Waltham Park Pavilion
PRK 1044 BLDG 005 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

30 Waltham Road, Waltham

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

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Quantitative Report
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30 Waltham Road, Waltham

Christchurch City Council

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

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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 and visual inspections on 12 December 2012.

Building Description

The building is located at 30 Waltham Road, Waltham. The date of construction is estimated to be during the early 1970s based on construction characteristics of the building. The building is currently used as changing rooms and for storage of sports equipment.

The building is approximately 13.1m in length by 5.3m in width with a height of 3.7m. The building occupies a footprint of approximately 70m^2. The site is approximately 100m north-west of the Heathcote River.

Key Damage Observed

Cracking in the mortar between the concrete masonry units was observed in the north-western end of the original building.

Cracking and lifting of sections of the concrete floor slab was observed near north-western wall of the original building structure.

Building Capacity Assessment

The building has been assessed to have a seismic capacity in the order of 9% NBS and is therefore Earthquake Prone.

Recommendations

It is recommended that Christchurch City Council proceed with developing potential strengthening concepts for the building.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the pavilion in Waltham Park.

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 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

Table 1 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 30 Waltham Road, Waltham. The date of construction is estimated to be during the early 1970s based on construction characteristics of the building. The building is currently used as changing rooms and for storage of sports equipment.

The building is approximately 13.1m in length by 5.3m in width with a height of 3.7m. The building occupies a footprint of approximately 70m^2. The site is approximately 100m north-west of the Heathcote River.

The structure of the building consists of unreinforced concrete masonry walls supporting a lightweight timber framed roof. The walls consist of unreinforced 190mm thick partially filled concrete masonry units.

The roof structure consists of an approximately 10 degree pitch hip roof formed by corrugated sheet metal on timber sarking supported by timber purlins and rafters. The timber rafters are supported by the concrete masonry walls. The foundations of the building consist of a concrete slab-on-grade and concrete strip footings beneath the external walls.

An extension to the north-western end of the original building is likely to have been constructed after the completion of the original structure. The concrete masonry walls in this section (Walls 1 and 14 in Figure 4.1 and Appendix B) of the building are reinforced with 12mm vertical bars at 1200mm centres.

Figures 4.1 and 4.2 show the construction details of the building.

