Project: Nationwide FLS Upgrades

Canterbury Block

Structural Calculations: Christchurch

Reference: 246313

Prepared for: Ministry of Education

Revision: 0

18/09/2015

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Document Control						
Report Title		Nationwide FLS Upgrades – Canterbury Block Structural Calculations: Christchurch				
Document ID		Canterbury Block FLS Upgrade Calculations Christchurch.docx	**Project Number**		232603	
File Path		P:\246313\03 Project Delivery\Design\Canterbury Block\CanterburyBlock FLS Upgrade Calculations Christchurch.docx				
Client		Ministry of Education	**Client Contact**			
Rev	**Date**	**Revision Details/Status**	**Prepared by**	**Author**	**Verifier**	**Approver**
0	18 September 2015	Issue to Client	P Don	P Don	S Randall	J Finnegan
Current Revision		0				

Approval			
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FLS Upgrade – Canterbury Block

Christchurch Region - Calculations

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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 an FLS for a Canterbury 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	Christchurch

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 size of the openings between the classrooms and the use of space in the annex structures at the rear of the main classrooms. 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.

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1. Building Overview

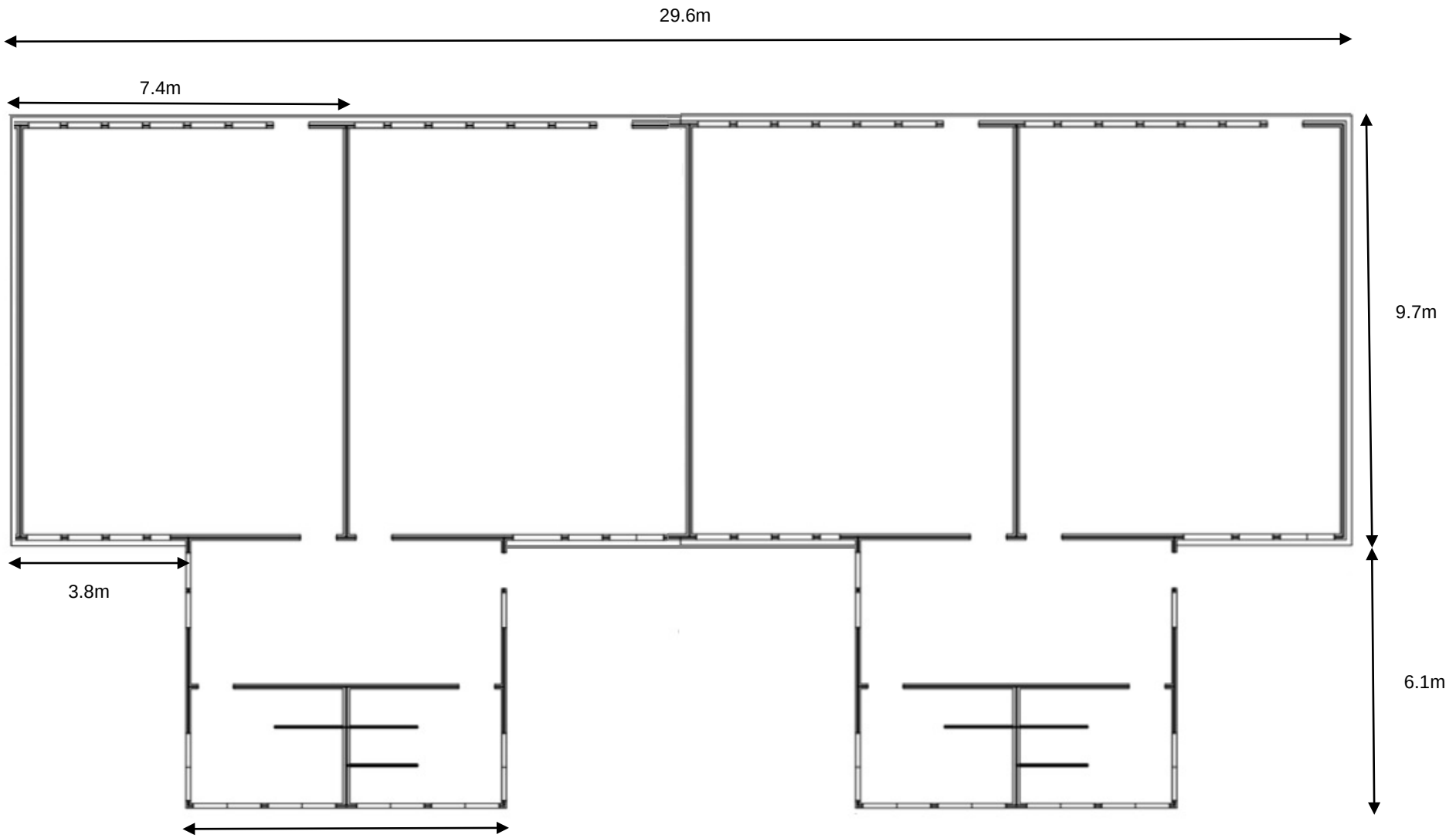
1.1 Original Structure

Standard Canterbury Blocks are rectangular, single-storey structures with attached toilet/cloakroom blocks centred at the rear of the building.

These blocks feature sheet-lined, timber-framed interior and exterior walls which provide lateral bracing in both the longitudinal and transverse directions. Brick or block veneers are also typically present on the exterior of the structure. The timber framed walls support a duo-pitched timber roof that spans transversely across the structure.

Typical Canterbury Block Building

The floor plan of a 4-classroom unit is shown in the figure below:



Floor plan of a 4-classroom unit showing wall layout

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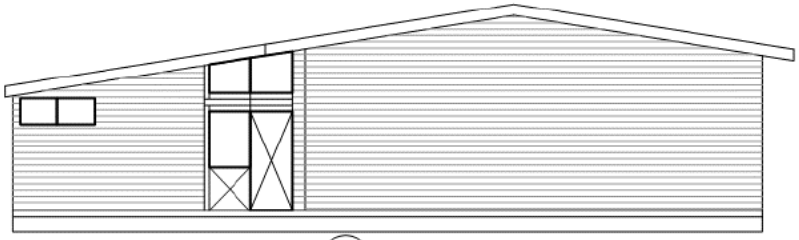
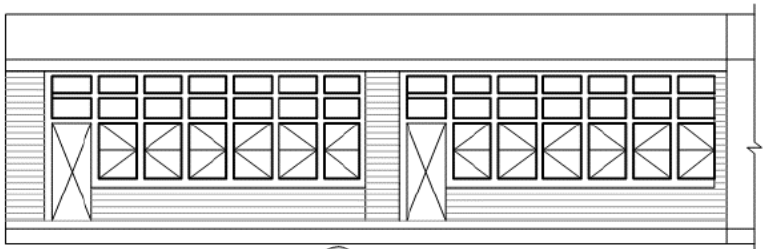
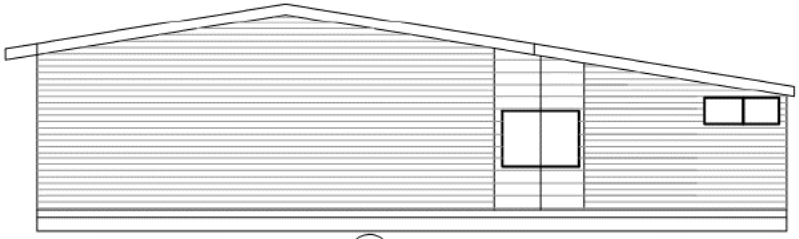


END ELEVATION
N.T.S



FRONT ELEVATION
N.T.S



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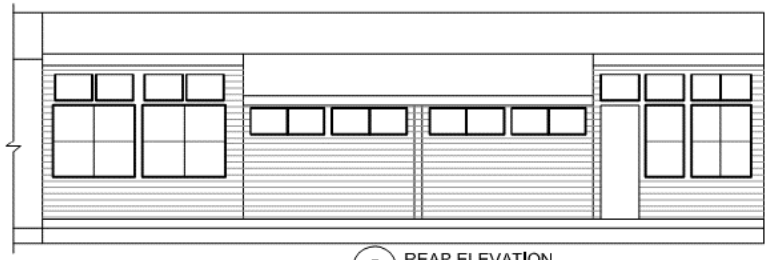
N.T.S



REAR ELEVATION
N.T.S

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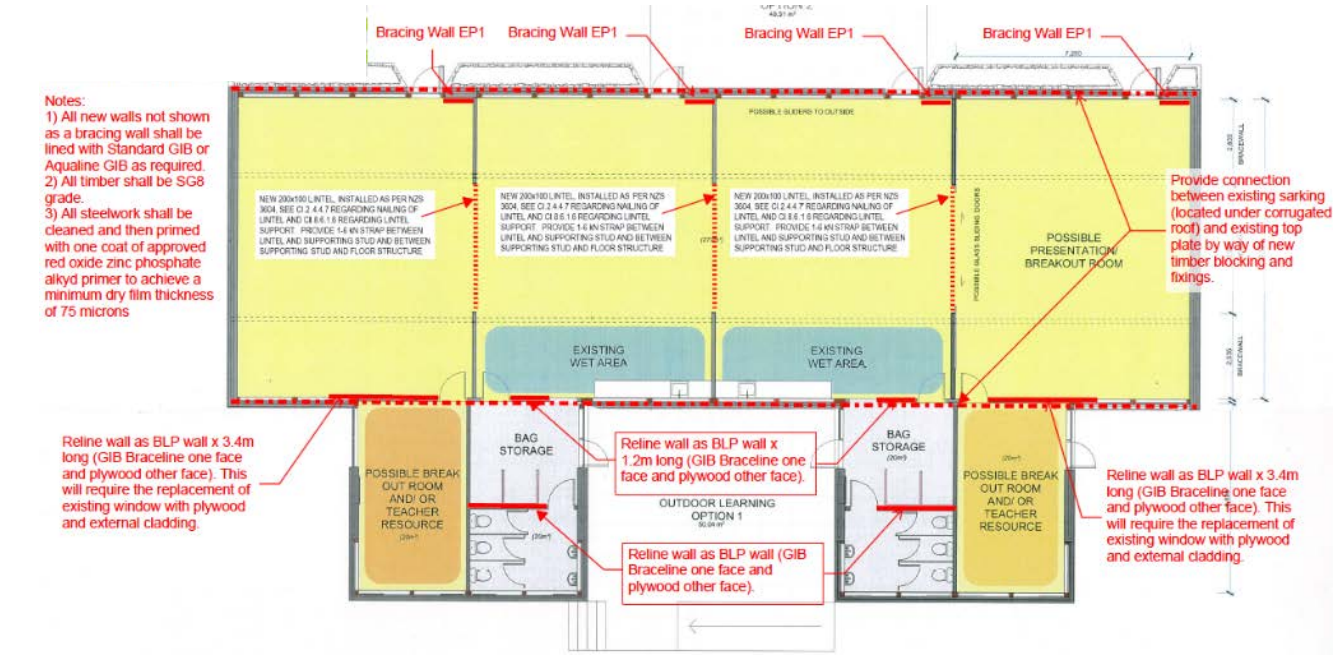
1.2 FLS Upgrade – Option 1

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 room for break out spaces and bag storage.

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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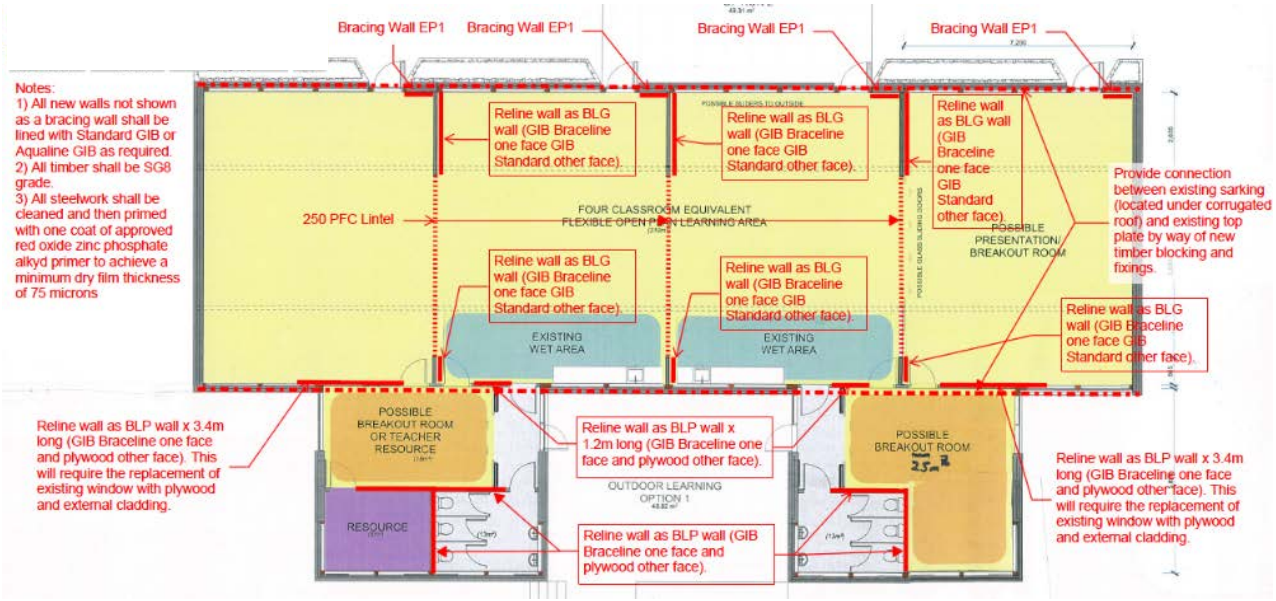
1.3 FLS Upgrade – Option 2

Option 2 for the FLS upgrade varies from Option 1 through there being 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.

