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Structural Concepts

Detailed Engineering Evaluation

Quantitative Report

Kapuatohe Museum

665 Main North Road, Christchurch

Prepared For:

Christchurch City Council

Ref: 1599-304
23 January 2013

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KAPUATOHE MUSEUM

665 MAIN NORTH ROAD, CHRISTCHURCH

DETAILED ENGINEERING EVALUATION

23 January 2013

FOR:

CHRISTCHURCH CITY COUNCIL

Appendices

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DETAILED ENGINEERING EVALUATION

23 January 2013

FOR:

KAPUATOHE MUSEUM

AT:

665 MAIN NORTH ROAD, CHRISTCHURCH

1.0 Preamble

This report covers our assessment of the structural condition of the Kapuatohe Museum building at 665 Main North Road, Christchurch, following the magnitude 6.3 earthquakes on 2nd February 2011 and that of the 13 June 2011. Our assessment is based on a visual inspection of the outside and inside where it was deemed to be safe to enter. This was carried out in July 2012.

This report describes the damage observed, and comments on remedial work options for both temporary securing of the building, and long term repair where appropriate. This report does not cover **structural strength details in full detail or detailed specification** of remedial works but does provide some investigation and assumptions that will allow an assessment to be made as to whether to reconstruct or demolish.

A Detailed Engineering Evaluation Procedure (DEEP) has recently been developed by CERA to provide consistent, comprehensive and auditable guidelines which help restore confidence in the remaining building stock in Canterbury. We have used these guidelines to form the basis for our detailed engineering evaluation.
The DEEP process follows a two step process, firstly a qualitative assessment then a quantitative assessment, if necessary.
The qualitative assessment involves visual review of the structure and its conditions in order to ascertain whether the structure does or does not fall within required capacity limitations without completing any complex analysis.

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The quantitative assessment involves analytically calculating the capacity of the structure in terms of the current code requirements ie. to estimate the percentage of new building strength available (%NBS).
The overall objective of this assessment is to determine if a strengthening solution is required or not.

More specifically, this report covers:

- Describes the existing building, its construction, and structural system
- Outlines the level of investigation undertaken and where information was obtained
- Summarises earthquake damage caused by the recent Canterbury earthquakes
- Reviews the building's performance in the recent Canterbury earthquakes
- Identifies critical structural weaknesses
- Assesses the building's seismic strength relative to New Building Standard (NBS), commonly referred to as "current code"
- Proposes earthquake strengthening work to bring the building as close as practically possible to 67% of current code

2.0 Scope of Investigation

In July 2012, we visually inspected the building including:

- The exterior from ground level
- The interior including the roof space

The following records have also been obtained and reviewed:

- Drawings showing structural detail of the building.
- Intrusive investigations have not been completed on this occasion.

This report is based on our assessment of the building at the time stated. Photos attached in Appendix A are indicative of the damage and findings. Any subsequent loading by aftershocks, or high winds, may initiate further damage.

3.0 Building Description

General description:
The Kapuatohe Museum is constructed from partially filled reinforced masonry in two parts including a light roof and ceiling structure. It has a nominal ceiling diaphragm nominally connected to the top of the walls. The building was built in 1986.

The building was being used as a Museum, but is currently unoccupied due to earthquake damage. The main occupancy classification in NZS1170 is C3 and importance level of 2. The occupant load is calculated at 19 as classified by the Building Code Clause C table 2.2. The building area is approximately 75m².

Roof construction:
Galvanised iron roofing on purlins and timber trusses and framing.

Ceiling construction:
Gib ceiling linings over battens fixed to underside of trusses.

Outer External Wall construction:
15 series partially filled masonry blockwork.

Internal Wall construction:
Plasterboard linings over timber framing.

Floor construction:
Reinforced concrete slab with reinforced concrete foundations.

4.0 Structural System

Gravity Structural System:
The gravity structural system can be described as simple beam and post/wall support.

Lateral Structural System:
The lateral structural system can be described as face loaded walls supported at foundation level and ceiling level with nominal diaphragms in the form of plasterboard linings taking loads back to the blockwalls and cantilever block columns acting in-plane. Loads are then transferred to reinforced concrete foundations to the ground.

5.0 Strength

The strength of the building has been determined as a % NBS using methodologies provided by NZSEE. A "Preliminary Detailed Assessment", which includes calculations, has been completed as opposed to an IEP ("Initial Evaluation Procedure") as this is deemed more accurate and the IEP would provide an inaccurate %NBS purely because of the buildings age. Our assessments are as follows:

Before September 2010:
The strength of the building before September 2010 is estimated as

Across	
Hazard factor 0.22 (pre 19th May 2011)	64% NBS
Hazard factor 0.3 (post 19th May 2011)	47% NBS
Along	
Hazard factor 0.22 (pre 19th May 2011)	64% NBS
Hazard factor 0.3 (post 19th May 2011)	47% NBS

The Building as a Whole prior to 22 Feb 2011 47% NBS

On day of inspection:
The strength of the building on the day of inspection is estimated as

Across		
Hazard factor 0.3 (post 19th May 2011)	35% NBS	(estimated only)
Along		
Hazard factor 0.3 (post 19th May 2011)	40% NBS	(estimated only)

The Building as a Whole on day of inspection 35% NBS

It must be understood that this strength is based on the overall building strength and not individual elements. Furthermore this estimate is based on the fact that there is now significant cracking and lose of connection between blockwork elements thus making the structure vulnerable.

6.0 Areas of Structural Vulnerability

The building is reasonably sound, however some structural vulnerabilities were found in parts and are in need of strengthening which includes:

- Specifically designed ceiling diaphragms
- Lack of proper connection between ceiling diaphragm and walls
- Lack of continuity between blockwalls and centre columns
- Lowly reinforced central columns

7.0 Damage Description

Damage caused by the February and June earthquakes to the Kapuatohe Museum is described below. Damage described is that observed on the day. Refer to Appendix B for marked-up drawings indicating damaged locations.

i. General Damage to Exterior Blockwalls:
General damage includes minor cracking of block walls.

ii. Damage between Blockwalls and Columns:
Cracking has occurred at the junction between the blockwalls and columns.

iii. Damage to Ceiling diaphragm:
Ceiling diaphragm has dropped and become dislodged from the walls in the South West corner.

8.0 Immediate Securing of the Building

It is our opinion that no immediate securing is necessary at this stage.

9.0 Long Term Repair

This section of the report outlines options for repair to restore the building to a minimum 67% NBS. Options for repair and/or strengthening will ultimately need to be discussed with the owner, and will be subject to revised local authority legislation.

Requirements to bring the building up to 67%NBS include:

i. Internal Masonry Columns:

Install new D16 (600crs) pins from outer face of blockwall into the internal column and epoxy in place. Epoxy grout the joint between both elements internally using **SIKADUR 52**.

ii. Ceiling Diaphragms:

Install new 10mm ultraline diaphragm over damage areas.
Refix all edges @ 100mm crs to top plate.

iii. Plywood Shear Wall:

Install new plywood shear wall to line of internal columns.

iv. General Damage:

Repair general cracking to blockwork with sikadur injectokit or similar.

Requirements to bring the building up to 33%NBS include:

i. Internal Masonry Columns:

Install new D16 (600crs) pins from outer face of blockwall into the internal column and epoxy in place. Epoxy grout the joint between both elements internally using **SIKADUR 52**.

ii. Ceiling Diaphragms:

Install new 10mm ultraline diaphragm over damage areas.
Refix all edges @ 100mm crs to top plate.

iii. General Damage:

Repair general cracking to blockwork with sikadur injectokit or similar.

The costs associated with the repairs would require the appropriate professional to visit the site to view the extent of damage. At this stage we have not provided any specific detailing for repair works but can so at your request.

10.0 Elements Not Inspected

The following is a list of elements not specifically inspected:

- Foundations below ground level (there is no sign of damage or movement to this area due to seismic activity)
- Soils (Although the building appears to be founded on rock a geotechnical investigation or assessment and report by an experienced Geotechnical Engineer is recommended)

11.0 Applicability

Recommendations and opinions in this report are based on data and records obtained from Christchurch City Council and nondestructive visual inspections. Although there is nothing to suggest otherwise, as the nature and continuity of the structure hidden from sight (e.g. reinforcing steel, bolt depths etc.) is inferred, it must be appreciated that actual conditions could vary.
Findings presented in this report are for the sole use of the client. The findings may not contain sufficient information for use by other parties, and as such should not be relied upon unless discussed with Structural Concepts Ltd. We have exercised our services in a professional manner using a degree of care and skill normally, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

Prepared By:

Garry Newton
BE (Civil), MIPENZ, CPEng, IntPE, APEC Engineer
Director

On behalf of Structural Concepts Ltd

APPENDIX A

KAPUATOHE MUSEUM
CHRISTCHURCH

PHOTOGRAPHS

Please note that the photographs provided in this report are not high quality and are for providing information that shows the indicative damage found around the building for structural engineering assessment only.

