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Waimairi Cemetery Toilets/Shed/Office PRK 0291 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

195a Grahams Road, Burnside

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

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195a Grahams Road, Burnside

Christchurch City Council

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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 and visual inspections on 1 February 2013.

Building Description

The building is located at 195a Grahams Road, Burnside and is used as public toilets and storage space. The date of construction is estimated to be during the 1960s based on construction characteristics of the building. A storage room extension added to the south-west of the original building is estimated to have been constructed during the 1980s. The overall building is approximately 13.5m in length by 4.7m in width with a height of 3.1m and occupies a footprint of approximately 65m^2. The site is approximately 150m west of the Wairarapa River.

The structure consists of concrete and brick masonry walls supporting a lightweight timber framed roof. The roof structure consists of a mono-pitch roof formed by corrugated sheet metal supported by timber purlins and rafters with a hardboard panel ceiling.

Key Damage Observed

No residual displacements of the structure were observed during inspection of the building. An existing crack at the intersection between a transverse concrete masonry wall and a longitudinal concrete masonry wall was observed in the original section of the building. The damage observed is unlikely to be a result of recent seismic activity

Building Capacity Assessment

The building has been assessed to have a seismic capacity in the order of 11% 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 public toilets in Waimairi Cemetery.

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 building's 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 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.

Table 1 %NBS compared to relative risk of failure

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

4. Building Descriptions

4.1 General

The building is located at 195a Grahams Road, Burnside and is used as public toilets and storage space. The date of construction is estimated to be during the 1960s based on construction characteristics of the building. A storage room extension added to the south-west of the original building is estimated to have been constructed during the 1980s.

The building is approximately 13.5m in length by 4.7m in width with a height of 3.1m and occupies a footprint of approximately 65m^2. The site is approximately 150m west of the Wairarapa River.

The original building consists of unreinforced concrete masonry walls supporting a lightweight timber framed roof. The north-west, north-east and south-west external walls of the original structure consist of unreinforced 100mm thick partially filled concrete masonry units clad externally with a 100mm brick veneer. The rear conceret masonry wall along the south-eastern side of the original building and the internal transverse concerte masonry walls (Walls 13 and 14, Figure 2) are 150mm thick and unreinforced.

Unrestrained partial height walls form the toilet cubicles in the original building. These walls consist of 100mm thick unreinforced concrete masonry.

The roof structure consists of an approximately 10 degree mono-pitch roof formed by corrugated sheet metal supported by timber purlins and rafters. The ceiling is formed by fixed hardboard panels. The timber rafters are supported by cast-in-situ concrete infill above the unreinforced concrete masonry walls. The foundations of the building consist of a concrete slab-on-grade and concrete strip footings beneath the external walls.

The structure of the extension to the south-west of the original building consists of partially filled concrete masonry walls reinforced with 12mm vertical reinforcing bars at 800mm centres. The rear south-eastern wall of the extension is 150mm thick. The other walls are 200mm thick. There is a large door opening on the north-western wall of the extension. The roof structure is similar to that of the original structure, consisting of corrugated sheet metal supported by timber purlins and rafters and a hardboard panel ceiling. Lightweight timber framed infills above the walls support the mono-pitch roof.

Figure 2 and Figure 3 show the construction details of the building.