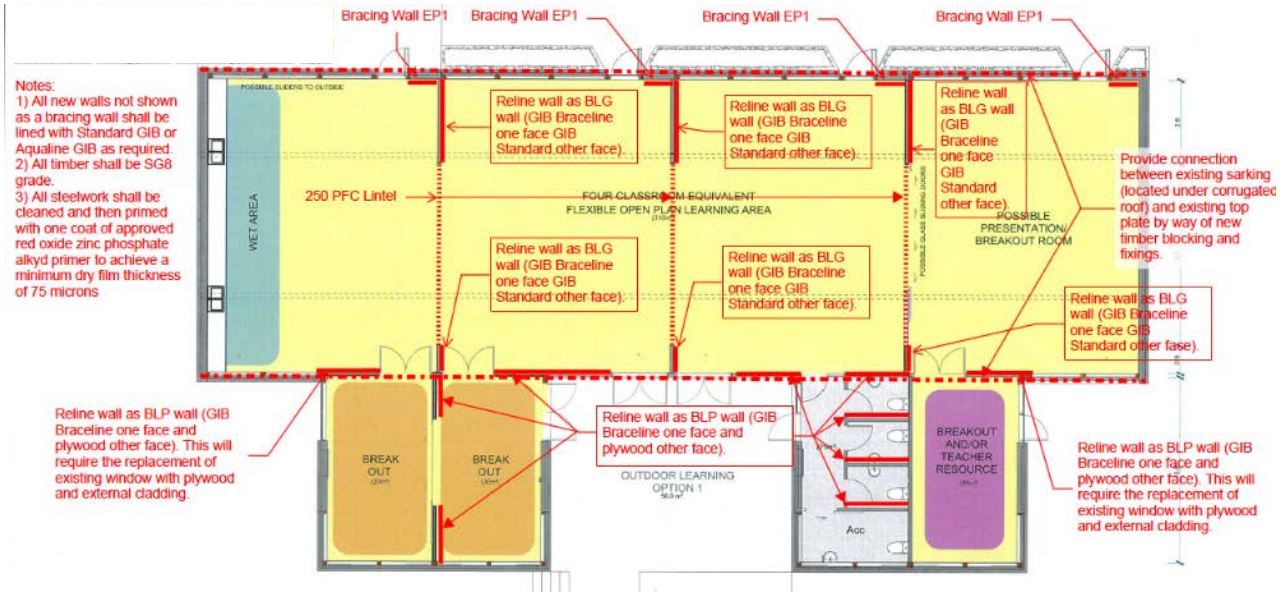


FLS Layout and Structural Scheme Option 2

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 the proposed wall layout is different, therefore requiring a different bracing layout.

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 (Typical) – All timber walls and windows will be assumed to have a weight of 0.3 kPa

Brick/Block Veneer – Brick/block veneer will be assumed to have a weight of 1.8 kPa (19kN/m^3 up to 100mm thick)

Imposed Actions:

Live Floor Loads – Live loads on classroom floors will be taken as 3.0 kPa as per NZS1170.1 Table 3.1.

Live Roof Loads – Live loads on non-accessible roofs will be taken as 0.25 kPa as per NZS1170.1 Table 3.1. This will be ignored in the seismic weight calculations (negligible) but will be included in the lintel design.

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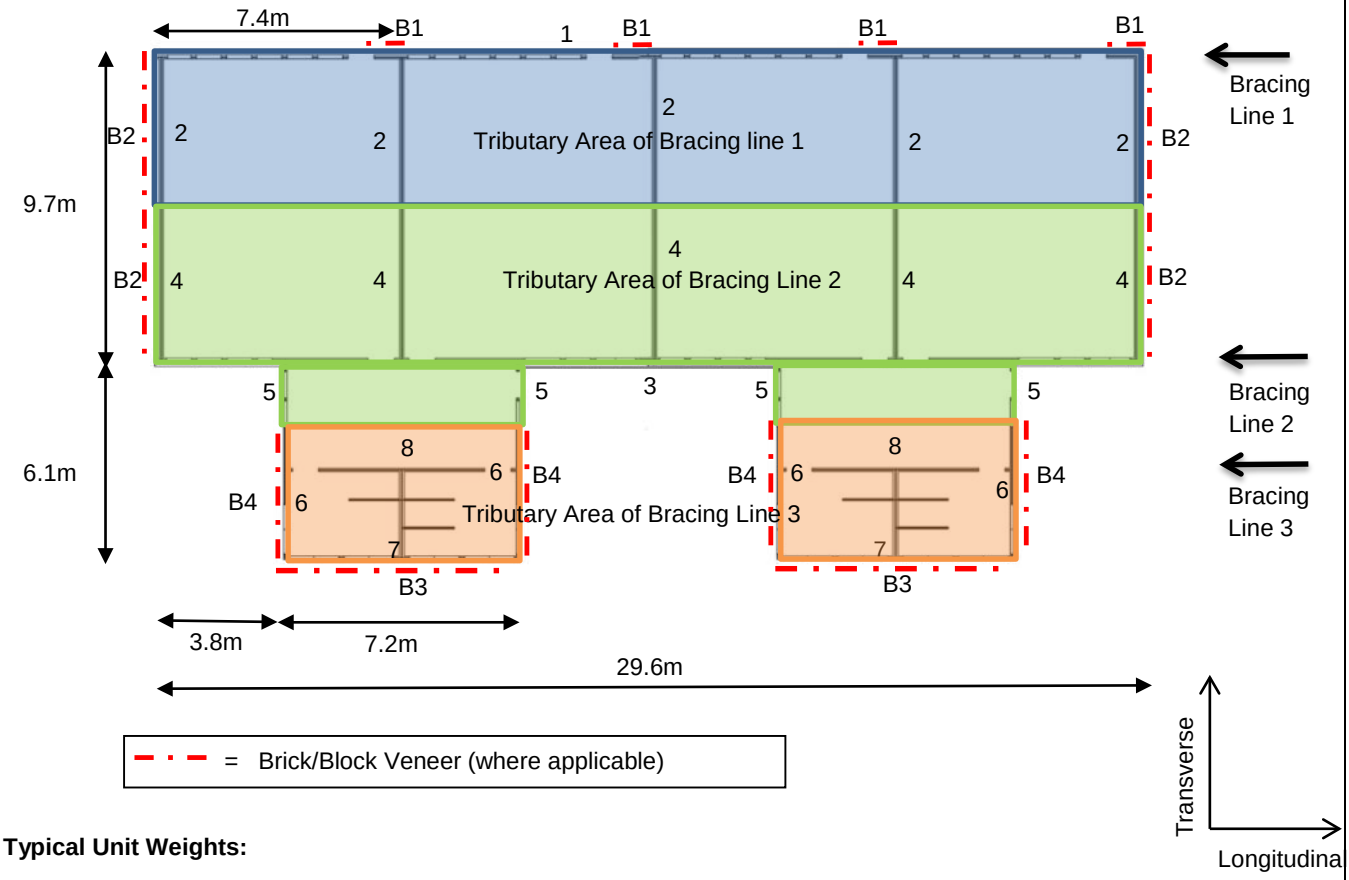
2.1.2 Seismic Weight of Classroom Section

The seismic weight will be calculated on the basis of tributary area of each bracing line.



Typical Unit Weights:

Wall weight	WW = 0.3kPa	Timber framed walls and glazing
	BW = 1.80kPa	Brick/Block Veneer
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

Lightweight Wall Weight

Wall	Number of Members	Length	Height	
1	1	29.6m	3.20m	W1 = **28.42 kN**
2	5	4.85m	3.60m	W2 = **26.19 kN**
	Total			**W_{BL1} = 54.61 kN**

Masonry/Block Wall Weight

Wall	Number of Members	Length	Height	
B1	4	1.00m	3.20m	BW1 = **23.04 kN**
B2	2	4.85m	3.60m	BW2 = **62.86 kN**
	Total			**BW_{BL1} = 85.90 kN**

Roof Weight

Plan Area

$Area_{BL1}$ = 29.6 m × 4.85 m
$Area_{BL1}$ =143.56 m^2

Total Roof Weight

$Roof_{BL1}$ = 0.4kPa × $Area_{BL1}$
$Roof_{BL1}$ = 57.42kN

Tributary Weight – Bracing Line 2

Lightweight Wall Weight

Wall	Number of Members	Length	Height	Weight
3	1	29.6m	3.20m	W3 = **28.42 kN**
4	5	4.85m	3.60m	W4 = **26.19 kN**
5	4	1.50m	3.00m	W5 = **5.40 kN**
	Total			**W_{BL2} = 60.01 kN**

Masonry/Block Wall Weight

Wall	Number of Members	Length	Height	
B2	2	4.85m	3.60m	BW2 = **62.86 kN**
	Total			**BW_{BL2} = 62.86 kN**

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Roof Weight

Plan Area — $Area_{BL2}$ = 29.6m×4.85 m + (1.5 m×7.2 m)×2

$Area_{BL2}$ = 165.16 m^2

Total Roof Weight — $Roof_{BL2}$ = 0.4kPa × $Area_{BL2}$

$Roof_{BL2}$ = 66.06kN

Tributary Weight – Bracing Line 3

Lightweight Wall Weight

Wall	Number of Members	Length	Height	Weight
6	4	4.60m	2.60m	W6 = **14.35 kN**
7	2	7.20m	2.40m	W7 = **10.37 kN**
8	2	6.00m	2.80m	W8 = **10.08 kN**
	Total			**W_{BL3} = 34.80 kN**

Masonry/Block Wall Weight

Wall	Number of Members	Length	Height	
B3	2	7.20m	2.40m	BW3 = **62.21 kN**
B4	4	4.60m	2.60m	BW4 = **86.11 kN**
	Total			**BW_{BL3} = 148.32 kN**

Roof Weight

Plan Area — $Area_{BL3}$ = 7.2m×4.6 m×2

$Area_{BL3}$ = 66.24 m^2

Total Roof Weight — $Roof_{BL3}$ = 0.4kPa × $Area_{BL3}$

$Roof_{BL3}$ = 26.50 kN

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2.1.3 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 – In both directions**
Site Subsoil Class (Section 3.1.3):	**D – Deep or Soft Soil (to be verified for a specific site)**
Site Location (Table 3.3):	**Site Location 102 - Christchurch**

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 Sp 0.7) **$C_dT_{3.0}$ = 0.2940**

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

Seismic Coefficient (ductility of 2.5 Sp 0.5) **$C_dT_{2.5}$ = 0.2423**

For the assessment of existing elements

Seismic Coefficient (ductility of 1.25 Sp 0.925) **$C_dT_{1.25}$= 0.7284**

For the design and assessment of non-ductile elements

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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:			

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)

102

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.3
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	0.3	1.0	0.9000	0.700	2.143	**0.2940**
	MRSM	3.0000			0.9000			0.2940
Y	ESM	3.0000	0.3	1.0	0.9000	0.700	2.143	**0.2940**
	MRSM	3.0000			0.9000			0.2940

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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:			

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)

102

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.3
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	0.3	1.0	0.9000	0.500	1.857	**0.2423**
	MRSM	3.0000			0.9000			0.2423
Y	ESM	3.0000	0.3	1.0	0.9000	0.500	1.857	**0.2423**
	MRSM	3.0000			0.9000			0.2423

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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:			

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)

102

Hazard factor and distance to nearest fault (Clause 3.1.4)

Hazard factor Z		0.3
Shortest distance D	Min.	N/A
	Max.	N/A

Return period factor (clause 3.1.5)

R	1.00

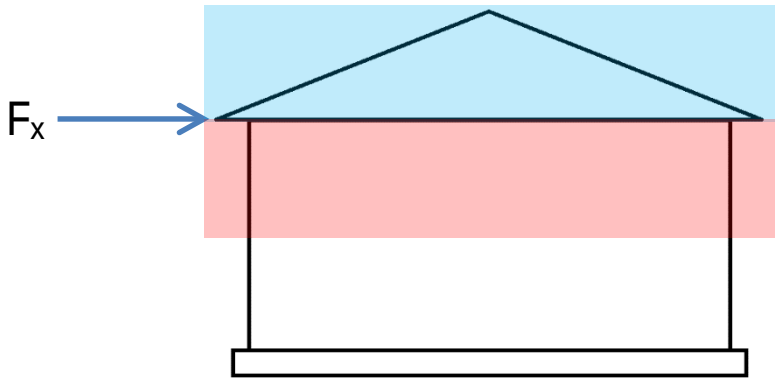
Horizontal design action coefficient (part 5, clause 5.2)

Direction	Analysis method	$C_h(T)$	ZR	N(T,D)	C(T)	S_p	k_μ	$C_d(T)$
X	ESM	3.0000	0.3	1.0	0.9000	0.925	1.143	**0.7284**
	MRSM	3.0000			0.9000			0.7284
Y	ESM	3.0000	0.3	1.0	0.9000	0.925	1.143	**0.7284**
	MRSM	3.0000			0.9000			0.7284

2.1.4 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.



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

- **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

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

Dem_{BL1} =30.94kN

Demand on Bracing Line 2

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

Dem_{BL2} = 30.89kN

Demand on Bracing Line 3

$$Dem_{BL3} = C_dT_{2.5} \times [(W_{BL3}/2)+(BW_{BL3}/2)+Roof_{BL3}]$$

Dem_{BL3} = 28.60kN

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

$$F_EQ_{Long} = Dem_{BL1}+Dem_{BL2}+Dem_{BL3}$$

F_EQ_{Long} = 90.43kN

- **Transverse Demand**

The transverse demand will be the same as the longitudinal demand.

Total Lateral Demand $\mathbf{F_EQ_{Trans} = F_EQ_{Long}}$

$\mathbf{F_EQ_{Trans} = 90.43kN}$

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	=	A7
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 Canterbury Block with different site parameters.

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

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AS1170.2 Design Wind Speed V8 — AS Standard References

Region Data

Region	A7		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	0.90	0.90	0.80	0.90	0.90	0.90	1.00	1.00	1.00	AS 1170.2 Table 3.2

Terrain/Height Multiplier $M_{z,cat}$ — 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 — AS 1170.2 Section 4.3

M_s	1.00		AS 1170.2 Section 4.3

Topographical Multiplier - M_t — 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
$V_{sit,\beta,Serv}$	27.64	27.64	24.57	27.64	27.64	27.64	30.71	30.71	30.71
$V_{sit,\beta,Ult}$	33.62	33.62	29.88	33.62	33.62	33.62	37.35	37.35	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°]

AS 1170.2 Figure 2.2

ms^{-1}	Wall 1	Wall 2	Wall 3	Wall 4	Any
$V_{des,\theta,Serv}$	30.7	27.6	27.6	30.7	30.7
$V_{des,\theta,Ult}$	37.4	33.6	33.6	37.4	37.4

INPUTS VALID

Basic Wind Pressure - p_{basic} $p = (0.5 \rho_{air}) [V_{des,\theta}]^2$ — AS 1170.2 Section 2.4

Multiply by C_{fig} and C_{dyn} to obtain Design Wind Pressures

kPa	Wall 1	Wall 2	Wall 3	Wall 4	Any	
$p_{serv.basic}$	0.57	0.46	0.46	0.57	0.57	
$p_{ult.basic}$	0.84	0.68	0.68	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 = 3.2m (avg – Including Annex Rooms)

d = 29.6m

b = 15.8m (Including Annex Rooms)

d/b = 1.9

Cp,e (L)Long = -0.32 (Worst Case - Table 5.2(B) roof pitch less than 10 Degrees)

Transverse Loading

h = 3.2m (avg– Including Annex Rooms)

d = 15.8m

b = 29.6m

d/b = 0.54

Cp,e (L)Trans = -0.50 (Worst Case Table 5.2(B) roof pitch less than 10 Degrees)

See over the page for the Formula Block dimensions.