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APPENDIX B

KAPUATOHE MUSEUM
CHRISTCHURCH

MARKED-UP DRAWING INDICATING DAMAGED LOCATIONS

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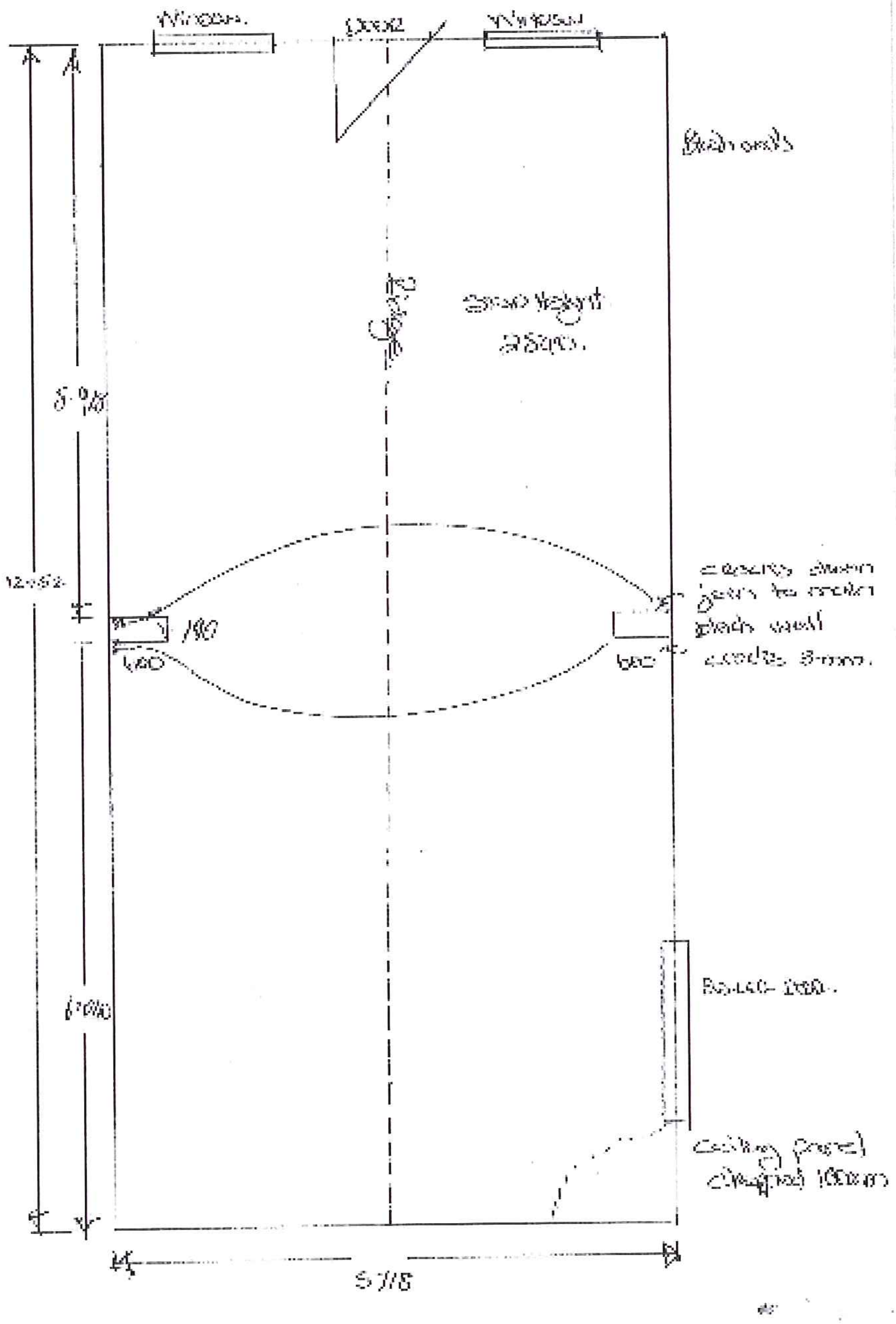
EAST

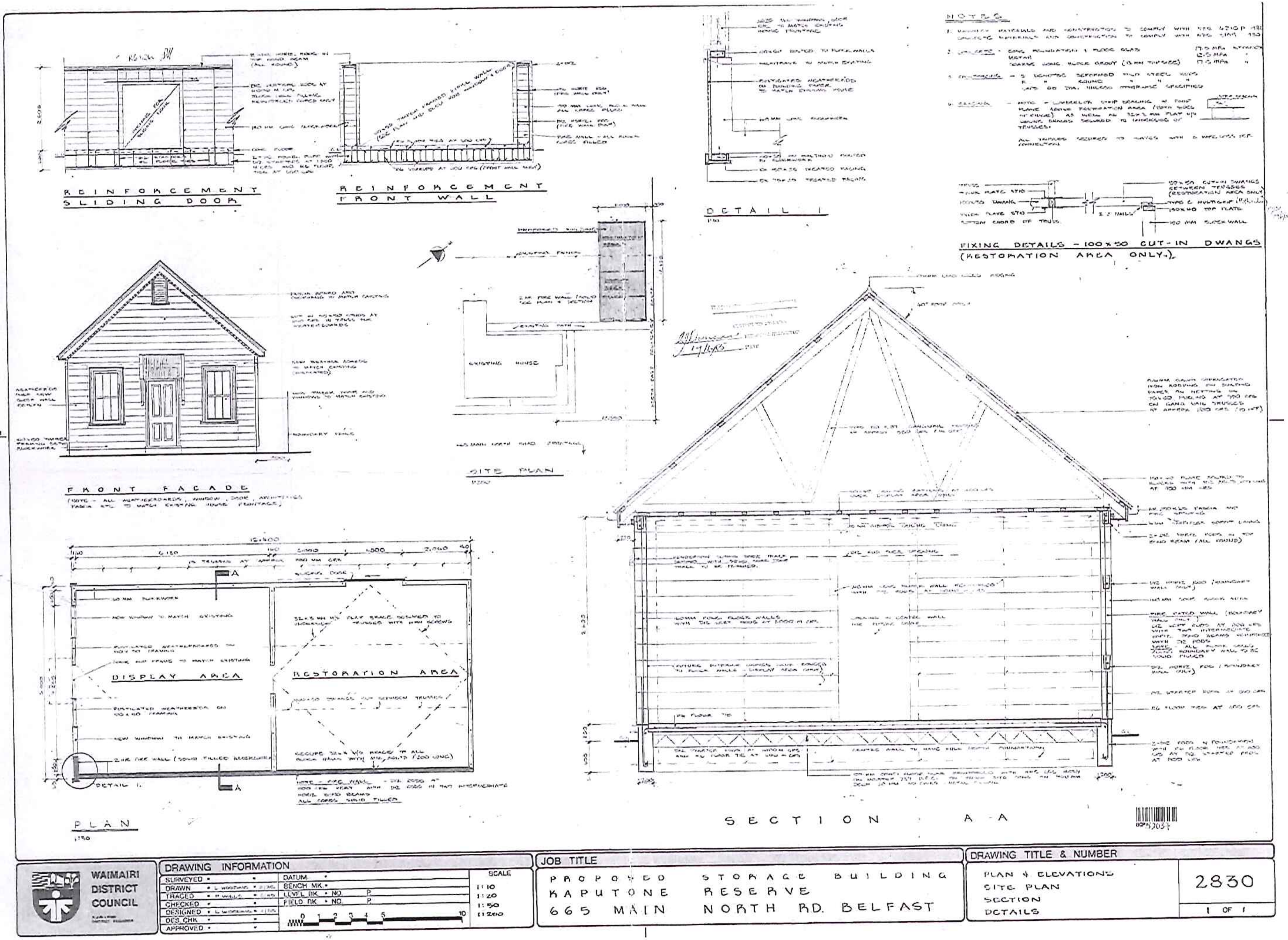
KEYSTONE MUSEUM BUILDING



APPENDIX C

KAPUATOHE MUSEUM
CHRISTCHURCH

FLOOR PLANS


REINFORCEMENT SLIDING DOOR
REINFORCEMENT FRONT WALL
DETAIL 1
NOTES
FIXING DETAILS - 100 x 50 CUT-IN DWANGS (RESTORATION AREA ONLY.)
FRONT FACADE
SITE PLAN
EXISTING HOUSE
PLAN
DISPLAY AREA
RESTORATION AREA
DETAIL 1
SECTION A-A
WAIMAIRI DISTRICT COUNCIL
DRAWING INFORMATION
SURVEYED
DRAWN
TRACED
CHECKED
DESIGNED
DES. CHK.
APPROVED
DATUM
BENCH MK.
LEVEL BK. NO. P.
FIELD BK. NO. P.
SCALE
1:10
1:20
1:50
1:200
JOB TITLE
PROPOSED STORAGE BUILDING
KAPUTONE RESERVE
665 MAIN NORTH RD. BELFAST
DRAWING TITLE & NUMBER
PLAN & ELEVATIONS
SITE PLAN
SECTION
DETAILS
2830
1 OF 1

APPENDIX D

KAPUATOHE MUSEUM
CHRISTCHURCH

DESIGN FEATURES REPORT

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Philosophy

Scope

In general terms, the scope of work is as follows:
Provide seismic assessment of existing building

Means of compliance

The following standards have been used:
- NZS 1170.0:2002
- NZS 1170.1:2002

- NZS 1170.5:2004
- NZS 3101:1995
- NZS 3602:2003
- NZS 3603:1993
- NZS 4203:PART 1:2004

THE STRUCTURE

General

The building is constructed using a partially filled reinforced masonry walls on a NZS3604 reinforced concrete foundation/floor slab. The roof is to consist of pressed plate trusses (by others) and timber purlins and battens with plasterboard ceiling. The location of the building is 655 Main North Road, Christchurch. The importance level for the building has been assessed as Importance Level 2. The design life of the structure is 50 years. For the purpose of analysis, the across and along directions are as per the geometric shape.

Chosen Design Life	50 Years
Chosen Importance Level	2
Annual Probability of exceedance (inverse) Ultimate	500
Annual Probability of exceedance (inverse) Service	25

Gravity structure

Load paths: → → → →
Trusses/rafters Walls foundations

Lateral load resisting structure

Across the building
Roof → In-plane Masonry walls → → → foundations

Along the building
Roof → In-plane Masonry walls → → → foundations

Significant Design Features
the masonry is partially filled with sparse reinforcement. Diaphragm design and fixings are nominal with no specific design.

SOIL CONDITIONS

LDE Ltd have carried out a Soils investigation and report with recommendations. We have used 150Kpa Ultimate or 50Kpa working for The bearing pressure of the Ground this is as per the report carried out by LDE However, we do recomend that an engineer is to be engaged to inspect the open foundation to verify.

DESIGN LOADS

Vertical loads

All Dead loads are listed on the gravity loads sheet at the front of these calculations.

All Live loads are listed on the gravity loads sheet at the front of these calculations.

Lateral Loads
Wind

Site wind speed NA Ult (m/s)

Further information on wind speeds, internal pressures etc are on the main wind load sheets contained in these calculations.

Seismic loads
Analysis methodology
The seismic analysis has been completed in accordance with NZS 1170.5:2004. Design Spectra are in accordance with NZS 1170.5:2004 for site sub soil class D. Analysis has been completed using the Equivalent Static Method for bracing.

Across the building		
Structural ductility factor (Ultimate)	μ	1.25
Structural Performance factor (Ultimate)	Sp	0.70
Along the building		
Structural ductility factor (Ultimate)	μ	1.25
Structural Performance factor (Ultimate)	Sp	0.70

SERVICEBILITY CRITERIA

The following servicebility criteria have been chosen for the project:
Note: These are generally in line with those recommanded in NZS1170.2 Table C1.

<u>**Seismic deflections/storey drift**</u>	<u>**Criteria**</u>	<u>**Phenomenon controlled**</u>
Maximum allowable deflection (SLS)	spacing/200	Damage to cladding
Maximum allowable storey Drift (ULS)	height/40	Soft storey protection
<u>**Wind deflections**</u>		
Maximum allowable lateral deflection (SLS)	spacing/200	Damage to cladding
Maximum allowable vertical deflection (SLS)	span/200	Damage to cladding/finishes
<u>**Gravity deflections**</u>		
Maximum allowable deflection (SLS)	span/500	Visual sag

SOFTWARE

The following computer applications were used for the design:

Analysis type	Software used
Stuctural analysis	Excel 2009
Structural design	Excel 2009

Significant or Special Construction Features

Reiforcing spacing and diaphragm fixings.

104 Assumptions

We have assumed 300Kpa Ultimate or 100Kpa working for the bearing pressure of the Ground.

The building has been constructed as per the original plans in general terms.

