West Lake Park Toilets and Changing Rooms
PRK 0489 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

50 Westlake Drive, Halswell

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

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50 Westlake Drive, Halswell

Christchurch City Council

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Date

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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50 Westlake Drive, Halswell

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 13 December 2012.

Building Description

The single storey building is located at 50 Westlake Drive, Halswell and is used as public toilets and changing rooms. The date of construction is estimated to be during the 1980s based on the construction characteristics and materials.

The structure consists of reinforced concrete masonry walls supporting a lightweight timber framed roof consisting of nail plate timber trusses. The building is approximately 9.0m in length by 5.4m in width with a height of 4.6m. The building occupies a footprint of approximately 45m^2 and is located over 100m from the nearest structure. The site is approximately 25m east of West Lake.

Key Damage Observed

The building was observed to generally be in good condition during the inspection. Minor cracking in a concrete masonry unit was observed on the south-eastern side of the building at the connection point between an external timber roof beam and a concrete masonry wall. Minor step cracking was observed in the mortar in the concrete masonry wall on the south-eastern side of the building.

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

Building Capacity Assessment

The building has been assessed to have a seismic capacity in the order of 50% NBS and is therefore Earthquake Risk. The building is not Earthquake Prone.

Recommendations

No further action is required by Christchurch City Council to comply with the Building Code; however 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 and changing rooms in West Lake 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 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 single storey building is located at 50 Westlake Drive, Halswell and is used as public toilets and changing rooms. The date of construction is estimated to be during the 1980s based on the construction characteristics and materials.

The building is approximately 9.0m in length by 5.4m in width with a height of 4.6m. The building occupies a footprint of approximately 45m^2 and is over 100m from the nearest structure. The site is approximately 25m east of West Lake.

The structure of the building consists of reinforced concrete masonry walls supporting a lightweight timber framed roof. The roof structure consists of a 45 degree duo-pitch roof formed by corrugated sheet metal on timber sarking supported by nail plate timber trusses as shown in Photograph 1. A timber beam spanning longitudinally between concrete masonry walls supports the timber trusses for a portion of the south eastern elevation. 50mm square timber posts support the verandah roof around the north-eastern corner of the building.

The walls consist of 190mm thick fully filled concrete masonry units reinforced with 12mm diameter bars placed centrally at 600mm centres vertically and 800mm centres horizontally. The concrete masonry walls are clad internally with ceramic tiles. A bond beam reinforced with a 12mm diameter bar runs along the length of the masonry walls at eaves level. The transverse walls have plasterboard lined, timber framed infill panels above the concrete masonry walls.

Unrestrained partial height walls form the shower area in the changing room section of the building. These walls consist of 190mm thick reinforced concrete masonry units and are 1.8m high.

The foundations of the buildings are assumed to consist of a concrete slab-on-grade and concrete strip footings beneath the external and internal concrete masonry walls.

Figures 2 and 3 show the construction details.

