exceedance (part 3, table 3.3)

Limit state	ULS	SLS1	SLS2
1 / P	500	25	-

Limit state for this analysis

	ULS
1 / P	500

Natural periods

T_n^x =	0.4000	sec
T_n^y =	0.4000	sec

Ductility factors

Along X	1.25
Along Y	1.25

Site subsoil class (part 5, clause 3.1.3)

Deep or soft soil	Category	D

Site location (Tables 3.3 and 3.4)

7

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.13
Shortest distance D	Min.	N/A
	Max.	N/A

Return period factor (clause 3.1.5)

R	1.00

Horizontal design action coefficient (part 5, clause 5.2)

Direction	Analysis method	$C_h(T)$	ZR	N(T,D)	C(T)	S_p	k_μ	$C_d(T)$
X	ESM	3.0000	0.1	1.0	0.3900	0.925	1.143	**0.3157**
	MRSM	3.0000			0.3900			0.3157
Y	ESM	3.0000	0.1	1.0	0.3900	0.925	1.143	**0.3157**
	MRSM	3.0000			0.3900			0.3157

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Seismic Data - NZ1170:2002 Parts 0 and 5

Job Name:	MOE ILS	**Date:**	17/08/2015
Job No:	246313	**Designer:**	PGD
General Notes:	Formula Block		

Importance level (part 3, table 3.1)

2	Normal structures

Design working life

50

Required annual probablities of exceedance (part 3, table 3.3)

Limit state	ULS	SLS1	SLS2
1 / P	500	25	-

Limit state for this analysis

	ULS
1 / P	500

Natural periods

T_n^x =	0.4000	sec
T_n^y =	0.4000	sec

Ductility factors

Along X	1.00
Along Y	1.00

Site subsoil class (part 5, clause 3.1.3)

Deep or soft soil	Category	D

Site location (Tables 3.3 and 3.4)

7

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.13
Shortest distance D	Min.	N/A
	Max.	N/A

Return period factor (clause 3.1.5)

R	1.00

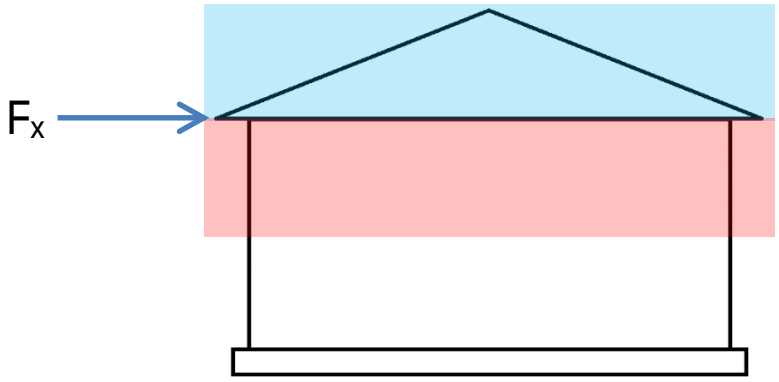
Horizontal design action coefficient (part 5, clause 5.2)

Direction	Analysis method	$C_h(T)$	ZR	N(T,D)	C(T)	S_p	k_μ	$C_d(T)$
X	ESM	3.0000	0.1	1.0	0.3900	1.000	1.000	**0.3900**
	MRSM	3.0000			0.3900			0.3900
Y	ESM	3.0000	0.1	1.0	0.3900	1.000	1.000	**0.3900**
	MRSM	3.0000			0.3900			0.3900

2.1.5 Demand on Wall Bracing Elements

Demand will be calculated as per the equivalent static method given in NZS1170.5. For buildings of this type it is overly conservative to add 8% of the total demand to roof level as per clause 6.2.1.3, so this will not be carried out.

Current building codes require that the lateral bracing elements are able to withstand forces generated at roof level. This includes the weight of the roof and the upper half of the walls.



$$F_x = C_d(T)_{\mu=x}(\boldsymbol{Roof\ Weight + Upper\ Wall\ Weight})$$

- **Demand on Longitudinal Bracing Lines**

The demand on each longitudinal wall bracing element will be calculated from its tributary seismic weight and the horizontal design action coefficient for a ductility of μ=2.5 (Sp=0.5). This ductility and Sp factor is a commonly accepted value as per MOE guidelines for timber framed walls which is the primary bracing element in this direction.

Include the weight of the brick veneer at each end wall also (worst case scenario).

Demand on Bracing Line 1

$$Dem_{BL1} = C_dT_{2.5} \times (W_{BL1}/2 + Roof_{BL1} + (W_{Brick} \times 2))$$

Dem_{BL1} = 12.28kN

Demand on Bracing Line 2

$$Dem_{BL2} = C_dT_{2.5} \times (W_{BL2}/2 + Roof_{BL2} + (W_{Brick} \times 2))$$

Dem_{BL2} = 15.70kN

The total base shear in the longitudinal direction is therefore equal to:

$$F_EQ_{Long} = Dem_{BL1} + Dem_{BL2}$$

F_EQ_{Long} = 27.98kN

- **Transverse Demand**

Brick veneer end walls will have a stiff in-plane response and will be able to support the in-plane lateral loads resulting from their own weight. The seismic weight contributed by these will therefore not be included in the transverse demand calculations.

Bracing is provided by a combination of timber shear walls and steel/timber portal frames in the transverse direction. A ductility of 2.5 is a commonly accepted value for existing timber framed walls, however 1.25 is a more suitable ductility factor for the steel/timber portal frames. Calculate the demand for both ductility values.

Total Lateral Demand

$F_EQ_{Trans2.5} = C_d T_{2.5} \times ((W_{BL1}+W_{BL2}) / 2 + (Roof_{BL1}+Roof_{BL2}))$

$\mathbf{F_EQ_{Trans2.5}} = \mathbf{22.61}$kN

$F_EQ_{Trans1.25} = C_d T_{1.25} \times ((W_{BL1}+W_{BL2}) / 2 + (Roof_{BL1}+Roof_{BL2}))$

$\mathbf{F_EQ_{Trans1.25}} = \mathbf{67.98}$kN

2.2 Wind Loading

Determine lateral wind loading demands (in accordance with NZS 1170.2) on the structure to confirm which lateral loading form governs.

2.2.1 Basic Wind Pressure Calculation

Wind Region	=	A6
Importance Level	=	2
Terrain Category	=	3
Shielding Multiplier (Ms)	=	1.00
Topographical Multiplier(Mt)	=	1.00
V_{des} SLS	=	30.7m/s
V_{des} ULS	=	37.4m/s
P_{Basic} SLS	=	0.57kPa
P_{Basic} ULS	=	0.84kPa

These wind parameters have been calculated for a terrain category 3 site with no topographical and shielding factors. These may need to be altered when a design is carried out for a specific Formula Block with different site parameters.

The spreadsheet calculations to determine the basic wind pressures are shown below:

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Client	MOE	Date	17-Aug-15
Project	ILS	Job No:	246313
Subject	Auckland	Sheet No:	By: PGD

AS1170.2 Design Wind Speed V8 — AS Standard References

Region Data

Region	A6		AS 1170.2 Figure 3.1
Design Working Life	50 Years	[Input 50 Years to conform with BCA]	AS 1170.0 Table F2
Importance Level	2		AS 1170.0 Table F1

Average Recurrence Intervals - R — AS 1170.0 Section 3.4

Retrieved From	AS1170.0		
$R_{Service}$	25	Years	
$R_{Ultimate}$	500	Years	AS 1170.0 Table F2

Region Wind Speed - V_r

$V_{r,Serv}$	37.0	ms^{-1}	AS 1170.2 Section 3.2
$V_{r,Ult}$	45.0	ms^{-1}	AS 1170.2 Section 3.2

Wind Direction Multiplier - M_d — AS 1170.2 Section 3.3

	North	North-East	East	South-East	South	South-West	West	North-West	Any	
M_d	0.85	0.95	1.00	0.95	0.85	0.95	1.00	0.95	1.00	AS 1170.2 Table 3.2

Terrain/Height Multiplier $M_{z,cat}$ — Simple — AS 1170.2 Section 4

Building Height - (z)	4	m	AS 1170.2 Section 4.2.1
Terrain Category	3.0		AS 1170.2 Table 4.1
$M_{z,cat\ Serv}$	0.83		
$M_{z,cat\ Ult}$	0.83		

Shielding Multiplier - M_s — Simple — AS 1170.2 Section 4.3

M_s	1.00	AS 1170.2 Section 4.3

Topographical Multiplier - M_t — Simple — AS 1170.2 Section 4.4

M_t	1.00	AS 1170.2 Section 4.4

Site Wind Speed - $V_{sit,\beta}$ $V_{sit,\beta} = V_R M_d (M_{z,cat} M_s M_t)$ — AS 1170.2 Section 2.2

ms^{-1}	North	North-East	East	South-East	South	South-West	West	North-West	Any
$V_{sit,\beta,Serv}$	26.10	29.17	30.71	29.17	26.10	29.17	30.71	29.17	30.71
$V_{sit,\beta,Ult}$	31.75	35.48	37.35	35.48	31.75	35.48	37.35	35.48	37.35

Design Wind Speed - $V_{des,\theta}$ — AS 1170.2 Section 2.3

Building Bearing (Angle from North)	0	Degrees	[Input range of 0° to 360°]

ms^{-1}	Wall 1	Wall 2	Wall 3	Wall 4	Any	
$V_{des,\theta,Serv}$	29.2	30.7	29.2	30.7	30.7	INPUTS VALID
$V_{des,\theta,Ult}$	35.5	37.4	35.5	37.4	37.4	

Basic Wind Pressure - p_{basic} $p = (0.5\rho_{air})[V_{des,\theta}]^2$ — Multiply by C_{fig} and C_{dyn} to obtain Design Wind Pressures — AS 1170.2 Section 2.4

kPa	Wall 1	Wall 2	Wall 3	Wall 4	Any	
$p_{serv.basic}$	0.51	0.57	0.51	0.57	0.57	
$p_{ult.basic}$	0.76	0.84	0.76	0.84	0.84	AS 1170.2 Figure 2.3

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2.2.2 Design Wind Pressure Calculation

As per Cl 2.4.1 of NZS1170.2, the design wind pressure is calculated by the following equation:

P_{design} = $P_{basic} \times C_{fig} \times C_{dyn}$

P_{Basic} ULS = 0.84kPa

C_{dyn} = 1.0 (Low rise single storey structure with natural frequency above 1Hz)

Determine Cfig for different loading faces and directions:

Cp,i = NA (Internal pressures cancel each other out)

Windward Face:

Cp,e (W) = 0.7 (Table 5.2(A) - h<25m, wind speed doesn't vary with height)

Leeward Face:

Longitudinal Loading

h = 3.0m (avg)

d = 33m

b = 10m

d/b = 3.3

Cp,e (L)Long = -0.30 (Worst Case - Table 5.2(B) Roof Pitch Equals 10 Degrees)

Transverse Loading

h = 3.0m (avg)

d = 10m

b = 33m

d/b = 0.30

Cp,e (L)Trans = -0.50 (Worst Case Table 5.2(B) Roof Pitch Equals 10 Degrees)

See over the page for the Formula Block dimensions.