APPENDIX E

KAPUATOHE MUSEUM
CHRISTCHURCH

PRELIMINARY CALCULATIONS

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Client: Christchurch City Council

Project: Kapuatohe Museum
665 Main North Road, Christchurch

Ref: 1599-0304

Date: 20-Jan-13

CALCULATIONS

BY GARRY NEWTON

BE (Civil) , MIPENZ, CPEng, IntPE(NZ), APEC Engineer

CONTENTS

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Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Gravity Loads**		

Sheet No.: **2**

Loads

Roof

Corr/Trimdek CS	0.059	
Trusses (Rafter 150).(	0.110	
Battens 05 .4	0.034	
Purlins 05 .4	0.034	
Rockwool Insu.	0.002	
Gib Board 13	0.120	
	0.359	kPa
0.359 / Cos 30 =	0.415	kPa

External Walls

15S 3rd core	2.600	
	2.600	kPa

Live loads

R2 Roofs	0.25	kPa

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Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum 665 Main North Road, Christchurch	Date:	20/1/13
Subject:	Philosophy	BY:	GN

Sheet No.: 3

Philosophy

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Gravity structure

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Lateral load resisting structure

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Along the building
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Wind deflections		
Maximum allowable lateral deflection (SLS)	spacing/200	Damage to cladding
Maximum allowable vertical deflection (SLS)	span/200	Damage to cladding/finishes
Gravity deflections		
Maximum allowable deflection (SLS)	span/500	Visual sag

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Client: **Christchurch City Council**
Project: **Kapuatohe Museum**
665 Main North Road, Christchurch
Subject: **EQ Static 1170.5 Blockwork**

Ref: **1599-0304**
Date: **20/1/13**
BY: **GN**

Sheet No.: **7**

Seismic Loads to NZS 1170.5

Ref:	Design	Output

Design working live **50 Years**

Importance level 2

Annual Probability of exceedance (inverse) Ultimate 500

Annual Probability of exceedance (inverse) Service 25

Element			Area/length	Load Kpa	Total kN
Roof			73.20	0.41	30.37
External Walls			47.58	2.60	123.71
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
	1.00	0.40	0.00	0.00	0.00

154.07 kN

Live load reduction

Total floor area 0.0

$$.3+\frac{3}{\sqrt{A}} = 1.000$$

But not less than .5

Total building weight

154.07 kN

Ref:	Design				Output
	Soil type				
	D. Deep or soft soil				
	Across the building				
	Period of building across the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☑
	Structural ductility factor (Ultimate)	μ =	1.25		
	Structural ductility factor (Service SLS1)	μ =	1.25		
	Hazard Factor Christchurch	Z =	0.3		
	Return period factor	Ru =	1.00		
	Return period factor	Rs =	0.25		
	Structural Performance factor (Ultimate)	Sp =	0.70		
	Structural Performance factor (Service)	Sp =	0.70		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0	n/a	
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.23		
	Ultimate	kμ =	1.14		
	Service	kμ =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.55	But not less than 0.030Ru	
	Ultimate force across the building	Cd(T1) x Wi =	84.93	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.14		
	Service force across the building	Cd(T1) x Wi =	21.23	kN Total	
	Along the building				
	Period of building along the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☑
	Structural ductility factor (Ultimate)	μ =	1.25		
	Structural ductility factor (Service SLS1)	μ =	1.25		
	Structural Performance factor (Ultimate)	Sp =	0.70		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0		
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.23		
	Ultimate	kμ =	1.14		
	Service	kμ =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.55	But not less than 0.030Ru	
	Ultimate force along the building	Cd(T1) x Wi =	84.93	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.14		
	Service force across the building	Cd(T1) x Wi =	21.23	kN Total	

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Client: **Christchurch City Council**
Project: **Kapuatohe Museum**
665 Main North Road, Christchurch
Subject: **EQ Static 1170.5 Blockwork (2)**

Ref: **1599-0304**
Date: **20/1/13**
BY: **GN**

Sheet No.: **9**

Seismic Loads to NZS 1170.5

Ref:	Design	Output

Design working live **50 Years**

Importance level 2

Annual Probability of exceedance (inverse) Ultimate 500

Annual Probability of exceedance (inverse) Service 25

Element			Area/length	Load Kpa	Total kN
Roof			73.20	0.41	30.37
External Walls			31.72	2.60	82.47
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
			0.00	0.00	0.00
	1.00	0.40	0.00	0.00	0.00
					112.84 kN

Live load reduction

Total floor area 0.0

$$.3+\frac{3}{\sqrt{A}} = 1.000$$

But not less than .5

Total building weight
112.84 kN

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Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**EQ Static 1170.5 Blockwork (2)**	Sheet No.:	**10**

Ref:	Design				Output
	Soil type				
	D. Deep or soft soil				
	Across the building				
	Period of building across the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☑
	Structural ductility factor (Ultimate)	μ =	1.25		
	Structural ductility factor (Service SLS1)	μ =	1.25		
	Hazard Factor — Christchurch	Z =	0.3		
	Return period factor	Ru =	1.00		
	Return period factor	Rs =	0.25		
	Structural Performance factor (Ultimate)	Sp =	0.70		
	Structural Performance factor (Service)	Sp =	0.70		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0	n/a	
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.23		
	Ultimate	kμ =	1.14		
	Service	kμ =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.55	But not less than 0.030Ru	
	Ultimate force across the building	Cd(T1) x Wi =	62.20	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.14		
	Service force across the building	Cd(T1) x Wi =	15.55	kN Total	
	Along the building				
	Period of building along the building		0.40		
	Does the seismic bracing have ductile capabilities but is designed as nominally ductile				☑
	Structural ductility factor (Ultimate)	μ =	1.25		
	Structural ductility factor (Service SLS1)	μ =	1.25		
	Structural Performance factor (Ultimate)	Sp =	0.70		
	Spectral Shape Factor (across)	Ch(T) =	3.00		
	Near Fault factor	N(T,D) =	1.0		
	Elastic site spectra (Ultimate)	C(T) =	0.90		
	Elastic site spectra (Service)	C(T) =	0.23		
	Ultimate	kμ =	1.14		
	Service	kμ =	1.14		
	Ultimate				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.55	But not less than 0.030Ru	
	Ultimate force along the building	Cd(T1) x Wi =	62.20	kN Total	
	Service				
	Horizontal design action coefficients (Across)	Cd(T1) =	0.14		
	Service force across the building	Cd(T1) x Wi =	15.55	kN Total	

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Client: **Christchurch City Council**
Project: **Kapuatohe Museum**
665 Main North Road, Christchurch
Subject: **Load Distribution**

Ref: **1599-0304**
Date: **20/1/13**
BY: **GN**

Sheet No.: **11**

Ref:	Design	Output

Assume load is distributed on a tributary area basis

			V* kN	M* kNm
Total Base Shear Walls	84.93	kN		
Total Base Shear Columns	62.20	kN		
Side Shear Walls	42.47	kN	42.47	110.4128
End Shear Walls	10.62	kN per wall	10.62	27.60319
Internal Columns	15.55	kN per column	15.55	40.43101

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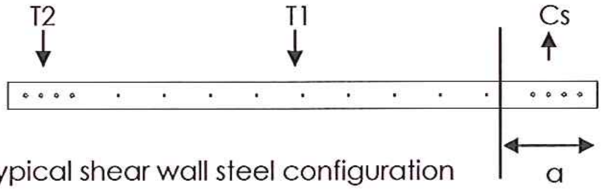
Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Masonry In-Plane Shear Wall**		

Design of elastic Masonry shear wall for In - Plane loads — Sheet No.: **12**

Ref:	Design					Output
	Design moment		M*	110	kNm	
	Design Shear force		V*	42	kN	
	Ductility factor used		μ	1.25	<= 1.25	



Typical shear wall steel configuration

The moment capacity is based on concrete theory, as found in any concrete text book, i.e. ccanz "Red Book"

Clear storey height			2600	mm
Length of wall		Lw	12000	mm
Width of wall web region		bw	140	mm
Concrete grade		Fc'	4	Mpa
Steel reinforcement yield stress (Yeilding steel)		Fy	300	Mpa
Steel reinforcement yield stress (Shear steel)		fyt	300	Mpa
<u>Tension steel For T1</u>				
Number of bars		No.	10	
Diameter of bars		dia	12	mm
Area of bars at T1		As1	1131	mm²
Tension capacity	As x Fy =	T1	339.3	kN
<u>Tension steel For T2</u>				
Number of bars		No.	1	
Diameter of bars		dia	12	mm
Area of bar: As x Fy =		As2	113	mm²
Tension capacity	As x Fy =	T2	33.9	kN
<u>Compression steel For Cs</u>				
Number of bars		No.	1	
Diameter of bars		dia	12	mm
Area of bars at T1		AsCs	113	mm²
Tension capacity	As x Fy =	Cs	33.9	kN
Axial load on wall				
Self weight of wall	12 x 2.6 x 0.14 x 22 =		96.10	
Other dead load			0.00	
		Nc/.85 =	113.05	
C = T1 + T2 + Nn - Cs = 339.34 x 33.93 + 113.05 - 33.93 =			452	

<u>Depth of equivalent stress block a</u>

$$a = \frac{C}{.85 \times Fc' \times b} = 950 \text{ mm}$$

therefore c =	950 / 0.85 =	1118	mm (distance to n/a from RHS)		
Centroid of T1 from LHS =		6000	mm Internal leverarm	Jd =	5524.8
Centroid of T2 from LHS =		792	mm Internal leverarm	Jd =	10732.8

Ref:	Design				Output
	Mn for T1 = AS x Fy x Jd x 10^-6	1874.8	kNm		
	Mn for T2 = AS x Fy x Jd x 10^-6	364.2	kNm		
	Mn for N* = N* x Jd x 10^-3 =	624.6	kNm		
		2863.6	kNm		
	ØMn = 2434 kNm	> 110 kNm therefore OK			PASS
	Shear Steel design				
	Ultimate shear force on wall	V*wall	42	kN	
	Nominal shear stress				
11.3.10.3.3	Note d = 80% of actual length				
	Masonry grade		B		
	Thickness of wall	bw	140	mm	
	Length of wall	Lw	12000	mm	
	Length of shear plane — Lw x .8 =	d	2.4	m	
	Shear Steel Bar diameter		1	mm	
	Spacing		1000	mm	
	Area of steel per bar	Av	1	mm²	
	Area of steel per m	Av	1	mm²/m	
	Height of wall (he)	he	2.6	m	
	Height to length ratio	he / Lw	0.21667		
	Ratio of steel to concrete — Av/ bw x he =	pw	0.00001		
	Shear Stress 42.4664465673535 x 10^-3 / (2.4 x 0.14) =	vn	0.126	Mpa	
	Shear Strength provided by Masonry				
T 10.1	Dependent nominal strengh	vbm	0.300	Mpa	
		C1	1.000		
		C2	1.500		
Eq. 10-6	Shear provided by masonry — (C1+C2) x vbm =	vm	0.750	Mpa	
	Minimum Reinforcement only is required				
	Shear strength provided by masonry under axial load				
	Axial load on wall (Dead Load only)		96.10	kN	
	Approximate angle of strut in wall		23.4	Degrees	
	Axial load shall not be taken greater than — 0.1 x f'm x Ag =		134	kN	
	(.9 x N* / bw x d) x TAN =	vp	0.112	Mpa	
	Maximum value of vp	vp Max	0.4	Mpa	
	Design vp	vp	0.112	Mpa	