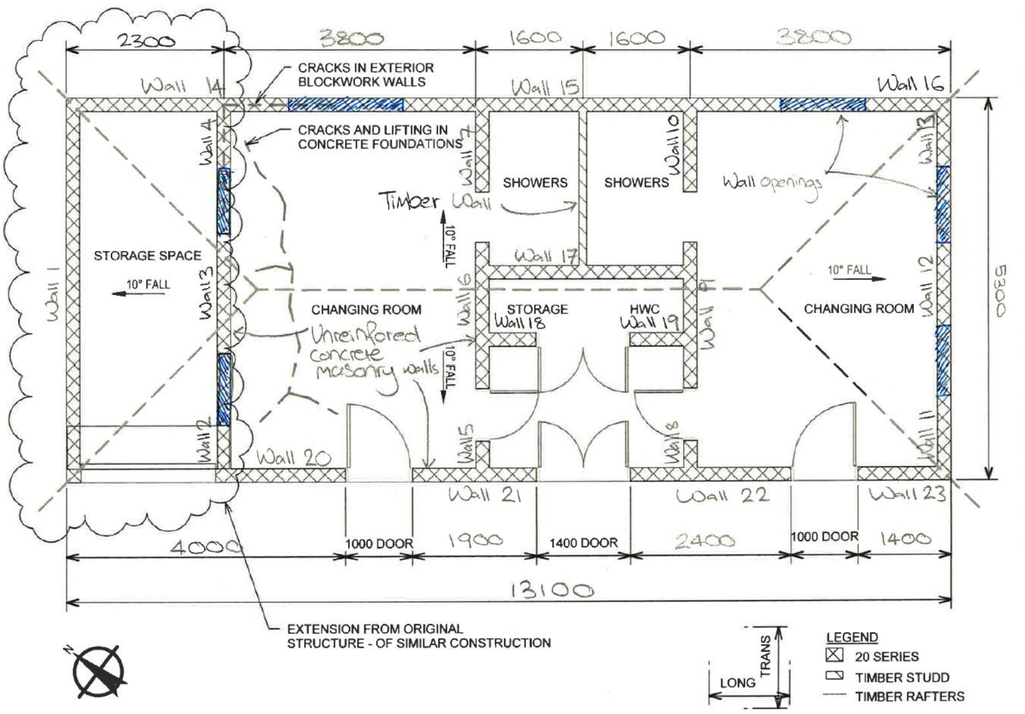


Figure 4.1 Floor plan of the building

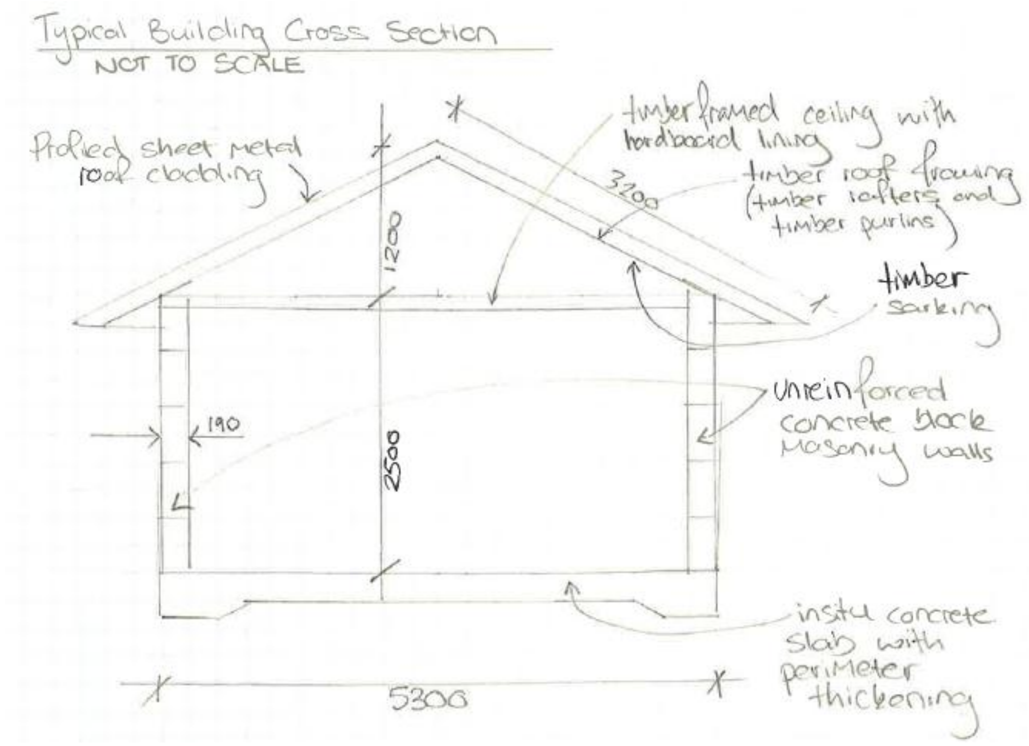


Figure 4.2 Sketch showing section through the building

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 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 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 unreinforced 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 190mm 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 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.

A Hilti PS 200 Ferroscan was used to confirm the position, depth and diameter of the reinforcement in the concrete masonry walls. The scans showed no reinforcement is present in the original section of the structure. 12mm diameter vertical bars centrally placed were shown on the scans of the concrete masonry walls in the extension to the building. The results of the reinforcement scanning were used as part of the element capacity calculations.

5.2 Available Drawings

The construction drawings of the original structure were not available.

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

5.3 Damage Assessment

5.3.1 Surrounding Buildings

No damage to surrounding buildings was observed during inspections.

5.3.2 General Observations

Cracking in the mortar between the concrete masonry units was observed in the north-western end of the original building. The observed damage is shown in Photographs 7 and 8. The location of the cracking observed in Walls 3 and 4 is shown in Figure 4.1 and Appendix B. The calculated load resisting capacity of Walls 3 and 4 has been reduced by 25% to account for the damage observed. These reduced wall capacities were not found to be the critical %NBS for the structure.

Cracking and lifting of sections of the concrete floor slab was observed near north-western wall of the original building structure.

5.3.3 Ground Damage

No evidence of ground damage was observed in Waltham Park during inspections.

6. Geotechnical Consideration

6.1 Site Description

The site is situated in the suburb of Waltham, in south east Christchurch. It is relatively flat at approximately 6 m above mean sea level. It is approximately, 100 m northwest of the Heathcote River, 4.5 km west of the estuary and 8.5 km 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 Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits. The map also indicates that the site is situated on an old stream bed (Jacksons Creek).

Shallow ground water is anticipated due to the low lying location of the site.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that seven boreholes are located within a 130 m radius of the site with four greater than 2 m depth (Table 6.1). Of these boreholes, one has an adequate lithographic log (borehole M36/1194).

The site geology described in the borehole M36/1194 log indicates the area is predominantly layers of sand and clay to a depth of ~23 m bgl. Varying amounts of gravel and silt are also indicated to be present.