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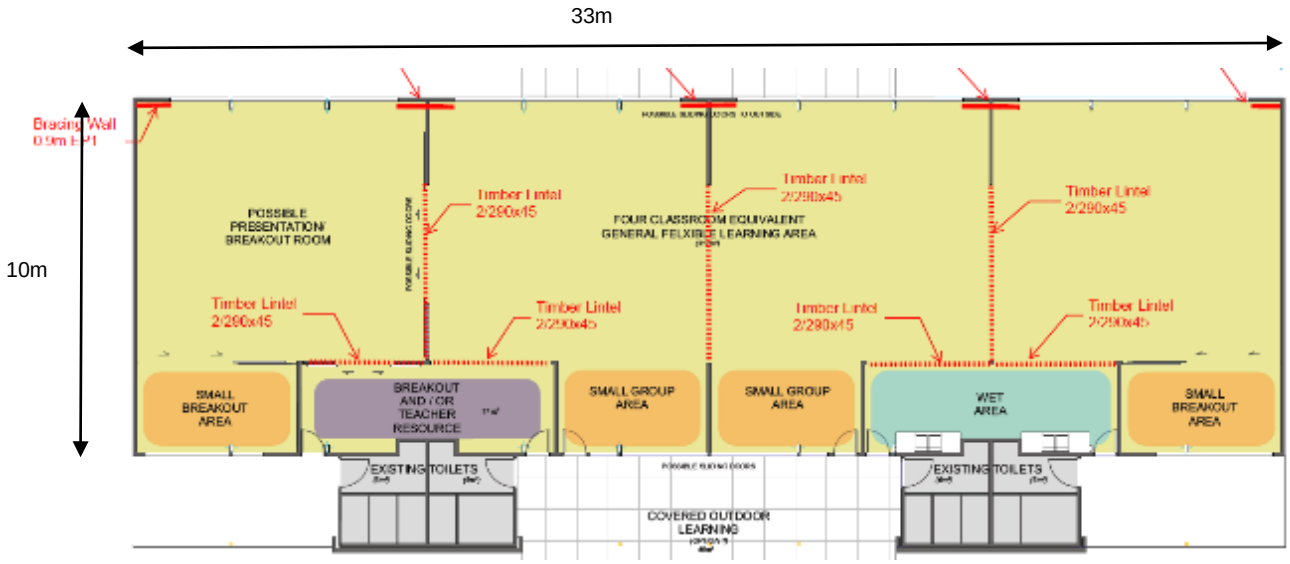

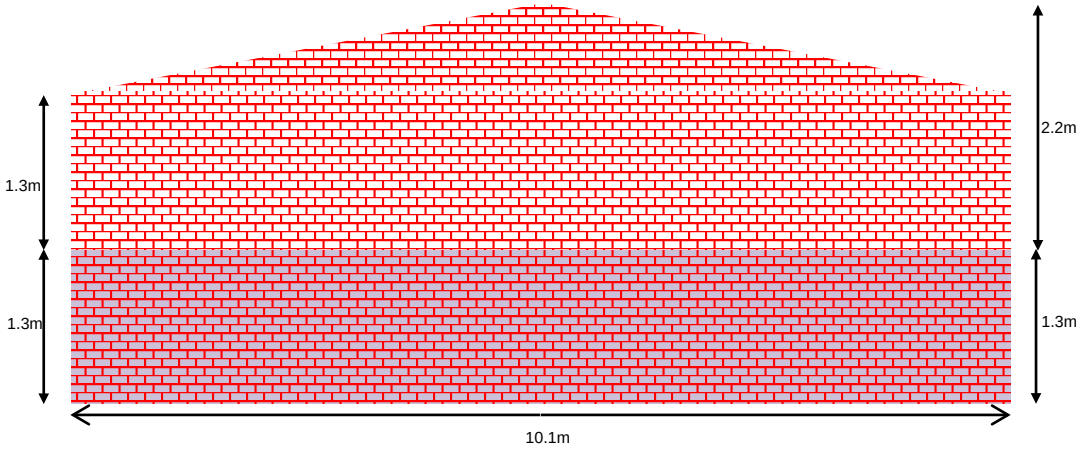

Kc,e	=	0.8 (Table 5.5 - 3 Effective Surfaces)
Ka	=	0.9 (Table 5.4 – 25m^2 Tributary Area – Worst Case)
Kp	=	1.0
Kl	=	1.0 (Bracing elements not directly supporting the cladding)
Cfig,e	=	Cp,e x Ka x Kc,e x Kl x Kp
Cfig,e W	=	0.7 x 0.9 x 0.8 x 1.0 x 1.0
	=	0.50 (All directions)

Cfig,e L Long	=	-0.30 x 0.9 x 0.8 x 1.0 x1.0	
	=	-0.22	
Cfig,e L Trans	=	-0.50 x 0.9 x 0.8 x 1.0 x1.0	
	=	-0.36	

Hence, the net Cfig factors are as follows:

Cfig net Long	=	0.5+0.22	(Windward and leeward)
	=	0.72	
Cfig net Trans	=	0.5+0.36	(Windward and leeward)
	=	0.86	

The ULS design pressures in both directions are therefore equal to the following:

PdesULSLong	=	0.72 x 0.84kPa
	=	0.60 kPa
PdesULSTrans	=	0.86 x 0.84kPa
PdesULSTrans	=	0.72 kPa

Wind Shear Forces

F_Wind_{Long}	=	0.60kPa × ((10m × 1.3m) + (10m×0.9m×0.5))
F_Wind_{Long}	**=**	**10.50kN**

F_Wind_{Trans}	=	0.72kPa × (33m × 2.6m × 0.5)
F_Wind_{Trans}	=	**30.89**kN

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2.3 Design Loading For Lateral Bracing

Section 2.1 and 2.2 show the lateral design force calculations for Earthquake and Wind Loading respectively. Compare these demands to determine the critical force to design the lateral bracing to.

Since the majority of the lateral bracing elements in this system will be from new Gib bracing walls. For these elements, it would be more suitable to use a ductility of 3.0 and an Sp factor of 0.7. The seismic forces will therefore have to be scaled up as the seismic coefficient of ductility 3.0 is higher than for a ductility of 2.5 with an Sp factor of 0.5.

In the transverse direction, compare wind loading with ductility 1.25 and ductility 3.0 demands as a combination of timber walls and portal frames will be resisting lateral loads.

2.3.1 Longitudinal Loading

$F_EQ_{Long3.0}$ = $F_EQ_{Long} \times (C_dT_{3.0}/C_dT_{2.5})$

$F_EQ_{Long3.0}$ = **33.95 kN**

F_Wind_{Long} = **10.50kN**

Hence, seismic loading governs the lateral bracing design in this direction. It is to be ensured the new bracing elements have no less capacity that what is currently in place in the existing structure and they achieve a minimum capacity of 67%NBS.

2.3.2 Transverse Loading

$F_EQ_{Trans3.0}$ = $F_EQ_{Trans2.5} \times (C_dT_{3.0}/C_dT_{2.5})$

$F_EQ_{Trans3.0}$ = **27.43kN**

$F_EQ_{Trans1.25}$ = **67.98kN**

F_Wind_{Trans} = **30.89 kN**

In this direction, seismic loading typically governs as the majority of the lateral bracing elements are portal frames. It needs to be ensured that the transverse bracing elements can resist the full wind loading demand and a minimum of 67%NBS seismic loading.

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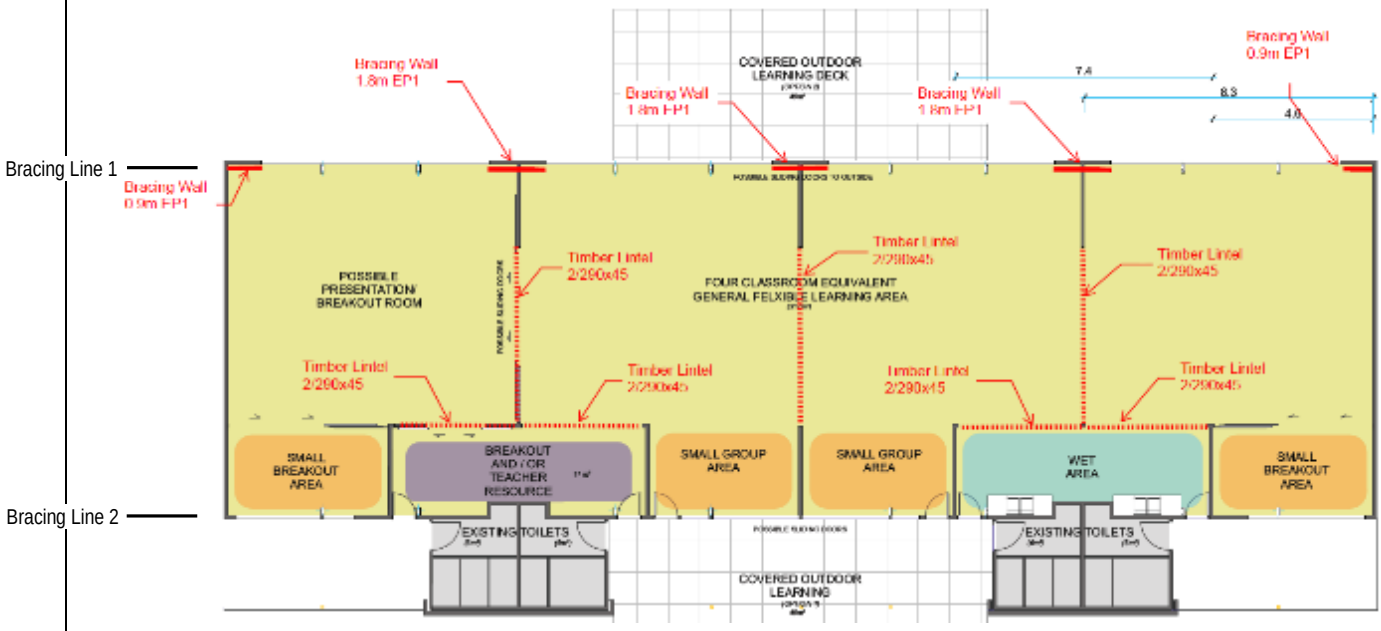
3. Lateral Bracing Design

3.1 Option 1

3.1.1 Overview/Bracing Scheme

The preliminary bracing scheme is shown in the figure below.



Preliminary Bracing Scheme – Option 1

Justify the preliminary structural scheme being adequate to resist lateral loading demand. The critical areas to check are as follows:

- Longitudinal Direction:
  - New bracing walls on Bracing Line 1
  - Existing bracing walls on Bracing Line 2
- Transverse Direction:
  - Existing bracing walls on the exterior bracing lines
  - Existing portal frames spaced evenly across the structure.

3.1.2 Transverse Direction

Lateral Demands

Utilise the existing timber framed shear walls at the ends as well as the 8 steel/timber portal frames spaced evenly across the structure.

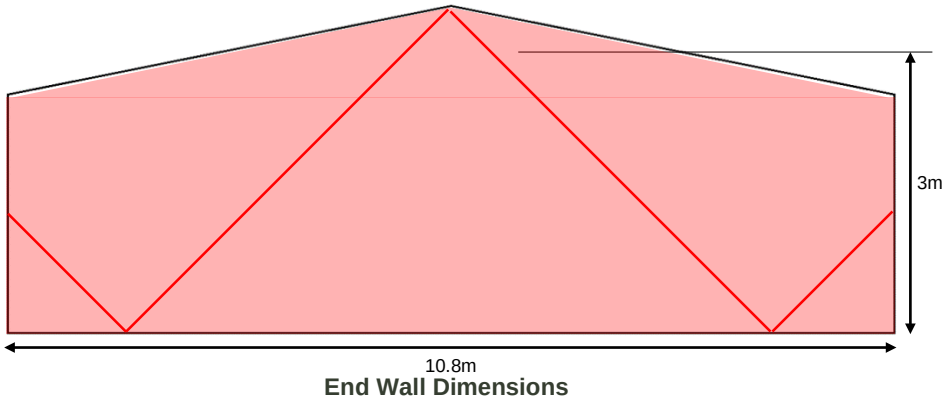
Since the timber framed walls are not being re-lined with a new Gib bracing material, use ductility 2.5, Sp 0.5 demands for these elements. Use a ductility of 1.25 for the portal frames.

The critical transverse lateral loading demands on the structure are equal to the following:

$F_EQ_{Trans2.5}$	**=**	**22.61kN**
$F_EQ_{Trans1.25}$	**=**	**67.98kN**
F_Wind_{Trans}	**=**	**30.89kN**

The seismic demands at ductility 1.25 are higher than the ductility 2.5 demands by a factor of 3.0.

Cross Braced Timber Shear Wall Capacity



End Wall Dimensions

Capacity of Timber Walls

Panel Length	L_T = 10.1 m	
Panel Height	H_T = 3 m	
Height Multiplier	f_{hmT} = 1.0	MOE Guidelines V2 (H/L < 2)
Bracing Capacity per metre	Bu_T = 2.75 kN/m	
Timber Wall Capacity	$Cap_T = L_T \times Bu_T \times f_{hmT}$	
	Cap_T = 27.78 kN	

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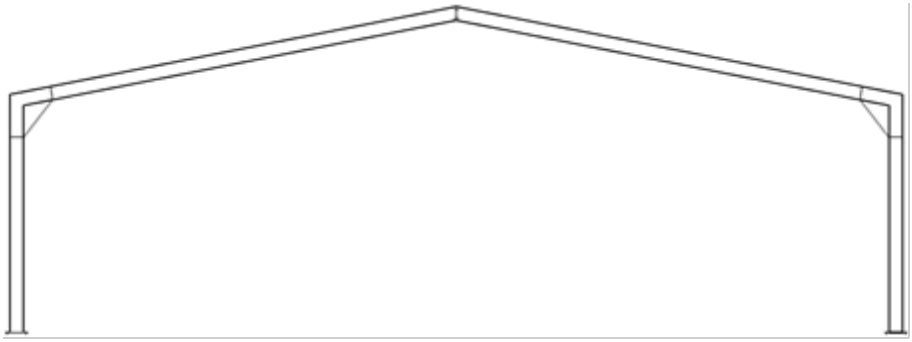
Steel Portal Frame Capacity

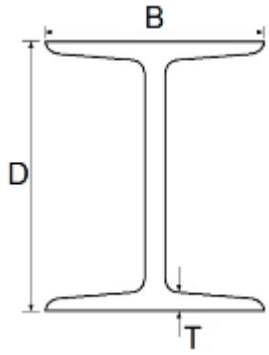


Steel portal frame elevation

Steel transverse portal frames are similar to 7x4x16 RSJ sections. These portal frames have pinned connection at the base and apex, so the moment capacity at the knee governs performance.