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Structural Concepts

Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	Masonry In-Plane Shear Wall		

Sheet No.: 14

Design of elastic concrete shear wall for In - Plane loads continued

Ref:	Design				Output
	Shear strength provided by shear steel				
	Shear stress provided by steel	$C3\frac{Av.Fy}{bw.S}$ = Vs		0.00 Mpa	
	Minimum area of shear steel	1.5bw x S / Fy =	Av	700 mm²/m	
	Total shear stress provided	vm + vp + vs =	vn	0.863 Mpa	
	Total shear strength				
T. 10.1	Maximum total shear stress		vg	1.500 Mpa	
	Design total shear stress		vn	0.863 Mpa	
	Shear strength of wall		Vn	1160 kN	
	Strength reduction			0.75	
	Shear strength of wall		ϕV	870 kN	PASS

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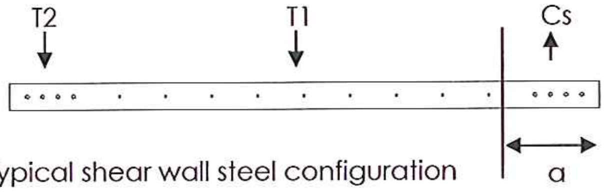
Structural Concepts

Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	Masonry In-Plane Shear Wall (2)		

Design of elastic Masonry shear wall for In - Plane loads — Sheet No.: 15

Ref:	Design					Output
	Design moment		M*	28	kNm	
	Design Shear force		V*	11	kN	
	Ductility factor used		μ	1.25	<= 1.25	



Typical shear wall steel configuration

The moment capacity is based on concrete theory, as found in any concrete text book, i.e. ccanz "Red Book"

Clear storey height			3000	mm
Length of wall		Lw	2000	mm
Width of wall web region		bw	140	mm
Concrete grade		Fc'	4	Mpa
Steel reinforcement yield stress (Yeilding steel)		Fy	300	Mpa
Steel reinforcement yield stress (Shear steel)		fyt	300	Mpa
<u>Tension steel For T1</u>				
Number of bars		No.	1	
Diameter of bars		dia	12	mm
Area of bars at T1		As1	113	mm²
Tension capacity	As x Fy =	T1	33.9	kN
<u>Tension steel For T2</u>				
Number of bars		No.	1	
Diameter of bars		dia	12	mm
Area of bar: As x Fy =		As2	113	mm²
Tension capacity	As x Fy =	T2	33.9	kN
<u>Compression steel For Cs</u>				
Number of bars		No.	1	
Diameter of bars		dia	12	mm
Area of bars at T1		AsCs	113	mm²
Tension capacity	As x Fy =	Cs	33.9	kN
Axial load on wall				
Self weight of wall	2 x 3 x 0.14 x 22 =		18.48	
Other dead load			0.00	
		Nc/.85 =	21.74	
C = T1 + T2 + Nn - Cs = 33.93 x 33.93 + 21.74 - 33.93 =			56	

<u>Depth of equivalent stress block a</u>

$$a = \frac{C}{.85 \times Fc' \times b} = 117 \text{ mm}$$

therefore c =	117 / 0.85 =	138	mm (distance to n/a from RHS)	
Centroid of T1 from LHS =		1000	mm Internal leverarm Jd =	941.5
Centroid of T2 from LHS =		97	mm Internal leverarm Jd =	1844.0

Design of elastic concrete shear wall for In - Plane loads continued

Ref:	Design				Output
	Mn for T1 = AS x Fy x Jd x 10^-6	31.9	kNm		
	Mn for T2 = AS x Fy x Jd x 10^-6	62.6	kNm		
	Mn for N* = N* x Jd x 10^-3 =	20.5	kNm		
		115.0	kNm		
	ØMn = 98 kNm	> 28 kNm therefore OK			PASS
	Shear Steel design				
	Ultimate shear force on wall	V*wall	11	kN	
	Nominal shear stress				
11.3.10.3.3	Note d = 80% of actual length				
	Masonry grade		C		
	Thickness of wall	bw	140	mm	
	Length of wall	Lw	2000	mm	
	Length of shear plane Lw x .8 =	d	1.6	m	
	Shear Steel Bar diameter		12	mm	
	Spacing		1000	mm	
	Area of steel per bar	Av	113	mm²	
	Area of steel per m	Av	113	mm²/m	
	Height of wall (he)	he	3.0	m	
	Height to length ratio	he / Lw	1.50000		
	Ratio of steel to concrete Av/ bw x he =	pw	0.00081		
	Shear Stress10.6166116418384 x 10^-3 / (1.6 x 0.14) =	vn	0.047	Mpa	
	Shear Strength provided by Masonry				
T 10.1	Dependent nominal strengh	vbm	0.300	Mpa	
		C1	0.027		
		C2	1.000		
Eq. 10-6	Shear provided by masonry (C1+C2) x vbm =	vm	0.077	Mpa	
	Minimum Reinforcement only is required				
	Shear strength provided by masonry under axial load				
	Axial load on wall (Dead Load only)		18.48	kN	
	Approximate angle of strut in wall		71.6	Degrees	
	Axial load shall not be taken greater than	0.1 x f'm x Ag =	90	kN	
	(.9 x N* / bw x d) x TAN =	vp	0.223	Mpa	
	Maximum value of vp	vp Max	0.4	Mpa	
	Design vp	vp	0.223	Mpa	

Design of elastic concrete shear wall for In - Plane loads continued

Ref:	Design				Output
	Shear strength provided by shear steel				
	Shear stress provided by steel	$C3\frac{Av.Fy}{bw.S}$	= Vs	0.19 Mpa	
	Minimum area of shear steel	1.5bw x S / Fy =	Av	700 mm²/m	
	Total shear stress provided	vm + vp + vs =	vn	0.494 Mpa	
	Total shear strength				
T. 10.1	Maximum total shear stress		vg	0.800 Mpa	
	Design total shear stress		vn	0.494 Mpa	
	Shear strength of wall		Vn	111 kN	
	Strength reduction			0.75	
	Shear strength of wall		ϕV	83 kN	PASS

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Client: **Christchurch City Council**
Project: **Kapuatohe Museum**
665 Main North Road, Christchurch
Subject: **External Column**

Ref: **1599-0304**
Date: **20/1/13**
BY: **GN**

Sheet No.: **18**

REINFORCED MASONRY RECTANGULAR BEAM DESIGNED TO NZS4230:PART 1:2004

Ref:	Design				Output
	Span of beam to centre of supports		L	2.6 m	
	Dead load not including the beam		G1	0.00 kN/m	
	Beam load		G2	0.00 kN/m	
	Live load UDL		Q	0 kN/m	
	Live load Plt		E	7.8 kN	
	Dead load factor		DF	0	
	Live load factor		LF	1	
	Design load per meter width				
		((G1+G2) x DF) + (Q x LF) =	W	0.0 kN/m	
	Cantilever				
	Maximum moment for UDL is		M*	20.2 kNm	
	Maximum shear is		V*	7.8 kN	
	Beam dimensions and materials				
Table 3.1	Masonry grade		Fc'	4 Mpa	
3.4.5	Steel reinforcement yield stress		Fy	300 Mpa	
	Shear steel yield stress		Fyt	300 Mpa	
	Cover to reinforcement		C	90 mm	
	Depth of beam		D	540 mm	
	Width of beam		bw	140 mm	
	Effective depth	H-C-DIA/2 =	d	442 mm	
3.4.7	Strength reduction factor flexural		ϕ	0.85	
	Strength reduction factor Shear		ϕ	0.75	
	Lever arm	$d - \frac{As.Fy}{1.7Fc'b} =$	Jd	379 mm	
	Main bar diameter		DIA	16 mm	
	Number of bars		N	1	
	Area of steel provided		As	201 mm²	PASS
	Minimum & Maximum area of tension steel				
8.3.6.3 (a)	Min. area of tension steel	.7 bw x d / fy =	As min	144 mm²	
8.3.6.3 (b)	Alternatively may be 1/3 greater than what is required by analysis				
8.3.6.4	Maximum area of reinforcement	.75Pb =	As Max	459 mm²	
8.3.6.2	Maximum area of reinforcement in a grouted space				
		8 x tc x hu / Fy =	As Max	280 mm²	
			As Max	280 mm²	
	Moment capacity	ø x AS x Fy x Jd x 10^-6 =	øMn	19.4 kNm	> 20.22 kNm **FAILED** **96% NBS**
	Shear Check				
10.3.2.1	Total nominal shear stress	V* / (bw.d) =	vn	0.126 Mpa	
10.3.2.4	Maximum shear stress vn shall be less than				
		Table 10.1	vg	0.9 Mpa	> 0.126 Mpa PASS

Ref:	Design				Output
10.3.2.6	**Shear stress provided by masonry**			For beams	
	$vm \approx (C1 + C2)vbm$	Where	$C1 = 33\rho \frac{fy}{300}$	$C2 = 1.0$	
10.1	Ratio of tension reinfrocement	As / bw.d =	ρ	0.0032	
	Basic shear strength table 10.1		vbm	0.70 Mpa	
			C1	0.107	
	Shear stress provided by masonry	(C1+C2)vbm =	vm	0.775 Mpa	
10.3.2.1	Nominal shear strength of masonry	vn = vm			
		vn x bw x d =	Vn	47.96 kN	
	Shear strength of masonry		ØVn	36.0 kN	> 7.78 kN PASS
	Shear steel not required by calculation (min steel only)				
	Stirrup & tie reinforcement provided				
	Bar dia		dia	12 mm	
	No. legs			1	
	Spacing		S	400 mm	< 270 mm
	Min Area		Av	23.3 mm²	**NA**
	Area provided			113.1 mm²	> 23.33 mm²
8.3.11	**Minimum stirrup reinforcement**				PASS
8.3.11.4	Shear steel spacing shall be equal to or less than		Smax	270.0 mm	
	Min diameter of shear reinforcement			6 mm	PASS
10.3.2.11	**Shear reinforcement contribution to strength**				
	$vs = C3\frac{Av.fy}{bw.S}$	Where	C3 = 1.0 for beams		
			vs	0.61 Mpa	
		vs + vm =	vn	1.4 Mpa	
10.3.2.11	Nominal shear strength of masonry				
		vn x bw x d =	Vn	85.5 kN	
	Shear strength of masonry		ØVn	64.1 kN	> 7.78 kN
	Where shear reinforcement is required by 10.3.2.11(a)				PASS
	The min area of steel shall be:-				
		$\frac{.15 bw.S}{fy}$	Av	28.0 mm²	< 113.1 mm² PASS
8.3.4	From 8.3.4 (b) we have L x 0.083				
	and the minimum depth is			216 mm	< 540 mm PASS