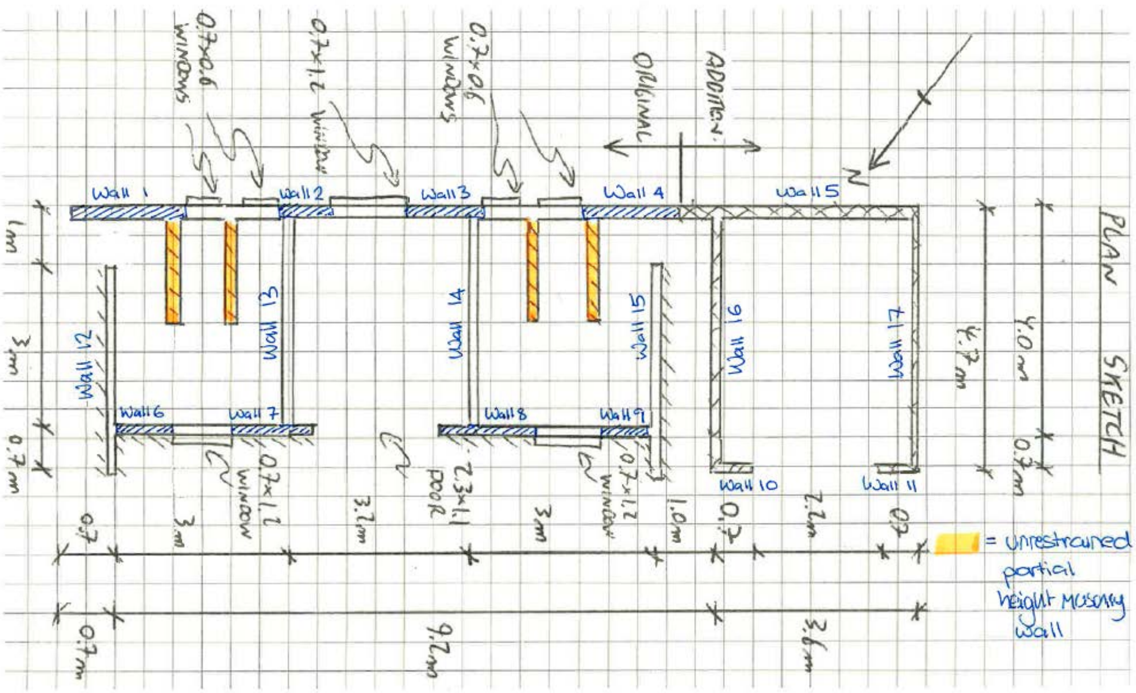


Figure 2 Floor plan of the building

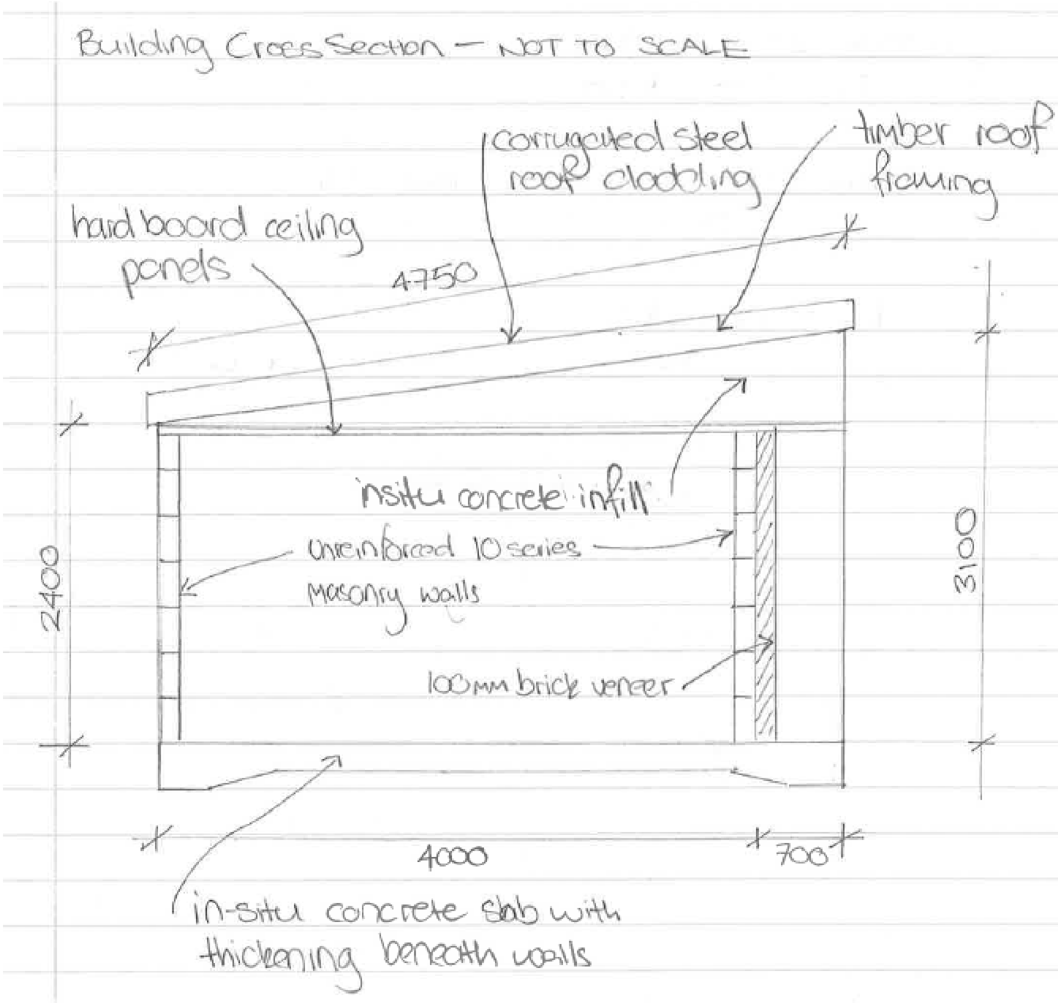


Figure 3 Sketch showing section through the building

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 purlins and 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 hardboard panel ceiling lining provides a diaphragm to transfer seismic forces through the roof structure to the walls in the plane of loading. Lateral seismic loads in both the transverse and longitudinal direction are resisted by the concrete masonry walls in the plane of loading. 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 unreinforced and reinforced concrete masonry walls are restrained at eaves level by the timber framed roof structure. Concrete masonry walls perpendicular to the direction of seismic loading transfer the lateral seismic forces via diaphragm action of the hardboard panel ceiling to the concrete masonry walls in the plane of loading. The unrestrained partial height concrete masonry walls in the original unreinforced section of the building resist out-of-plane seismic forces through cantilever action.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 1st of February 2013. Both the interior and exterior of the building were 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 presence of reinforcement in the concrete masonry walls. Where reinforcement was detected, the position, depth and diameter of the reinforcement were recorded. 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

Drawings of the building 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

No residual displacements of the structure were observed during inspection of the building.

An existing crack at the intersection between a transverse concrete masonry wall and a longitudinal concrete masonry wall was observed in the original section of the building (shown in Photograph 7). The damage observed is unlikely to be a result of recent seismic activity.

5.3.3 Ground Damage

No evidence of ground damage was observed during inspections.

6. Geotechnical Consideration

6.1 Site Description

The site is situated within the Waimairi Cemetery, within the suburb of Burnside in western Christchurch, and is relatively flat at approximately 20m above mean sea level. It is approximately 150m west of the Wairarapa River, and 12km 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.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that one shallow borehole is located within 200m of the site (see Table 2). The site geology described in this log indicates the area is predominantly sand to a depth of ~1.4m bgl. Varying amounts of shingle and silt are also indicated to be present.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/16775	~1.4m	N/A	195m NE

