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Owen Mitchell Reserve Toilet
PRK 0086 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

100 Grimseys Road, Redwood

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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100 Grimseys Road, Redwood

Christchurch City Council

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14 March 2013

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Quantitative Report Summary

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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Background

This is a summary of the Quantitative report for the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 2 November 2012 and available drawings itemised in Section 5.2.

Building Description

The building is located at 100 Grimseys Road, Redwood. The date of construction is estimated to be during the early 1990s based on the construction date observed for other similar structures. The sole use of the building is a public toilet.

The structure of the building consists of reinforced concrete masonry walls supporting a lightweight timber framed roof. The building is approximately 4.0m in length by 2.5m in width with a height of 3.6m. The building occupies a footprint of approximately 10m^2 and is over 50m from the nearest structure. The site is approximately 1.7km south of the Heathcote River.

Key Damage Observed

The building was observed to be in good condition during inspections. No damage to the structure was observed.

Building Capacity Assessment

The building has been assessed to have a seismic capacity in the order of 87% NBS and is therefore not Earthquake Prone or Earthquake Risk.

Recommendations

No further action is required by Christchurch City Council to comply with the Building Code.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the public toilet in Owen Mitchell Reserve.

This report is a Quantitative Assessment of the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A quantitative assessment involves a full site measure of the building which is used to determine the buildings bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined and the percentage of New Building Standard (%NBS) is assessed.

At the time of this report, no intrusive site investigation or modelling of the building structure had been carried out.

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 3.1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Figure 3.1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Figure 3.2 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

Figure 3.2 %NBS compared to relative risk of failure

4. Building Descriptions

4.1 General

The building is located at 100 Grimseys Road, Redwood. The date of construction is estimated to be during the early 1990s based on the construction date observed for other similar structures. The sole use of the building is a public toilet.

The building is approximately 4.0m in length by 2.5m in width with a height of 3.6m. The building occupies a footprint of approximately 10m^2 and is over 50m from the nearest structure. The site is approximately 1.7km south of the Heathcote River.

The structure of the building consists of reinforced concrete masonry walls supporting a lightweight timber framed roof. The walls consist of 140mm thick fully filled concrete masonry units reinforced with 12mm diameter bars placed centrally at 600mm centres both horizontally and vertically. The concrete masonry walls are clad internally with ceramic tiles. A bond beam reinforced with 1 No. 12mm diameter bar runs along the length of the masonry walls at eaves level.

A 290mm thick timber partition wall along the centreline of the buildings in the transverse direction separates the male and female toilets. The wall has timber studs at 400mm centres. The timber wall supports gravity loads from the roof structure. The wall is clad with ceramic tiles.

The roof structure consists of a 45 degree duo-pitch roof formed by corrugated sheet metal on timber sarking supported by timber purlins and rafters. A timber ridge beam running the length of the structure is supported at each end and at midpoint by timber posts. The timber posts are supported by the transverse concrete masonry end walls and the transverse central timber wall. The central post supporting the timber ridge beam is shown in Photograph 6.

The foundations of the buildings consist of a concrete slab-on-grade reinforced with centrally placed 665 Mesh and concrete strip footings beneath the external walls reinforced with 2 No. 12mm diameter bars with 10mm diameter stirrups at 300mm centres. The reinforced concrete masonry walls are tied to the foundations by starter bars lapped 450mm on to the vertical wall reinforcement.

Figures 4.1 and 4.2 show the construction details.