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Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**External Foundation beam**		

REINFORCED CONCRETE RECTANGULAR BEAM DESIGNED TO NZS3101:PART 1:2006 — Sheet No.: **20**

Ref:	Design				Output
	Design bending moment from analysis		M*	20 kNm	
	Shear from analysis		V*	7 kN	
	Beam dimensions and materials				
5.2.1	Concrete grade		Fc'	20 Mpa	
5.3.3	Steel reinforcement yield stress		Fy	300 Mpa	
	Shear steel yield stress		Fyt	300 Mpa	
	Cover to reinforcement		C	75 mm	
	Depth of beam		D	600 mm	
	Width of beam		bw	150 mm	
	Effective depth	H-C-DIA/2 =	d	517 mm	
2.3.2.2	Strength reduction factor flexural		ϕ	0.85	
	Strength reduction factor Shear		ϕ	0.75	
	Lever arm	$d - \frac{As.Fy}{1.7Fc'b} =$	Jd	505 mm	
	Main bar diameter		DIA	16 mm	
	Number of bars		N	1	
	Area of steel provided		As	201 mm²	
	Minimum area of tension steel				
9.3.8.2.1	Min. area of tension steel	$\frac{\sqrt{Fc'}}{4Fy} bw.d =$	As min	289 mm²	
	But equal to or greater than	1.4 bw.d/Fy =	As min	362 mm²	
9.3.8.2.3	Alternatively may be 1/3 greater than what is required by analysis				
	Moment capacity	ø x AS x Fy x Jd x 10^-6 =	øMn	25.9 kNm	> 20.22 kNm PASS **128% NBS**
	Shear Check				
7.5.1	Total nominal shear stress	V* / (bw.d) =	vn	0.087 Mpa	
7.5.2	Maximum shear stress vn shall be less than				
		.2Fc' or 8Mpa		4.0 Mpa	> 0.087 Mpa PASS
9.3.9.3.4	**Shear stress provided by concrete**				
	$Vc = vc.Acv$ Where $vc = kd.ka.vb$				
9.1	Ratio of tension reinfrocement	As / bw.d =	ρ	0.0026	
	vb = smaller of $(.07+10\rho)\sqrt{Fc'}$	OR $.2\sqrt{Fc'}$			
	But not less than .08 x Fc'^.5		vb	0.429 Mpa	
	Aggregate size factor		ka	1.0	
	Effective depth factor		kd	0.94	
		vb x ka x kd =	vc	0.402 Mpa	
	Nominal shear strength provided by concrete				
		vc.Acv =	Vc	31.2 kN	
		Shear steel not required			

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Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**External Foundation beam**		

REINFORCED CONCRETE RECTANGULAR BEAM DESIGNED CONTINUED — Sheet No.: **21**

Ref:	Design				Output
9.3.9.4.15	**Minimum shear steel requirement**				
		Bar dia	dia	6 mm	
		No. legs		1	
	$Av = \frac{1}{16}\sqrt{f'c}\frac{bw.s}{fyt}$	Spacing	S	200 mm	< 258.5 mm
		Min Area	Av	28.0 mm²	PASS
		Area provided		28.3 mm²	> 27.95 mm²
9.3.9.3.6	**Shear reinforcement required when vn>vc**				PASS
	$Vs = Av.fyt.\frac{d}{S}$		Vs	21.9 kN	
7.5.3	Shear strength	(Vc + Vs) x ϕ =	Vn	39.8 kN	> 6.74 kN
					PASS

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Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	Internal Column		

REINFORCED MASONRY RECTANGULAR BEAM DESIGNED TO NZS4230:PART 1:2004 — Sheet No.: 22

Ref:	Design				Output
	Span of beam to centre of supports		L	2.6 m	
	Dead load not including the beam		G1	0.00 kN/m	
	Beam load		G2	0.00 kN/m	
	Live load UDL		Q	0 kN/m	
	Live load Plt		E	15.6 kN	
	Dead load factor		DF	0	
	Live load factor		LF	1	
	Design load per meter width				
		((G1+G2) x DF) + (Q x LF) =	W	0.0 kN/m	
	Cantilever				
	Maximum moment for UDL is		M*	40.4 kNm	
	Maximum shear is		V*	15.6 kN	
	Beam dimensions and materials				
Table 3.1	Masonry grade		Fc'	4 Mpa	
3.4.5	Steel reinforcement yield stress		Fy	300 Mpa	
	Shear steel yield stress		Fyt	300 Mpa	
	Cover to reinforcement		C	90 mm	
	Depth of beam		D	540 mm	
	Width of beam		bw	140 mm	
	Effective depth	H-C-DIA/2 =	d	442 mm	
3.4.7	Strength reduction factor flexural		ϕ	0.85	
	Strength reduction factor Shear		ϕ	0.75	
	Lever arm	$d - \frac{As.Fy}{1.7Fc'b} =$	Jd	379 mm	
	Main bar diameter		DIA	16 mm	
	Number of bars		N	1	
	Area of steel provided		As	201 mm²	PASS
	Minimum & Maximum area of tension steel				
8.3.6.3 (a)	Min. area of tension steel	.7 bw x d / fy =	As min	144 mm²	
8.3.6.3 (b)	Alternatively may be 1/3 greater than what is required by analysis				
8.3.6.4	Maximum area of reinforcement	.75Pb =	As Max	459 mm²	
8.3.6.2	Maximum area of reinforcement in a grouted space				
		8 x tc x hu / Fy =	As Max	280 mm²	
			As Max	280 mm²	
	Moment capacity	ø x AS x Fy x Jd x 10^-6 =	øMn	19.4 kNm	> 40.43 kNm FAILED 48% NBS
	Shear Check				
10.3.2.1	Total nominal shear stress	V* / (bw.d) =	vn	0.251 Mpa	
10.3.2.4	Maximum shear stress vn shall be less than				
		Table 10.1	vg	0.9 Mpa	> 0.251 Mpa PASS

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Structural Concepts

Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	Internal Column		

REINFORCED MASONRY BEAM CONTINUED — Sheet No.: 23

Ref:	Design			Output
10.3.2.6	**Shear stress provided by masonry**		For beams	
	$vm \approx (C1 + C2)vbm$ Where $C1 = 33\rho \frac{fy}{300}$		$C2 = 1.0$	
10.1	Ratio of tension reinfrocement — As / bw.d =	ρ	0.0032	
	Basic shear strength table 10.1	vbm	0.70 Mpa	
		C1	0.107	
	Shear stress provided by masonry — (C1+C2)vbm =	vm	0.775 Mpa	
10.3.2.1	Nominal shear strength of masonry — vn = vm			
	vn x bw x d =	Vn	47.96 kN	
	Shear strength of masonry	ØVn	36.0 kN	> 15.55 kN PASS
	Shear steel not required by calculation (min steel only)			
	Stirrup & tie reinforcement provided			
	Bar dia	dia	12 mm	
	No. legs		1	
	Spacing	S	400 mm	< 270 mm
	Min Area	Av	23.3 mm²	NA
	Area provided		113.1 mm²	> 23.33 mm²
8.3.11	**Minimum stirrup reinforcement**			PASS
8.3.11.4	Shear steel spacing shall be equal to or less than	Smax	270.0 mm	
	Min diameter of shear reinforcement		6 mm	PASS
10.3.2.11	**Shear reinforcement contribution to strength**			
	$vs = C3 \frac{Av.fy}{bw.S}$ Where C3 = 1.0 for beams			
		vs	0.61 Mpa	
	vs + vm =	vn	1.4 Mpa	
10.3.2.11	Nominal shear strength of masonry			
	vn x bw x d =	Vn	85.5 kN	
	Shear strength of masonry	ØVn	64.1 kN	> 15.55 kN
	Where shear reinforcement is required by 10.3.2.11(a)			PASS
	The min area of steel shall be:- $\frac{.15.bw.S}{fy}$	Av	28.0 mm²	< 113.1 mm² PASS
8.3.4	From 8.3.4 (b) we have L x 0.083			
	and the minimum depth is		216 mm	< 540 mm PASS

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Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Internal Foundation beam**		

REINFORCED CONCRETE RECTANGULAR BEAM DESIGNED TO NZS3101:PART 1:2006 — Sheet No.: **24**

Ref:	Design				Output
	Design bending moment from analysis		M*	40 kNm	
	Shear from analysis		V*	13 kN	
	Beam dimensions and materials				
5.2.1	Concrete grade		Fc'	20 Mpa	
5.3.3	Steel reinforcement yield stress		Fy	300 Mpa	
	Shear steel yield stress		Fyt	300 Mpa	
	Cover to reinforcement		C	75 mm	
	Depth of beam		D	600 mm	
	Width of beam		bw	150 mm	
	Effective depth	H-C-DIA/2 =	d	517 mm	
2.3.2.2	Strength reduction factor flexural		ϕ	0.85	
	Strength reduction factor Shear		ϕ	0.75	
	Lever arm	$d - \frac{As.Fy}{1.7Fc'b} =$	Jd	505 mm	
	Main bar diameter		DIA	16 mm	
	Number of bars		N	1	
	Area of steel provided		As	201 mm²	
	Minimum area of tension steel				
9.3.8.2.1	Min. area of tension steel	$\frac{\sqrt{Fc'}}{4Fy} bw.d =$	As min	289 mm²	
	But equal to or greater than	1.4 bw.d/Fy =	As min	362 mm²	
9.3.8.2.3	Alternatively may be 1/3 greater than what is required by analysis				
	Moment capacity	ø x AS x Fy x Jd x 10^-6 =	øMn	25.9 kNm	> 40.43 kNm FAILED **64% NBS**
	Shear Check				
7.5.1	Total nominal shear stress	V* / (bw.d) =	vn	0.174 Mpa	
7.5.2	Maximum shear stress vn shall be less than				
		.2Fc' or 8Mpa		4.0 Mpa	> 0.174 Mpa PASS
9.3.9.3.4	**Shear stress provided by concrete**				
	$Vc = vc.Acv$ Where $vc = kd.ka.vb$				
9.1	Ratio of tension reinfrocement	As / bw.d =	ρ	0.0026	
	vb = smaller of $(.07+10\rho)\sqrt{Fc'}$ OR $.2\sqrt{Fc'}$				
	But not less than .08 x Fc'^.5		vb	0.429 Mpa	
	Aggregate size factor		ka	1.0	
	Effective depth factor		kd	0.94	
		vb x ka x kd =	vc	0.402 Mpa	
	Nominal shear strength provided by concrete				
		vc.Acv =	Vc	31.2 kN	
			Shear steel not required		