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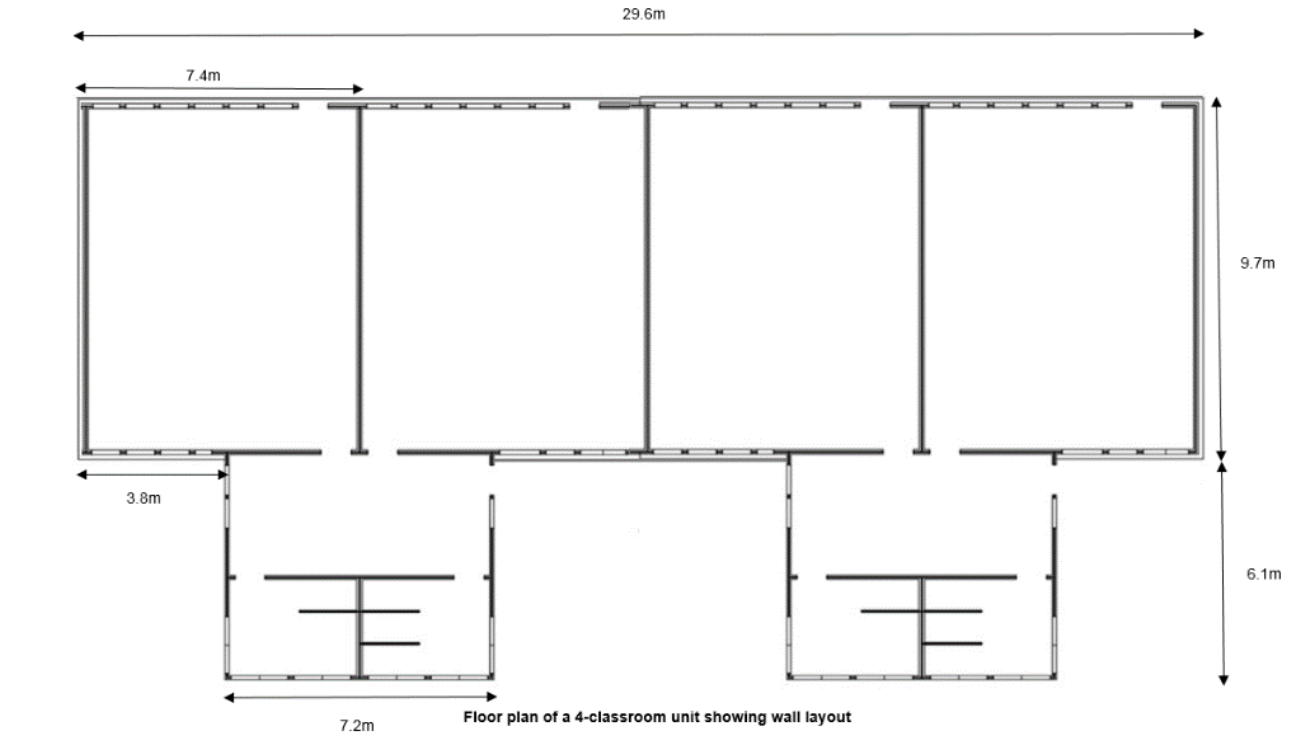
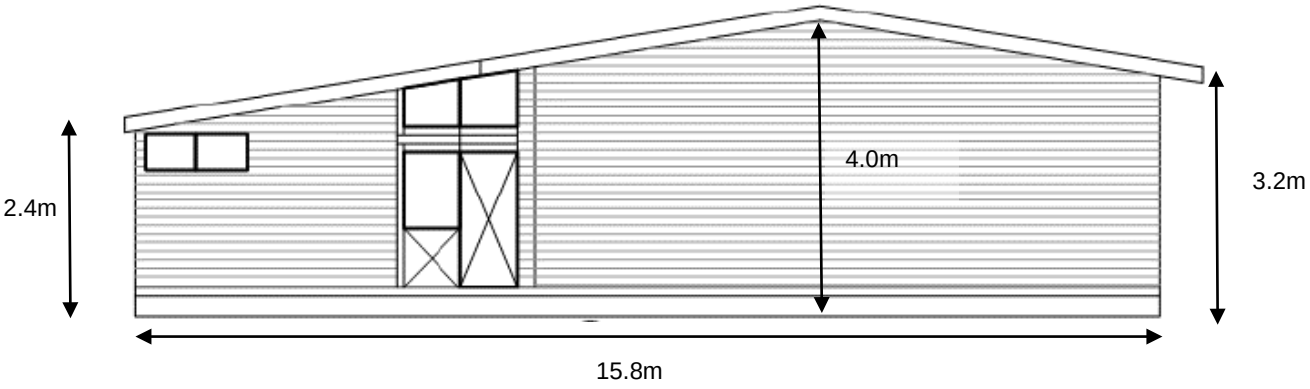


Floor plan of a 4-classroom unit showing wall layout



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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	=	-0.32 x 0.9 x 0.8 x 1.0 x1.0	
	=	-0.23	
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+0.23	(Windward and leeward)
	=	0.73	
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	=	0.73 x 0.84kPa
	=	0.61 kPa
PdesULSTrans	=	0.86 x 0.84kPa
PdesULSTrans	=	0.72 kPa
Wind Shear Forces		
F_Wind$_{Long}$	=	0.61kPa × (15.8m×3.2m×0.5)
F_Wind$_{Long}$	**=**	**15.42kN**
F_Wind$_{Trans}$	=	0.72kPa × (29.6m × 3.2m × 0.5)
F_Wind$_{Trans}$	=	**34.10**kN

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

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

$\mathbf{F_EQ_{Long3.0}} = \mathbf{109.73\ kN}$

$\mathbf{F_Wind_{Long}} = \mathbf{15.42kN}$

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

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

$\mathbf{F_EQ_{Trans3.0}} = \mathbf{109.73kN}$

$\mathbf{F_Wind_{Trans}} = \mathbf{34.10\ kN}$

Hence, seismic loading governs the lateral bracing design in this direction. 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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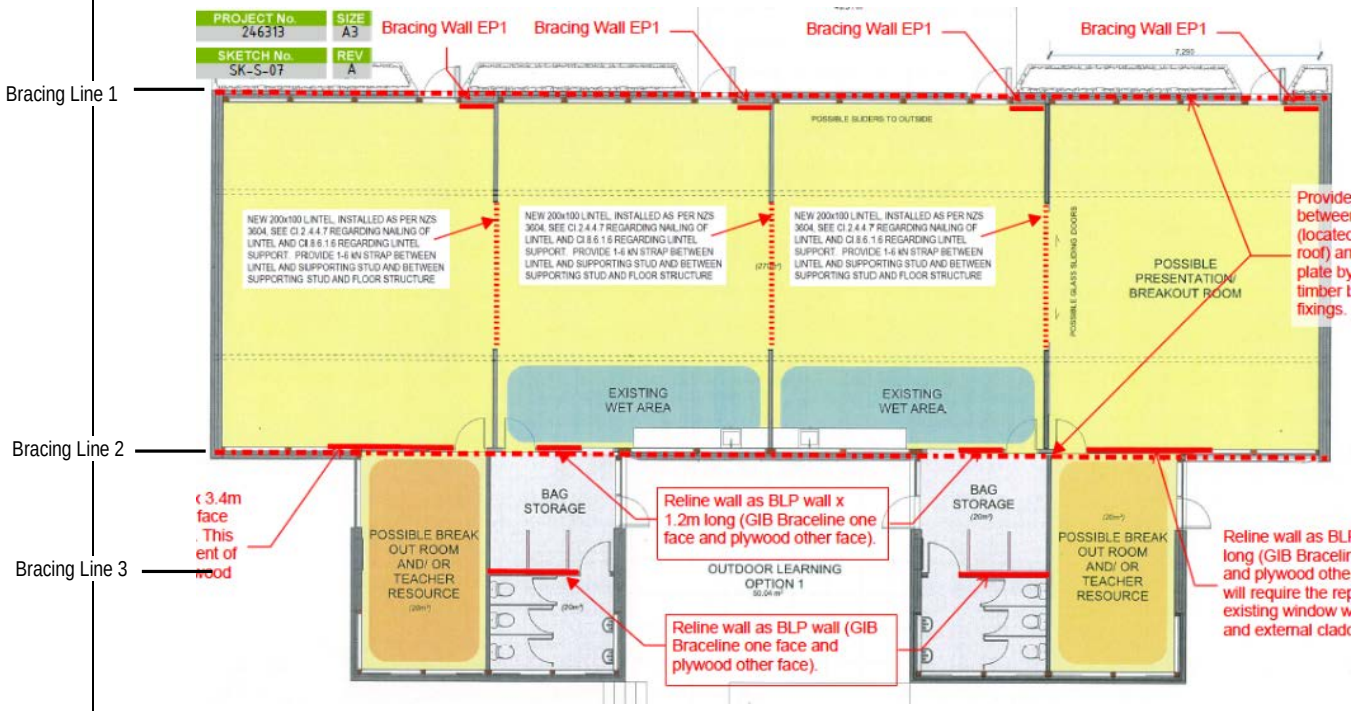
3. Lateral Bracing Design

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 Lines 1-3
- Transverse Direction:
  - Existing bracing walls on all transverse bracing lines

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3.1.2 Transverse Direction

Lateral Demands

Separate the demands in the main classroom area (Bracing Line 1 and Bracing Line 2 demands) with the demands in the end blocks.

Check the capacity of the existing timber framed walls to see if they are adequate at resisting seismic loads without requiring re-lining.

Some of the transverse walls in the main classroom areas will be cut out in the centre. Check the remaining walls can resist 100%NBS. If not, some re-lining may be required.

Since the intention is for the timber framed walls to not be re-lined with a new Gib bracing material, use ductility 2.5, Sp 0.5 demands for these elements.

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

Main Classroom

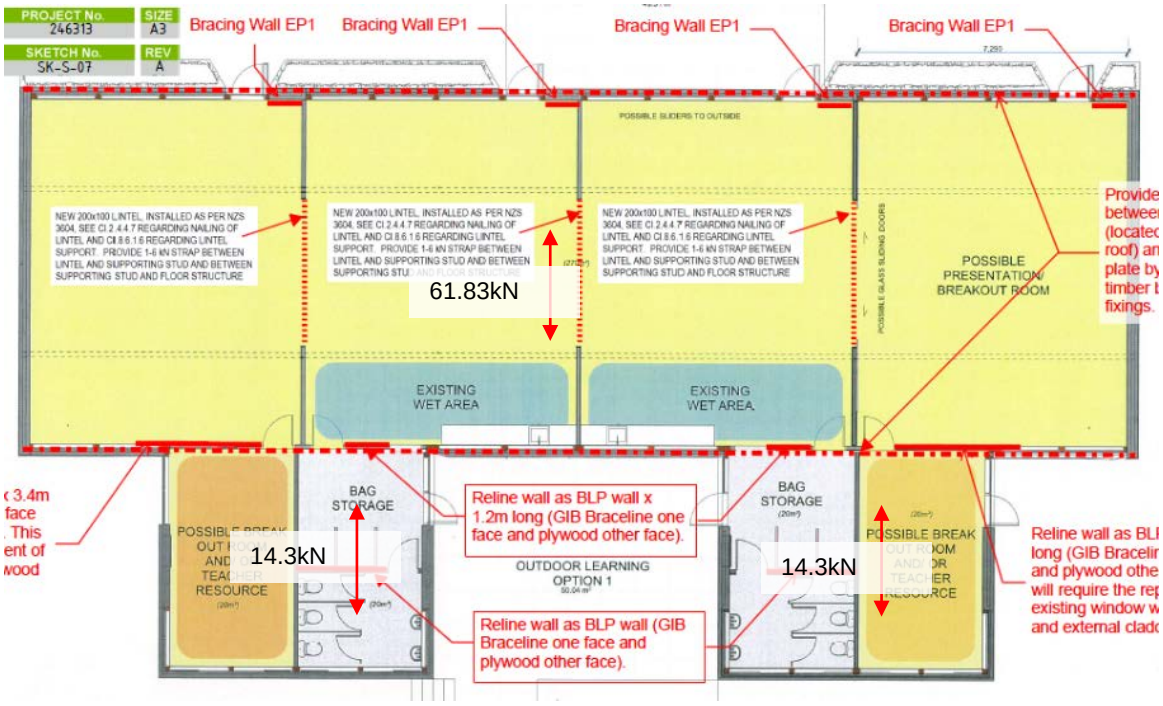
$$F_EQ_{MainClass2.5} = Dem_{BL1}+Dem_{BL2}$$

$$F_EQ_{MainClass2.5} = 61.83kN$$

End Block Extensions

$$F_EQ_{Extension2.5} = Dem_{BL3}$$

$$F_EQ_{Extension2.5} = 28.60kN$$



Based on tributary widths, distribute 1/4 of the seismic demand to the three internal wall lines in the Main Classroom and 1/8 of the demand to the end walls.

FTransIntWall = $(1/4)\times F_EQ_{MainClass2.5}$

FTransIntWall = **15.46**kN

FTransExtWall = $(1/8)\times F_EQ_{MainClass2.5}$

FTransExtWall = **7.73**kN

In the end block extensions, there are three evenly spaced gridlines. Compare the total demand in these areas with the total capacity.

Wall Capacities – Main Classroom:

Base the bracing capacities of the existing walls on the information provided in Table 7.1 in the MOE guidelines for the seismic evaluation of timber framed buildings.

Table 7.1: Wall Bracing Ratings for Assessing Pre-1978 Timber Framed Buildings

Bracing Element	Rating (BU/m)	Basis
Walls with diagonal braces within framing – lathe and plaster.	55	NZSEE 2006 Table 11.1 (ductility 2.0)
Walls with diagonal braces within framing – sheet material on one face.	50	Based on NZS 3604:1981
Walls with diagonal braces within framing – sheet material on both faces.	74	Based on NZS 3604:1981
Walls with no diagonal brace - match lining on one or both faces.	25	From NZSEE 2011, based on University of Auckland testing.
Walls with no diagonal brace - sarking on one or both faces.		
One nail per stud	50	From calculation
Minimum 2 nails per stud	84	NZSEE 2006 Table 11.1
Walls with no diagonal brace – sheet bracing (excluding Softboard)		
One face	50	From BRANZ 1992
Both faces	60	

- For H/L > 3.5: Factor = 0
- For 2 > H/L > 3.5: Factor = 2L/H
- For H/L < 2: Factor = 1

The available drawings do not provide detailed information on the wall framing. Assume the walls have no diagonal bracing but have sheet lining on 2 faces for the internal walls and 1 face for the external walls. This gives the exterior walls a bracing capacity of 50Bu/m (2.5kN/m) and the interior walls a bracing capacity of 60Bu/m (3kN/m). This is a reasonable lower bound capacity.