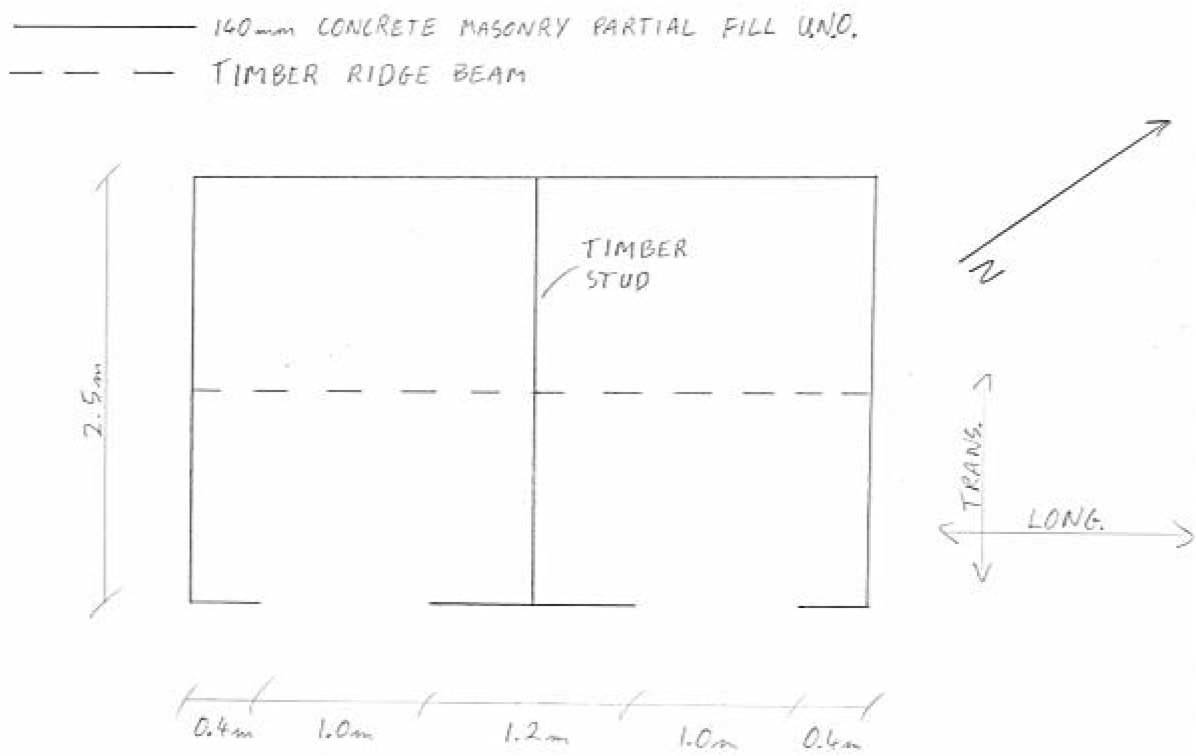

Figure 4.1 Typical Plan of Blocks A & B

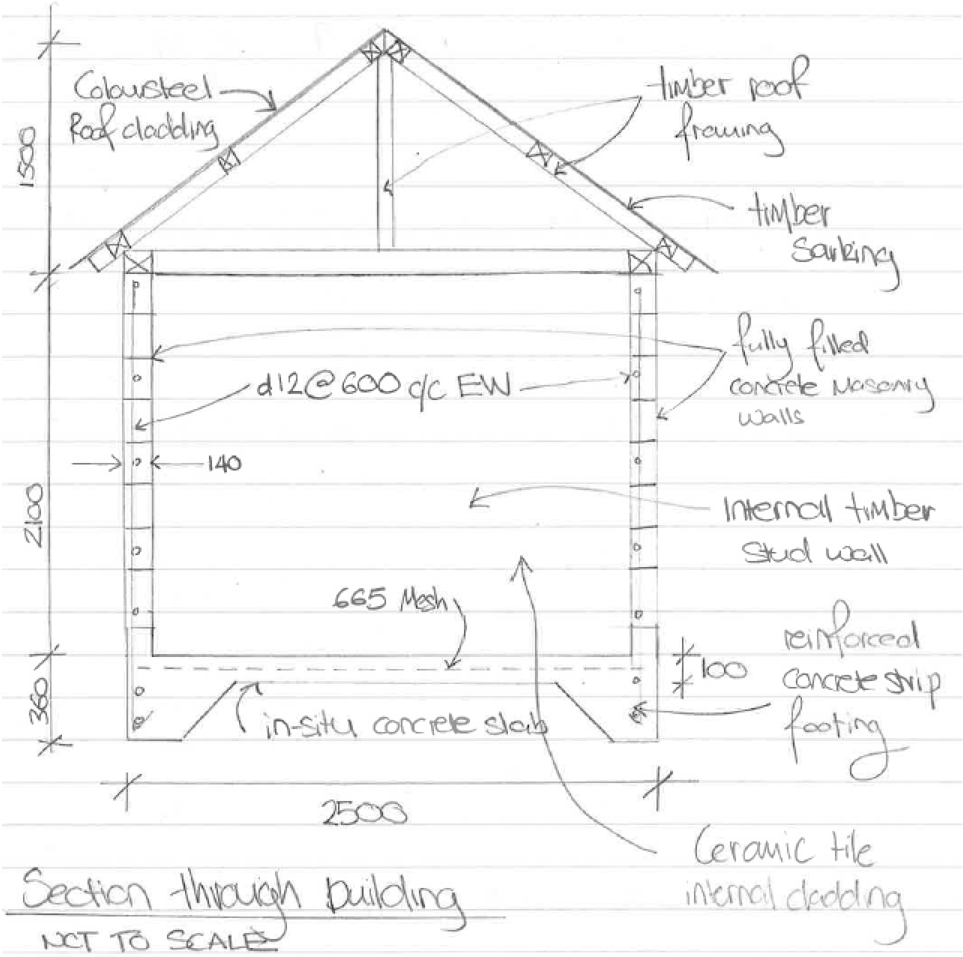

Figure 4.2 Typical Section of a Housing Unit

4.2 Gravity Load Resisting Systems

Gravity loads acting on the building are resisted by load bearing concrete masonry walls. Gravity loads from the corrugated steel roof are transferred via the timber rafters and the propped timber ridge beam to the concrete masonry walls. The gravity loads are transferred through the concrete masonry walls to the concrete strip footings where they are distributed into the ground. Floor gravity loads are transferred through the reinforced concrete slab to the underlying ground.

4.3 Lateral Load Resisting Systems

The timber sarking over the timber roof framing provides a diaphragm to transfer seismic forces in the roof structure to the lateral load resisting walls supporting the diaphragm. Lateral seismic loads in both the transverse and longitudinal direction are resisted by the perimeter reinforced concrete masonry walls. The lateral forces are resisted by the panel action of concrete masonry units. Loads are transferred to the foundations through shear and bending of the concrete masonry walls.

The 140mm thick concrete masonry walls are restrained at eaves level by the timber framed roof structure. Concrete masonry walls perpendicular to the direction of loading transfer the lateral seismic forces via diaphragm action of the roof to the concrete masonry walls in the plane of loading.

Due to the relatively stiff nature of the concrete masonry end walls compared to the central timber framed wall in the transverse direction, it is likely that the majority of lateral seismic loads will be taken by the concrete masonry walls. As a result, the contribution of the timber framed wall in the transverse direction to the overall lateral load resisting capacity is negligible.

5. Assessment

5.1 Site Inspection

An inspection of the buildings was undertaken on the 12th of December 2012. Both the interior and exterior of the building was inspected. It should be noted that inspection of the foundations of the structure was limited to the top of the external strips exposed above ground level.

The inspection consisted of observing the building to determine the structural systems and likely behaviours of the building during earthquake. The site was assessed for damage, including observing the ground condition, checking for damage areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

5.2 Available Drawings

The construction drawings of the original structure were not available. Drawings of a similar building constructed in Abberley Park have been used as the basis for assumptions regarding the structure.

The building at Abberley Park has identical dimensions and layout. The Abberley Park toilet has steel framed trusses above the concrete masonry transverse end walls and the central transverse timber partition wall which support the roof structure. The building described in this report has timber framing in place of the steel trusses.

Sketches of the key structural features of the building are attached as Appendix B.