- **Steel Section Capacity**

The steel portal frames are constructed from a section similar to that of a 7×4×16 RSJ. This section has the following geometrical properties:



Steel portal frame section

f_y = 250 MPa

B = 101.6 mm

D = 177.8 mm

T = 6.35 mm

A_n = 3064.5 mm^2

I_{xx} = 16.455 × 10^6 mm^4

Z_{xx} = 185 × 10^3 mm^3

S_{xx} = 211.4 × 10^3 mm^3

r_x = 73.4 mm

Section moment capacity is calculated as follows.

$Z_e = \min(1.5 \times Z_{xx}, S_{xx}) =$ **211400.00** mm^3

$M_s = Z_e \times f_y$

$M_s =$ **52.85** kNm

Section axial capacity is calculated as follows.

k_f = 1.0

$N_s = k_f \times A_n \times f_y$

N_s = 766.13 kN

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Steel Member Capacity

The combination of axial load and bending moment

L_s = 2.5 m

k_e = 1.2

$L_e = k_e \times L_s$

$\lambda_n = (L_e/r_x) \times \sqrt{(k_f)} \times \sqrt{(f_y/250)}/1000$

λ_n = **40.87**

From NZS 3404, Table 6.3.3(2), with α_b = 0 and λ_n = 40.87:

α_c = 0.905

$N_c = \alpha_c \times N_s$

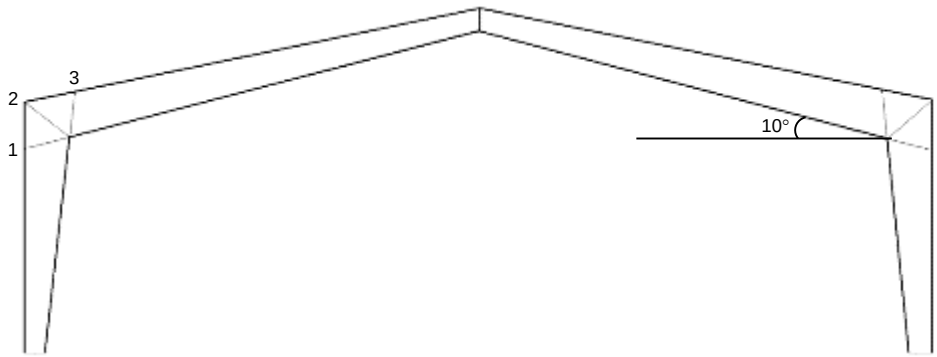
N_c = **693.34** kN

The portal frames are restrained against out-of-plane buckling by timber framing in the adjacent walls and windows. As the axial load is very low (~12 kN) in comparison to this axial load limit, it will be ignored. The bending moment limit is thus:

$\phi M_c = 0.9 \times M_s$

ϕM_c = **47.57** kNm

Glulam Timber Portal Frame Capacity



Glulam portal frame elevation

These portal frames have pinned connection at the base and apex, so the moment capacity at the knee governs performance.

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Cross Lapped, Glued Joint Capacity

The knee joints are comprised of 6 wood layers, with the joints alternating between the 3 configurations shown in the figure above. Both the rafter and column members are 15.5" by 6" at the apex so the nominal moment capacity (from chapter 19 of the Timber Design Guide) is:

$$M_n = k_1 \frac{d^2}{2} \sqrt{nEHb}$$

where

k_1 = duration of load

d = depth of rafter or column

n = number of gluelines

E = modulus of elasticity

$H = 240(1 + \sin^2\theta)$

θ = slope of rafter

b = breadth of rafter

$k_1 = 1.0$ d = 0.3937m n = 5 E = 8000MPa

b = 0.1524m $\theta = 10°$ $H = 240(1 + \sin(10)^2)$; H = 247.24 Nm/m^2

$M_n = k_1 \times (d^2/2) \times \sqrt{(n \times E \times H \times b)}$

M_n = **95.14** kNm

Section Capacity

b = 0.1524m

h = 0.3937m

f_b = 14MPa

Section Modulus $Z = (b \times h^2) / 6$ = **3936992.13** mm^3

Bending capacity **$M_n = f_b \times Z$**

M_n = **55.12** kNm

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Lateral Deflection Limit

Deflection is limited to 2.5% of the storey height. As the ductility of the portal frames will be limited to μ =1.25, deflections will be scaled up by a factor of 1.25 (NZS1170.5, Section 7.2.1.1). A drift modification factor of k_{dm} = 1.2 will also be used as the structure is less than 15m tall (NZS1170.5, Section 7.3.1.1).

Ductility Factor	μ = 1.25	
Drift Modification Factor	k_{dm} = 1.2	Table 7.1, NZS1170.5
Story Height	h = 2.6 m	
Unfactored Deflection Limit	d_{limit} = 2.5% × h = **65.00** mm	
Factored Deflection Limit	$d_{max} = d_{limit} / (\mu \times k_{dm})$	
	d_{max} = 43.33mm	

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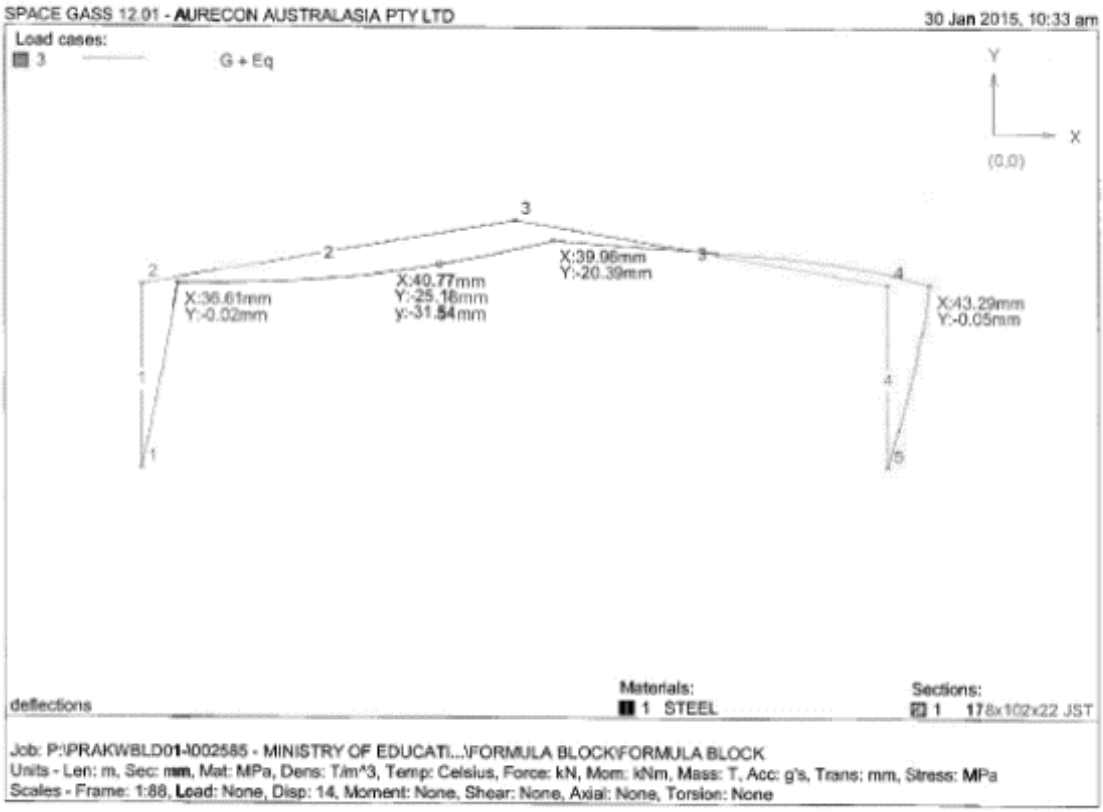
Corresponding Lateral Force

Both frame types have similar moment capacities. Deflection of the steel portal frame will govern the maximum lateral load that it can carry. The load corresponding to this deflection has been found using the analysis programme Space Gass. This lower bound load, along with the corresponding member forces, is shown in the Space Gass output below.



Maximum deflection almost reached in Space Gass

The maximum deflection of 43.33mm is close to being reached. The corresponding knee joint loads and member actions are included below.

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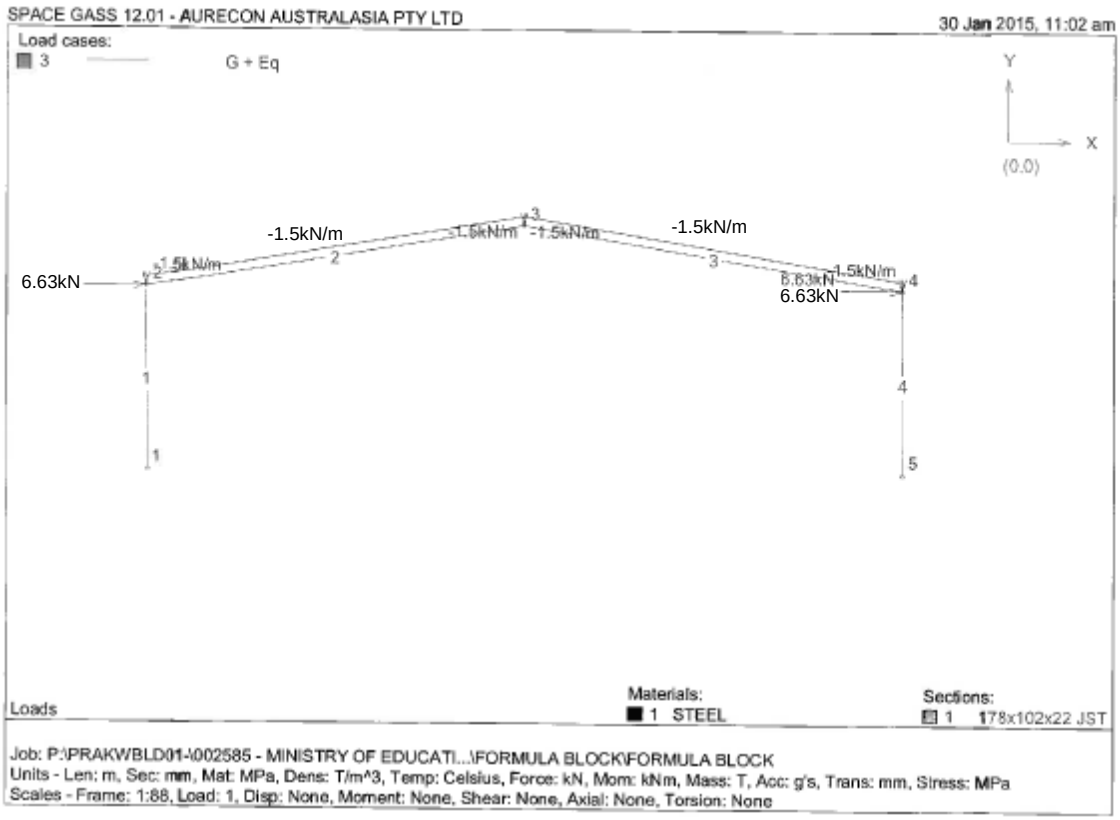
Load Diagram



Resulting loads from given deflection

This gives a total lateral load of 2 × 6.63 kN = 13.26 kN per portal frame.