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Structural Concepts

Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Internal Foundation beam**		

REINFORCED CONCRETE RECTANGULAR BEAM DESIGNED CONTINUED — Sheet No.: **25**

Ref:	Design				Output
9.3.9.4.15	**Minimum shear steel requirement**				
		Bar dia	dia	6 mm	
		No. legs		1	
	$$Av = \frac{1}{16}\sqrt{f'c}\,\frac{bw.s}{fyt}$$	Spacing	S	200 mm	< 258.5 mm
		Min Area	Av	28.0 mm²	PASS
		Area provided		28.3 mm²	> 27.95 mm²
9.3.9.3.6	**Shear reinforcement required when vn>vc**				PASS
	$$Vs = Av.fyt.\frac{d}{S}$$		Vs	21.9 kN	
7.5.3	Shear strength	(Vc + Vs) x ϕ =	Vn	39.8 kN	> 13.48 kN
					PASS

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Client: **Christchurch City Council**
Project: **Kapuatohe Museum**
665 Main North Road, Christchurch
Subject: **117 EQ Parts 1170.5 diaphragm**

Ref: **1599-0304**
Date: **20/1/13**
BY: **GN**

Sheet No.: **26**

Seismic Loads of parts to NZS 1170.5

Ref:	Design				Output
	Design working live		50 Years		
	Importance level		2		
	Annual Probability of exceedance (inverse) Ultimate		500		
	Soil type				
	D. Deep or soft soil				
	For Parts				
	Height of the upper most seismic mass	hn	2.7	m	
	Height of support for part (from ground level)	hi	2.7	m	
	Floor acceleration is such to causing yielding of part See table C8.2				
	Structural ductility of part (Table C8.2)	μp=	2.00		
T 3.3	Hazard Factor	Z =	0.3		
T 3.5	Return period factor	Ru =	1.00		
T 3.1	Spectral Shape Factor for parts	Ch(0) =	1.12		
T 3.7	Near Fault factor	N(T,D) =	1.0		
	Site Hazard coefficient Ch(0) x Z x R x N(T,D) =	C(0) =	0.34		
T. 8.1	Part risk factor	Rp	1.0		
8.3	Floor height coefficient				
	Eq 8.3(1) $\left(1+\frac{hi}{6}\right)$	Chi	1.450		
	Eq 8.3(2) $\left(1+10\frac{hi}{hn}\right)$	Chi	11.0		
		Chi	1.450		
	Period of part	Tp	0.4	Sec	
8.4	Part spectral shape coefficient	Ci(Tp)	2.0		
8.2	Design response coefficent for part				
	C(0).Chi.Ci(Tp) =	Cp(Tp)	0.97		
8.6	Part horizontal response factor				
		Cph	0.55		
8.5.1	Horizontal design coefficient	Cp(Tp).Cph.Rp =	0.54		

Seismic Loads to NZS 1170.5

Ref:	Design	Output

Weight of the part Wp

Element		Area/length		Load KPa	Total kN
Roof		36.6		0.41	15.18
External Walls		16.5		2.60	42.82
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
	0.0	0.4	36.6	0.00	0.00
				Total kN	58.00 kN

8.5.1 Horizontal design action Cp(Tp).Cph.Rp.Wp = Fph 31.1 kN

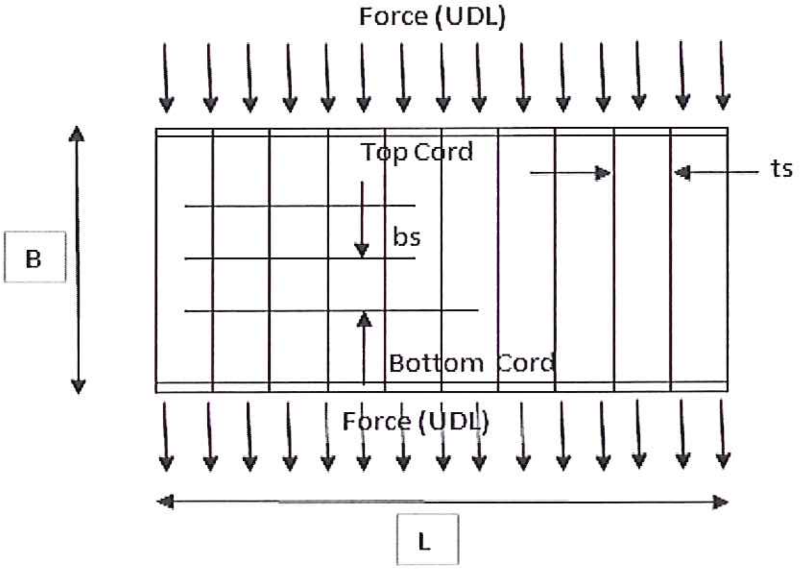
emajineer Structural Concepts

Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Gib Diaphragm Across**		

GIB Ceiling Diaphragm, Designed to report SC 5014 Winstone Wallboards — Sheet No.: **28**

Ref:	Design	Output

For Gib ceilings outside of NZS3604



Bracing Material

Material type		10 Ultraline	
Joint type		Fully back-blocked with cut edge	
Control joint positions		None	
Dimensions			
Diaphragm width	B	6.0	m
Diaphragm length	L	6.1	m
Truss spacing	ts	900	mm
Batten spacing	bs	400	mm
Chord area	A	4900	mm²
Chord MOE	E	8.0	GPa
Bracing Fasteners			
Fastener type		screws	
Spacing around perimeter	ps	150	mm
Spacing along battens	fs	300	mm

Horizontal loads		**Seismic**	**Wind**	
Applied to upper chord	Fu	2.106	0	kN/m
Applied to lower chord	Fl	2.106	0	kN/m
Applied along trusses	Ft	0.2214	0	kN/m
Total load	F	34.6968	0	kN

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Structural Concepts

Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Gib Diaphragm Across**		

GIB Ceiling Diaphragm, Designed to report SC 5014 Winstone Wallboards — Sheet No.: **29**

Ref:	Design			Output
	Strength			
	Ceiling end fastener force			
	Seismic	F / (2 x 1000 x B / ps) =	0.43371 kN	
	Wind	F / (2 x 1000 x B / ps) =	0 kN	
T4	Gib Constants (Table 4)	C	0.800	
T5	Average fastener strength (Table 5)	FS	0.739 kN	
	Fastener strength	C x FS =	0.591 kN	PASS
	Batten to ceiling fastener force			
	Seismic	F / L / B x bs / 1000 x fs / 1000 =	0.114 kN	
	Wind	F / L / B x bs / 1000 x fs / 1000 =	0.000 kN	
	Fastener strength	C x FS =	0.591 kN	PASS
	Chord to ceiling fastener force			
	Seismic	MAX(Fu,Fl) x ps / 1000 =	0.316 kN	
	Wind	MAX(Fu,Fl) x ps / 1000 =	0 kN	
	Fastener strength	.25 x FS x C =	0.148 kN	**FAILED**
	Maximum sheet shear			**47% NBS**
	Seismic	F / (2 x B) =	2.89	
	Wind	F / (2 x B) =	0.00	
T2	Sheet shear strength (table 2)		7.60 kN	PASS
	Maximum sheet joint shear			
	Seismic	V* x (1-2.4/L) =	1.75 kN	
	Wind	V* x (1-2.4/L) =	0.00 kN	
T3	Fully back-blocked with cut edge (Table 3)		4.00 kN	PASS
	Mid Span deflections			
	Chord bending			
	Seismic	5000 / 192 x F x L^3 / EC / A / B^2 = Δ_1	0.145 mm	
	Wind	Δ_1	0.000 mm	
	Boards Modulus of Rigidity	G	1.00	
	Boards thickness	t	9.50 mm	
	Panel shear			
	Seismic	F x L / (8 x G x B x t) = Δ_2	0.464 mm	
	Wind	Δ_2	0.000 mm	
	Ceiling end fastener slip			
T4	Nail slip constant (Table 4)	B	0.179555	
T4		A	0.803	
	Seismic	Fastener Force x B /(A - Fastener Force) = Δ_3	0.211 mm	
	Wind	Δ_3	0.000 mm	
	Control joint slip			
	Seismic	2 x C1 x EfF x B / (A - C1 x EfF) = Δ_4	0.000 mm	
	Wind	2 x C1 x EfF x B / (A - C1 x EfF) = Δ_4	0.000 mm	
	C1 (Joint loaction) = 0.0, .33, .5, 1.0 for None, 1/3 span, 1/4 span, Ends		Total Seismic Δ	0.82 mm
	EfF = End fastener force (top of this page)		Total Wind Δ	0.00 mm

Ref:	Design

GIB ceiling Diaphragm Summary for detailing.

Dimensions

Diaphragm width	B	6.0
Diaphragm length	L	6.1
Truss spacing	ts	900
Batten spacing	bs	400

Bracing Material

Material type	10 Ultraline
Joint type	Fully back-blocked with cut edge
Control joint positions	None
Continuous top plate area	4900 mm²

Top plate is typically 90x45 + 140x45 MSG8, as GIB details

Bracing Fasteners

Fastener type	**screws**
Spacing around perimeter	150 mm
Spacing along battens	300 mm

Use only GIB approved fixings for diaphragms, usually
GIB Grabber screws which have the extra large head
in the case of screw fixings. Nails usually have washers if specified.