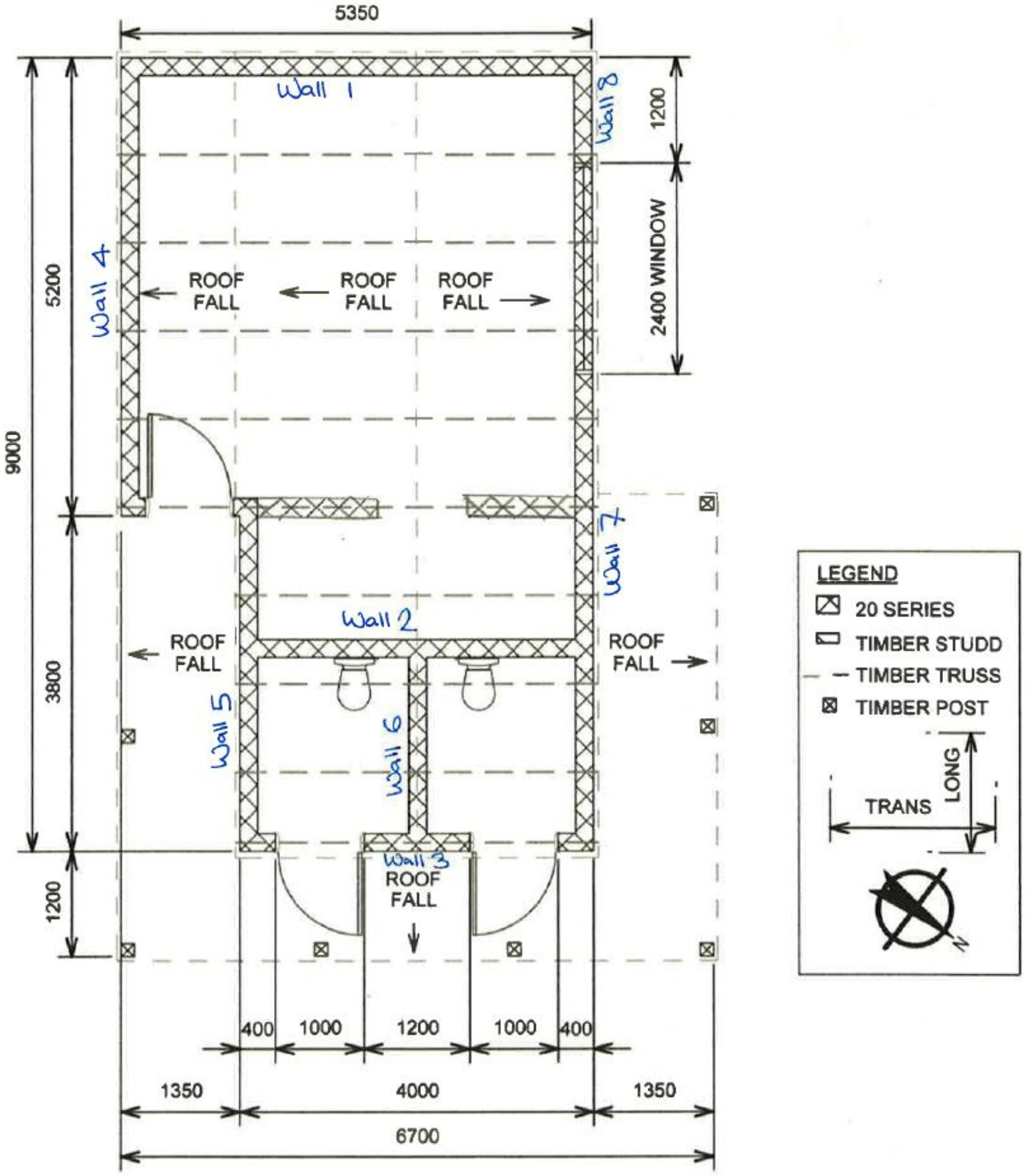


Figure 2 Floor plan of building

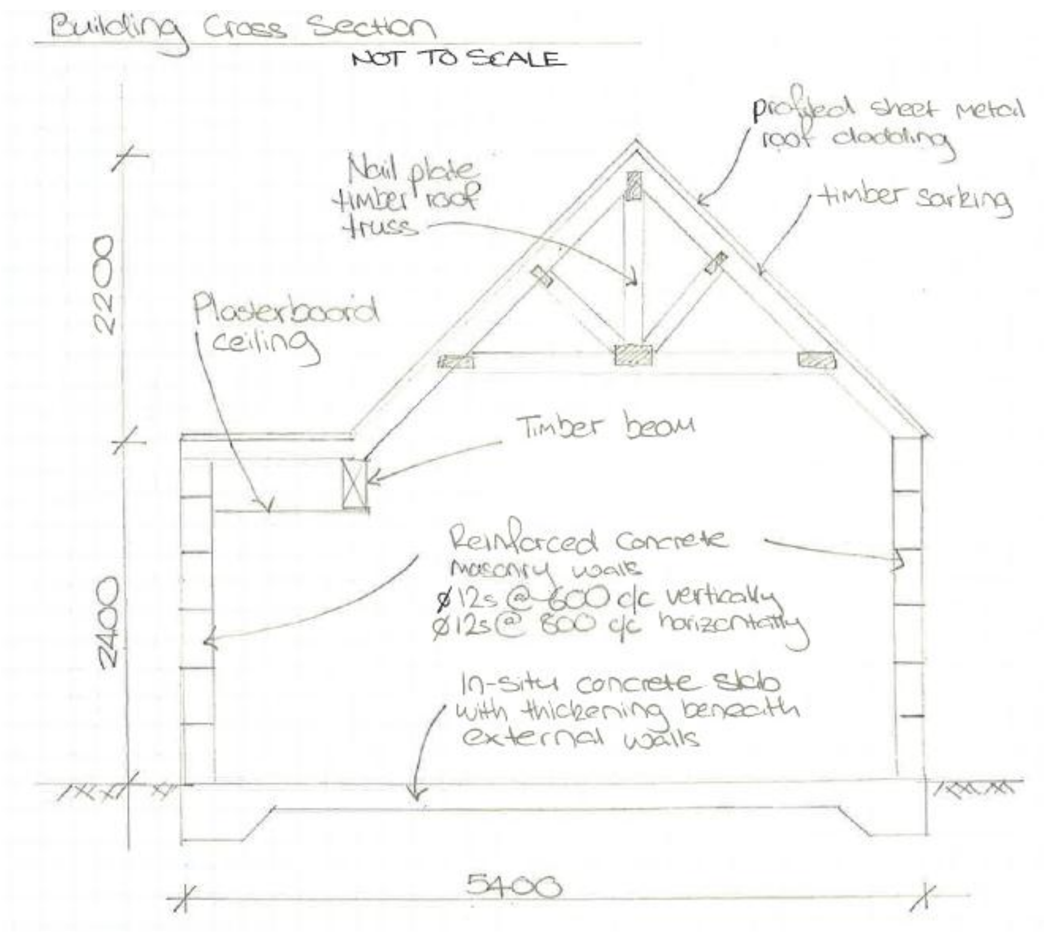


Figure 3 Building cross section

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 nail plate timber trusses to the longitudinal 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

In the longitudinal direction, lateral seismic roof loads are transferred to the nail plate timber roof trusses by panel action of the timber sarking lining. The forces are then transferred through the timber wall plate connection to the in-plane reinforced concrete masonry walls. Panel action of the in-plane longitudinal concrete masonry walls resists the lateral seismic roof loads. The longitudinal walls transfer the seismic loads to the foundations through bending and shear where they are distributed into the ground.

In the transverse direction, lateral seismic loads are transferred to the reinforced concrete masonry walls. Panel action of the transverse concrete masonry walls resists the lateral seismic roof loads. The transverse walls transfer the seismic loads to the foundations through bending and shear where they are distributed into the ground.

The concrete masonry walls are not restrained out-of-plane along their top edge due to the absence of an effectively connected roof diaphragm.

The roof structure is unlikely to act as a diaphragm due to the steep pitch of the roof and the collar tied nail plate timber trusses. There is also no bracing is present between the nail plate timber trusses.

The transverse walls are connected to the roof structure by lightweight timber framed gable infill walls. NZS 4229:1999 requires that a diaphragm is connected directly to a bond beam at the top of a concrete masonry wall to ensure diaphragm forces can be adequately transferred.

The nail plate timber trusses are nailed to the timber wall plate as shown in Photograph 7. The connection between the roof structure and walls is inadequate and is unlikely to have sufficient capacity to transfer forces from the nail plate timber trusses to the in-plane concrete masonry walls.

As a result, the concrete masonry walls resist out-of-plane bending through cantilever action or spanning vertically between the floor slab and bond beams spanning horizontally between return walls. The unrestrained partial height concrete masonry walls forming the shower area resist out-of-plane seismic forces through cantilever action.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 13th 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.

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 the site inspection.