The exterior walls are 9.7m long, with an average height of 3.6m. An opening out of the interior walls will result in there being two, 2.7m long walls on each gridline. These walls will have an average height of 3.4m.

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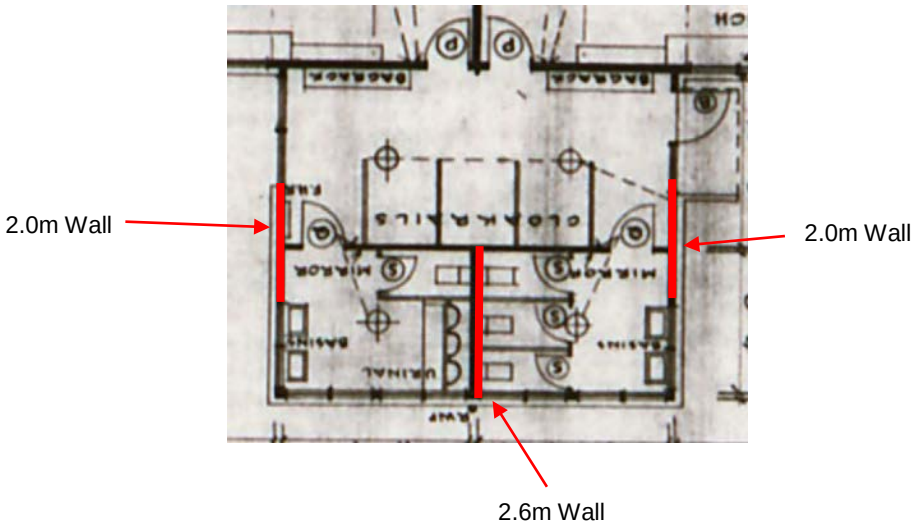
The walls therefore have the following bracing capacities:

FnExtWall	=	1.0×2.5kN/m×9.7m		(H/L<2)
FnExtWall	**=**	**24.25kN**		
	>	**FTransExtWall**	**OK**	
FnIntWall	=	1.0×3.0kN/m×2.7m×2		(H/L<2)
FnIntWall	**=**	**16.20kN**		
	>	**FTransIntWall**	**OK**	

The walls can therefore resist a seismic demand greater than or equal to 100%NBS. The walls will therefore not have to be re-lined with a new sheet bracing material.

Wall Capacities – End Blocks:

Check the capacity of the original bracing elements. The Option 1 wall layout shows a continuous wall running through the centre of the End Block. Originally however, only a 2.6m wall exists. Check that this 2.6m wall, along with the 2.0m walls on the sides have adequate seismic capacity alone.



FnEndBlock	=	1.0×[(2.5kN/m×2.0m×2)+(3.0kN/m×2.6m)]		(H/L<2)
FnEndBlock	**=**	**17.80kN**		
	>	**$F_EQ_{Extension2.5}/2$**	**OK**	

These walls are adequate as they stand to resist seismic loading equal to 100%NBS. No re-lining is required.

Note: It is fair to assume that load re-distribution can occur between the three wall lines, justifying comparing the total demand with the total capacity.

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3.1.3 Longitudinal Direction

Bracing Line 1

Re-line the existing timber framed walls with Ecoply EP1. 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.

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

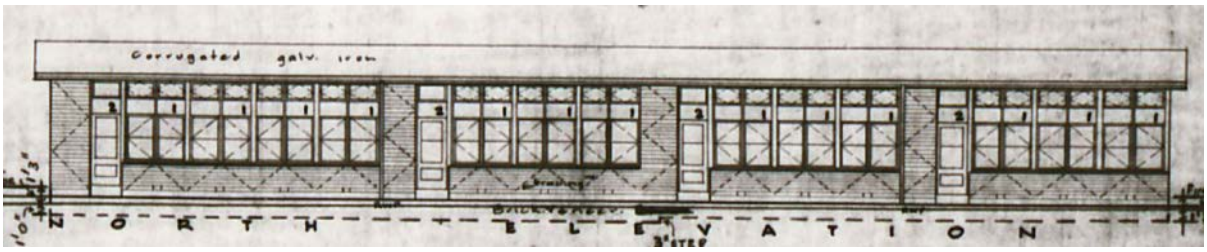
$$\mathbf{F_BL1Duct3_{67\%} = 25.15kN}$$

Ecoply EP1 has a bracing capacity of 105Bu/m (5.25kN/m) for bracing walls under 1.2m in length as per the latest Ecoply technical guidelines.

Specification No.	Minimum Wall Length	Lining Requirements	BUs/m Wind	BUs/m Earthquake
EP1_0.4	0.4 m	Ecoply® one side	80	95
EP1_0.6	0.6 m		95	105
EP1_1.2	1.2 m		120	135

The length of the existing walls are approximately 1.0m. The wall height along this bracing line is equal to 3.2m.

There are four walls along this bracing line (see figure below).



Bracing Line 1 Elevation

Check EP1 is adequate to resist the demand.

The capacity of Bracing Line 1 when lining the walls with EP1 is as follows:

$$FnBL1 = 5.25kN/m \times (2.4m/3.2m) \times 1.0m \times 4$$

$$\mathbf{FnBL1 = 15.75kN}$$

$$< \mathbf{F_BL1Duct3_{67\%}} \quad \mathbf{NG}$$

The capacity of these walls is less than the 67%NBS demand. Instead of removing additional windows, look at distributing the remaining demand across to Bracing Line 2. The excess demand that needs to be transferred to Bracing Line 2 is equal to the following:

$$F_BL1Excess = F_BL1Duct3_{67\%} - FnBL1$$

$$F_BL1Excess = \mathbf{9.40}kN$$

Design Bracing Line 2 to resist the demand shown above as well as the demand induced based on its tributary seismic weight.

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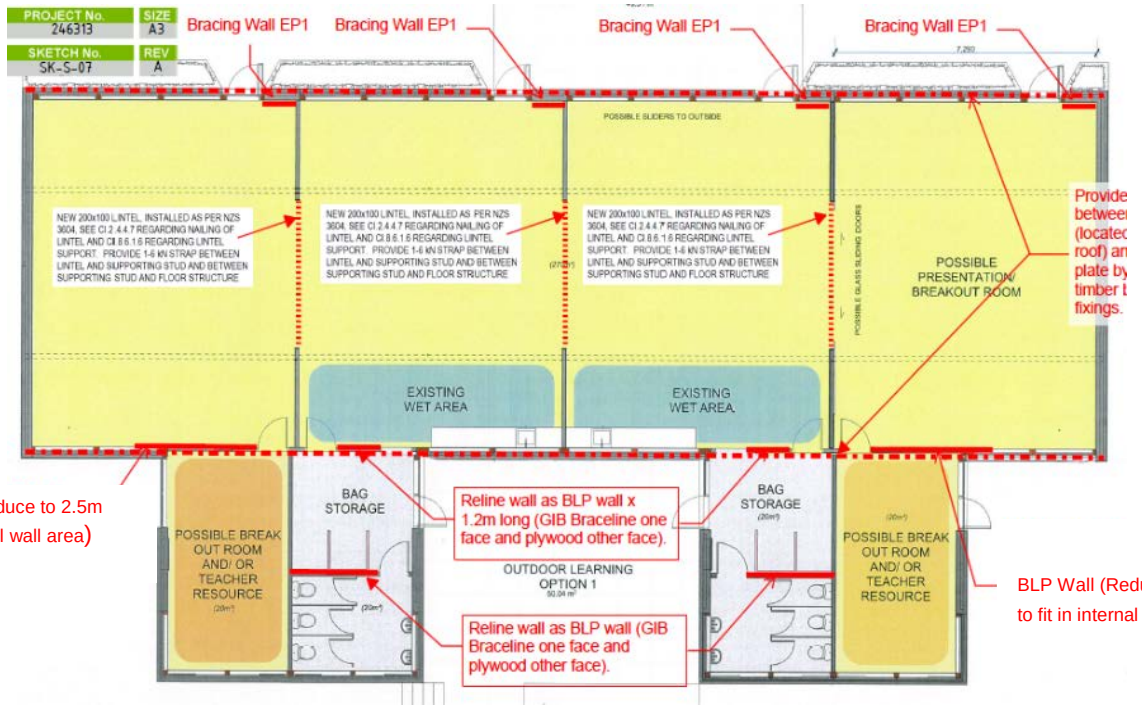
Bracing Line 2

As per the preliminary structural scheme, Bracing Line 2 is to have two 3.4m long and two 1.2m long BLP walls (see below). Check this bracing line has adequate capacity to resist 67%NBS plus the excess demand from Bracing Line 1.



$F_BL2Duct3_{67\%} = Dem_{BL2} \times (C_dT_{3.0}/C_dT_{2.5}) \times 0.67 + F_BL1Excess$

$F_BL2Duct3_{67\%}$ = 34.51kN

Gib BLP has a bracing capacity of 150Bu/m (7.5kN/m) for bracing walls 1.2m in length and longer. The walls will be fixed to a concrete foundation wall below, therefore the full bracing capacity can be achieved. The wall height along this bracing line is equal to 3.2m.

The capacity of Bracing Line 2 when lining the walls with BLP is as follows:

FnBL2 = 7.5kN/m×(2.4m/3.2m)×[(1.2m×2)+(2.5m×2)]

FnBL2 = 41.62kN

> $F_BL2Duct3_{67\%}$ OK

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Bracing Line 3

Half of the existing wall length is to be removed. Re-line the remaining timber wall lengths on this bracing line with Gib BLP. The walls are 2.3m long and there is one in each extension. The wall height is 2.7m in this location.

The 67%NBS demand on this bracing line is as follows:

$F_BL3Duct3_{67\%} = Dem_{BL3} \times (C_dT_{3.0}/C_dT_{2.5}) \times 0.67$

$F_BL3Duct3_{67\%}$ = 23.25kN

The capacity of Bracing Line 3 when lining the walls with BLP is as follows:

FnBL3 = 7.5kN/m×(2.4m/2.7m)×(2.3m×2)

FnBL3 = 30.67kN

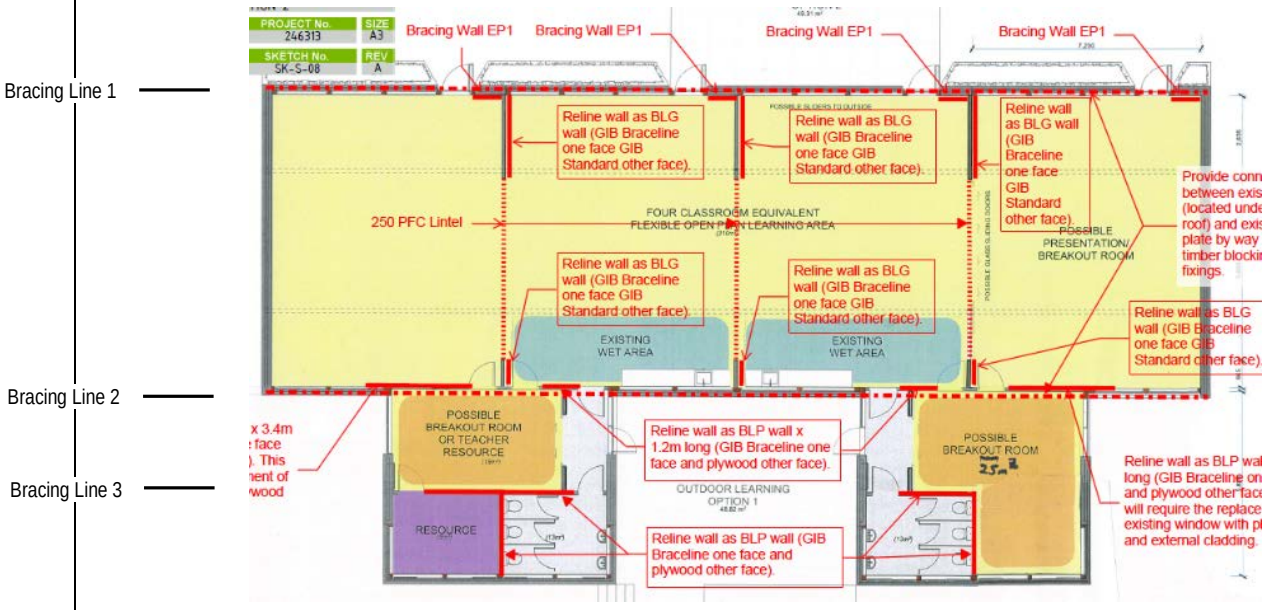
> **$F_BL3Duct3_{67\%}$** **OK**

This wall line lands on a concrete foundation wall with an adjacent concrete slab on grade, therefore the full 150BU/m capacity can be used here also.

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 1-3
- Transverse Direction:
  - Existing bracing walls on the exterior bracing lines
  - New bracing walls on the interior bracing lines

3.2.2 Transverse Direction

Main Classroom Area

As was found in Option 1 above, the external walls will have adequate bracing capacity and therefore no re-lining will be required.

The internal walls in Option 1 however were found to be at 100%NBS with the wall lengths reduced. The existing wall lengths will be further reduced and therefore strengthening will be required to ensure the bracing capacity remains at 100%NBS.

As the walls will be re-lined with Gib, scaled up internal bracing wall demands calculated in 3.1.2) for a ductility of 3.0 and an Sp of 0.7.

FTransIntWall3.0 = FTransIntWall $\times C_dT_{3.0}/C_dT_{2.5}$

FTransIntWall3.0 = 18.75kN

The transverse wall lengths will be 2.7m and 0.9m respectively. The 2.7m wall will have an average height of 3.4m and the 0.9m wall will be 3.2m in height. Try re-lining these with Gib BLG. Gib BLG has a bracing capacity of 145Bu/m (7.25kN/m) for walls 1.2m in length or greater, and a capacity of 115 Bu/m (5.75kN/m) for walls under 0.4m in length. A 0.9m wall therefore has a bracing capacity of 134 Bu (6.7kN/m) (can interpolate as per the Gib manual). The walls will be fixed to the concrete foundation walls below, therefore can justify achieving the full bracing capacities stated above.