Boreholes slightly further away indicate the area is underlain by gravel and sandy gravel.

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.

6.2.3 EQC Geotechnical Investigations

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

6.2.4 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 TC1 (grey), TC2 (yellow) and TC3 (blue). These categories describe how the land in expected to perform in future earthquakes.

[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.

The site is zoned as technical category "N/A – Urban Non-residential".

The surrounding land is generally categorised as Technical Category 1 (TC1) - future land damage from liquefaction is unlikely.

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 4.

Figure 4 Post February 2011 Earthquake Aerial Photography[2]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to be sand with varying amounts of silt and gravel.

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

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.

Table 3 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	18 km	SW	7.1	~15,000 years
Hope Fault	100 km	NW	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

Recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath central Canterbury, 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

This 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.

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.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Burnside, a flat suburb in western 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 liquefaction potential for this site is considered low. This is based on:

- no effects of liquefaction were reportedly observed at the ground surface in Burnside; 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

- the surrounding residential land is zoned TC1.

However, the published boreholes do not extend deep enough to identify liquefaction prone soils.

6.6 Conclusions & Summary

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 of sand with some gravel and silt with a low to moderate liquefaction potential.

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted. Specific testing details can be provided upon commission of the quantitative assessment phase.

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.

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

$$S_P = 1.3 - 0.3\mu$$

The seismic weight coefficient was then calculated in accordance with Clause 5.2.1.1 of NZS 1170.5:2004 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 brittle. 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 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 Clause 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.