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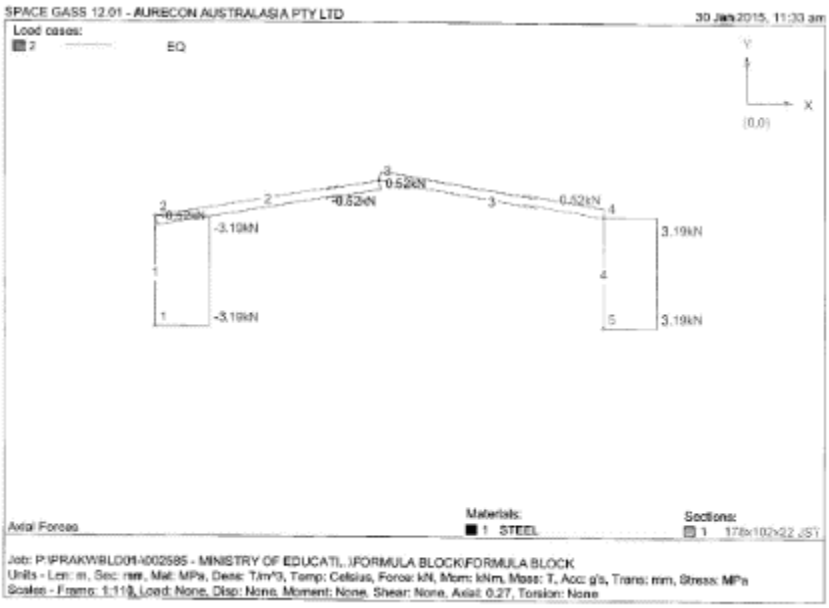
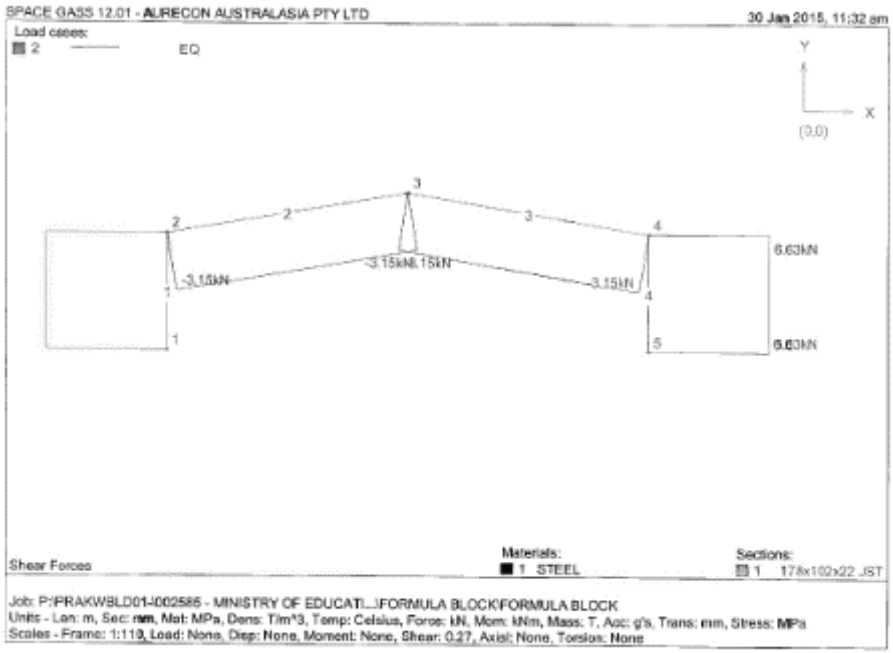
Axial Load



Resulting axial forces from given deflection

Shear Force Diagram



Resulting shear forces from given deflection

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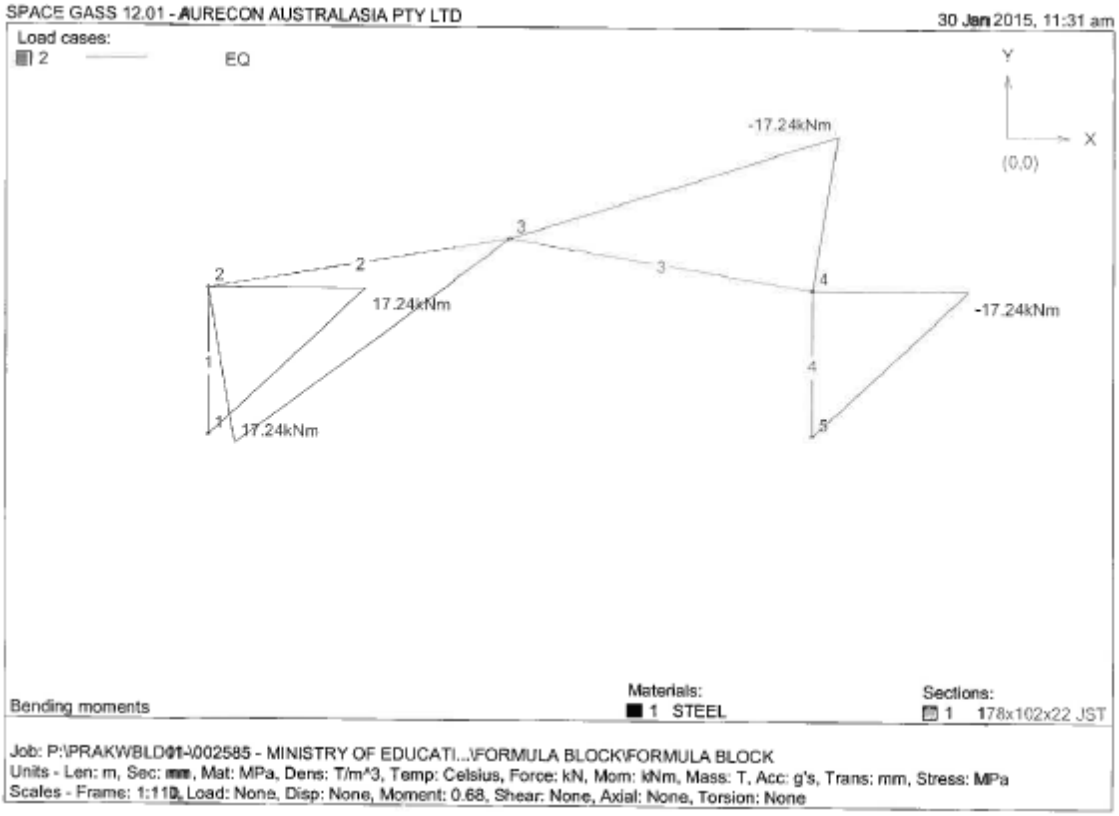
Bending Moment Diagram



Resulting bending moments from given deflection

All of the actions are below the calculated member capacities.

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%NBS Capacity – Transverse Direction

There are a total of 8 portal frames which can each carry a demand of 13.26kN at a ductility of 1.25.

The portal frames therefore have the following total capacity:

Cap_{Portal} = 13.26kN×8

Cap_{Portal} = **106.08kN**

The capacity of the portal frames is greater than the total demand in the transverse direction (wind and earthquake).

NBS Trans > 100%NBS

3.1.3 Longitudinal Direction

Bracing Line 1

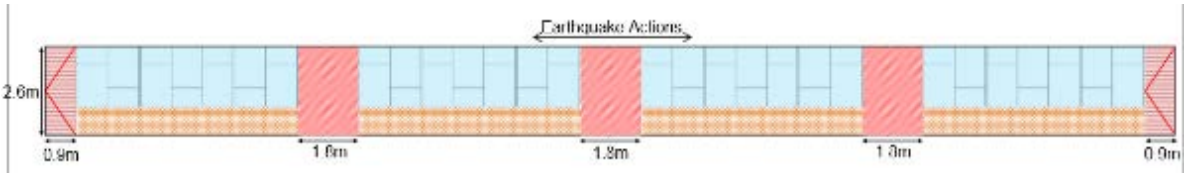
Re-line the existing timber framed walls with Gib. For new timber framed bracing walls, a ductility of 3.0 and Sp of 0.7 is a commonly accepted value. In past DSAs on this classroom type, the longitudinal bracing was found to be less than 67%NBS. Strengthen the structure to a minimum of 67%NBS.

$$F_BL1Duct3_{67\%} = Dem_{BL1} \times (C_dT_{3.0}/C_dT_{2.5}) \times 0.67$$

$$\mathbf{F_BL1Duct3_{67\%}} = \mathbf{9.98kN}$$

Try Gib GS1. This has a bracing capacity of 60Bu/m (3.0kN/m). The wall height along this bracing line is equal to 2.6m.

There are three 1.8m walls and two 0.9m walls along this bracing line (see figure below).



Bracing Line 1 Elevation

Check GS1 is adequate to resist the demand.

The capacity of Bracing Line 1 when lining the walls with GS1 is as follows:

$$FnBL1 = 3.0kN/m \times (2.4m/2.6m) \times [(0.9m \times 2)+(1.8m \times 3)]$$

$$\mathbf{FnBL1} = \mathbf{19.94kN}$$

$$> \mathbf{F_BL1Duct3_{67\%}} \quad \mathbf{OK}$$

It is therefore acceptable to line all of the walls on Bracing Line 1 with GS1. GS1 has sufficient capacity to resist 100%NBS on this Bracing Line.

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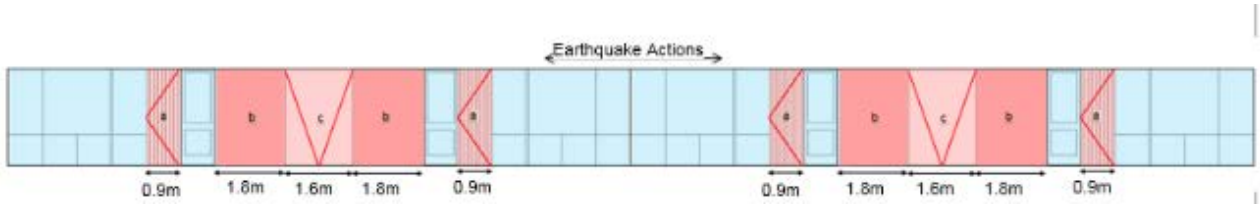
Bracing Line 2

Bracing line 2 has significant lengths of bracing as shown below. Check the existing capacity of these elements to confirm they are adequate to carry the seismic demand without any re-lining.



Bracing Line 2 Elevation

Bracing Line 2 contains four type ‘a’ cross braced panels, four type ‘b’ sarked panels and two type ‘c’ cross braced panels. As the cloakroom wall is to become glass, these walls have been ignored in the analysis, also the top half of the external wall of the toilets is glass so has also been ignored in this analysis.

Since the original timber walls are utilised here. Use the seismic demand for a ductility 2.5 (Sp 0.5).

Bracing Line 2 has the following demand at ductility 2.5:

F_BL2Duct2.5 = Dem_{BL2}

F_BL2Duct2.5 = 15.70kN

Capacity of Panel Type ‘a’

Number of Panels	$n_a = 4$	
Panel Length	$L_a = 0.9$ m	
Panel Height	H = 2.6 m	
Height Multiplier	$f_{hma} = 2L / H$	MOE Guidelines V2 (2 < H/L < 3.5)
	$f_{hma} = 0.69$	
Bracing Capacity per metre	$Bu_a = 2.75$ kN/m	
Contribution to Bracing Line 2	$Cap_{BLa} = n_a \times L_a \times Bu_a \times f_{hma}$	
	$Cap_{BLa} =$ **6.83** kN	

Capacity of Panel Type ‘b’

Number of Panels	$n_b = 4$	
Panel Length	$L_b = 1.8$ m	
Panel Height	H = 2.6 m	
Height Multiplier	$f_{hmb} = 1.0$	MOE Guidelines V2 (H/L < 2)
Bracing Capacity per metre	$Bu_b = 4.2$ kN/m	
Contribution to Bracing Line 2	$Cap_{BLb} = n_b \times L_b \times Bu_b \times f_{hmb}$	
	$Cap_{BLb} =$ **30.24** kN	

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Capacity of Panel Type 'c'

Number of Panels	$n_c = 2$	
Panel Length	$L_c = 1.6$ m	
Panel Height	$H = 2.6$ m	
Height Multiplier	$f_{hmc} = 1.0$	MOE Guidelines V2 (H/L < 2)
Bracing Capacity per metre	$Bu_c = 2.75$ kN/m	
Contribution to Bracing Line 2	$Cap_{BLc} = n_c \times L_c \times Bu_c \times f_{hmc}$	
	$Cap_{BLc} =$ **8.80** kN	

Total Bracing Capacity on Bracing Line 2

Total Bracing Capacity	$Cap_{BL2} = Cap_{BLa} + Cap_{BLb} + Cap_{BLc}$	
	$Cap_{BL2} =$ **45.87** kN	