5.3 Damage Assessment

5.3.1 Surrounding Buildings

There are no buildings in the immediate vicinity of the toilet. The distance to the nearest structure is approximately 50 m.

5.3.2 General Observations

The building was observed to be in good condition during inspections. No damage to the structure was observed.

5.3.3 Ground Damage

No evidence of ground damage was observed in Owen Mitchell Reserve during inspections.

6. Geotechnical Consideration

6.1 Site Description

The site is situated within a recreational reserve, within the suburb of Redwood in northern Christchurch. It is relatively flat at approximately 10m above mean sea level. It is approximately 1.7km south of the Heathcote River, and 8km west of the coast (Pegasus Bay).

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by Holocene alluvial soils of the Brown & Weeber, 1992 describes the site geology as:

- Dominantly alluvial sand and silt overbank deposits, being alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, Holocene in age;
- Underlying sediments (younger than 6500 years) surface or near surface peat underlain by alluvial silt and sand;
- The Riccarton gravels are located approximately 20 m below ground level (bgl); and
- Groundwater is likely 2 m below ground level.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that three boreholes are located within 200m of the site (see Table 6.1), all of which contain lithographic logs, however none of these borehole logs extend beyond 4m below ground level (bgl). These logs indicate that the subsoil conditions to 4m bgl comprise clay/silt/sand with occasional peat lenses to the southwest of the subject structure.

Boreholes further from site indicate the fine to medium grained material continues beyond 4m bgl.

Table 6.1 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site	Log Summary
M35/13314	3.96m		160m SW	0.0 – 0.5 Topsoil 0.5 – 2.7 Clayey SILT with PEAT lenses 2.7 – 4.0 Clayey SILT
M35/13315	3.66m	-	70m SW	0.0 – 0.3 Topsoil 0.3 – 0.9 SAND

[1] Brown, L. J. and Weeber, J.H. 1992: Geology of the Christchurch Urban Area. Institute of Geological and Nuclear Sciences 1:25,000 Geological Map 1. Lower Hutt. Institute of Geological and Nuclear Sciences Limited.

Bore Name	Log Depth	Groundwater	Distance & Direction from Site	Log Summary
				0.9 – 1.4 Clay 1.4 – 3.7 Silty CLAY
M35/14887	2.1m	-	140m E	0.0 – 0.1 GRAVEL 0.1 – 1.1 SAND 1.1 – 1.9 Sandy SILT

It should be noted that the logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the subject site.

6.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land in expected to perform in future earthquakes.

The site has been categorised as “N/A” – Urban Non-residential”. However, neighbouring residential properties have been categorised as TC2 (yellow), indicating minor to moderate land damage from liquefaction is possible in future significant earthquakes.

6.2.5 Post February Aerial Photography

Aerial photography taken following the 4 September 2011 and 22 February 2011 (Figure 6.1) earthquake shows no obvious signs of liquefaction outside the building footprint or adjacent to the site. However, minor liquefaction was evident in the surrounding area.

Figure 6.1 Post February 2011 Earthquake Aerial Photography [2]

6.2.6 Summary of Ground Conditions

From the information presented above, ground conditions are indicated to comprise predominantly silt, sand and clay, with some lenses of peat.

6.3 Seismicity

6.3.1 Nearby Faults

There are many faults in the Canterbury region, however only those considered most likely to have an adverse effect on the site are detailed below.

[2] Aerial Photography Supplied by Koordinates sourced from http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

Table 6.2 Summary of Known Active Faults[3,4]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	25 km	SW	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	~150 years
Porters Pass Fault	60 km	NW	7.0	~1100 years
Port Hills Fault (2011)	12 km	S	6.3	Not Estimated

The recent earthquake sequence since 4 September 2010 has identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; this includes the Greendale Fault and Port Hills Fault listed in Table 6.2. Research and published information on this system is in development and the average recurrence interval is yet to be established for the Port Hills Fault.