7.3.5 Reinforced Masonry In-Plane Moment Capacity

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

$$\emptyset M_n = \emptyset \left[\sum F_{si}(x_i - c) + C_m\left(c - \frac{a}{2}\right) + N\left(\frac{L_w}{2} - c\right) \right]$$

Where

$$\sum F_{si} - C_m + N = 0$$

F_{si} = tension or compression force in the vertical wall reinforcement

x_i = vertical reinforcing bar position

c = neutral axis depth

C_m = masonry compressive force

a = βc = masonry compression block parameter

N = axial load

7.3.6 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.

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

$$a = {A_s f_{yt}}/{0.85 f'_m b}$$

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 shear and bending moment capacities of the reinforced and unreinforced concrete masonry walls were compared to their respective demands to determine the overall %NBS for the 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) 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 **11% 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 **13% NBS** under in-plane shear seismic loading. The critical in-plane shear failure mode for the majority of the walls is rocking failure.

<u>In-Plane Moment</u>

The unreinforced concrete masonry walls achieve **17% NBS** when considering in-plane bending of the walls.

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

The unreinforced concrete masonry walls achieve **34% NBS** when considering out-of-plane bending of the walls.

The concrete masonry walls are restrained out-of-plane by the timber framed roof diaphragm. 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.

The unrestrained partial height concrete masonry walls achieve **11% NBS** when considering out-of-plane bending. These walls are the critical structural elements in the structure and govern the overall assessed %NBS score for the building.

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.

<u>In-Plane Moment</u>

The reinforced concrete masonry walls achieve **95% NBS** when considering in-plane bending of the walls.

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

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

8.2 Summary

Table 4 Summary of %NBS scores

Element	Seismic Action	%NBS
Longitudinal Direction		
Unreinforced Concrete Masonry Walls	In-Plane Shear	14
	In-Plane Bending	17
	Out-of-Plane Bending	34
Reinforced Concrete Masonry Wall	In-Plane Shear	100
	In-Plane Bending	95
	Out-of-Plane Bending	100
Transverse Direction		
Unreinforced Concrete Masonry Wall	In-Plane Shear	13
	In-Plane Bending	19
	Out-of-Plane Bending	45
Reinforced Concrete Masonry Wall	In-Plane Shear	100
	In-Plane Bending	100
	Out-of-Plane Bending	100
Partial Height Unrestrained Concrete Masonry Walls	Out-of-Plane Bending	11

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 1960s and was likely designed in accordance with 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 in the original section of the building are unreinforced and as a result, there is a significant risk of wall failure 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 11% 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 North-western elevation of the building