5.3.2 General Observations

The building was observed to generally be in good condition during the inspection. Minor cracking in a concrete masonry unit was observed on the south-eastern side of the building at the connection point between an external timber roof beam and a concrete masonry wall. This is shown in Photograph 8. Minor step cracking was observed in the mortar in the concrete masonry wall on the south-eastern elevation of the building. This is shown in Photograph 9.

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

5.3.3 Ground Damage

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

6. Geotechnical Consideration

6.1 Site Description

The site is situated in the suburb of Halswell, southwest of Christchurch City centre. The site is relatively flat at approximately 20m above mean sea level. It is approximately 25m east of West Lake, 900m southwest of Heathcote River, 3.3km south of the Main South Line Railway, 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:

- Yaldhurst member of the Springston Formation, dominantly alluvial gravel, sand and silt of historic river flood channels, Holocene in age.

Due to the low lying location of the site, elevated ground water table is considered likely.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there is one borehole located within 200m of the site. However, there is no bore log available.

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 the Tonkin & Taylor Report for Haslwell[2]. One shallow borehole was undertaken within 200m of the site but considered insignificant for this assessment.

6.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has classified Westlake Park, Haslwell as “Green Zone – N/A – Urban Nonresidential” category. Land in this zone is generally considered suitable for residential construction, though some areas may require stronger foundations or design where rebuilding or repairs are required. “Not Applicable – Urban Nonresidential” technical category is the classification given for nonresidential properties in urban area beyond the extent of land damage mapping.

However, properties to the north and south of the site where are classified as “Green Zone, Technical Category 2 – yellow.” Land in this zone is generally considered suitable for residential construction, though some areas may require stronger foundations or design where rebuilding or repairs are required. Technical Category 2, yellow means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

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

[2] Tonkin & Taylor Ltd., 2011: Christchurch Earthquake Recovery, *Geotechnical Factual Report, Halswell.*

6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of minor to moderate signs of liquefaction at road corridors and nearby properties, as shown in Figure 4.

Figure 4 Post February 2011 Earthquake Aerial Photography[3]

6.2.6 Summary of Ground Conditions

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

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 2 Summary of Known Active Faults[4,5]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	125 km	NW	~8.3	~300 years
Greendale (2010) Fault	16 km	W	7.1	~15,000 years
Hope Fault	109 km	NW	7.2~7.5	120~200 years
Kelly Fault	109 km	NW	7.2	150 years
Porter Pass Fault	70 km	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

Given the site's location in Halswell, 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 minor to moderately susceptible to liquefaction, due to the following reasons:

- Signs of minor to moderate liquefaction at road corridors and properties near the site (evidence from the post-earthquake aerial photograph);
- Anticipated presence of saturated sand and silt layers beneath the site; and
- Properties south of the site are classified by CERA as "Green Zone, Technical Category 2 – yellow."

[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, June 2002, pp. 1878-1903.

[5] GNS Active Faults Database, http://maps.gns.cri.nz/website/af/viewer

Due to the site's proximity to the lake, lateral spreading is considered low to moderate. Further investigation should be carried out to determine the site-specific lateral spreading potential.

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 alluvial deposits. Associated with this the site also has a minor to moderate liquefaction potential, in particular where sands and/or silts are present.

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.

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 based on the tributary areas of each wall. This is because the lightweight timber framed roof structure is unlikely to act as a diaphragm as it is not effectively fixed to the transverse concrete masonry walls.

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 ≤ 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 In-Plane 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 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 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 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

7.3.3 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

7.3.1 %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 **50% NBS** and is therefore **Earthquake Risk.** The building and is **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.

<u>In-Plane Moment</u>

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

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

The reinforced concrete masonry walls achieve **50% NBS** when considering out-of-plane bending of the walls.

The external concrete masonry walls are not restrained out-of-plane due to the absence of an effectively connected roof diaphragm. The transverse walls are connected to the roof structure by lightweight timber framed gable infill walls. The longitudinal walls are connected to the nail plate timber trusses by a timber wall plate which is intermittently fixed to the concrete masonry walls. As a result, the critical concrete masonry walls resist out-of-plane bending through cantilever action.