6.3.2 Ground Shaking Hazard

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

Since September 2010, seismic activity has produced earthquakes of Magnitude 6.3 with significant peak ground accelerations (PGA) across large parts of the city.

Conditional PGA's from the CGD indicate the PGA to be 0.2g during the 4 September 2010 earthquake, 0.24g on 22 February 2011, and 0.14g on 13 June 2011.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Redwood, a flat suburb in northern Christchurch, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

6.5 Liquefaction Potential

The site is considered to be low to moderate susceptibility to liquefaction, due to the following reasons:

- No signs of liquefaction in post-earthquake aerial photography;
- Neighbouring residential properties have been classified as TC2; and,

[3] Stirling, M.W, McVerry, G.H, and Berryman K.R. (2002) A New Seismic Hazard Model for New Zealand, Bulletin of the Seismological Society of America, Vol. 92 No. 5, pp 1878-1903, June 2002.

[4] GNS Active Faults Database

- Presence of clay and peat layers is likely to reduce the development of liquefaction.

6.6 Summary & Recommendations

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010.

The site appears to be situated on predominantly silt, sand and clay, with some lenses of peat.

A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for the site.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted.

7. Structural Analysis

7.1 Seismic Parameters

Seismic loading on the structure has been determined using New Zealand Standard 1170.5:2004.

◗ Site Classification	D
◗ Seismic Zone factor (Z)	
(Table 3.3, NZS 1170.5:2004 and NZBC Clause B1 Structure)	0.30 (Christchurch)
◗ Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/500 (ULS) Importance Level 2
◗ Return Period Factor (R_u)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)
Longitudinal Direction	
◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (S_p)	0.925
Transverse Direction	
◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (S_p)	0.925

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations.

The structural performance factor, S_P, was calculated in accordance with CL 4.4.2 NZS 1170.5.

$$S_P = 1.3 - 0.3\mu$$

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4 was assumed for both direction of the building. The coefficient was then calculated using Equation 5.2(1);

$$C_d(T_1) = \frac{C(T_1)S_P}{k_\mu}$$

Where

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1$$

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. The lateral seismic forces have been distributed to the concrete masonry walls assuming that the roof structure behaves as a rigid diaphragm and that the stiffness of each wall is directly proportional to the length. An accidental eccentricity of 10% has been assumed in each direction. The structure is considered to be nominally ductile. As a result, 30% loading from the other orthogonal direction has been included when determining the loading on the masonry walls for an earthquake in a particular direction as per NZS 1170.5:2004 requirements.

A ductility factor of 1.25 has been assumed in both the longitudinal and transverse direction based on reinforced concrete masonry walls that resist lateral seismic loading. The structure is expected to have nominally ductile behavior given the relatively lightly reinforced concrete masonry construction.

The elastic site hazard spectrum for horizontal loading:

$C(T_1)=C_h \cdot Z \cdot R \cdot N(T,D)$

$C_h = 3.0$ – Value from Table 3.1 ($T \leq 0.4s$)

$Z = 0.3$ – Hazard factor determined from Table 3.3 (NZS 1170.5:2004)

$R = 1.0$ – Return period factor determined from Table 3.5 (NZS 1170.5:2004)

$N(T,D) = 1.0$ – Near fault factor from Clause 3.1.6 (NZS 1170.5:2004)

$C(T_1) = 3.0 \cdot 0.3 \cdot 1.0 \cdot 1.0 = 0.9$

The horizontal design action coefficient:

$$C_d(T_1)=\frac{C(T_1)\cdot S_p}{k_\mu}=\frac{0.9\times 0.925}{1.14}=0.73$$

7.3 Capacity of Structural Elements

7.3.1 Reinforced Masonry Shear Capacity

The shear capacity of the reinforced filled masonry wall was determined using NZS 4230:2004. As there are no details as to the level of supervision during the construction stage, an Observation Type of B was used in accordance with Table 3.1. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.85 in accordance with NZSEE guidelines. The overall shear capacity of the wall was calculated from Cl 10.3.2.1, Equation 10-4.