The transverse capacity of an internal bracing line is therefore equal to the following:

FnIntWall = [(2.4m/3.4m)×7.25kN/m×2.7m] + [(2.4m/3.2m)×6.70kN/m×0.9m]

FnIntWall = 18.34kN

< FTransIntWall3.0 NG

The bracing wall capacity is slightly short of the total demand. By inspection, this wall however will in fact be adequate to resist 100%NBS for the following reasons:

- The weight of the brick veneers on the external transverse walls has been included in the demand calculations for the internal walls. These demands are therefore conservative as the internal walls will not be subject to the demand from this weight in reality. The weight of the wall openings also has also not been removed in the demand calculations.
- Some load redistribution will be able to occur out to the end walls that have a larger bracing capacity.
- The total bracing capacity of these re-lined internal walls are greater than the existing wall arrangement in Option 1.

End Blocks/Extensions

The transverse wall lengths are not being shortened from the original structure. As was found in the Option 1 calculations, the existing wall layout is adequate to resist 100%NBS. Can therefore justify no additional strengthening is required.

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3.2.3 Longitudinal Direction

Bracing Line 1

The longitudinal bracing system is the same as Option 1.

Bracing Line 2

The longitudinal bracing system is the same as Option 1.

Bracing Line 3

Half of the existing wall length is to be removed in one of the extensions. The full existing wall length is to remain in the other extension. Re-line these remaining timber walls with Gib BLP. The wall is 4.6m long in one extension and 2.3m long in the other. The wall heights are 2.7m. Check the extension with the 2.3m wall (worst case).

The 67%NBS demand on this bracing line is as follows:

$$\mathbf{F_BL3Duct3_{67\%Half}} = [Dem_{BL3} \times (C_dT_{3.0}/C_dT_{2.5}) \times 0.67]/2$$

$$\mathbf{F_BL3Duct3_{67\%Half}} = \mathbf{11.63kN}$$

The capacity of Bracing Line 3 when lining the walls with BLP is as follows:

$$FnBL3 = 7.5kN/m \times (2.4m/2.7m) \times (2.3m)$$

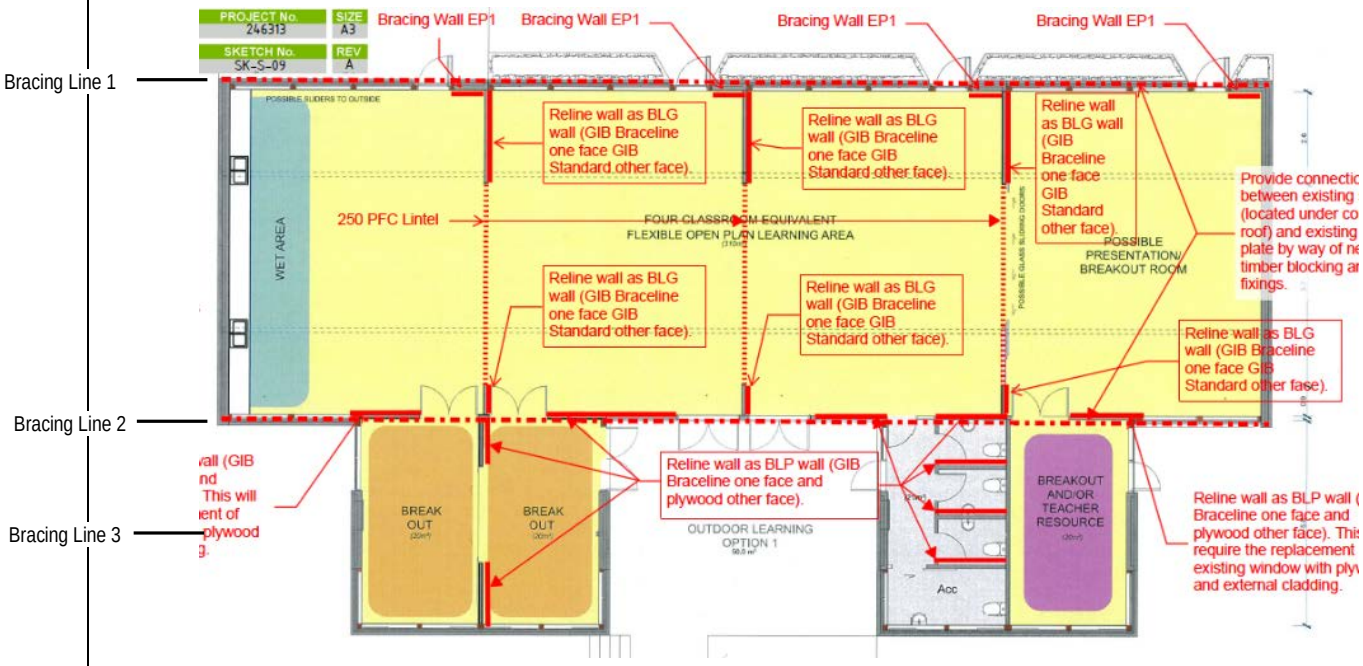
$$\mathbf{FnBL3 = 15.33kN}$$

$$> \mathbf{F_BL3Duct3_{67\%Half}}\ \mathbf{OK}$$

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 Lines 1-3
- Transverse Direction:
  - Existing bracing walls on the exterior bracing lines
  - New bracing walls on the interior bracing lines

3.3.2 Transverse Direction

Main Classroom Area

The transverse bracing system is the same as Option 2.

End Blocks/Extensions

One of the extensions is to have the existing wall layout, therefore will not require any alteration.

The other extension will have the existing central wall reduced in length and another wall installed. These walls will be 1.8m and 1.3m in length with an average height of 2.8m. Line these walls with Gib BL1 (5.25kN/m).

The total capacity of this central wall line is equal to the following:

FnCentralWall = (2.4m/2.8m)×5.25kN/m×(1.8m+1.3m)

FnCentralWall **= 13.95kN**

This is greater than the capacity of the existing 2.6m central wall (7.8kN). Lining the central walls with Gib BL 1 is therefore adequate. It is making the bracing capacity no weaker than it currently stands in its original state. In the Option 1 calculations, the original capacity was found to be above 100%NBS in this direction in the extensions.

3.3.3 Longitudinal Direction

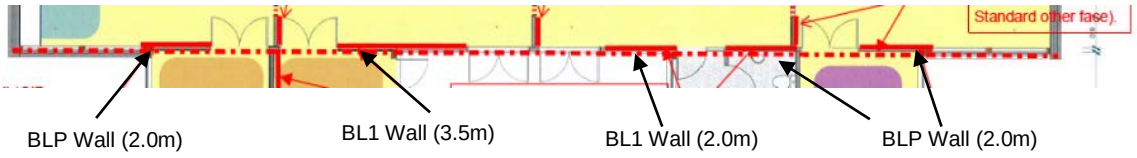
Bracing Line 1

The longitudinal bracing system is the same as Option 1.

Bracing Line 2

The longitudinal bracing system will involve re-lining a selection of walls with Gib BLP and Gib BL1.

Check the bracing arrangement shown below has adequate capacity to resist 67%NBS.



BLP has a capacity of 150Bu/m (7.5kN/m) and BL1 has a bracing capacity of 105Bu/m (5.25kN/m)

The 67%NBS demand on this bracing line is as follows (including the excess demand from Bracing Line 2):

$F_BL2Duct3_{67\%} = Dem_{BL2} \times (C_dT_{3.0}/C_dT_{2.5}) \times 0.67 + F_BL1Excess$

$F_BL2Duct3_{67\%}$ = 34.51kN

The capacity of Bracing Line 2 when lining the walls with BLP is as follows:

FnBL2 = [7.5kN/m×(2.4m/2.7m)×2.0m×3]+[5.25kN/m×(2.4m/2.7m)×(3.5m+2.0m)]

FnBL2 = 65.67kN

> **$F_BL3Duct3_{67\%}$** **OK**

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Bracing Line 3

As can be seen in 3.3.1) above, one of the extensions has three new bracing walls installed. These are 2.0m in length. Try lining these with Gib GS2 (4.25kN/m). The average height is 2.8m.

As these walls are being re-lined, scale the demands up for ductility 3, Sp 0.7. The 67%NBS demand in one extension is therefore equal to the following:

$\mathbf{F_BL3Duct3_{67\%Half}}$ = $[Dem_{BL3} \times (C_dT_{3.0}/C_dT_{2.5}) \times 0.67]/2$ Demand in one extension

$\mathbf{F_BL3Duct3_{67\%Half}}$ = **11.63kN**

The capacity of the walls when lining with GS2 is as follows:

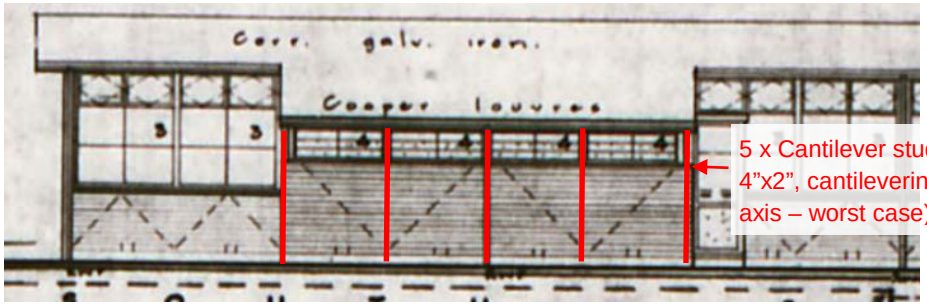
FnBL3 = 4.25kN/m×(2.4m/2.8m)×2m×3

FnBL3 = 21.86kN

> $\mathbf{F_BL3Duct3_{67\%Half}}$ **OK**

On the other extension however, all of the longitudinal bracing walls have been removed. Check to see the loading can be carried by the end wall of the extension.

The bracing element at the end of the extension consists of a timber framed wall with diagonal timber members. A small clerestorey (approximately 400mm exists above the walls). Lateral seismic resistance across this clerestorey area is provided by cantilever action from 5 studs (see elevation below). This will govern the capacity.



Cantilever Timber Studs Parameters

Stud breadth	b = 4in	
	b = 101.60mm	Nominal Dimensions
Stud depth	d = 2in	
	d = 50.80mm	
Bending Strength	f_b = 19.8MPa	Estimated Bending Strength - Rimu
Number of Studs	n = 5	5 studs between 4 classrooms

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k factors	$k_1 = 1$	Earthquake Load
	$k_4 = 1.2$	5 members conservative
	$k_5 = 1.0$	no grid system
	$k_8 = 1.0$	Bending about its weak axis

Max Lateral Bracing Capacity

Lever arm	L = 0.4 m
Section Modulus	$Z_x = (b \times d^2)/6$
	$Z_x = 43699mm^3$
Moment Capacity	$M_n = k_1 \times k_4 \times k_5 \times k_8 \times f_b \times Z_x$
	M_n = 1.04kNm
Max Roof Force	$Fmax = n \times M_n / L$
	Fmax = 12.98 kN

The 67%NBS Bracing Line 3 demand in the one extension is equal to the following (at ductility 2.5, Sp 0.5):

$Dem_{BL3}67\%_{\mathbf{Half}}$	=	**(**$Dem_{BL3} \times 0.67)/2$
$Dem_{BL3}67\%_{\mathbf{Half}}$	**=**	**9.58kN**
	<	Fmax OK

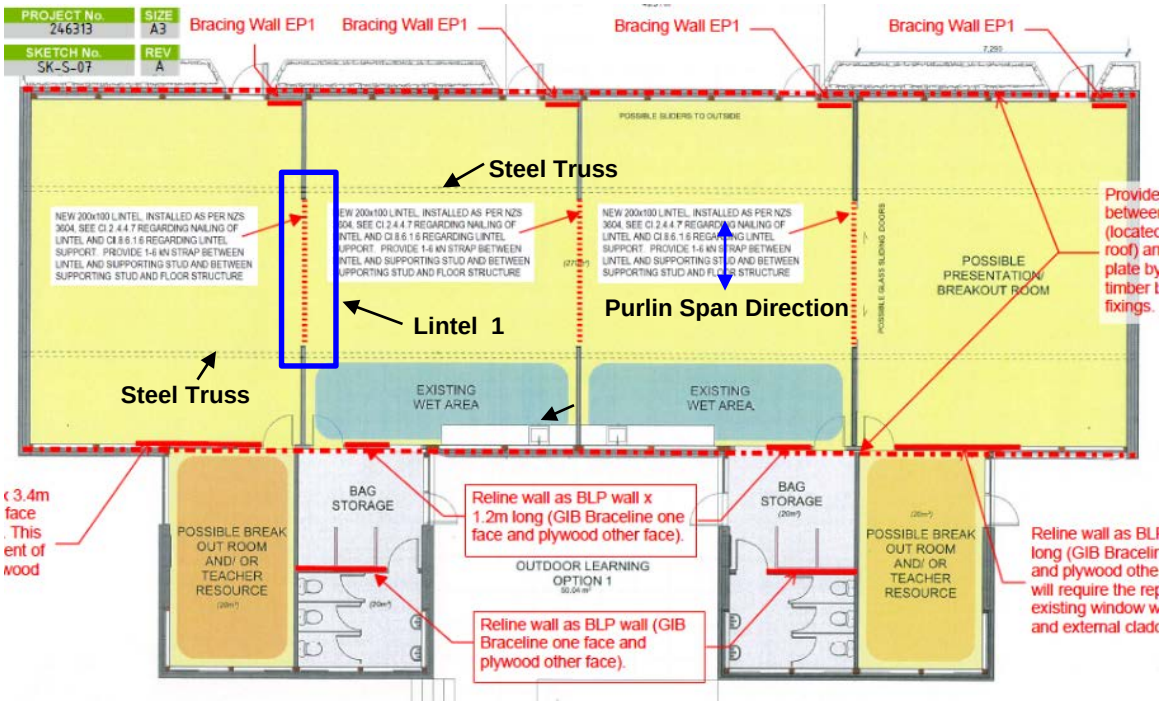
Hence, the full 67%NBS Bracing Line 3 demand in the one extension with no bracing walls can be carried by the end wall and clerestorey system. There is also excess capacity in Bracing Line 2 to carry any residual demand from Bracing Line 2. Based on tributary widths it is likely close to half of the Bracing Line 3 demand will get transferred to Bracing Line 3 in this extension.

4. Roof Lintel Design

4.1 Option 1

4.1.1 Lintel Layout

There is one lintel type that is 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. This lintel is to be labelled Lintel 1.



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 less than 10 degrees across the entire structure.