Table 6.1 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M36/1194	~34.1 m	~1.3 m bgl	~95 m E
M36/9705	~3.5 m	N/A	~92 m S
M36/9334	~3.71 m	N/A	~110 m SE
M36/9335	~2.44 m	N/A	~124 m S

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. 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.

[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.

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in Tonkin and Taylor Report[2]. Two investigation points were undertaken within close proximity of the site, the results of which are summarised below in Table 6.2.

Table 6.2 EQC Geotechnical Investigation Summary Table

Bore Name	Grid Reference	Depth (m bgl)	Log Summary
CPT-WTM-21	2481726 mE 5739636 mN	0 – 5.0	SILT and SAND mixtures.
		5.0 – 7.8	Silty CLAY.
		7.8 – 18.5	Fine to coarse SAND; dense to very dense.
		18.5 – 26.8 m	Silt, sandy SILT and clayey SILT.
			(WT at 3.0 m bgl)
CPT-STM-11	2481703 mE 5739201 mN	0 – 15.0 m	Layers of clayey SILT, sandy SILT and silty SAND
		15.0 - 20.0 m	Dense SAND and silty SAND
			(WT at 0.5 m bgl)

Initial observations of the CPT results indicate the soils are predominantly layers of loose to medium dense sand and silt underlain by dense to very dense sand. This would infer that liquefaction is possible and likely in a significant seismic event. Groundwater was encountered between 0.5 m and 3.0 m below ground level.

6.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has classified the site as “Green Zone, N/A – Urban Non-residential” category. Land in the green zone is generally considered suitable for residential construction. An ‘N/A’ technical category indicates the site is a non-residential property in urban area beyond the extent of land damage mapping. However, the neighbouring properties are classified as ‘Technical Category 2 (TC2, yellow) means that 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 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 6.1.

[2] Tonkin and Taylor. September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Waltham & St Martins.

Figure 6.1 Post February 2011 Earthquake Aerial Photography [3]

6.2.6 Summary of Ground Conditions

From information on ECan borehole logs and EQC CPT data subsoils at the site are anticipated to be layers of sands (with some gravel) and silts (with some sand and clay). These soils are consistent with the Springston formation (Yaldhurst member); being stratified alluvial deposits of predominantly sand and silt overbank deposits. Groundwater can be expected from a depth of 0.5 m below ground level.

It is anticipated that the site is situated on an old stream bed. Associated with this is an increased potential for subsoil liquefaction beneath the site.

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.

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

Table 6.3 Summary of Known Active Faults[45]

Known Active Fault	Distance from Site (km)	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130	NW	8.3	~300 years
Greendale (2010) Fault	24	W	7.1	~15,000 years
Hope Fault	110	NW	7.2~7.5	120~200 years
Kelly Fault	110	NW	7.2	~150 years
Porters Pass Fault	60	NW	7.0	~1100 years

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains, including Christchurch City, and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

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.

The recent seismic activity has produced earthquakes of Magnitude-6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

6.4 Slope Failure and/or Rockfall Potential

The site is located within Waltham, a flat suburb in southeast Christchurch. Global slope instability risk is considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

6.5 Liquefaction Potential

The site is at risk from minor to moderate liquefaction in future earthquake events this is based on:

- Due to the presence of alluvial sand and silt deposits,
- CERA's classification of the site as TC2,
- No evidence of liquefaction from the post-earthquake aerial photography.
- Given the site's proximity to the Heathcote River, it is considered likely that lateral spreading has occurred within or adjacent to the site. In future seismic events the site is considered at risk of lateral spreading.

[4] 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.

[5] GNS Active Faults Database

6.6 Conclusions & 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 stratified alluvial deposits, predominantly comprising sand and silt. Associated with this the site also has a minor to moderate potential for liquefaction including the potential for lateral spreading.

Should a more comprehensive ground condition assessment be required, we recommend that an intrusive investigation be conducted.

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

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)	1.0
◗ Ductility Scaling Factor (k_μ)	1.0
◗ Performance Factor (S_p)	1.0
Transverse Direction	
◗ Ductility Factor (μ)	1.0
◗ Ductility Scaling Factor (k_μ)	1.0
◗ Performance Factor (S_p)	1.0

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 directions 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 lateral load resisted by each wall is proportional to the stiffness of each wall. 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 1.0 has been assumed in both the longitudinal and transverse direction based on the unreinforced concrete masonry walls that resist lateral seismic loading. Seismic loading is also resisted by reinforced masonry walls however; a ductility factor of 1.0 has been selected as the overall seismic performance of the building is likely to be governed by the response of the unreinforced masonry walls during an earthquake.