Photograph 2 View of the building from the north

Photograph 3 Brick veneer on the exterior concrete masonry walls

Photograph 4 Extension to the south-west of the original building

Photograph 5 Unrestrained partial height concrete masonry walls

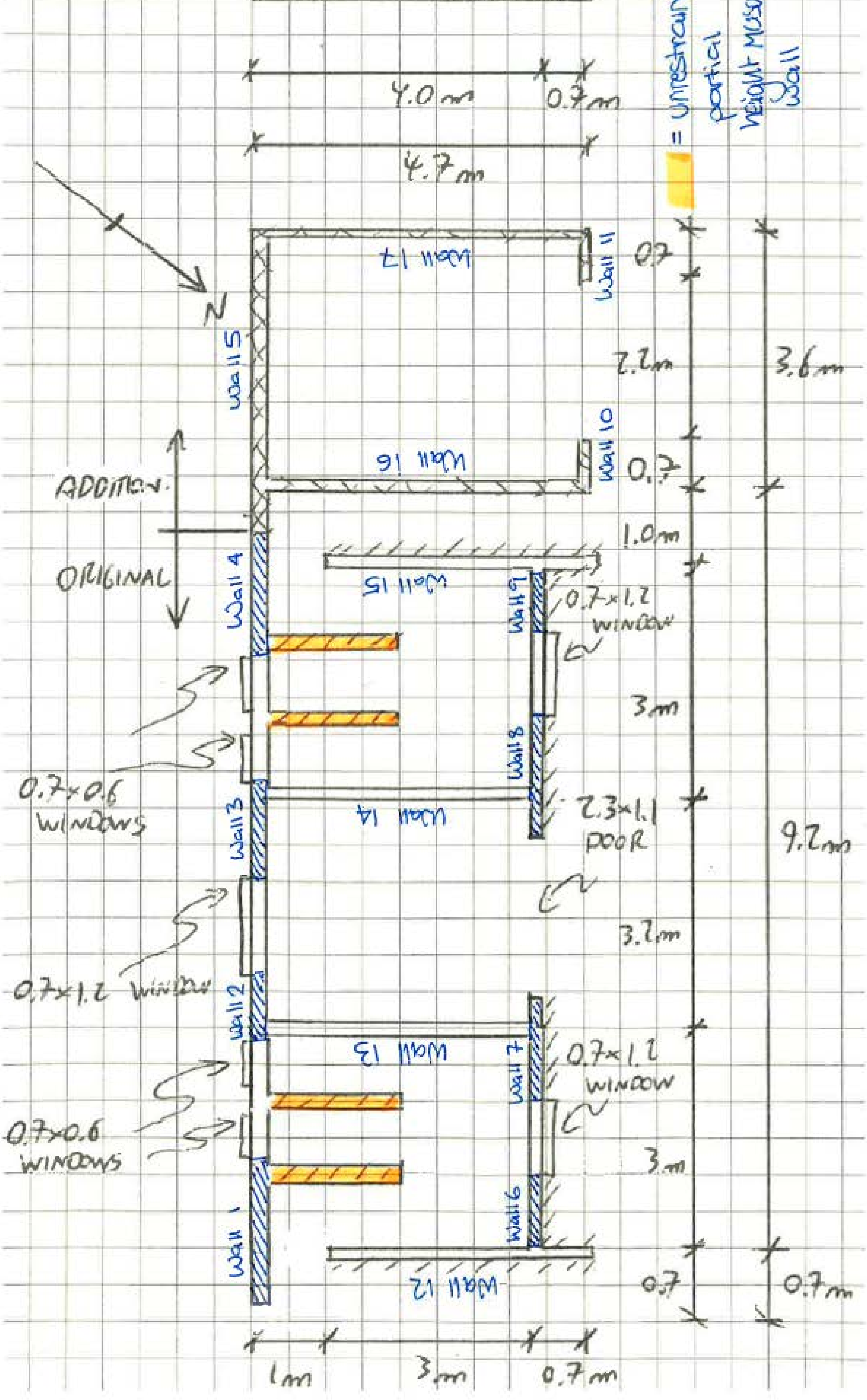
Photograph 6 Concrete masonry lintel beam

Photograph 7 **Existing cracking at connection between a transverse wall and a longitudinal wall**