Longitudinal Loading

h = 3.2m (avg)

d = 29.6m

h/d = 0.11

Cp,e (R)Long = -0.9 and

= 0.2 (Table 5.3(C) – Worst Cases)

Transverse Loading

h = 3.2m (avg)

d = 15.8m

h/d = 0.20

Cp,e (U/D)Trans = -0.9 and

= 0.2 (Table 5.3(C) – Worst Cases)

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The wind loading demands will therefore be the same in both directions. Design all lintels for the worst case uplift and downwards wind loading demand cases.

Ka	=	0.9 (Lintel tributary area approximately 25m^2)
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 Up	=	Cp,e x Ka x Kc,e x Kl x Kp
Cfig,e Up	=	-0.9 x 0.9 x 0.8 x 1.0 x 1.0
	=	-0.648
Cfig,e Down	=	Cp,e x Ka x Kc,e x Kl x Kp
Cfig,e Down	=	0.2 x 0.9 x 0.8 x 1.0 x 1.0
	=	0.144
Cfig,i	=	Cp,i x Kc,i
Cfig,i	=	0.3 x 0.8
	=	0.24 Or
	=	0.00

Hence, the net Cfig factors are as follows:

Cfig net Up	=	-0.648
Cfig net Down	=	0.144+0.240
	=	0.384

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

PdesULS	=	-0.648 x 0.84kPa
	=	-0.544 kPa (Up)
	=	0.384 x 0.84kPa
	=	0.322 kPa (Down)
PdesSLS	=	-0.648 x 0.57kPa
	=	-0.369 kPa (Up)
	=	0.384 x 0.57kPa
	=	0.219 kPa (Down)

***Note: These design wind pressures are lower than that calculated for the Wellington region design. The same lintel members will be used for the Christchurch 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 = 3.9m

Tributary Width = 0.6m (Rafter Spacing)

The rafters span transversely across the structure and are supported at the longitudinal wall lines as well as two intermediate longitudinal steel trusses. These trusses are supported by the transverse existing walls in this option, therefore the lintel will not have to support point loads from these trusses in this instance.

An allowance for the lintel to support 1m of wall above will also be allowed for in the design.

It is also assumed that the rolling doors to be installed across the openings are supported directly on the floor and are therefore not required to be supported by the new lintel. The sliding door requirements/details are to be confirmed by the architect in a specific building design. Additional load will need to be applied should the architect wish to hang the door off the lintel.

2x190x45 timber members are suitable for this lintel. See spreadsheet calculations below:

Structural Timber Beam Calculation Sheet: Simply Supported

Job Name:	MoE ILS Canterbury Block	Date:	3-Sep-15
Job Number:	246313	Designer:	PGD
Description:	Lintel 1 - Uplift Wind		

1. Beam Properties

Depth d	190	mm	Nominal d	200	mm	Species	Radiata pine	Bending Strength fb	14	
Thickness b	90	mm	Nominal b	100	mm			Mpa		user defined
Length	3.90	m	d override		mm	Condition	Dry		14	use for calcs.
Unsupp. L_{ay} downwards	3.90	m	b override		mm			Shear Strength fs	3.8	
Unsupp. L_{ay} uplift	3.90	m	only enter overrides for non-standard			Grade	MSG8			user defined
I	5.14E+07	mm^4	dimensional values.					Mpa	3.8	use for calcs.
Z	5.42E+05	mm^3								
A	1.71E+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	0.09	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)

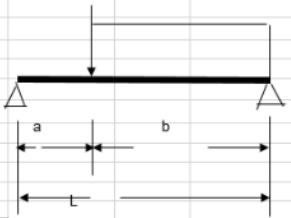
case3 - double element beam or lintel such as where 2x50mm wide timber elements are used to make up a 100mm wide element 9C2.4.2.3)

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	0.60	m
Floor		kPa	Roof	0.40	kPa
Ceiling	0.00	kPa	Ceiling		kPa
Walls	0.00	kN/m	Parapet	0.30	kN/m
SDL	0.00	kPa	SDL		kPa
G_{total}	0.09	kN/m	G_{total}	0.54	KN/m
Q		kPa	Q	0.25	kPa
Q	0.00	kN/m	Q	0.15	KN/m
			Ws/Wu	0.70	
			Wu	-0.70	kPa
			Wu	-0.42	KN/m

Load combinations to: AS/NZS 1170:2002



Combination			Combination			Total kN/m	Reactions (kN)	Mmax (kNm)	Δmax (mm)
1.35G	0.1154	kN/m	1.35G	0.73	kN/m	0.84	1.6	1.6	7.38
1.2G +1.5Q	0.1026	kN/m	1.2G +1.5Q	0.87	kN/m	0.98	1.9	1.9	8.53
G +0.4Q	0.09	kN/m	G +0Q	0.54	kN/m	0.63	1.2	1.2	5.47
G +0.7Q	0.09	kN/m	G +0.7Q	0.65	kN/m	0.73	1.4	1.4	6.38
1.2G +0.4Q +Wu	0.10	kN/m	1.2G +0Q +Wu	0.23	kN/m	0.33	0.6	0.6	2.89
0.9G +Wu	0.08	kN/m	0.9G +Wu	0.07	kN/m	0.14	0.3	0.3	1.25
Ws	0.00	kN/m	Ws	-0.29	kN/m	-0.29	-0.6	-0.6	-2.57

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3. Design Load Capacity

Bending Capacity		
Str. Reduction Factor	0.8	
Duration Factor K_1	0.8	
K_4	1.14	
K_5	1.0	
Unsupported Length L_{uy}	3900.0	mm
Slenderness Ratio S_1	12.1	
Slenderness Factor K_8	1.0	
Bending Stress fb	14.0	MPa
Fact. Bend. Str ϕM_{nx}	5.438	kNm
Bending Demands		
Ult. Mom. M*	1.9	kNm
Ult. Mom. M* Wind	0.629	kNm
C/D Ratio	2.9	
Check	OK	

Shear Capacity		
Str. Reduction Factor	0.80	
Shear Stress f_s	3.80	MPa
Shear Area A_s	11400	mm^2
Fact Shear Str. ϕV_n	31.61	kN
Shear Demands		
Ult. Shear V*	1.90	kN
Ult. Shear V* Wind	0.6	kN
C/D Ratio	16.61	
Check	OK	
Reactions		
Left (KN)	1.90	kN
Right (KN)	1.90	kN

	Summary Reaction	
	Left (KN)	Right (KN)
1.35G	1.6	1.6
1.2G +1.5Q	1.9	1.9
G +0.4Q	1.2	1.2
G +0.7Q	1.4	1.4
1.2G +0.4Q +Wu	0.6	0.6
0.9G +Wu	0.3	0.3
Ws	-0.6	-0.6

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
11.2	8.5	-2.6	3.59	7.8	13.0	16.3	19.5	0.7
L/345	*L/455*	*L/1480*		NG	OK	OK	OK	NG

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Structural Timber Beam Calculation Sheet: Simply Supported

Job Name:	MoE ILS Canterbury Block	Date:	3-Sep-15
Job Number:	246313	Designer:	PGD
Description:	Lintel 1 - Downwards Force Wind		

1. Beam Properties

			Nominal					
Depth	d	190	mm	Nominal d	200	mm	Species	Radiata pine
Thickness	b	90	mm	Nominal b	100	mm		
Length		3.90	m	d override		mm	Condition	Dry
Unsupp. L_{ay} downwards		3.90	m	b override		mm		
Unsupp. L_{ay} uplift		3.90	m	only enter overrides for non-standard			Grade	MSG8
I		5.14E+07	mm^4	dimensional values.				
Z		5.42E+05	mm^3					
A		1.71E+04	mm^2					
Self Weight		0.09	KN/m					
E		6700	MPa	use for calcs	☑			

Bending Strength fb Mpa	14	user defined / use for calcs.
Shear Strength fs Mpa	3.8	user defined / use for calcs.

☐ At least 4 elements in the system which constrained to deformations similar to that of their neighbours (2.4.2.2)
☐ system consists of a single timber element (2.4.2.3)
☑ system consists of two or three elements acting together (2.4.2.3)

Tick a box. Choose only one option

2. Load Data

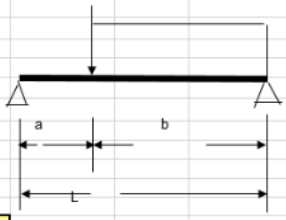
case2 - single element beam or lintel (C2.4.2.3)
case3 - double element beam or lintel such as where 2x50mm wide timber elements are used to make up a 100mm wide element 9C2.4.2.3)

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	0.60	m
Floor		kPa	Roof	0.40	kPa
Ceiling	0.00	kPa	Ceiling		kPa
Walls	0.00	kN/m	Parapet	0.30	kN/m
SDL	0.00	kPa	SDL		kPa
G_{total}	0.09	kN/m	G_{total}	0.54	KN/m
Q		kPa	Q	0.25	kPa
Q	0.00	kN/m	Q	0.15	KN/m
			Ws/Wu	0.70	
			Wu	0.42	kPa
			Wu	0.25	KN/m

Load combinations to: AS/NZS 1170:2002



Combination			Combination			Total kN/m	Reactions (kN)	Mmax (kNm)	Δmax (mm)
1.35G	0.1154	kN/m	1.35G	0.73	kN/m	0.84	1.6	1.6	7.38
1.2G +1.5Q	0.1026	kN/m	1.2G +1.5Q	0.87	kN/m	0.98	1.9	1.9	8.53
G +0.4Q	0.09	kN/m	G +0Q	0.54	kN/m	0.63	1.2	1.2	5.47
G +0.7Q	0.09	kN/m	G +0.7Q	0.65	kN/m	0.73	1.4	1.4	6.38
1.2G +0.4Q +Wu	0.10	kN/m	1.2G +0Q +Wu	0.90	kN/m	1.00	1.9	1.9	8.74
0.9G +Wu	0.08	kN/m	0.9G +Wu	0.74	kN/m	0.81	1.6	1.5	7.10
Ws	0.00	kN/m	Ws	0.17	kN/m	0.17	0.3	0.3	1.52

3. Design Load Capacity

Bending Capacity		
Str. Reduction Factor	0.8	
Duration Factor K_1	0.8	
K_4	1.14	
K_5	1.0	
Unsupported Length L_{uy}	3900.0	mm
Slenderness Ratio S_1	12.1	
Slenderness Factor K_8	1.0	
Bending Stress fb	14.0	MPa
Fact. Bend. Str ϕM_{nx}	5.438	kNm
Bending Demands		
Ult. Mom. M*	1.9	kNm
Ult. Mom. M* Wind	1.900	kNm
C/D Ratio	2.9	
Check	OK	

Shear Capacity		
Str. Reduction Factor	0.80	
Shear Stress f_s	3.80	MPa
Shear Area A_s	11400	mm^2
Fact Shear Str. ϕV_n	31.61	kN
Shear Demands		
Ult. Shear V*	1.90	kN
Ult. Shear V* Wind	1.9	kN
C/D Ratio	16.21	
Check	OK	
Reactions		
Left (KN)	1.95	kN
Right (KN)	1.95	kN

	Summary Reaction Left (KN)	Right (KN)
1.35G	1.6	1.6
1.2G +1.5Q	1.9	1.9
G +0.4Q	1.2	1.2
G +0.7Q	1.4	1.4
1.2G +0.4Q +Wu	1.9	1.9
0.9G +Wu	1.6	1.6
Ws	0.3	0.3

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
11.2	8.5	1.6	3.59	7.8	13.0	16.3	19.5	0.7
L/345	*L/455*	*L/2495*		NG	OK	OK	OK	NG

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4.1.4 Lintel Fixings and Stud Supports

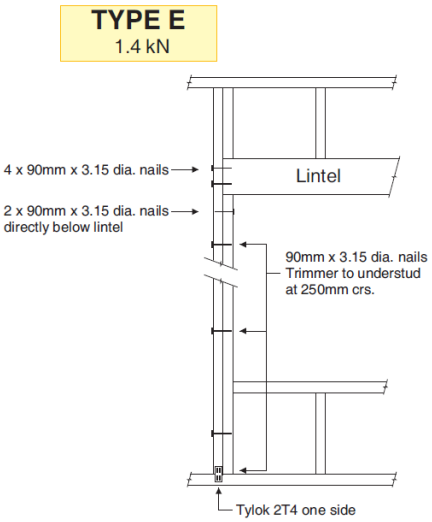
From the lintel calculations shown above, the critical reactions are as follows:

N*t = 0kN (Uplift)

N*c = 1.9kN (Compression – 1.2G+Wu, 1.2G+1.5Q)

Design a lintel fixing to resist these loads.

Use a standard Lumberlok lintel fixing detail. Use a standard 1.4kN connection as the uplift is 0kN.