%NBS Calculation

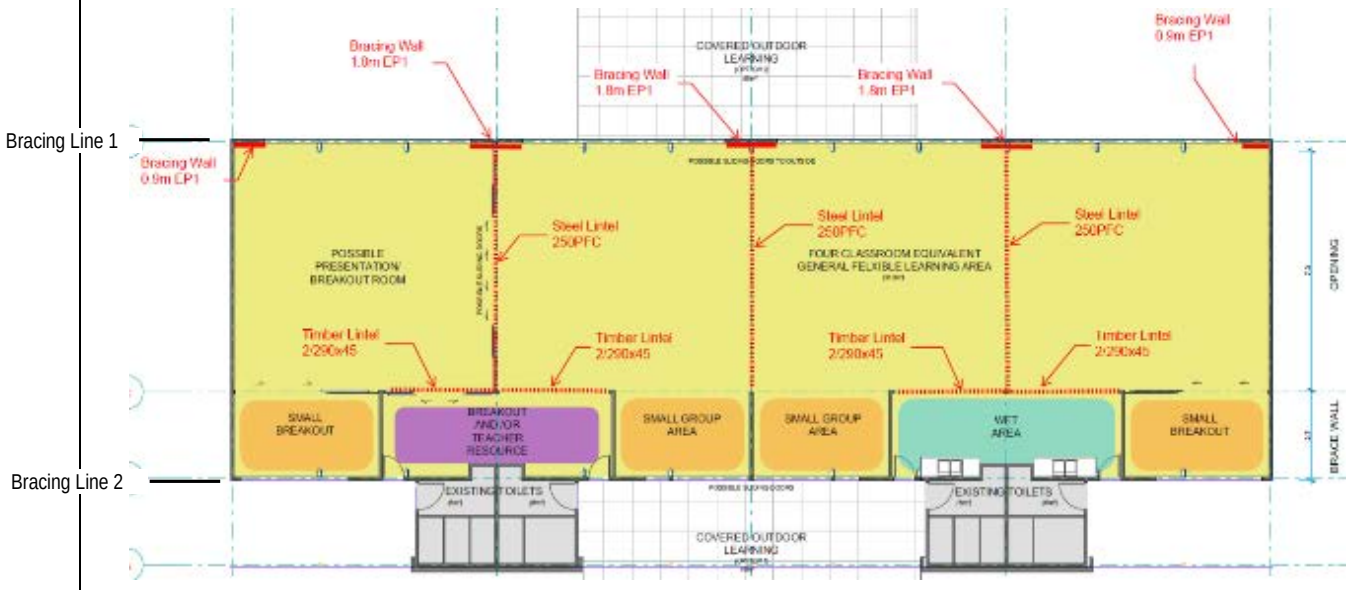
%NBS	NBS = Cap_{BL2} / F_BL2Duct2.5	
	NBS = **292.13**%NBS	

The capacity exceeds the demand, therefore no further strengthening is required on this Bracing Line.

3.2 Option 2

3.2.1 Overview/Bracing Scheme

The preliminary bracing scheme is shown in the figure below.



Preliminary Bracing Scheme – Option 2

Justify the preliminary structural scheme being adequate to resist lateral loading demand. The critical areas to check are as follows:

- Longitudinal Direction:
  - New bracing walls on Bracing Line 1
  - Existing bracing walls on Bracing Line 2

- Transverse Direction:
  - Existing bracing walls on the exterior bracing lines
  - Existing portal frames spaced evenly across the structure.

3.2.2 Transverse Direction

The transverse bracing layout is to remain unaltered similar to Option 1, with the lateral load resistance being provided by the 8 portal frames and the two timber framed end walls.

These elements were found to be adequate to resist seismic loading greater than 100%NBS. See 3.1.2 for the bracing capacity calculations in this direction.

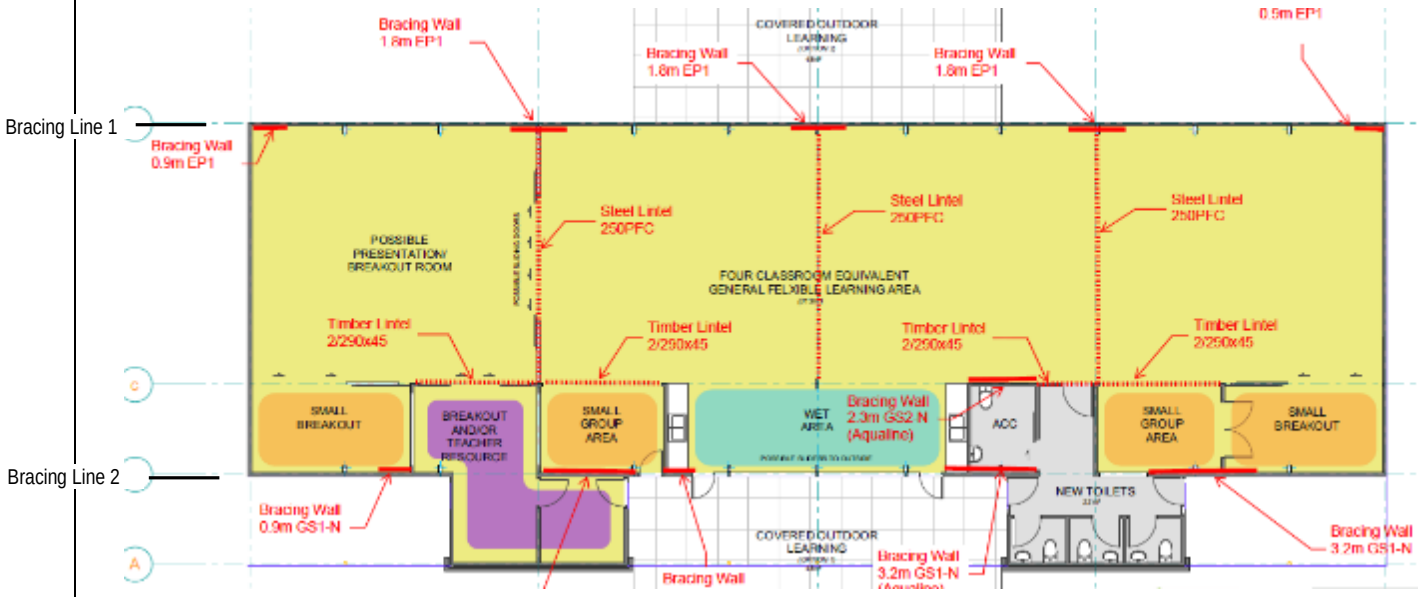
3.2.3 Longitudinal Direction

The longitudinal bracing system is the same in this option as Option 1. Refer to 3.1.3 for the bracing calculations.

3.3 Option 3

3.3.1 Overview/Bracing Scheme

The preliminary bracing scheme is shown in the figure below.



Preliminary Bracing Scheme – Option 3

Justify the preliminary structural scheme being adequate to resist lateral loading demand. The critical areas to check are as follows:

- Longitudinal Direction:
  - New bracing walls on Bracing Line 1 and 2
- Transverse Direction:
  - Existing bracing walls on the exterior bracing lines
  - Existing portal frames spaced evenly across the structure.

3.3.2 Transverse Direction

The transverse bracing layout is to remain unaltered similar to Option 1, with the lateral load resistance being provided by the 8 portal frames and the two timber framed end walls.

These elements were found to be adequate to resist seismic loading greater than 100%NBS. See 3.1.2 for the bracing capacity calculations in this direction.

3.3.3 Longitudinal Direction

Bracing Line 1

The existing walls are to be re-lined with GS1 similar to Option 1 and Option 2. See 3.1.3 for the bracing calculations.

Bracing Line 2

Some of the existing walls on this bracing line are to be removed. Re-line the remaining walls on this Bracing Line with a combination of Gib GS1 or GS2 as per the preliminary scheme shown below:

Since the walls will be re-lined with Gib, use a ductility of 3.00 (Sp 0.7) along this Bracing Line.

The seismic demand is therefore equal to the following:

$FBL2_{Duct3} = (C_dT_{3.0}/C_dT_{2.5}) \times Dem_{BL2}$

$FBL2_{Duct3} =$ **19.05**kN

As was found in 3.1.3 above, this demand is similar to the total capacity of the existing bracing line. Design the new bracing to resist this demand.

Gib GS1 has a capacity of 60Bu/m (3kN/m) and GS2 has a capacity of 85 Bu/m (4.25kN/m). Directly on Bracing Line 2, utilise 2x0.9m, 1x3.2m and 1x2.7m GS1 walls and a 2.6m GS2 wall directly on BL 2. There will also be a new bracing wall line on GL C (see bracing plan above). Utilise a GS2 wall on this grid line also (approximately 2.0m). This wall will be approximately 3.0m high.

The capacity on this Bracing Line is equal to the following:

FnBL2 = 2.4m/2.6m×[3kN/m×(0.9m+0.9m+3.2m+2.7m)+4.25kN/m×(2.6m)] +[(2.4m/3.0m)×4.25kN/m×2.0m]

FnBL2 = **38.32**kN

> $FBL2_{Duct3}$ **OK**

Also, Clause 5.4.7 in NZS 3604:2011 states that no bracing line shall have a bracing capacity less than the greater of 100 BUs or 50% of the total bracing demand divided by the number of bracing lines. Check the additional bracing line on GL C meets this criteria.

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Total Demand

$$\mathbf{F_EQ_{Long3.0}} = \mathbf{33.95\ kN}$$

Minimum allowable capacity per bracing line

$$Capmin = (\mathbf{F_EQ_{Long3.0}}/3)\times 0.5$$

$$Capmin = \mathbf{5.66}kN$$

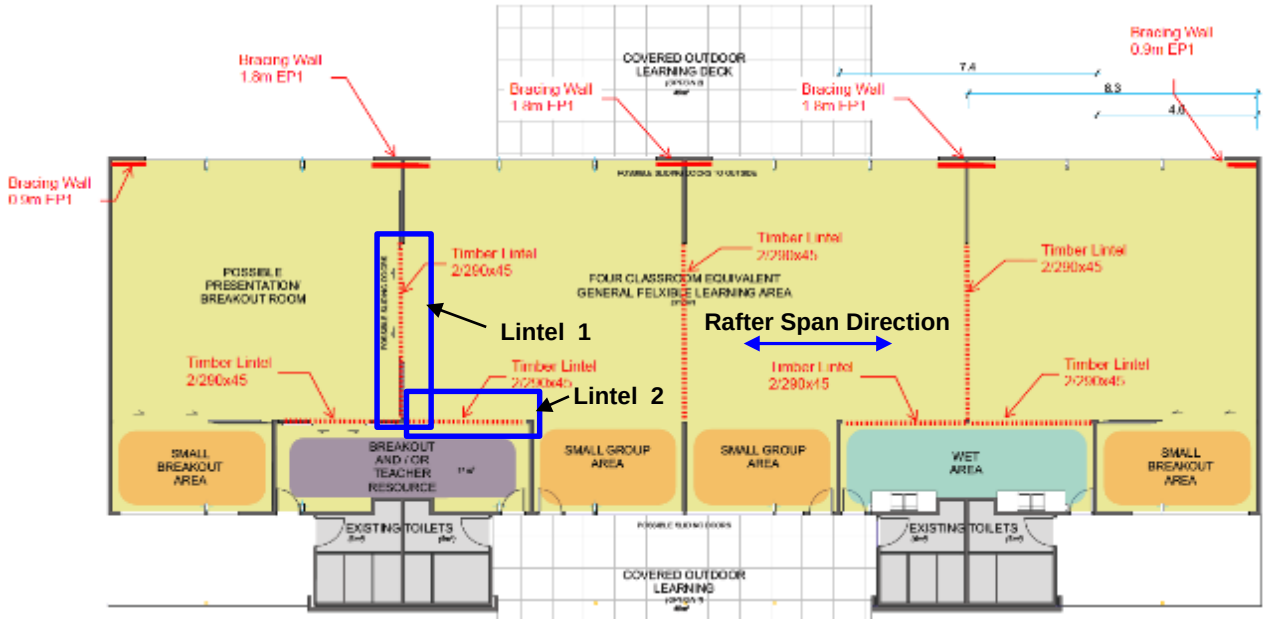
A 2.0m long, 3.0m high wall lined with Gib GS2 has a bracing capacity equal to 6.8kN, this is greater than the minimum capacity as per NZS3604 requirements.

4. Roof Lintel Design

4.1 Option 1

4.1.1 Lintel Layout

Three lintel types are to be designed to support the roof purlins in the areas where walls have been taking out. The locations of the lintels are shown in the plan below. These lintels are to be labelled Lintel 1 and Lintel 2.



4.1.2 Design Loads

Load Combinations

Check the lintels under the following load combinations from NZS 1170.0

ULS:

1.35G
1.2G+1.5Q
1.2G+Wu+ψcQ
0.9G+wu

SLS:

G+ψsQ
G+ψlQ
G+ψsQ+Ws
G+ψlQ+Ws

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Gravity Loading:

G = 0.4kPa (Roof – As was used for seismic weight calculations)

Q = 0.25kPa (Non Accessible Roof)

Wind Loading:

From 2.2.1), the following basic pressures were calculated:

P_{Basic} ULS = 0.84kPa

P_{Basic} SLS = 0.57kPa

Determine the design wind pressures on the lintels. Conservatively take the worst case wind loading for all lintels.