Appendix B

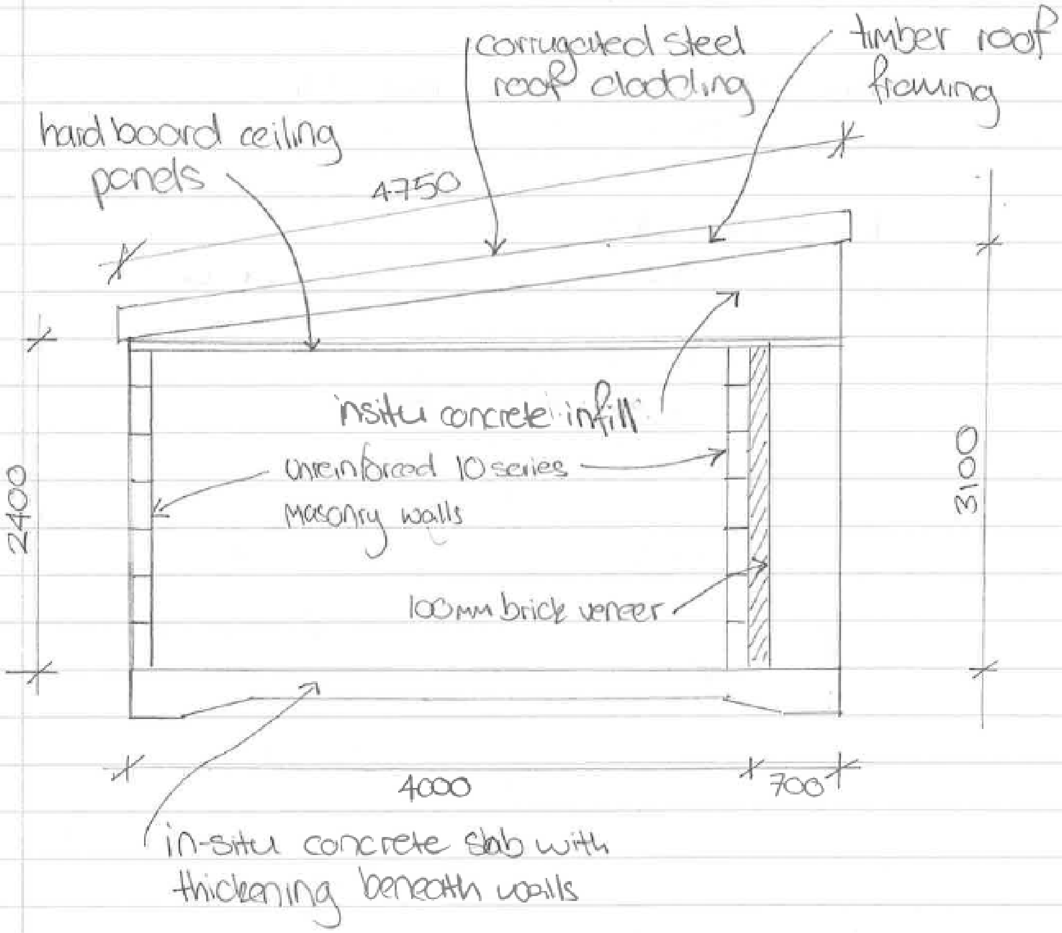
Sketches

PLAN SKETCH



4.0 m
0.7 m
4.7 m
= Unrestrained partial height masonry wall
N
Wall 17
Wall 11
0.7
Wall 5
2.2 m
3.6 m
Wall 10
Wall 16
0.7
ADDITION
1.0 m
ORIGINAL
Wall 4
Wall 15
Wall 9
0.7×1.2 WINDOW
3 m
Wall 8
0.7×0.6 WINDOWS
Wall 3
Wall 14
2.3×1.1 DOOR
9.2 m
3.2 m
0.7×1.2 WINDOW
Wall 2
Wall 13
Wall 7
0.7×1.2 WINDOW
0.7×0.6 WINDOWS
3 m
Wall 1
Wall 6
Wall 12
0.7
0.7 m
1 m
3 m
0.7 m

Building Cross Section — NOT TO SCALE



Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Waimairi Cemetery Toilets	Reviewer:	Stephen Lee
Building Address:	Unit: / No: 195a / Street: Grahams Road, Burnside	CPEng No:	1006840
Legal Description:		Company:	GHD
		Company project number:	51309213
		Company phone number:	04 472 0799
GPS south:	Degrees / Min / Sec	Date of submission:	
GPS east:		Inspection Date:	1-Feb-13
		Revision:	
Building Unique Identifier (CCC):	PRK_0291_BLDG_002 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	20.00

Building

No. of storeys above ground:	1	single storey = 1 / Ground floor elevation (Absolute) (m):	
Ground floor split?	no	Ground floor elevation above ground (m):	
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	3.10	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	65		
Age of Building (years):	50	Date of design:	1935-1965
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):	Public Toilets		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	Steel corrugate on 140 x 45 rafters
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	in-situ lintel beams
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	90

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - brick		Note: Define along and across in detailed report!	
Ductility assumed, μ:	1.00		note wall thickness and cavity	
Period along:	0.40	0.40	from parameters in sheet / estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	unreinforced masonry bearing wall - brick			
Ductility assumed, μ:	1.00		note wall thickness and cavity	
Period across:	0.40	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):		
south (mm):		
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Metal	describe	
Glazing:	timber frames		
Ceilings:	Other		Hardboard Ceiling Panels
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	None Observed
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Minor damage observed
	Describe (summary):	Minor damage observed		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage observed		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

$$Damage\_Ratio = \frac{(\%NBS(before) - \%NBS(after))}{\%NBS(before)}$$

Recommendations

	Level of repair/strengthening required:		Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	14%	#### %NBS from IEP below / If IEP not used, please detail assessment methodology:	Detailed Assessment
	Assessed %NBS after e'quakes:	14%		
Across	Assessed %NBS before e'quakes:	11%	#### %NBS from IEP below	
	Assessed %NBS after e'quakes:	11%		

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Alex Baylis	Stephen Lee		Nick Waddington		19/3/13