For reinforced concrete masonry;

$$V_m = 0.8 d b_w v_m$$

$$v_m = (C_1 + C_2) v_{bm}$$

$$C_2 = 33 p_w \frac{f_y}{300}$$

$$p_w = A_s/b_w d$$

Where

C_1 = wall proportion factor;

v_m = shear strength of masonry;

b_w = t wall thickness when fully filled;

d = 0.8 x length of wall,

A_s = area of reinforcement.

The shear capacity component from the reinforcing steel, V_S, was calculated using equation below;

$$V_S = A_V \, f_{yt} \frac{d}{s}$$

Where

A_V = area of transverse (horizontal) reinforcing at spacing s;

f_{yt} = characteristic yield strength of the transverse steel;

7.3.2 Reinforced Masonry Out-of-Plane Moment Capacity

The following method was used to calculate the out of plane moment capacity of the reinforced masonry walls.

$$\emptyset M_n = \emptyset \left(\frac{t}{2} - \frac{a}{2}\right)\left(f_{yt} A_s\right)$$

$$a = {}^{A_s f_{yt}}\!\big/_{0.85 f'_m b}$$

Where

t = thickness of the masonry wall

b = unit width of wall

A_s = area of steel reinforcement

A_m = area of masonry

f'_m = specified compressive strength of masonry

f_y = the strength of steel as specified by the NZSEE guidelines

1.1.1 %NBS

The in-plane shear capacity, the in-plane bending moment capacity and the out-of-plane bending moment capacity of the concrete masonry walls were compared to their respective demands to determine the overall %NBS for each building.

$$\%\mathrm{NBS} = \frac{\mathrm{V_n}}{\mathrm{V^*}} \text{ x } 100$$

$$\%\mathrm{NBS} = \frac{\mathrm{M_n}}{\mathrm{M^*}} \text{ x } 100$$

8. Results

The New Zealand Society for Earthquake Engineering (NZSEE) publication “Assessment & Improvement of Structural Performance of Buildings” (2006) and the relevant New Zealand material standards were used to provide a framework and method for the analysis. Our analysis applied live loads, imposed dead loads and seismic loads to the structure. The elements were then assessed against their respective load capacities.

Our calculations show that the structure achieves **87% NBS** and is therefore **not Earthquake Prone.**

The structural analysis results are discussed in the following sections.

8.1.1 Reinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

The reinforced concrete masonry walls achieve **100% NBS** under in-plane shear seismic loading.

The reinforced concrete masonry walls are significantly stiffer than the timber framed wall in the transverse direction. As a result, the concrete masonry walls are likely to resist the majority of lateral seismic loads in the transverse direction. The contribution of the timber framed wall to the lateral load resisting capacity has been ignored in the calculations. The layout of the reinforced concrete masonry walls in the transverse direction is regular.

In the longitudinal direction, there is a stiffness eccentricity due to the door openings along the south-eastern side of the building. As a result, the wall running the full length of the building on the north-west side of the building resists the majority of the lateral load for an earthquake occurring in the longitudinal direction.

The layout of the walls is shown below in Figure 8.1.

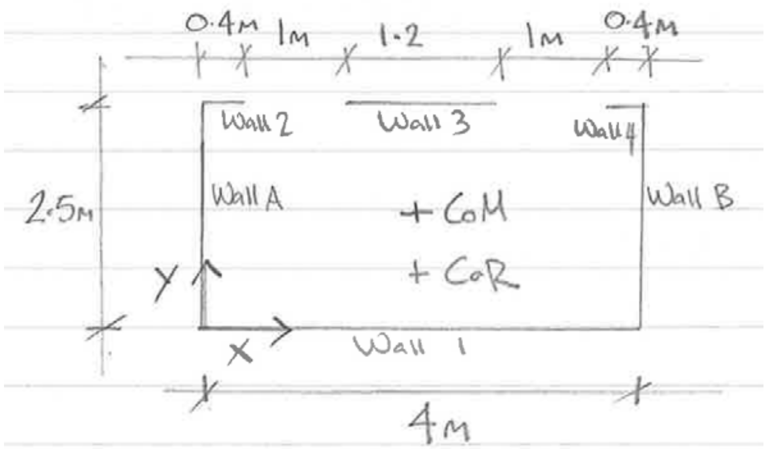

Figure 8.1 Concrete masonry wall layout and labels

The in-plane shear demand and capacity for each wall is summarised in Table 8.1.