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:

$$C_d(T_1)=\frac{C(T_1) \cdot S_p}{k_\mu}=\frac{0.9\times1.0}{1.0}=0.9$$

7.3 Capacity of Structural Elements

7.3.1 Unreinforced Masonry In-Plane Shear Capacity

The in-plane shear capacity of the unreinforced concrete masonry walls was determined using Section 8.4 of the NZSEE guidelines "Assessment & Improvement of Unreinforced Masonry Building for Earthquake Performance (2011)". 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 each wall was evaluated considering four shear failure modes. These are diagonal tension failure, rocking failure, bed-joint sliding failure and toe crushing failure. The in-plane shear capacity of each wall is,

$$V_n = min(V_{dt}, V_r, V_s, V_{tc})$$

7.3.2 Unreinforced Masonry In-Plane Moment Capacity

The in-plane flexural capacity of the unreinforced concrete masonry walls was calculated as,

$$M_n = N_b \left[Z - \frac{1}{2} \times \frac{N_b}{0.85 f'_m t_w}\right]$$

$$Z = {}^{L_w}/_2$$

Where

N_b = normal force acting at wall base

f'_m = compressive strength of masonry

t_w = wall thickness

L_w = wall length

7.3.3 Unreinforced Masonry Out-of-Plane Capacity

The out-of-plane flexural capacity of the unreinforced concrete masonry walls was determined using Section 10.3.4 of the NZSEE guidelines "Assessment & Improvement of the Structural Performance of Buildings in Earthquakes (2006)". The overall out-of-plane capacity of each wall was evaluated by comparing the likely displacement of the wall during an earthquake and the displacement that would cause instability of the wall. The out-of-plane capacity of each wall is,

$$\%NBS = 0.72 \frac{\Delta_i}{D_{ph}}$$

Where

Δ_i = out-of-plane deflection that would cause instability

D_{ph} = out-of-plane displacement response demand for a wall panel

7.3.4 Reinforced Masonry Shear Capacity

The shear capacity of the reinforced masonry walls 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 and from applied loading, V_p, was ignored in the calculations.

7.3.1 Reinforced Masonry In-Plane Moment Capacity

The in-plane flexural capacity of the reinforced concrete masonry walls was calculated as,

$$M_n = A_s f_y \left[d - \frac{a}{2}\right]$$

$$a = \frac{A_s f_y}{0.85 f_m' b_w}$$

Where

A_s = area of tension steel

f'_m = compressive strength of masonry

b_w = wall thickness

d = $0.8L_w$ = wall length

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

f'_m = specified compressive strength of masonry

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

7.3.1 %NBS

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

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

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

8. Results

The New Zealand Society for Earthquake Engineering (NZSEE) publications “Assessment & Improvement of Structural Performance of Buildings” (2006) and “Assessment & Improvement of Unreinforced Masonry Buildings for Earthquake Resistance” (2011) along with 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 **9% NBS** and is therefore **Earthquake Prone.**

The structural analysis results are discussed in the following sections.

8.1.1 Unreinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

The unreinforced concrete masonry walls achieve **9% NBS** under in-plane shear seismic loading. The critical in-plane shear failure mode for the majority of the walls is rocking failure.

The unreinforced concrete masonry walls are significantly stiffer than the central 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.

<u>In-Plane Moment</u>

The unreinforced concrete masonry walls achieve **12% NBS** when considering in-plane bending of the walls.

<u>Out-of-Plane Moment</u>

The unreinforced concrete masonry walls achieve **68% NBS** when considering out-of-plane bending of the walls.

The 190mm thick concrete masonry walls are restrained out-of-plane at eaves level by the timber framed roof diaphragm at a height of 2.5m. The walls have been assumed to span vertically between the concrete floor slab and the timber roof. Out-of-plane displacement demands and capacities were evaluated per metre width of wall.

8.1.2 Reinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

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

The layout of the concrete masonry walls in the longitudinal and transverse directions is relatively regular and symmetrical. The extension to the north-western end of the building creates a stiffness eccentricity in the lateral load resisting system due to the presence of the long external wall with no penetrations.

In-Plane Moment

The reinforced concrete masonry walls achieve **68% NBS** when considering in-plane bending of the walls. This was calculated assuming a single 12mm diameter vertical bar yielding in tension.

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **29% NBS** when considering out-of-plane bending of the walls.

The 190mm thick, long concrete masonry wall in the extension area of the building (Wall 1) is not restrained by a return wall at the south-western end of the wall. It is unlikely that the roof diaphragm will provide sufficient restraint at eaves level to this wall. As a result, the wall has been assumed to cantilever from the foundations.