As per Cl 2.4.1 of NZS1170.2, the design wind pressure is calculated by the following equation:

$P_{design} = P_{basic} \times C_{fig} \times C_{dyn}$

C_{dyn} = 1.0 (Low rise single storey structure with natural frequency above 1Hz)

Determine Cfig for different loading faces and directions:

Internal:

Cp,i = -0.3, 0 (All walls equally permeable)

Roof:

The roof slope is equal to 10 degrees across the entire structure.

Longitudinal Loading

h = 3.0m (avg)

d = 33m

h/d = 0.09

Cp,e (R)Long = -0.3 (Table 5.3(C))

Transverse Loading

h = 3.0m (avg)

d = 10m

h/d = 0.3

Cp,e (U)Trans = -0.74 (Table 5.3(B)) Critical

Cp,e (D)Trans = -0.34 (Table 5.3(C))

The critical case will be on the upwind roof slope under transverse loading. Design all lintels for this worst case.

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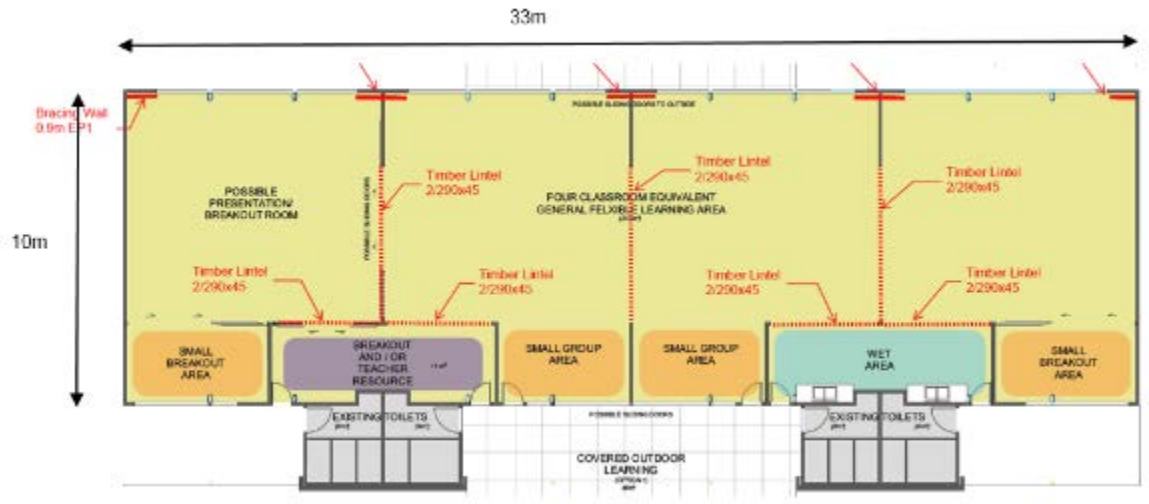

Ka	=	1.0 (Conservative)
Kc,e/Kc,i	=	0.8 (4 Effective Surfaces Table 5.5)
Kp	=	1.0
Kl	=	1.0 (Lintels not directly supporting the cladding)
Cfig,e	=	Cp,e x Ka x Kc,e x Kl x Kp
Cfig,e Up	=	-0.74 x 1.0 x 0.8 x 1.0 x 1.0
	=	-0.59
Cfig,i	=	Cp,i x Kc,i
Cfig,i	=	0 x 0.8
	=	0 (worst case)

Hence, the net Cfig factors are as follows:

Cfig net	=	-0.59

The ULS and SLS design pressures in both directions are therefore equal to the following:

PdesULS	=	-0.59 x 0.84kPa
	=	-0.50 kPa (Upwards)
PdesSLS	=	-0.59 x 0.57kPa
	=	-0.34 kPa (Upwards)

***Note: These design wind pressures are lower than that calculated for the Wellington region design. The same lintel members will be used for the Auckland design. The Wellington design loads therefore govern. The lintel design calculations shown on the pages to follow are for the Wellington region. Refer to the Wellington calculations for the wind load derivation.**

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4.1.3 Lintel 1 Design

Span = 5.1m

Tributary Width = 2.8m

2x290x45 timber members are suitable for this lintel. See spreadsheet calculations below:

Structural Timber Beam Calculation Sheet: Simply Supported

Job Name:	MoE ILS Formula Block	Date:	20-Aug-15
Job Number:	246313	Designer:	PGD
Description:	Lintel 1 - Uplift Wind		

1. Beam Properties

Depth d	290	mm	Nominal d	300	mm	Species	Radiata pine	Bending	14	
Thickness b	90	mm	Nominal b	100	mm			Strength fb		user defined
Length	5.10	m	d override		mm	Condition	Dry	Mpa	14	use for calcs.
Unsupp. L_{ay} downwards	1.00	m	b override		mm			Shear	3.8	
Unsupp. L_{ay} uplift	5.10	m	only enter overrides for non-standard			Grade	MSG8	Strength fs		user defined
I	1.83E+08	mm^4	dimensional values.					Mpa	3.8	use for calcs.
Z	1.26E+06	mm^3								
A	2.61E+04	mm^2		☐	At least 4 elements in the system which constrained to deformations similar to that of their neighbours (2.4.2.2)					
Self Weight	0.13	KN/m		☐	system consists of a single timber element (2.4.2.3)					
E	6700	MPa	use for calcs	☑	system consists of two or three elements acting together (2.4.2.3)					

Tick a box. Choose only one option

2. Load Data

case2 - single element beam or lintel (C2.4.2.3)

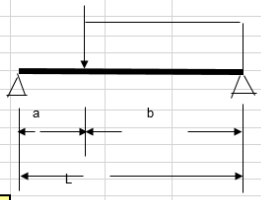
case3 - double element beam or lintel such as where 2x50mm wide timber elements are used to make up a 100mm wide element 9C2.4.2.3)

E=	8000	Mpa case1
Elb=	5400	Mpa case2
(E+Elb)/2=	6700	Mpa case3
E=		Mpa user defined

(1) Uniform Load

1. Floor			2.Roof		
Tribu Width	0.00	m	Tribu Width	2.80	m
Floor		kPa	Roof	0.40	kPa
Ceiling	0.00	kPa	Ceiling		kPa
Walls	0.00	kN/m	Parapet		kN/m
SDL	0.00	kPa	SDL		kPa
G_{total}	0.13	kN/m	G_{total}	1.12	KN/m
Q		kPa	Q	0.25	kPa
Q	0.00	kN/m	Q	0.70	KN/m
			Ws/Wu	0.72	
			Wu	-0.64	kPa
			Wu	-1.79	KN/m

Load combinations to: AS/NZS 1170:2002



Combination			Combination			Total kN/m	Reactions (kN)	Mmax (kNm)	Δmax (mm)
1.35G	0.1762	kN/m	1.35G	1.51	kN/m	1.69	4.3	5.5	12.13
1.2G +1.5Q	0.1566	kN/m	1.2G +1.5Q	2.39	kN/m	2.55	6.5	8.3	18.33
G +0.4Q	0.13	kN/m	G +0Q	1.12	kN/m	1.25	3.2	4.1	8.99
G +0.7Q	0.13	kN/m	G +0.7Q	1.61	kN/m	1.74	4.4	5.7	12.51
1.2G +0.4Q +Wu	0.16	kN/m	1.2G +0Q +Wu	-0.45	kN/m	-0.29	-0.7	-0.9	-2.09
0.9G +Wu	0.12	kN/m	0.9G +Wu	-0.78	kN/m	-0.67	-1.7	-2.2	-4.79
Ws	0.00	kN/m	Ws	-1.29	kN/m	-1.29	-3.3	-4.2	-9.27

3. Design Load Capacity

Bending Capacity		
Str. Reduction Factor	0.8	
Duration Factor K_1	0.6	
K_4	1.14	
K_5	1.0	
Unsupported Length L_{uy}	1000.0	mm
Slenderness Ratio S_1	9.7	
Slenderness Factor K_8	1.0	
Bending Stress fb	14.0	MPa
Fact. Bend. Str ϕM_{nx}	9.664	kNm
Bending Demands		
Ult. Mom. M*	8.3	kNm
Ult. Mom. M* Wind	-2.167	kNm
C/D Ratio	1.2	
Check	OK	

Shear Capacity		
Str. Reduction Factor	0.80	
Shear Stress f_s	3.80	MPa
Shear Area A_s	17400	mm^2
Fact Shear Str. ϕV_n	36.18	kN
Shear Demands		
Ult. Shear V*	6.50	kN
Ult. Shear V* Wind	-1.7	kN
C/D Ratio	5.56	
Check	OK	
Reactions		
Left (KN)	6.50	kN
Right (KN)	6.50	kN

	Summary Reaction	
	Left (KN)	Right (KN)
1.35G	4.3	4.3
1.2G +1.5Q	6.5	6.5
G +0.4Q	3.2	3.2
G +0.7Q	4.4	4.4
1.2G +0.4Q +Wu	-0.7	-0.7
0.9G +Wu	-1.7	-1.7
Ws	-3.3	-3.3

4. Deflection

Deflection (mm)				Limits (mm)				
G+0.4Q Includes K_2=2 (Accounts for long term sag)	G+0.7Q K_2=1.3	Ws K_2=1.0	Defl. under 1KN Point Load	Floor (L/500)	Roof (L/300)	Lintel (L/240)	Roof Wind (L/200)	1KN
18.4	16.7	-9.5	2.25	10.2	17.0	21.3	25.5	0.7
L/275	*L/305*	*L/535*		NG	OK	OK	OK	NG

4.1.4 Lintel 2 Design

Span	=	3.5m
Tributary Width	<	1.0m (Spans parallel to rafters)

By inspection 2x290x45 timber members are suitable here also. Could justify not requiring any lintel to be installed here as this lintel runs parallel to the existing roof framing.

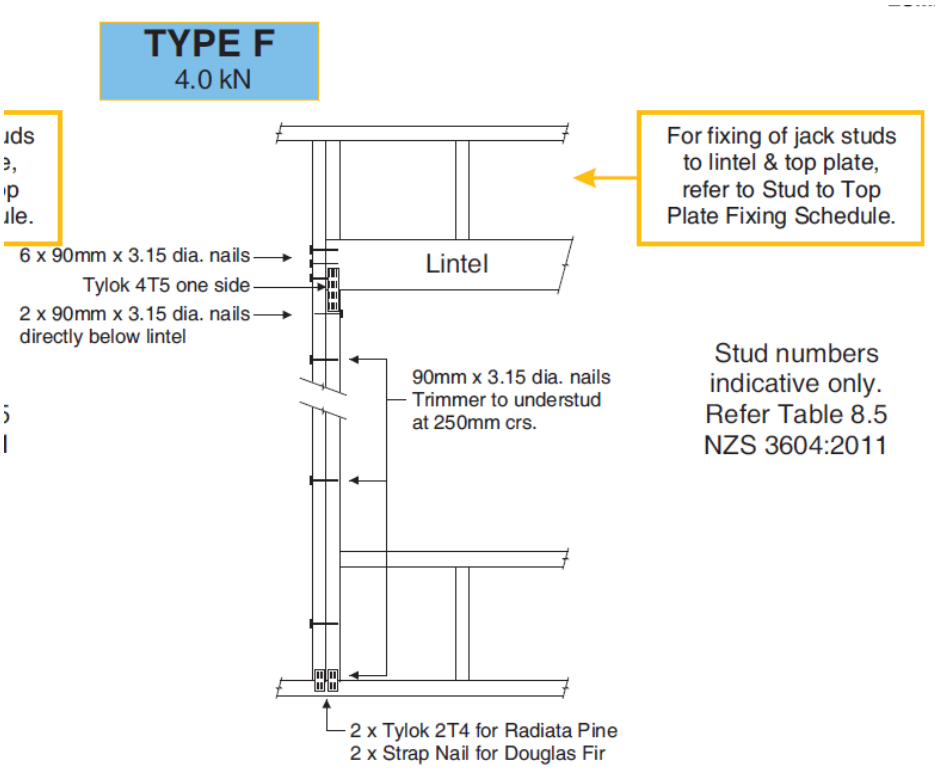
4.1.5 Lintel Fixings and Stud Supports

From the lintel calculations shown above, the critical reactions are as follows:

N*t	=	-1.7kN (Uplift – 0.9G+Wu)
N*c	=	6.5kN (Compression – 1.2G+1.5Q)

Design a lintel fixing to resist these loads.