The unrestrained concrete masonry walls that form the shower areas achieve 100% NBS. These walls perform better than the external concrete masonry walls as they are shorter and do not support any axial loads.

8.2 Summary

Element	Seismic Action	%NBS
Longitudinal Direction		
Concrete Masonry Walls	In-Plane Shear	100
	In-Plane Bending	100
	Out-of-Plane Bending	50
Transverse Direction		
Concrete Masonry Walls	In-Plane Shear	100
	In-Plane Bending	100
	Out-of-Plane Bending	52
Unrestrained Concrete Masonry Walls	Out-of-Plane Bending	100

Table 3 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 1980s and was likely designed in accordance with the previous loading standard, NZS 4203:1976. The design loads used are likely to have been less than those required by the current loading standard.

The building performs well in-plane in both the transverse and longitudinal directions with the concrete masonry walls achieving 100% NBS. The concrete masonry walls have a regular layout and are sufficiently reinforced to resist the in-plane shear and bending demands during an earthquake.

Out-of-plane bending of the concrete masonry walls out-of-plane is the critical aspect of the seismic performance of the overall structure. The concrete masonry walls are not effectively restrained along the top edges due to the absence of a roof diaphragm and rely on cantilever action to resist lateral seismic loads.

9. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of 50% NBS and is therefore Earthquake Risk. The building is not Earthquake Prone. No further action is required by Christchurch City Council to comply with the Building Code; however 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 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 Nail plate timber trusses

Photograph 2 View of the building from the north

Photograph 3 View of the building from the east

Photograph 4 View of the building from the south

Photograph 5 Unrestrained concrete masonry shower partition walls

Photograph 6 Timber beam supporting the timber roof trusses

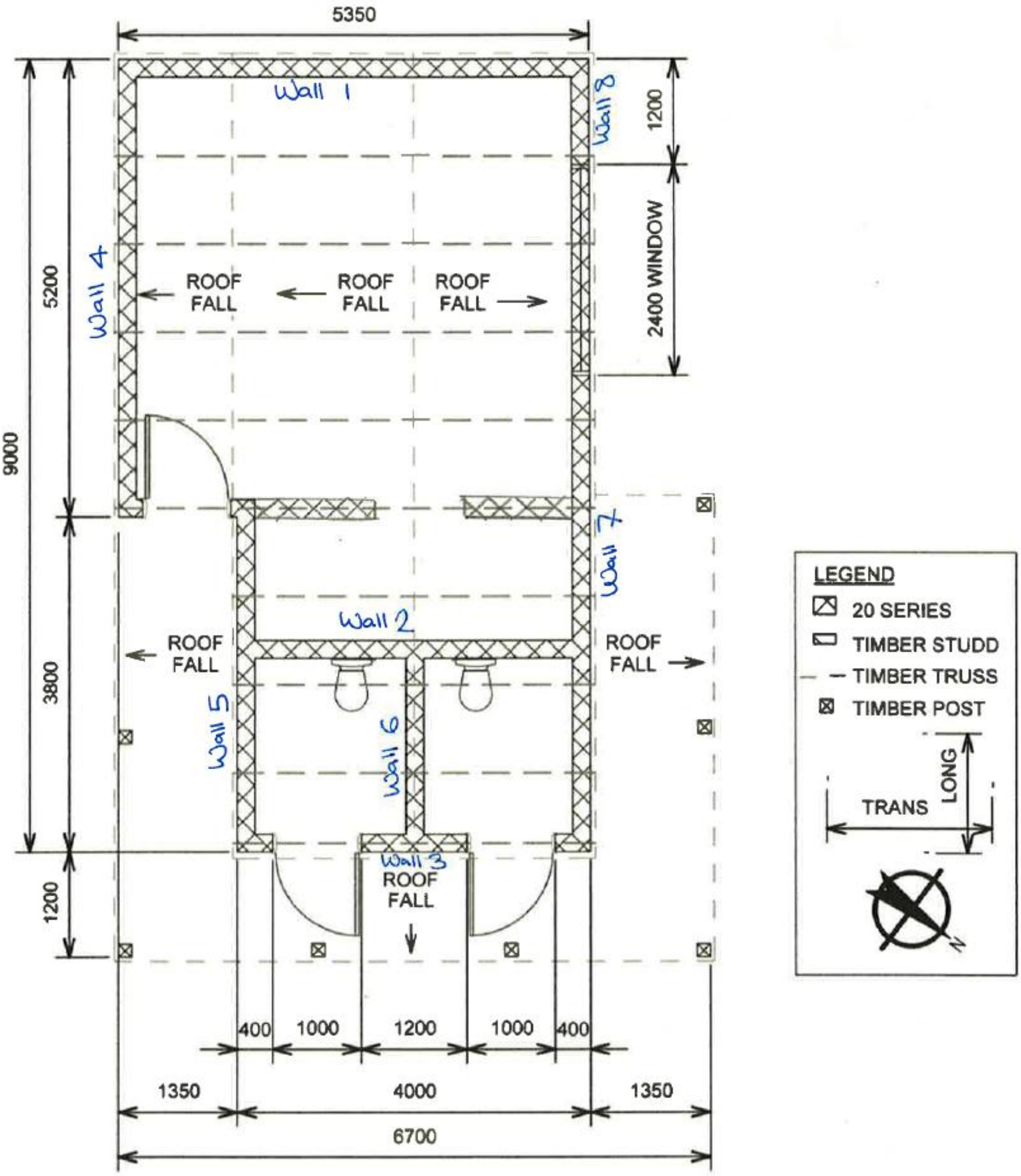
Photograph 7 **Wall plate connection between timber trusses and concrete masonry walls**