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

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}	=	3500mm	(Approximate stud height at support location)
L_{ay}	=	1000mm	(Conservative – Dwang spacing in wall the studs are built in)
b	=	90 mm	
d	=	90 mm	
S_2	=	3600/90	(Lax/d)
	=	40	Critical
S_3	=	1000/90	(Lay/b)
	=	11.11	

Hence: k_8 = 0.190 (Table 2.8 – NZS3603:1993)

A = 8100 mm^2

Hence: ϕNc_{Col} = 0.8 x 0.6 x 0.19 x 18.0 x 8100 x 10^{-3}

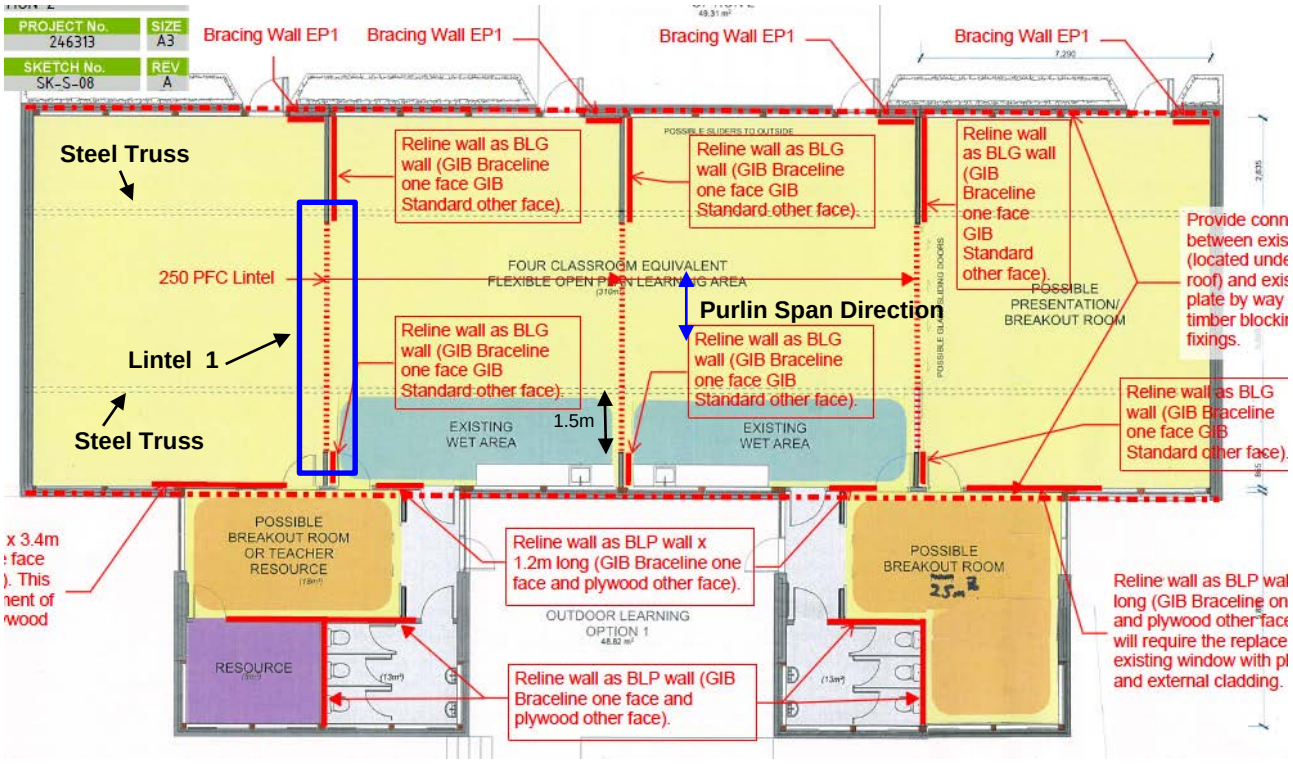
= 13.29 kN

> N*c OK

A double 90x45 stud is therefore adequate to support the lintel loads.

4.2 Option 2

4.2.1 Lintel Layout



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4.2.2 Design Loads

The demands will be the same as those shown in 4.1.2) for Option 1. This lintel will also be subject to a point load from the steel truss. This truss is positioned 1.5m from the end of the beam. The tributary area of the truss loads being supported by the lintel is equal to 26.6m^2 (7.4m x 3.6m). The design point loads are therefore as follows:

Gtruss = 0.4kPa x 26.6m^2

= 10.64kN

Qtruss = 0.25kPa x 26.6m^2

= 6.65kN

Wutruss = -0.7kPa x 26.6m^2

= -18.62 kN (Up)

= 0.415kPa x 26.6m^2

= 11.04 kN (Down)

Wstruss = -0.49kPa x 26.6m^2

= -13.03kN (Up)

= 0.292kPa x 26.6m^2

= 7.78 kN (Down)

4.2.3 Lintel 1 Design

Span = 5.7m

Tributary Width = 0.6m

The point loads listed above are also to be applied to the lintel 1.5m from the support edge. Also, an allowance for the lintel to support 1m of wall above will also be allowed for in the design.

It is also assumed that the rolling doors to be installed across the openings are supported directly on the floor and are therefore not required to be supported by the new lintel. The sliding door requirements/details are to be confirmed by the architect in a specific building design. Additional load will need to be applied should the architect wish to hang the door off the lintel.

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

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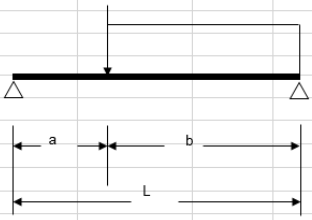
1. Beam Properties

Steel Beam	250PFC	
Span	5.70	m
Unsupp. L downwards	5700	mm
Unsupp. L uplift	5700	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		
Tribu Width	0.00	m	Tribu Width	0.60	m
Floor	0.00	kPa	Roof	0.40	kPa
Ceiling		kPa	Ceiling		kPa
Walls		kN/m	Parapet	0.30	kN/m
SDL	0.00	kPa	SDL		kPa
G_{total}	0.35	kN/m	G_{total}	0.54	KN/m
Q	0.00	kPa	Q	0.25	kPa
Q	0.00	kN/m	Q	0.15	KN/m
			Ws/Wu	0.70	
			Wu	-0.70	kPa
			Wu	-0.42	KN/m

Combination			Combination			Total kN/m	Reactions (kN)	Mmax (kNm)	Δmax (mm)
1.35G	0.47	kN/m	1.35G	0.73	kN/m	1.20	3.4	4.9	1.78
1.2G +1.5Q	0.42	kN/m	1.2G +1.5Q	0.87	kN/m	1.29	3.7	5.2	1.92
G +0.4Q	0.35	kN/m	G +0Q	0.54	kN/m	0.89	2.5	3.6	1.32
G +0.7Q	0.35	kN/m	G +0.7Q	0.65	kN/m	0.99	2.8	4.0	1.48
1.2G +0.4Q +Wu	0.42	kN/m	1.2G +0Q +Wu	0.23	kN/m	0.65	1.8	2.6	0.96
0.9G +Wu	0.31	kN/m	0.9G +Wu	0.07	kN/m	0.38	1.1	1.5	0.56
Ws	0.00	kN/m	Ws	-0.29	kN/m	-0.29	-0.8	-1.2	-0.44

(2) Point Load (Dead, Live and Wind)

NO	Point Load				a(From Left)	b	Combination		Reaction		Mmax	Δcenter
	G (KN)	Q_{FLOOR} (KN)	Q_{ROOF} (KN)	Wu (KN)	(m)	(m)		(KN)	Left (KN)	Right (KN)	(kNm)	(mm)
1 (PL)	10.64		6.65	-18.62	1.5	4.2	1.35G	14.36	10.6	3.8	15.9	4.30
	10.64	0	6.65	-18.62	1.5	4.2	1.2G +1.5Q	22.74	16.8	6.0	25.1	6.80
	10.64	0	6.65	-18.62	1.5	4.2	G +0.4Q	10.64	7.8	2.8	11.8	3.18
	10.64	0	6.65	-18.62	1.5	4.2	G +0.7Q	15.30	11.3	4.0	16.9	4.57
	10.64	0	6.65	-18.62	1.5	4.2	1.2G +0.4Q +Wu	-5.85	-4.3	-1.5	-6.5	-1.75
	10.64	0	6.65	-18.62	1.5	4.2	0.9G +Wu	-9.04	-6.7	-2.4	-10.0	-2.70
	10.64	0	6.65	-18.62	1.5	4.2	Ws	-13.03	-9.6	-3.4	-14.4	-3.90

3. Design Load Capacity

M^*_{max}	M^*_{wind}	Reaction	
KNm	KNm	Left (KN)	Right (KN)
29.2	-8.8	20.4	9.7

ϕM_{bx}	ϕM_{by}	Beam Span
kNm	kNm	m
47.3	28.9	5.7
OK		
162%		

	Total Reaction	
	Left (KN)	Right (KN)
1.35G	14.0	7.2
1.2G +1.5Q	20.4	9.7
G +0.4Q	10.4	5.3
G +0.7Q	14.1	6.9
1.2G +0.4Q +Wu	-2.5	0.3
0.9G +Wu	-5.6	-1.3
Ws	-10.4	-4.3

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
4.6	6.2	-4.4	0.42	11.4	19.0	28.5	0.7
L/1235	*L/915*	*L/1280*					
OK	OK	OK	OK				

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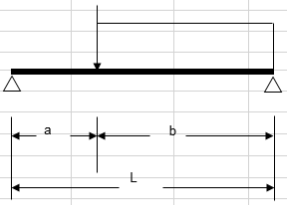
1. Beam Properties

Steel Beam	250PFC	
Span	5.70	m
Unsupp. L downwards	5700	mm
Unsupp. L uplift	5700	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		
Tribu Width	0.00	m	Tribu Width	0.60	m
Floor	0.00	kPa	Roof	0.40	kPa
Ceiling		kPa	Ceiling		kPa
Walls		kN/m	Parapet	0.30	kN/m
SDL	0.00	kPa	SDL		kPa
G_{total}	0.35	kN/m	G_{total}	0.54	KN/m
Q	0.00	kPa	Q	0.25	kPa
Q	0.00	kN/m	Q	0.15	KN/m
			Ws/Wu	0.70	
			Wu	0.42	kPa
			Wu	0.25	KN/m

Combination			Combination			Total kN/m	Reactions (kN)	Mmax (kNm)	Δmax (mm)
1.35G	0.47	kN/m	1.35G	0.73	kN/m	1.20	3.4	4.9	1.78
1.2G +1.5Q	0.42	kN/m	1.2G +1.5Q	0.87	kN/m	1.29	3.7	5.2	1.92
G +0.4Q	0.35	kN/m	G +0Q	0.54	kN/m	0.89	2.5	3.6	1.32
G +0.7Q	0.35	kN/m	G +0.7Q	0.65	kN/m	0.99	2.8	4.0	1.48
1.2G +0.4Q +Wu	0.42	kN/m	1.2G +0Q +Wu	0.90	kN/m	1.31	3.7	5.3	1.95
0.9G +Wu	0.31	kN/m	0.9G +Wu	0.74	kN/m	1.05	3.0	4.3	1.56
Ws	0.00	kN/m	Ws	0.17	kN/m	0.17	0.5	0.7	0.26

(2) Point Load (Dead, Live and Wind)

NO	Point Load				a(From Left)	b	Combination		Reaction		Mmax	Δcenter
	G (KN)	Q_{FLOOR} (KN)	Q_{ROOF} (KN)	Wu (KN)	(m)	(m)		(KN)	Left (KN)	Right (KN)	(kNm)	(mm)
1 (PL)	10.64		6.65	11.04	1.5	4.2	1.35G	14.36	10.6	3.8	15.9	4.30
	10.64	0	6.65	11.04	1.5	4.2	1.2G +1.5Q	22.74	16.8	6.0	25.1	6.80
	10.64	0	6.65	11.04	1.5	4.2	G +0.4Q	10.64	7.8	2.8	11.8	3.18
	10.64	0	6.65	11.04	1.5	4.2	G +0.7Q	15.30	11.3	4.0	16.9	4.57
	10.64	0	6.65	11.04	1.5	4.2	1.2G +0.4Q +Wu	23.81	17.5	6.3	26.3	7.12
	10.64	0	6.65	11.04	1.5	4.2	0.9G +Wu	20.62	15.2	5.4	22.8	6.16
	10.64	0	6.65	11.04	1.5	4.2	Ws	7.73	5.7	2.0	8.5	2.31

3. Design Load Capacity

M^*_{max}	M^*_{wind}	Reaction	
KNm	KNm	Left (KN)	Right (KN)
29.2	30.5	21.3	10.0

ϕM_{bx}	ϕM_{by}	Beam Span
kNm	kNm	m
47.3	28.9	5.7
OK		
155%		

	Total Reaction	
	Left (KN)	Right (KN)
1.35G	14.0	7.2
1.2G +1.5Q	20.4	9.7
G +0.4Q	10.4	5.3
G +0.7Q	14.1	6.9
1.2G +0.4Q +Wu	21.3	10.0
0.9G +Wu	18.2	8.4
Ws	6.2	2.5

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
4.6	6.2	2.6	0.42	11.4	19.0	28.5	0.7
L/1235	*L/915*	*L/2160*					
OK	OK	OK	OK				

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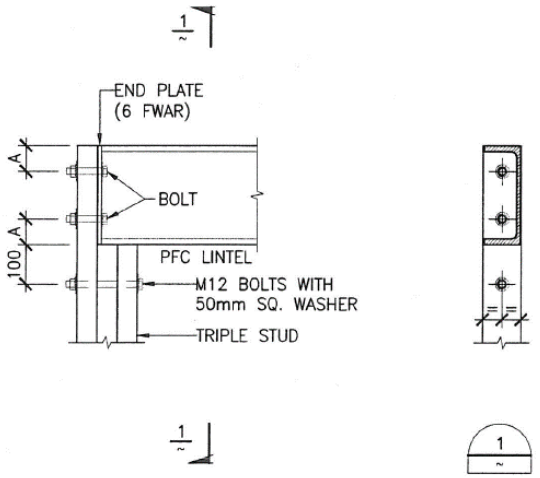
4.2.4 Lintel Fixings and Stud Supports

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

N*t	=	-6.7kN+1.1kN
	=	-5.6kN (Uplift – 0.9G+Wu)
N*c	=	17.5kN+3.7kN
	=	21.2kN (Compression – 1.2G+Wu)
	=	16.8kN + 3.7kN
	=	20.5kN (Compression – 1.2G+1.5Q)

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

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

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

ϕNtstud	=	ϕ x k_1 x k_4 x f_t x 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	

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Hence: $\phi Ntstud$ = $0.8 \times 1.0 \times 1.20 \times 6.00 \times 12150 \times 10^{-3}$

= 70.00kN

> N*t OK

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

$\phi Ncstud$ = $\phi \times k_1 \times k_8 \times f_c \times A$

ϕ = 0.80

k_1 = 0.60 (Gravity Case - Critical)

= 1.0 (Wind Case)

f_c = 18.00 MPa (SG8 Timber)

L_{ax} = 3300mm (Height of stud at most heavily loaded support)

L_{ay} = 1000mm (Conservative – Dwang spacing in wall the studs are built in)

b = 90 mm

d = 135 mm

S_2 = 3300/90 (Lax/d)

= 36.7 Critical

S_3 = 1000/135 (Lay/b)

= 7.41

Hence: k_8 = 0.23 (Table 2.8 – NZS3603:1993)

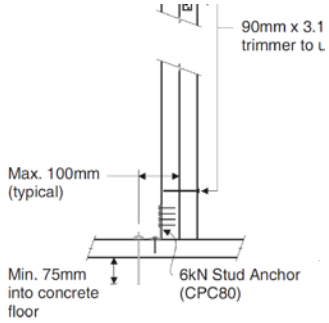
A = 12150 mm^2

Hence: ϕNc_{Col} = $0.8 \times 0.6 \times 0.23 \times 18.0 \times 12150 \times 10^{-3}$

= 24.14 kN

> N*c OK

A triple 90x45 stud is therefore adequate to support the lintel loads. At the base of the studs utilise a 7.5kN Lumberlok hold down fixing. 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.

Truss-Lintel Connection

The steel truss is to bear onto the PFC lintels. In order for the lintels to support the uplift loads from the trusses, the PFC is to be connected to the truss with a bolted connection or through welding on site. This is to be confirmed in the design of a specific Canterbury Block. In the specific design, the exact size and layout of the truss members will able to be determined on site, allowing a practical site specific connection to be detailed.