Use a standard Lumberlok lintel fixing detail. The critical uplift demand is 1.7kN, therefore use a standard 4.0kN detail.



These details show a double stud as the lintel support. Check a double 90x45 stud is adequate to support the reaction loads.

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Tensile Capacity (In accordance with NZS 3603:1993):

	ϕN_{tstud}	=	$\phi \times k_1 \times k_4 \times f_t \times A$	
	ϕ	=	0.80	
	k_1	=	1.00	(Wind Loading Critical)
	k_4	=	1.14	(Two Support Elements)
	f_t	=	6.00 MPa	(SG8 Timber)
	A	=	90mm x 45mm x 2	
		=	$8100mm^2$	
Hence:	ϕN_{tstud}	=	$0.8 \times 1.0 \times 1.14 \times 6.00 \times 8100 \times 10^{-3}$	
		=	44.3kN	
		>	N*t	OK

Compression Capacity (In accordance with NZS 3603:1993):

	ϕN_{cstud}	=	$\phi \times k_1 \times k_8 \times f_c \times A$	
	ϕ	=	0.80	
	k_1	=	0.60	(Critical gravity demand similar to critical wind demand)
	f_c	=	18.00 MPa	(SG8 Timber)
	L_{ax}	=	3000mm	(Approximate worst case)
	L_{ay}	=	1000mm	(Conservative – Dwang spacing in wall the studs are built in)
	b	=	90 mm	
	d	=	90 mm	
	S_2	=	3000/90	(Lax/d)
		=	33.33	Critical
	S_3	=	1000/90	(Lay/b)
		=	11.11	
Hence:	k_8	=	0.277	(Table 2.8 – NZS3603:1993)
	A	=	$8100\ mm^2$	
Hence:	ϕNc_{Col}	=	$0.8 \times 0.6 \times 0.277 \times 18.0 \times 8100 \times 10^{-3}$	
		=	19.39 kN	
		>	N*c	OK

A double 90x45 stud is therefore adequate to support the lintel loads. Note, Lintel 2 will be supporting minimal roof loads as it spans parallel to the roof framing. Could justify not installing this lintel type at all.

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4.1.6 Jack Stud Framing

It is likely that there will be some clearance between the top of the lintel and the roof level. As a result, jack stud framing will be required between the roof and the top of the lintel. The maximum jack stud framing height is expected to be no greater than 800mm.

Try 90x45 jack studs at 600mm. From the lintel design, the critical uplift wind UDL was calculated to be -0.67kN/m and the critical downwards UDL was calculated to be 2.55kN/m. This correlates to an axial tension load of 0.40kN and an axial compression load of 1.53kN in the jack studs. Check the studs can support these loads.

Tensile Capacity (In accordance with NZS 3603:1993):

	ϕN_{tstud}	=	$\phi \times k_1 \times k_4 \times f_t \times A$	
	ϕ	=	0.80	
	k_1	=	1.00	(Wind Loading Critical)
	k_4	=	1.00	(One Element)
	f_t	=	6.00 MPa	(SG8 Timber)
	A	=	90mm x 45mm	
		=	$4050mm^2$	
Hence:	ϕN_{tstud}	=	$0.8 \times 1.0 \times 1.0 \times 6.00 \times 4050 \times 10^{-3}$	
		=	19.44kN	
		>	N*t	OK

Compression Capacity (In accordance with NZS 3603:1993):

	ϕN_{cstud}	=	$\phi \times k_1 \times k_8 \times f_c \times A$	
	ϕ	=	0.80	
	k_1	=	0.60	(Critical gravity demand similar to critical wind demand)
	f_c	=	18.00 MPa	(SG8 Timber)
	L_{ax}	=	800mm	(Approximate worst case)
	L_{ay}	=	800mm	(Approximate worst case)
	b	=	45 mm	
	d	=	90 mm	
	S_2	=	800/90	(Lax/d)
		=	8.88	
	S_3	=	800/45	(Lay/b)
		=	17.77	Critical
Hence:	k_8	=	0.800	(Table 2.8 – NZS3603:1993)
	A	=	$4050\ mm^2$	

Hence: ϕNc_{Col} = $0.8 \times 0.6 \times 0.8 \times 18.0 \times 4050 \times 10^{-3}$

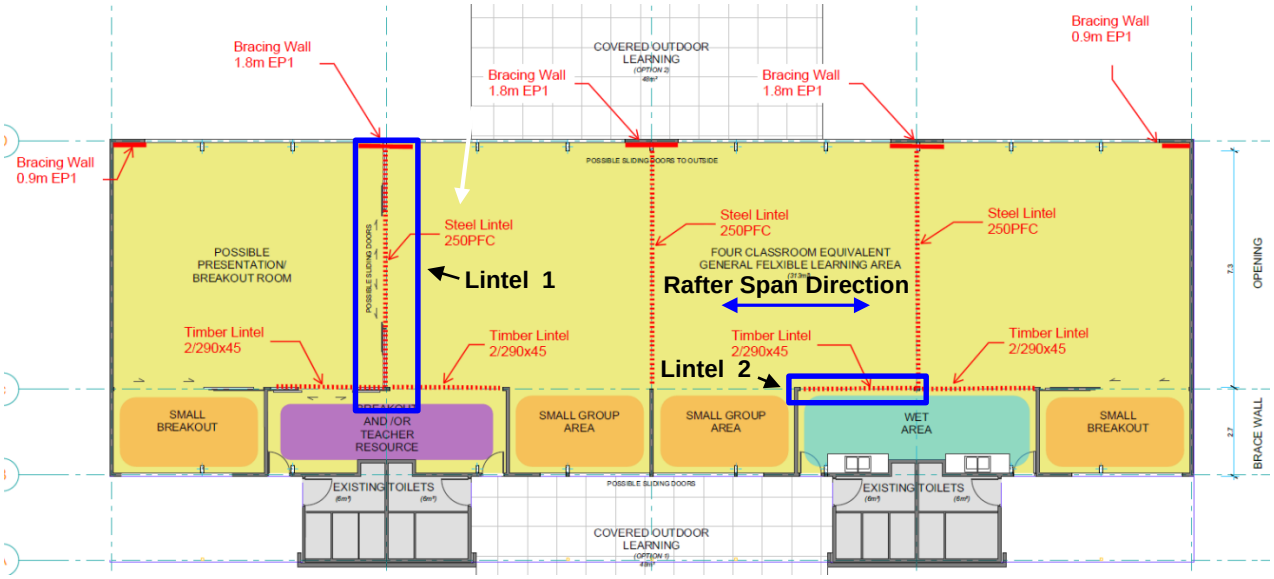
= 27.99 kN

> N*c OK

90x45 jack studs at 600 centres are therefore adequate to support the roof loads. By inspection it will be adequate to fix these jack studs to the lintels with Lumberlok CPC80 fixings.

4.2 Option 2

4.2.1 Lintel Layout



4.2.2 Design Loads

The demands will be the same as those shown in 4.1.2) for Option 1.

4.2.3 Lintel 1 Design

Span = 7.5m

Tributary Width = 2.8m

A 250 PFC is suitable for this lintel. See spreadsheet below for lintel capacity calculations.

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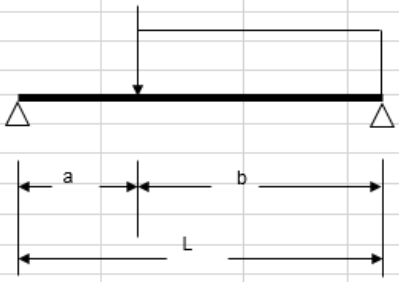
1. Beam Properties

Steel Beam	250PFC	
Span	7.50	m
Unsupp. L downward	7500	mm
Unsupp. L uplift	7500	mm
I	4.51E+07	mm^4
A	4.52E+03	mm^2
Self Weight	3.48E-01	KN/m
E	205000	MPa
Fy	300	MPa



2. Load Data

(1) Uniform Load

Load combinations to: AS/NZS 1170:2002

1. Floor			2. Roof		
Tributo Width		m	Tributo Width	2.80	m
Floor		kPa	Roof	0.40	kPa
Ceiling		kPa	Ceiling		kPa
Walls		kN/m	Parapet		kN/m
SDL		kPa	SDL		kPa
G_{total}	0.00	kN/m	G_{total}	1.12	KN/m
Q		kPa	Q	0.25	kPa
Q	0.00	kN/m	Q	0.70	KN/m
			Ws/Wu	0.72	
			Wu	-0.64	kPa
			Wu	-1.79	KN/m

Combination			Combination			Total kN/m	Reactions (kN)	Mmax (kNm)	Δmax (mm)
1.35G	0.00	kN/m	1.35G	1.51	kN/m	1.51	5.7	10.6	6.74
1.2G +1.5Q	0.00	kN/m	1.2G +1.5Q	2.39	kN/m	2.39	9.0	16.8	10.67
G +0.4Q	0.00	kN/m	G +0Q	1.12	kN/m	1.12	4.2	7.9	4.99
G +0.7Q	0.00	kN/m	G +0.7Q	1.61	kN/m	1.61	6.0	11.3	7.17
1.2G +0.4Q +Wu	0.00	kN/m	1.2G +0Q +Wu	-0.45	kN/m	-0.45	-1.7	-3.2	-2.00
0.9G +Wu	0.00	kN/m	0.9G +Wu	-0.78	kN/m	-0.78	-2.9	-5.5	-3.49
Ws	0.00	kN/m	Ws	-1.29	kN/m	-1.29	-4.8	-9.1	-5.75

3. Design Load Capacity

M^*_{max}	M^*_{wind}	Reaction		ϕM_{bx}	ϕM_{by}	Beam Span
KNm	KNm	Left (KN)	Right (KN)	kNm	kNm	m
16.8	-5.5	9.0	9.0	37.6	28.9	7.5
				OK		
				223%		

4. Deflection

Deflection (mm)				Limits (mm)			
G +0.4Q	G +0.7Q	Ws	Deflection under 1KN Point Load	Floor (L/500)	Roof (L/300)	Roof Wind (L/200)	1KN
5.1	7.4	-5.9	0.95	15.0	25.0	37.5	0.7
L/1465	*L/1015*	*L/1270*					
OK	OK	OK	NG				

4.2.4 Lintel 2 Design

Span	=	3.5m
Tributary Width	<	1m (spans parallel to the rafters)

As was found in Option 1, 2x290x45 timber members will be suitable here also.

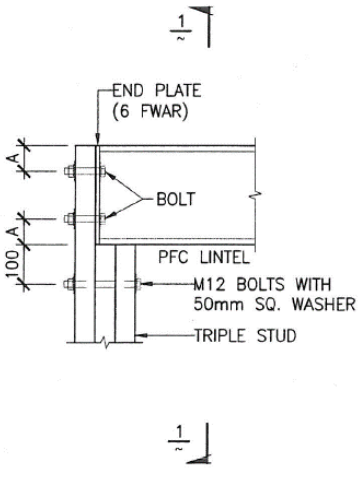
4.2.5 Lintel Fixings and Stud Supports

Steel PFC Lintels

From the lintel calculations shown above, the critical reactions are as follows for the PFC Lintels:

N*t	=	-2.9kN (Uplift – 0.9G+Wu)
N*c	=	9.0kN (Compression – 1.2G+Wu)

Design a lintel fixing to resist these loads. Utilise the following standard detail shown below with a triple stud and three M12 bolts. Check the stud is adequate to resist tension and compression loads and the bolts have adequate shear capacity to resist the uplift loading reactions.



MEMBER	END PLATE	BOLT	A
150PFC	6mm	M12	50mm
180PFC	6mm	M12	50mm
200PFC	10mm	M12	65mm
230PFC	10mm	M12	65mm
250PFC	12mm	M16	75mm
300PFC	12mm	M16	75mm

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Check a triple 90x45 stud is adequate to support the reaction loads.