8.2 Summary

Element	Seismic Action	%NBS
Longitudinal Direction		
Unreinforced Concrete Masonry Walls	In-Plane Shear	28
	In-Plane Bending	37
	Out-of-Plane Bending	68
Reinforced Concrete Masonry Wall	In-Plane Shear	100
	In-Plane Bending	68
	Out-of-Plane Bending	100
Transverse Direction		
Unreinforced Concrete Masonry Wall	In-Plane Shear	9
	In-Plane Bending	12
	Out-of-Plane Bending	68
Reinforced Concrete Masonry Wall	In-Plane Shear	100
	In-Plane Bending	79
	Out-of-Plane Bending	29

Table 8.1 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 1970s and was likely designed in accordance with the previous loading standard, NZS 1900:1965. The design loads used are likely to have been less than those required by the current loading standard.

The concrete masonry walls are unreinforced and as a result, there is a significant risk of the walls collapsing during a seismic event. It is therefore reasonable to expect the detailed assessment of the structure to indicate that the building is earthquake prone.

9. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of 9% NBS and is therefore Earthquake Prone.

It is recommended that Christchurch City Council proceed with developing potential strengthening concepts for the building.

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 Christchurch City Council 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

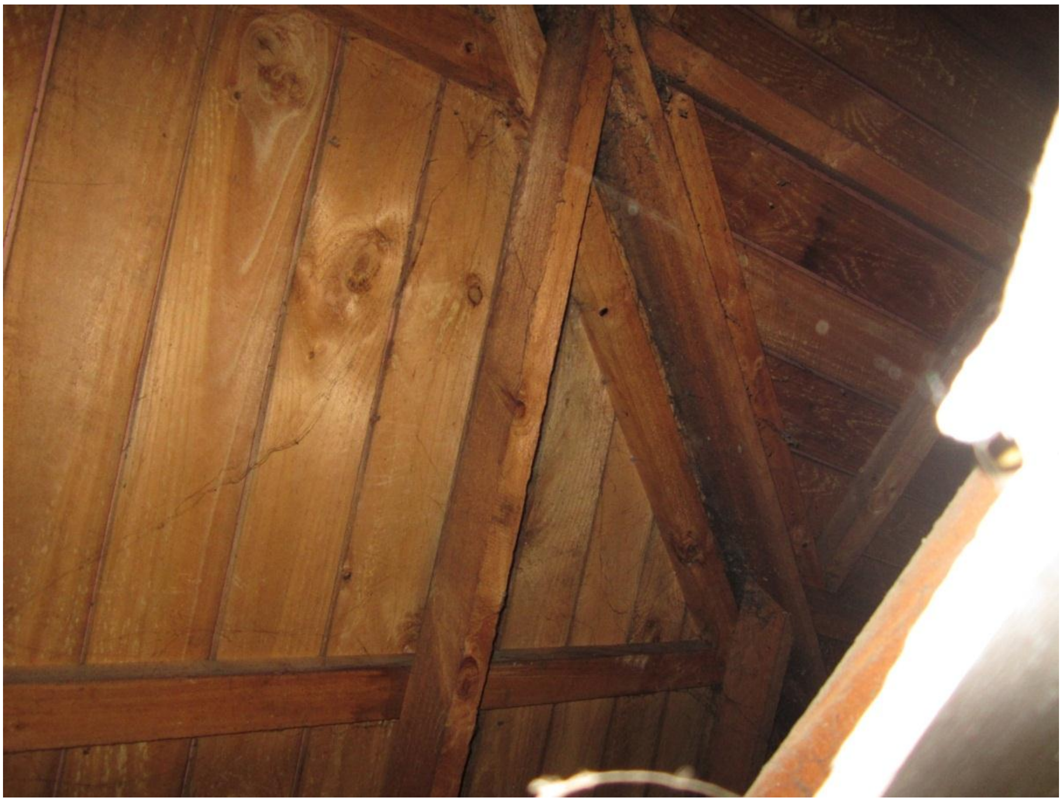
Photograph 1 View of the building from the north-east

Photograph 2 South-eastern elevation of the building

Photograph 3 South-western side of the building

Photograph 4 North-western end of the building

Photograph 5 Interior view of building



Photograph 6 Timber sarking supporting the profiled sheet metal roof cladding

Photograph 7 Cracking along the mortar lines in the concrete masonry walls

Photograph 8 Cracking occurring at the corner of a filled in opening in a concrete masonry wall

Appendix B

Sketch

Appendix C
CERA Form

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