Wall	V* (kN)	ΦV_n (kN)	%NBS
1	21	651	100
2	3	53	100
3	8	162	100
4	3	53	100
A	19	352	100
B	19	352	100

Table 8.1 In-plane shear %NBS summary

In-Plane Moment

The reinforced concrete masonry walls achieve **87% NBS** when considering in-plane bending of the walls.

The in-plane bending moment demand and capacity for each wall is summarised in Table 8.2.

Wall	M* (kNm)	ΦM_n (kNm)	%NBS
1	45	99	100
2	5.4	4.7	87
3	16	30	100
4	5.4	4.7	87
A	42	62	100
B	42	62	100

Table 8.2 In-plane bending moment %NBS summary

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering out-of-plane bending of the walls.

The 140mm thick reinforced concrete masonry walls are restrained out-of-plane at eaves level by the timber framed roof diaphragm at a height of 2.1m. The walls were assumed to have pinned connections at the top and bottom of the wall. Out-of-plane bending moment demands and capacities were evaluated per metre width of wall for walls 1, 3, A and B. For walls 2 and 4 (0.4m long) the overall capacity in kNm was determined.

The out-of-plane bending moment demand and capacity for each wall is summarised in Table 8.3.

Wall	M* (kNm)	ΦM_n (kNm)	%NBS
1	1.2	4.2	100
3	1.2	4.2	100
A	1.2	4.2	100
B	1.2	4.2	100
Wall	**M* (kNm)**	**ΦM_n (kNm)**	**%NBS**
2	2	2	100
4	2	2	100

Table 8.3 Out-of-plane bending moment %NBS summary

8.2 Summary

Element	Seismic Action	%NBS
Longitudinal Direction		
Concrete Masonry Walls	In-Plane Shear	100
	In-Plane Bending	87
	Out-of-Plane Bending	100
Transverse Direction		
Concrete Masonry Walls	In-Plane Shear	100
	In-Plane Bending	100
	Out-of-Plane Bending	100

Table 8.4 Summary of %NBS scores

8.3 Discussion of Results

The results obtained from the analysis are generally consistent with those expected for a building of this age and construction type.

The building is assumed to have been designed in the early 1990s and was likely designed in accordance with the previous loading standard, NZS 4203:1984. The design loads used are likely to have been less than those required by the current loading standard.

The building performs well in both the transverse and longitudinal directions with the concrete masonry walls achieving 87% NBS. The stiffness eccentricity is accommodated by the relatively high in-plane shear and bending moment capacities of the walls in the longitudinal direction.

The in-plane bending capacity of the 0.4m walls in the longitudinal direction has been assessed at 87% NBS and controls the assessed score of the overall building. These walls have a lower in-plane %NBS than the other walls as they are be reinforced with a centrally placed single vertical 12mm diameter bar. This results in a lower in-plane bending capacity than the other walls.

9. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of 87% NBS and is therefore not Earthquake Prone or Earthquake Risk. No further action is required by Christchurch City Council to comply with the Building Code.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No material testing has been undertaken.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this report.

10.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Ministry of Education and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

Appendix A
Photographs

Photograph 1 **View of building from the south**

Photograph 2 **Rear concrete masonry wall**

Photograph 3 View of duo-pitched timber roof structure

Photograph 4 View of timber sarking in roof structure

Photograph 5 Connection between timber roof framing and concrete masonry walls

Photograph 6 Central timber post supporting timber ridge beam

Appendix B

Sketches

Appendix C
CERA Form

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