Photograph 8 **Cracking in the concrete masonry unit at connection with an external timber roof beam**

Photograph 9 **Minor step cracking in the mortar on the south-eastern elevation of the building**

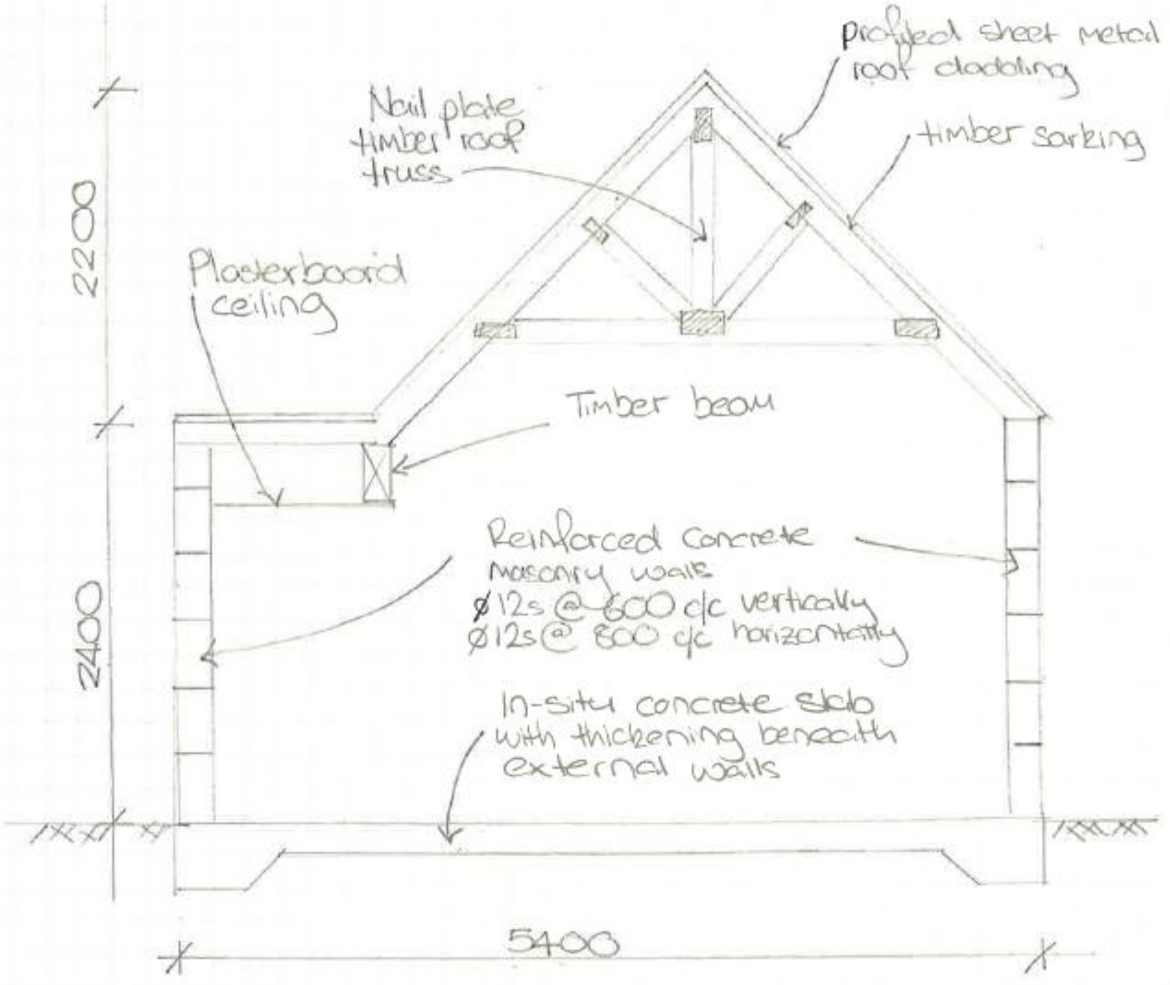
Appendix B

Sketches


5350
Wall 1
Wall 8
1200
2400 WINDOW
Wall 4
ROOF FALL
ROOF FALL
ROOF FALL
5200
9000
Wall 7
Wall 2
ROOF FALL
ROOF FALL
Wall 5
Wall 6
3800
Wall 3
ROOF FALL
1200
400
1000
1200
1000
400
1350
4000
1350
6700
LEGEND
20 SERIES
TIMBER STUDD
TIMBER TRUSS
TIMBER POST
LONG
TRANS

Building Cross Section

NOT TO SCALE



Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Westlake Park Toilet	Reviewer:	Stephen Lee
	Unit / No: / Street	CPEng No:	1006840
Building Address:	50 Westlake Drive, Halswell	Company:	GHD
Legal Description:	Lot 296 DP 61814	Company project number:	51/30902/41
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:	43 / 34 / 15.14	Date of submission:	7/03/2013
GPS east:	172 / 33 / 21.00	Inspection Date:	13/12/2012
		Revision:	FINAL
Building Unique Identifier (CCC):	PRK 0489 BLDG 001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	20	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):	20.15
Ground floor split?	no	Ground floor elevation above ground (m):	0.15
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	4.60	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	55		
Age of Building (years):	30	Date of design:	1976-1992
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 and Changing Rooms		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	metal cladding on timber truss
Floors:	concrete flat slab	slab thickness (mm)	Slab on grade
Beams:	timber	type	
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	20 series concrete block masonry

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	fully filled CMU	note total length of wall at ground (m):	
Ductility assumed, μ:	1.25	wall thickness (m):	
Period along:	0.40 ##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):		estimate or calculation?	
maximum interstorey deflection (ULS) (mm):		estimate or calculation?	
Lateral system across:	fully filled CMU	note total length of wall at ground (m):	
Ductility assumed, μ:	1.25	wall thickness (m):	
Period across:	0.40 ##### enter height above at H31	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:	tile (internal wall cladding)	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
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:	No ground damage 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:	No placard in place		
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:	No damage observed
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	No damage observed

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

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	50% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Detailed Assessment
	Assessed %NBS after:	50%		
Across	Assessed %NBS before:	52% ##### %NBS from IEP below		
	Assessed %NBS after:	52%		

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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		7/3/13