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Structural Concepts

Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**EQ Parts 1170.5 bolting**		

Seismic Loads of parts to NZS 1170.5 — Sheet No.: **31**

Ref:	Design				Output
	Design working live		50 Years		
	Importance level		2		
	Annual Probability of exceedance (inverse) Ultimate		500		
	Soil type				
	D. Deep or soft soil				
	For Parts				
	Height of the upper most seismic mass	hn	2.7	m	
	Height of support for part (from ground level)	hi	2.7	m	
	Floor acceleration is such to causing yielding of part See table C8.2				
	Structural ductility of part (Table C8.2)	μp=	1.00		
T 3.3	Hazard Factor	Z =	0.3		
T 3.5	Return period factor	Ru =	1.00		
T 3.1	Spectral Shape Factor for parts	Ch(0) =	1.12		
T 3.7	Near Fault factor	N(T,D) =	1.0		
	Site Hazard coefficient Ch(0) x Z x R x N(T,D) =	C(0) =	0.34		
T. 8.1	Part risk factor	Rp	1.0		
8.3	Floor height coefficient				
	Eq 8.3(1) $\left(1+\frac{hi}{6}\right)$	Chi	1.450		
	Eq 8.3(2) $\left(1+10\frac{hi}{hn}\right)$	Chi	11.0		
		Chi	1.450		
	Period of part	Tp	0.4	Sec	
8.4	Part spectral shape coefficient	Ci(Tp)	2.0		
8.2	Design response coefficent for part				
	C(0).Chi.Ci(Tp) =	Cp(Tp)	0.97		
8.6	Part horizontal response factor				
		Cph	1.00		
8.5.1	Horizontal design coefficient	Cp(Tp).Cph.Rp =	0.97		

Seismic Loads to NZS 1170.5

Ref:	Design	Output

Weight of the part Wp

Element		Area/length		Load KPa	Total kN
External Walls		1.5		2.60	3.90
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
	0.0	0.4	1.5	0.00	0.00
				Total kN	3.90 kN

8.5.1 Horizontal design action Cp(Tp).Cph.Rp.Wp = Fph 3.8 kN

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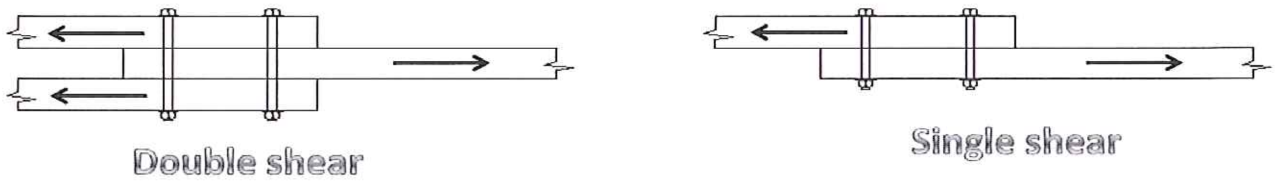
Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	Chord Bolt design		

Design of bolts in shear to NZS3603 — Sheet No.: 33

Ref:	Design	Output
	Bolt design	



Ref	Design				Output
	Ultimate load		3.80	kN	
	Shear type		Single		
	Timber group		J5		
	Effective thickness of timber Parallel		100	mm	
	Effective thickness of timber Perpendicular		100	mm	
	Bolt diameter		12		
	Ultimate bearing stress parallel	fcj	36.1	Mpa	
	Ultimate bearing stress perpendicular	fpj	12.9	Mpa	
	Duration of load	K1	1		
T 4.14	Green timber midification	K12	1.0		
4.4.3.2	Multiple number of fastners	K13	1.0		
	Strength reduction factor	ϕ	0.7		
	Number of fastners	n	1		
	For Parallel to the grain				
		k11 x fcj x da² =	10.40	kN	
		OR			
		.5 x be x fcj x da =	21.66	kN	
	Nominal strength	Qski	10.40	kN	
		Qski	10.40	kN	
	Design strength of bolt group		7.28	kN	PASS
	For Perpendicular to grain				
		k11 x fcj x da^1.5 =	7.99	kN	
		OR			
		.5 x be x fcj x da =	7.74	kN	
	Nominal strength	Qski	7.74	kN	
		Qski	7.74	kN	
	Design strength of bolt group		5.42	kN	PASS
	Edge & End distances				100% NBS

Loaded end distance	96	Unloaded end distance	60
Loaded edge distance	48	Unloaded edge distance	24
Loaded spacing	60		

Seismic Loads of parts to NZS 1170.5

Ref:	Design				Output
	Design working live		50 Years		
	Importance level		2		
	Annual Probability of exceedance (inverse) Ultimate		500		
	Soil type				
	D. Deep or soft soil				
	For Parts				
	Height of the upper most seismic mass	hn	2.7	m	
	Height of support for part (from ground level)	hi	2.7	m	
	Floor acceleration is such to causing yielding of part See table C8.2				
	Structural ductility of part (Table C8.2)	$\mu p=$	2.00		
T 3.3	Hazard Factor	Z =	0.3		
T 3.5	Return period factor	Ru =	1.00		
T 3.1	Spectral Shape Factor for parts	Ch(0) =	1.12		
T 3.7	Near Fault factor	N(T,D) =	1.0		
	Site Hazard coefficient Ch(0) x Z x R x N(T,D) =	C(0) =	0.34		
T. 8.1	Part risk factor	Rp	1.0		
8.3	Floor height coefficient				
	Eq 8.3(1) $\left(1+\frac{hi}{6}\right)$	Chi	1.450		
	Eq 8.3(2) $\left(1+10\frac{hi}{hn}\right)$	Chi	11.0		
		Chi	1.450		
	Period of part	Tp	0.4	Sec	
8.4	Part spectral shape coefficient	Ci(Tp)	2.0		
8.2	Design response coefficent for part				
	C(0).Chi.Ci(Tp) =	Cp(Tp)	0.97		
8.6	Part horizontal response factor				
		Cph	0.55		
8.5.1	Horizontal design coefficient	Cp(Tp).Cph.Rp =	0.54		

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Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	EQ Parts 1170.5 diaphragm along		

Seismic Loads to NZS 1170.5 — Sheet No.: 35

Ref:	Design	Output

Weight of the part Wp

Element		Area/length		Load KPa	Total kN
Roof		36.6		0.41	15.18
External Walls		15.9		2.60	41.24
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
	0.0	0.4	36.6	0.00	0.00
				Total kN	56.42 kN

8.5.1 Horizontal design action Cp(Tp).Cph.Rp.Wp = Fph 30.2 kN

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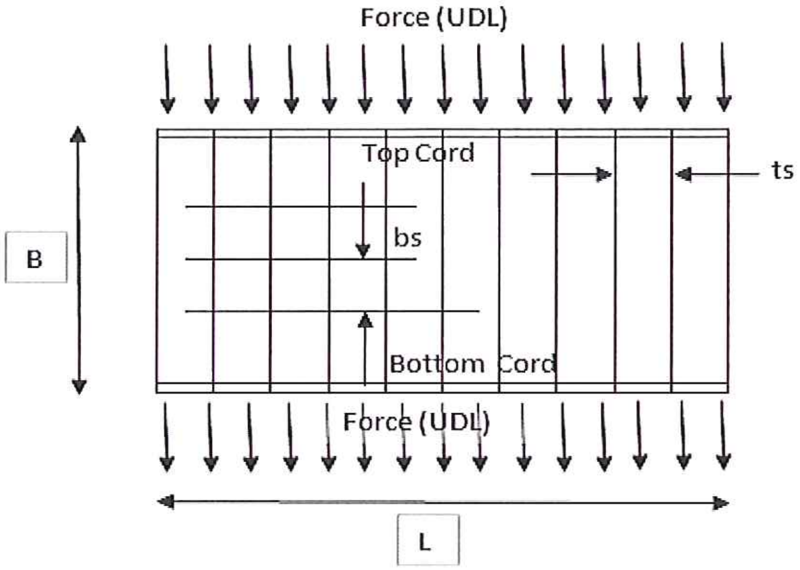
Client:	Christchurch City Council	Ref:	1599-0304
Project:	Kapuatohe Museum	Date:	20/1/13
	665 Main North Road, Christchurch	BY:	GN
Subject:	Gib Diaphragm Along		

GIB Ceiling Diaphragm, Designed to report SC 5014 Winstone Wallboards — Sheet No.: 36

Ref:	Design	Output

For Gib ceilings outside of NZS3604



Bracing Material				
Material type		10 Ultraline		
Joint type		Fully back-blocked with cut edge		
Control joint positions		None		
Dimensions				
Diaphragm width	B	12.0 m		
Diaphragm length	L	6.1 m		
Truss spacing	ts	900 mm		
Batten spacing	bs	400 mm		
Chord area	A	4900 mm²		
Chord MOE	E	8.0 GPa		
Bracing Fasteners				
Fastener type		screws		
Spacing around perimeter	ps	150 mm		
Spacing along battens	fs	300 mm		
Horizontal loads		**Seismic**	**Wind**	
Applied to upper chord	Fu	2.106	0	kN/m
Applied to lower chord	Fl	0.405	0	kN/m
Applied along trusses	Ft	0.2214	0	kN/m
Total load	F	24.3207	0	kN

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Structural Concepts

Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Gib Diaphragm Along**		

GIB Ceiling Diaphragm, Designed to report SC 5014 Winstone Wallboards — Sheet No.: **37**

Ref:	Design				Output
	Strength				
	Ceiling end fastener force				
	Seismic	F / (2 x 1000 x B / ps) =	0.152004	kN	
	Wind	F / (2 x 1000 x B / ps) =	0	kN	
T4	Gib Constants (Table 4)	C	0.800		
T5	Average fastener strength (Table 5)	FS	0.739	kN	
	Fastener strength	C x FS =	0.591	kN	PASS
	Batten to ceiling fastener force				
	Seismic	F / L / B x bs / 1000 x fs / 1000 =	0.040	kN	
	Wind	F / L / B x bs / 1000 x fs / 1000 =	0.000	kN	
	Fastener strength	C x FS =	0.591	kN	PASS
	Chord to ceiling fastener force				
	Seismic	MAX(Fu,Fl) x ps / 1000 =	0.316	kN	
	Wind	MAX(Fu,Fl) x ps / 1000 =	0	kN	
	Fastener strength	.25 x FS x C =	0.148	kN	**FAILED**
	Maximum sheet shear				**47% NBS**
	Seismic	F / (2 x B) =	1.01		
	Wind	F / (2 x B) =	0.00		
T2	Sheet shear strength (table 2)		7.60	kN	PASS
	Maximum sheet joint shear				
	Seismic	V* x (1-2.4/L) =	0.61	kN	
	Wind	V* x (1-2.4/L) =	0.00	kN	
T3	Fully back-blocked with cut edge (Table 3)		4.00	kN	PASS
	Mid Span deflections				
	Chord bending				
	Seismic	5000 / 192 x F x L^3 / EC / A / B^2 = Δ_1	0.025	mm	
	Wind	Δ_1	0.000	mm	
	Boards Modulus of Rigidity	G	1.00		
	Boards thickness	t	9.50	mm	
	Panel shear				
	Seismic	F x L / (8 x G x B x t) = Δ_2	0.325	mm	
	Wind	Δ_2	0.000	mm	
	Ceiling end fastener slip				
T4	Nail slip constant (Table 4)	B	0.179555		
T4		A	0.803		
	Seismic	Fastener Force x B /(A - Fastener Force) = Δ_3	0.042	mm	
	Wind	Δ_3	0.000	mm	
	Control joint slip				
	Seismic	2 x C1 x EfF x B / (A - C1 x EfF) = Δ_4	0.000	mm	
	Wind	2 x C1 x EfF x B / (A - C1 x EfF) = Δ_4	0.000	mm	
	C1 (Joint loaction) = 0.0, .33, .5, 1.0 for None, 1/3 span, 1/4 span, Ends	Total Seismic Δ			0.39 mm
	EfF = End fastener force (top of this page)	Total Wind Δ			0.00 mm

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Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Gib Diaphragm Along**		

GIB Ceiling Diaphragm, Designed to report SC 5014 Winstone Wallboards — Sheet No.: **38**

Ref:	**Design**

GIB ceiling Diaphragm Summary for detailing.