4.3 Option 3

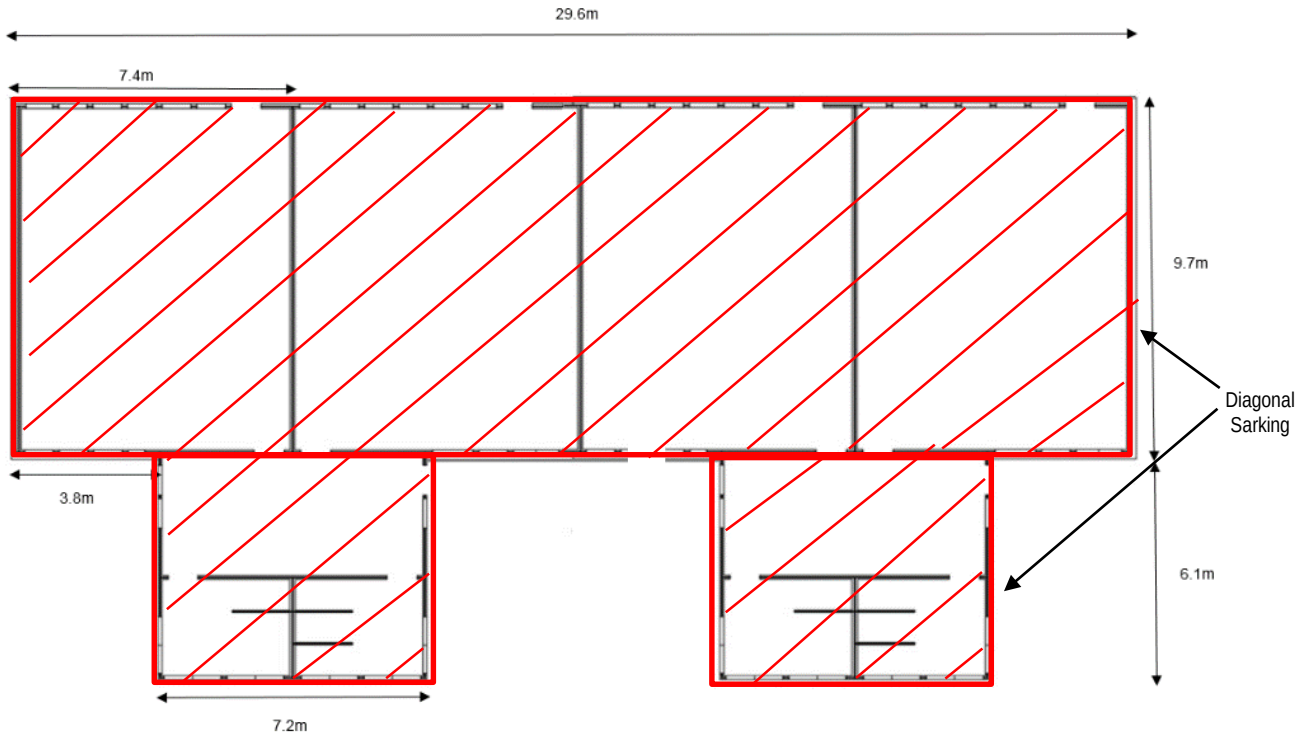
4.3.1 Lintel Layout

These lintels are similar to those in Option 2. A steel 250PFC will be adequate.

5. Roof Diaphragm Capacity Check

5.1 Roof Diaphragm System

The existing ceiling diaphragm consists of diagonal timber sarking in both the main classroom area and the two additional structures. The existing ceiling diaphragm is shown in the figure below:



Roof Sarking Layout

Note: The roof diaphragm was found to have the capacity to resist seismic loading greater than 67%NBS for the Wellington Region. The seismic forces in the Wellington Region (Soil Class C) are greater than the Christchurch Region (Soil Class D). The roof diaphragm will therefore be adequate in Christchurch also.

Below are the roof diaphragm capacity calculations for the Wellington loading case. Refer to the Wellington design for the load derivation.

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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.

Main Classroom Area:

Under longitudinal loading, the sarking diaphragm length is 29.6m, giving the diaphragm a total capacity of 124kN.

The largest shear demand in the diaphragm will be at Bracing Line 2 as this has to support some additional load that isn't able to be supported on Bracing Line 1. From 2.1.4) and 3.13) the critical shear demand in the diaphragm at ductility 2.5, Sp 0.5 is equal to the following:

$F_Dia_{LongMain} = Dem_{BL2}+(Dem_{BL1}-FnBL1)$

$\mathbf{F_Dia_{LongMain} = 49.11kN}$

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

$F_Dia_{LongMain}1.25 = F_Dia_{LongMain}\times(C_dT_{1.25}/C_dT_{2.5})$

$F_Dia_{LongMain}1.25 = \mathbf{147.63}kN$

$\%NBS = 124kN/F_Dia_{LongMain}1.25$

$\mathbf{\%NBS = 83.99\%}$

This is greater than 67%NBS, therefore no further strengthening is required.

End Blocks/Extension

Under longitudinal loading, the sarking diaphragm length is 7.2m, giving the diaphragm a total capacity of 30.24kN.

Half of the Bracing Line 3 demand will be transferred to each end block. Within the end block, the diaphragm is expected to be subject to half the total shear in a given location as there are a minimum of two bracing lines for all options. The critical demand is therefore equal to the following:

$F_Dia_{LongExt}1.25 = 1/2\times[(Dem_{BL3}/2)\times C_dT_{1.25}/C_dT_{2.5}]$

$\mathbf{F_Dia_{LongExt}1.25 = 22.55kN}$

%NBS > 100% OK

No further strengthening is required.

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5.3 Diaphragm Capacity Check – Transverse Direction

Main Classroom Area

Under transverse loading, the sarking diaphragm length is 9.7m, giving the diaphragm a total capacity of 40.74kN.

The internal wall lines will each be subject to 1/4 of the total demand in the Main Classroom. The critical shear in the diaphragm will therefore be 1/8 of the total demand as half the shear will come from either side of the wall.

The transverse diaphragm demand will therefore be equal to the following:

$F_Dia_{TransMain}1.25 = 1/8 \times [F_EQ_{MainClass2.5} \times (C_dT_{1.25}/C_dT_{2.5})]$

$F_Dia_{TransMain}1.25 =$ 24.37kN

%NBS > 100% OK

No further strengthening is required.

End Blocks/Extension

Under transverse loading, the sarking diaphragm length is 6.1m, giving the diaphragm a total capacity of 25.62kN.

Each end block will be subject to half of the Bracing Line 3 demand. In each end block, there are 3 bracing lines, evenly spaced in this area. The critical diaphragm demand is therefore equal to 1/4 of this demand.

The transverse diaphragm demand will therefore be equal to the following:

$F_Dia_{TransExt}1.25 = 1/2 \times 1/4 \times [F_EQ_{Extension2.5} \times (C_dT_{1.25}/C_dT_{2.5})]$

$F_Dia_{TransExt}1.25 =$ 11.28kN

%NBS > 100% OK

No further strengthening is required.

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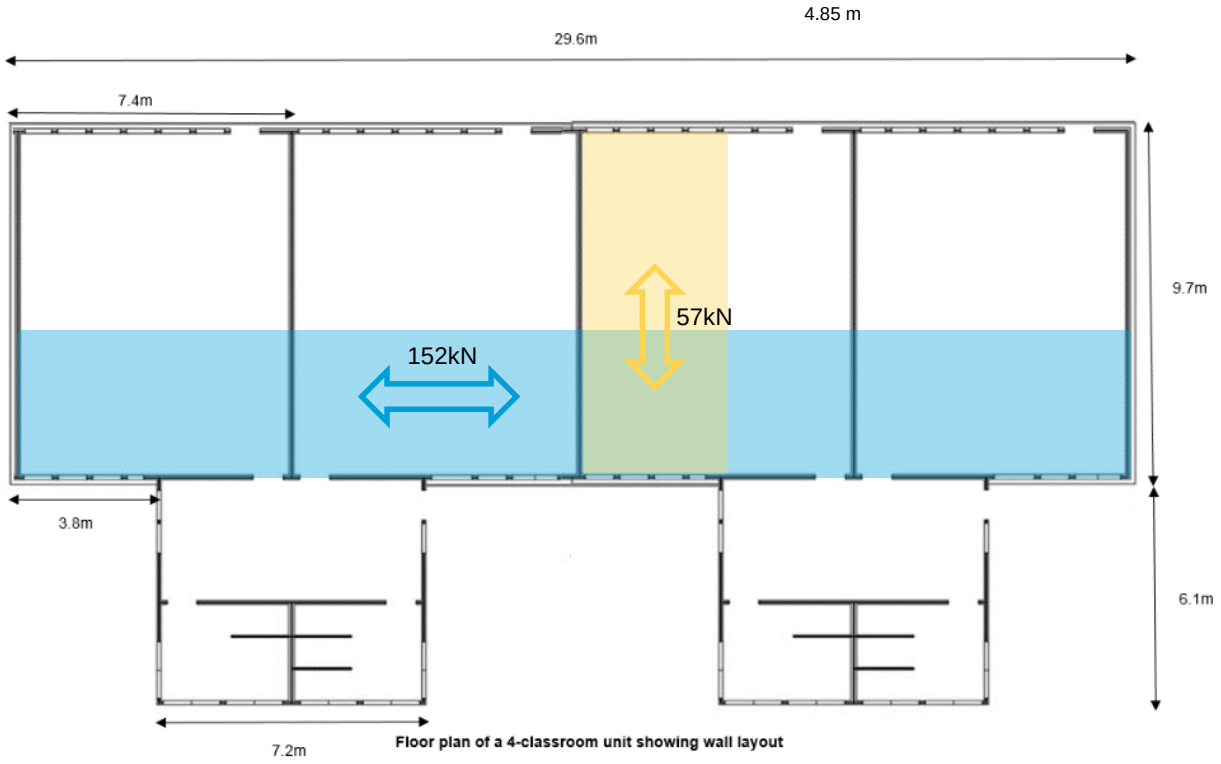
6. Floor Diaphragm/Foundations

The floor diaphragm was found to have the capacity to resist seismic loading greater than 67%NBS for the Wellington Region. The seismic forces in the Wellington Region (Soil Class C) are greater than the Christchurch Region (Soil Class D). The roof diaphragm will therefore be adequate in Christchurch also.

Below are the floor diaphragm capacity calculations for the Wellington loading case. Refer to the Wellington design for the load derivation.

The timber floor features sarking nailed to floor joists. The floor diaphragm will therefore have the same capacity in each direction as the ceiling diaphragms assessed above. The diaphragm will be subject to the weight of the floor plus its live load as well as the weight of the lower half of the walls. The floor diaphragm will need to transfer seismic load directly to the nearest adjacent foundation wall.

Half of the extensions contain a floor slab on grade. 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.



Floor plan of a 4-classroom unit showing wall layout

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Longitudinal Floor Diaphragm Capacity Check

The longitudinal diaphragm length is equal to 29.6m. This correlates to a diaphragm capacity of 124kN (4.2kN/m).

The floor has a similar weight to the roof diaphragm. The seismic load in the floor diaphragm is therefore equal to the roof diaphragm load, plus the additional load generated due to the live load on the floor.

The demand due to the self weight of the floor and lower half of the walls (at ductility 1.25) is therefore equal to the following :

$F_{GFloorLong} = Dem_{BL2} \times C_dT_{1.25}/C_dT_{2.5}$

$F_{GFloorLong}$ **= 97.42kN**

Calculate the live load in the floor:

Plan Area

$Area_{FloorL}$ = (9.7 m/2) × 29.6 m

$Area_{FloorL}$ = **143.56** m^2

Area Reduction Factor

$\Psi_a = 0.3 + 3 / \sqrt{(Area_{FloorL})}$ (Section 3.4.2, NZS 1170.1)

Ψ_a = **0.55**

Floor Live Load

$Q_{FloorL} = \Psi_a \times \Psi_E \times Q \times Area_{FloorL}$

Q_{FloorL} =71.06 kN

Total Demand

$F_{FloorL} = F_{GFloorLong} + C_dT_{1.25} \times Q_{FloorL}$

F_{FloorL} = 151.72kN

%NBS

%NBS Floor Diaphragm - Longitudinal

NBS_{FloorL} = Capacity / F_{FloorL}

NBS_{FloorL} = **124 / 152**

NBS_{FloorL} = **82%NBS**

This is greater than 67%NBS, no strengthening is required.

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Transverse Floor Diaphragm Capacity Check

The transverse diaphragm length is equal to 9.7m. This correlates to a diaphragm capacity of 40.74kN (4.2kN/m).

Similar to the roof diaphragm, the critical demand in the floor is equal to 1/8 of the total transverse demand as one wall line will be subject to 1/4 of the total demand. The demand due to the self weight of the floor and lower half of the walls (at ductility 1.25) is therefore equal to the following:

$F_{GFloorTrans} = (1/8)\times(Dem_{BL1}+Dem_{BL2})\times(C_dT_{1.25}/C_dT_{2.5})$

$F_{GFloorTrans}$ **= 24.37kN**

Calculate the live load in the floor:

Plan Area

$Area_{FloorT} = 7.4\ m \times 9.7\ m$

$Area_{FloorT}$ = **71.78** m^2

Area Reduction Factor

$\Psi_a = 0.3 + 3 / \sqrt{(Area_{FloorT})}$ (Section 3.4.2, NZS 1170.1)

Ψ_a = **0.65**

Floor Live Load

$Q_{FloorT} = \Psi_a \times \Psi_E \times Q \times Area_{FloorT}$

Q_{FloorT} = 41.99 kN

Total Demand

$F_{FloorT} = F_{GFloorTrans} + C_dT_{1.25}\times Q_{FloorT}$

F_{FloorT} = 56.46kN

%NBS

%NBS Floor Diaphragm - Transverse

NBS_{FloorT} = Capacity/**F_{FloorT}**

NBS_{FloorT} = 72%NBS

This is greater than 67%NBS, no strengthening is required.

Foundations

As mentioned above, foundation walls are present around the perimeter of the structure as well as underneath internal bracing walls. All timber bracing elements will be proprietary timber bracing products and will be fixed directly to the foundation walls. The loads are relatively small going through these bracing elements. It is not expected that foundation failure will cause collapse of the structure.

Client:	Ministry of Education		Date: 08/09/2015
Project/Job:	FLS Upgrade	Job No: 246313	
Subject:	Canterbury Block - Christchurch	Sheet No: 58	By: PGD

7. Brick/Block Veneers (External 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.