Tensile Capacity (In accordance with NZS 3603:1993):

	ϕN_{tstud}	=	$\phi \times k_1 \times k_4 \times f_t \times A$	
	ϕ	=	0.80	
	k_1	=	1.00	(Wind Loading Critical)
	k_4	=	1.20	(Three Support Elements)
	f_t	=	6.00 MPa	(SG8 Timber)
	A	=	90mm x 45mm x 3	
		=	$12150mm^2$	
Hence:	ϕN_{tstud}	=	$0.8 \times 1.0 \times 1.20 \times 6.00 \times 12150 \times 10^{-3}$	
		=	70.00kN	
		>	N*t	OK

Compression Capacity (In accordance with NZS 3603:1993):

	ϕN_{cstud}	=	$\phi \times k_1 \times k_8 \times f_c \times A$	
	ϕ	=	0.80	
	k_1	=	0.60	(Critical gravity demand similar to critical wind demand)
	f_c	=	18.00 MPa	(SG8 Timber)
	L_{ax}	=	3000mm	(Approximate worst case)
	L_{ay}	=	3000mm	(Approximate worst case)
	b	=	90 mm	
	d	=	135 mm	
	S_2	=	3000/90	(Lax/d)
		=	33.33	Critical
	S_3	=	3000/135	(Lay/b)
		=	22.22	
Hence:	k_8	=	0.277	(Table 2.8 – NZS3603:1993)
	A	=	$12150\ mm^2$	
Hence:	ϕN_{cCol}	=	$0.8 \times 0.6 \times 0.277 \times 18.0 \times 12150 \times 10^{-3}$	
		=	29.10 kN	
		>	N*c	OK

A triple 90x45 stud is therefore adequate to support the lintel loads. At the base of the studs utilise a similar base fixing to that shown in Option 1. As a triple stud is used, this will require nailing on both faces of the studs to ensure all three members are adequately fixed together.

Bolt Shear Capacity

	ϕQnbolt	=	ϕ x n x k1 x k12 x k13 x Qsk
	ϕ	=	0.7
	n	=	2 bolts (worst case)
	k1	=	1.0 (wind loading)
	k12	=	1.0 (dry timber)
	k13	=	1.0 (less than 4 fasteners)
	Qsk	=	10.4kN (M12 bolts, be = 90mm)
Hence:	ϕQnbolt	=	0.7 x 2 x 1.0 x 1.0 x 1.0 x 10.4
		=	14.56kN
		>	N*t OK

2 M12 bolts are adequate.

Timber Lintels

The same detailing as Option 1 could be used. Also, as mentioned in Option 1 above, could justify not requiring these lintels due to the timber rafters spanning parallel to these lintels.

4.2.6 Jack Stud Framing

The UDL demands on the lintels will be similar to Option 1, therefore 90x45 studs at 600 centres will be adequate here also.

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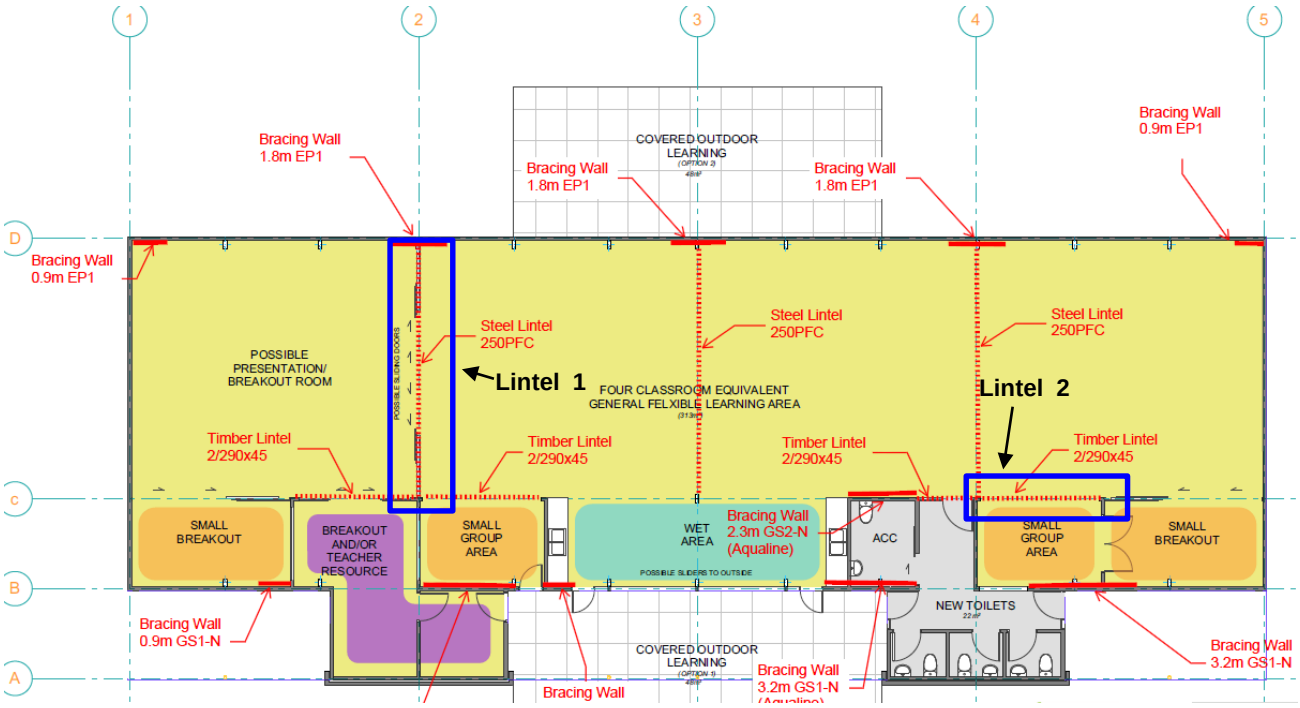
4.3 Option 3

4.3.1 Lintel Layout

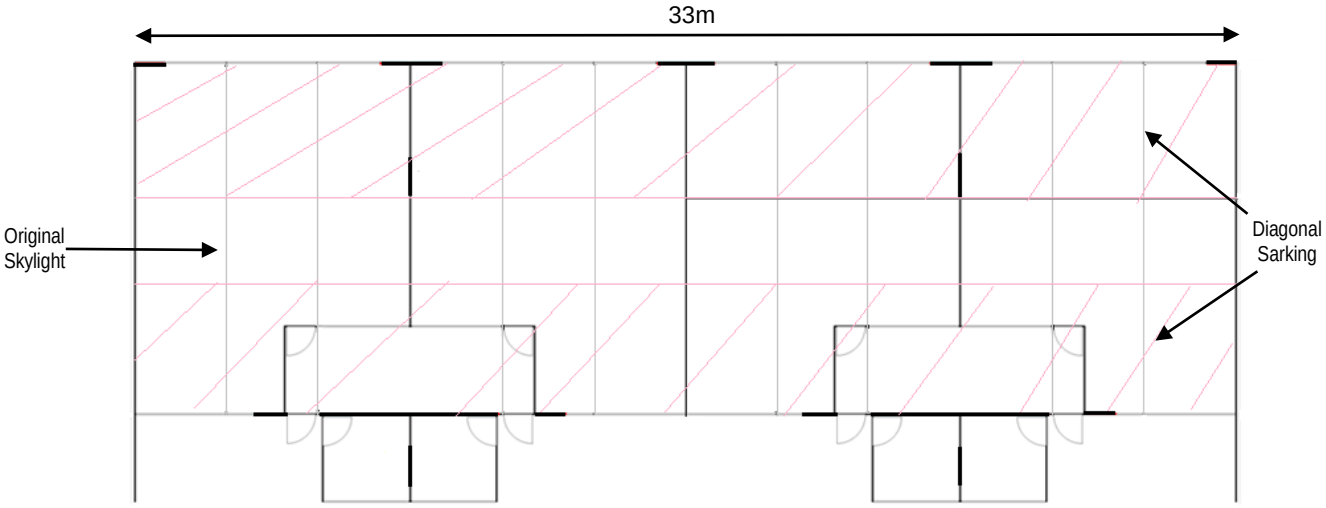


These lintels are similar to those in Options 1 and 2. A steel 200PFC minimum will be adequate for Lintel 1 and 2x290x45 timber members will be adequate for lintel 2.

5. Roof Diaphragm Capacity Check

5.1 Roof Diaphragm System

The existing ceiling diaphragm consists of diagonal timber sarking with discontinuity due to the original skylight across the centre of the structure. The existing ceiling diaphragm is shown in the figure below:



Original Sarking Layout

Although this skylight creates discontinuity in the diaphragm, it is likely to not be a significant issue. Under longitudinal loading, there will be minimal lateral seismic load going through the skylight as the seismic load is spread evenly between the two longitudinal bracing lines, based on tributary widths. The skylight will therefore only have to support seismic loading due to its self-weight which is insignificant.

5.2 Diaphragm Capacity Check – Longitudinal Direction

From the MOE and NZSEE seismic guidelines, sarking has been specified as having a capacity of 4.2kN/m.

Under longitudinal loading, the sarking diaphragm length is 33m, giving the diaphragm a total capacity of 138.6kN.

From 2.1.5, the total longitudinal seismic demand was found to be:

F_EQ_{Long} = **27.98**kN (Ductility 2.5, Sp 0.5)

As a roof diaphragm is expected to have minimal damage, scale the seismic demand up for a ductility of 1.25.

$F_EQ_{Long}1.25 = F_EQ_{Long} \times (C_dT_{1.25}/C_dT_{2.5})$

$F_EQ_{Long}1.25$ = **84.14**kN

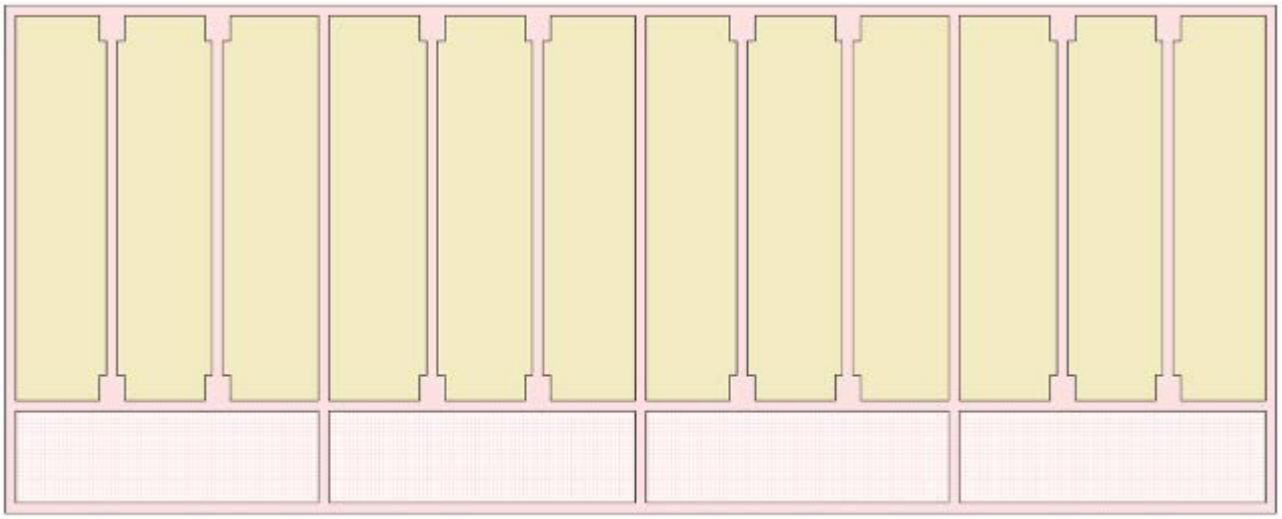
The sarking diaphragm will be adequate as the maximum shear in the diaphragm is expected to be a little over half the demand shown above. No further strengthening of the roof diaphragm will be required in this direction.

5.3 Diaphragm Capacity Check – Transverse Direction

In the transverse direction, there is a series of portal frames closely spaced together across the structure. By inspection, the roof diaphragm will be adequate in this direction. Minimum force will be required to be transferred across to these elements.

6. Floor Diaphragm / Foundations

The original foundations consist of a partly reinforced slab on hardfill. The sections of slab between the classrooms is largely unreinforced, but has reinforced thickenings to the perimeter and between the portal frames as shown in the figure below. The section of slab beneath the toilets and lean-to is reinforced with 665 HRC mesh. The perimeter ground beams provide some slab confinement.



Slab Layout

By inspection the slab is expected to perform satisfactorily unless the building is on a site that is prone to liquefaction, lateral spreading or differential settlement. Even in these situations failure of the slab is likely to result in out-of-plumbness and cracking, but not building collapse.

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7. Brick Veneer End Walls

We recommend that brick veneer end walls are replaced with lightweight timber weatherboards or similar. If these are required to remain then the following precautions should be taken. Note that it is likely that brick veneer will be damaged in a significant seismic event and require replacement.

Brick Restraints

Brick veneer end walls may present a fall hazard if not adequately tied to existing timber framing. The presence and condition of brick veneer ties should be checked by a qualified person. If brick ties are found to be inadequate the brick veneer should either be replaced with a lightweight alternative or new ties should be installed as outlined below:

If brick end walls are to be kept and existing ties are not adequate HeliFix stainless steel BowTie's or similar should be used to tie the veneer to the existing timber framing. Ties should be installed at no more than 600mm centres each way.