Dimensions

Diaphragm width	B	6.0
Diaphragm length	L	6.1
Truss spacing	ts	900
Batten spacing	bs	400

Bracing Material

Material type	10 Ultraline
Joint type	Fully back-blocked with cut edge
Control joint positions	None
Continuous top plate area	4900 mm²

Top plate is typically 90x45 + 140x45 MSG8, as GIB details

Bracing Fasteners

Fastener type	**screws**
Spacing around perimeter	150 mm
Spacing along battens	300 mm

Use only GIB approved fixings for diaphragms, usually
GIB Grabber screws which have the extra large head
in the case of screw fixings. Nails usually have washers if specified.

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Structural Concepts

Client: **Christchurch City Council**
Project: **Kapuatohe Museum**
665 Main North Road, Christchurch
Subject: **117 EQ Parts 1170.5 Walls**

Ref: **1599-0304**
Date: **20/1/13**
BY: **GN**

Sheet No.: **39**

Seismic Loads of parts to NZS 1170.5

Ref:	Design				Output
	Design working live		50 Years		
	Importance level		2		
	Annual Probability of exceedance (inverse) Ultimate		500		
	Soil type				
	D. Deep or soft soil				
	For Parts				
	Height of the upper most seismic mass	hn	3.0	m	
	Height of support for part (from ground level)	hi	3.0	m	
	Floor acceleration is such to causing yielding of part See table C8.2				
	Structural ductility of part (Table C8.2)	μp=	2.00		
T 3.3	Hazard Factor	Z =	0.3		
T 3.5	Return period factor	Ru =	1.00		
T 3.1	Spectral Shape Factor for parts	Ch(0) =	1.12		
T 3.7	Near Fault factor	N(T,D) =	1.0		
	Site Hazard coefficient Ch(0) x Z x R x N(T,D) =	C(0) =	0.34		
T. 8.1	Part risk factor	Rp	1.0		
8.3	Floor height coefficient				
	Eq 8.3(1) $\left(1+\frac{hi}{6}\right)$	Chi	1.500		
	Eq 8.3(2) $\left(1+10\frac{hi}{hn}\right)$	Chi	11.0		
		Chi	1.500		
	Period of part	Tp	0.4	Sec	
8.4	Part spectral shape coefficient	Ci(Tp)	2.0		
8.2	Design response coefficent for part C(0).Chi.Ci(Tp) =	Cp(Tp)	1.01		
8.6	Part horizontal response factor				
		Cph	0.55		
8.5.1	Horizontal design coefficient	Cp(Tp).Cph.Rp =	0.55		

Seismic Loads to NZS 1170.5

Ref:	Design	Output

Weight of the part Wp

Element		Area/length		Load KPa	Total kN
External Walls		2.6		2.60	6.76
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
		0.0		0.00	0.00
	0.0	0.4	2.6	0.00	0.00
				Total kN	6.76 kN

8.5.1 Horizontal design action Cp(Tp).Cph.Rp.Wp = Fph 3.7 kN

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Structural Concepts

Client:	**Christchurch City Council**	Ref:	**1599-0304**
Project:	**Kapuatohe Museum**	Date:	**20/1/13**
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Masonry wall**		

Stack bond wall design to NZS4230:2004 — Sheet No.: **41**

Ref:	Design			Output
	Loads			
	Axial load	N*	3.38 kN	
	Face moment	M*	1.25 kNm/m	
	Wall dimensions			
	Height		3.0 m	
	Length		1.0 m	
	Thickness		140 mm	
7.3.3	Wall slenderness		21.4 <20	NA
	Observation type		B	
	Concrete grade	Fc	12 Mpa	
	Steel reinforcement yield stress	Fy	300 Mpa	
	Cover to reinforcement	C	64 mm	
	Strength reduction factor	Ø	0.85	
	Effective depth	H-C-DIA/2 = d	70 mm	
	Flexure (Ignoring axial load) 10.2			
10.2.2.6	Lever arm (based on stress/strain relationship of 10.2.2.6)			
	$d - \frac{As \times Fy}{1.7 \times Fc \times 10}$	= Jd	70 mm	
	Main bar diameter	DIA	12 mm	
	Bar spacing	MC	600 mm	
	Area of each bar	AsB	113 mm²	
	Area at laps if the same size bar is used	AsL	226 mm²	
	Area of Vert steel per design width of wall	As	188 mm²/m	
	Flue area		12000 mm²	
7.3.4.3	Min. area of steel required			
	Min. area of vertical steel	0.07% x B x D = ASMIN	98 mm²/m	PASS
	Maximum area of steel to each flue	8/Fy x flue area	32000 mm²	PASS
	Max area of steel at laps to each flue	13/Fy x flue area	52000 mm²	PASS
	Distribution bar diameter	DIA'	12 mm	
	Bar spacing	MD	2400 mm	
	Area of Horiz steel per design width of wall	AsDIS	47 mm²/m	
	Min. area of Horizontal steel	0.07% x B x D =	98 mm²/m	NA
	Min. area of Horiz & Vert	0.2% x B x D =	280 mm²/m	NA
	Moment capacity of wall			
		ø x AS x Fy x Jd x 10^-6 = øMn	3.36 kNm/m	PASS
	Axial load 7.3.4.8			
	Nominal axial load strength			
	$.5.Fm.Ag\left(1-(Ln/40b)^2\right)$	Nnw	835.3 kN	
		øNnw	710.0 kN	PASS
				100% NBS

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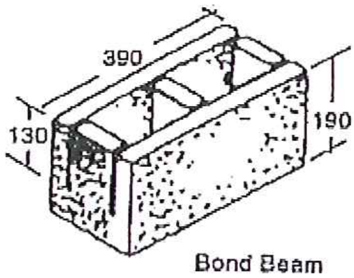
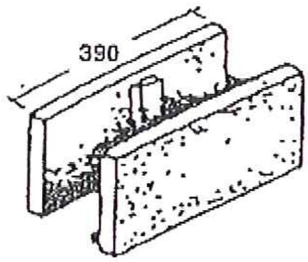
	665 Main North Road, Christchurch	BY:	**GN**
Subject:	**Masonry wall**		

Sheet No.: **42**

Ref:	**Design**	**Output**



Bond Beam



Open End Bond Beam

In stack bonded walls either use bond beam blocks and knock out all biscuits or use open ended bond beam blocks as shown above.

Use:-

15 Series blocks with 20Mpa concrete fill, reinforced with D12 at 600 Centres Vert. And D12 at 2400 Centres Horiz.

APPENDIX F

KAPUATOHE MUSEUM
CHRISTCHURCH

NEW WORKS AND EXAMPLES

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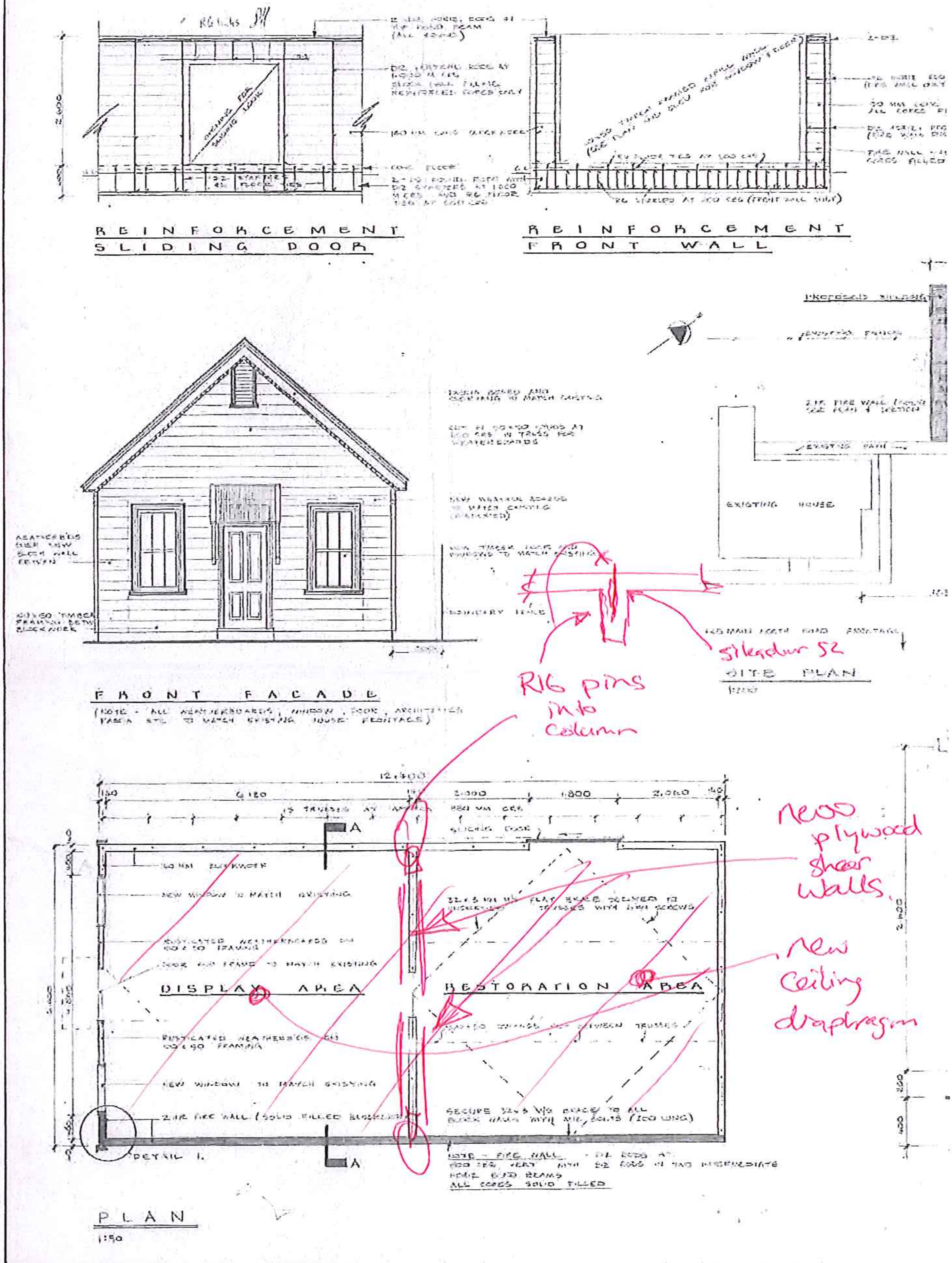
E info@structuralconcepts.co.nz



WAIMAIRI DISTRICT COUNCIL

DRAWING INFORMATION			
SURVEYED •	DATUM •		SCALE
DRAWN •	BENCH MK. •		1:10
TRACED •	LEVEL BK. • NO.	P.	1:20
CHECKED •	FIELD BK. • NO.	P.	1:50
DESIGNED •			1:200
DES. CHK. •			
APPROVED •			

JOB TITLE

PROPO
KAPUT
665 M