Birdsey Reserve Lockup Shed
PRK 1742 BLDG 003 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

40 Bridle Path Road, Heathcote Valley, Christchurch

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Valley, Christchurch

Christchurch City Council

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Date
05 April 2013

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Birdsey Reserve Lockup Shed

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

40 Bridle Path Road, Heathcote Valley, Christchurch

Background

This is a summary of the Quantitative report for the Birdsey Reserve Lockup Shed, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and inspections on the 05th September 2012.

Brief Description

The Lockup Shed at Birdsey Reserve is estimated to have been constructed in the 1970s. Plans for the shed were not available, so construction details have been observed from the site. The site is located on sloping land at the Birdsey Reserve in Heathcote Valley near Christchurch.

The site slopes to the west towards Bridle Path Road, with some variation in the topography in gently rolling hills. The site lies at the bottom of the Port hills, with some large boulders scattered around the nearby area.

The building is a single-storey shed with partially demolished concrete masonry storage bins adjacent to the south end of the shed. The single-storey shed is rectangular in plan with a pitched roof. The roof structure consists of timber trusses with timber purlins, clad in corrugated steel, with minimal timber cross-bracing between trusses. There is no internal lining to the ceiling. The roof structure is supported by external partially-filled external concrete masonry walls of 140mm thickness, with a window in the north wall, and a window and an entry door to the west wall. The south external wall extends to the west past the boundary of the existing shed, where the previous external boundary once stood before partial demolition. The floor is an on-grade concrete slab. The external walls are supported by low foundation walls around all four sides, and are assumed to be supported by reinforced concrete strip footings. There is an exterior concrete slab-on grade to the east of the shed used for timber storage. The dimensions of the building are approximately 5.7m long by 4.2m wide and 3.0m in height.

Indicative Building Strength

From a detailed assessment of all the walls, the building, excluding any structural damage, has been assessed as achieving 43 **%**NBS. Significant earthquake damage has been noted to the masonry walls in the north-west corner of the building that constitutes a failure of those structural elements. As a result of the failure of these elements the building has achieved a % NBS of 0%. Under the New Zealand

Society for Earthquake Engineering (NZSEE) guidelines the lockup shed is considered to be an Earthquake Prone building.

Recommendations

As the lockup shed has been assessed to have a %NBS less than 33%, it is deemed to be an Earthquake Prone building. It is recommended that replacement of critical elements or strengthening options be explored to bring the % NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Prone buildings.

1. Background

GHD Limited has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Birdsey Reserve Lockup Shed.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Concrete Masonry Buildings for Earthquake Resistance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006).

The quantitative assessment to the building comprises an investigation on in-plane and out-of-plane strength of the unreinforced masonry block walls of the lockup shed. The investigation is based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements is compared to the demand placed on the element to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the masonry walls. No reinforcing was identified and as such the building is being treated as unreinforced.

At the time of this report, no finite element modelling of the building structure has 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 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.

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

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The Lockup Shed at Birdsey Reserve is estimated to have been constructed in the 1970s. Plans for the shed were not available, so construction details have been observed from the site. The site is located on sloping land at the Birdsey Reserve in Heathcote Valley near Christchurch. The surrounding area mostly consists of hilly, rural lands with few structures.

The site slopes to the west to Bridle Path Road, with some variation in the topography in gently rolling hills. The site lies at the bottom of the Port hills, with some large boulders scattered around the nearby area.

The building is a single-storey shed with partially demolished concrete masonry storage bins adjacent to the south end of the shed. The concrete foundations and on-grade concrete slab of the storage bins have been left in place.

The single-storey shed is rectangular in plan with a duo-pitch roof. The roof structure consists of timber trusses with timber purlins, clad in corrugated steel, with minimal timber cross-bracing between trusses. There is no internal lining to the ceiling. The roof structure is supported by external partially-filled concrete masonry walls of 140mm thickness, with a window in the north wall, and a window and an entry door to the west wall. The south external wall extends to the west past the boundary of the existing shed, where the previous external boundary once stood before partial demolition. The floor is an on-grade concrete slab. The external walls are supported by low foundation walls at all four sides, and are assumed to be supported by reinforced concrete strip footings. There is a concrete slab-on grade to the east of the shed used for timber storage. The dimensions of the building are approximately 5.7m long by 4.2m wide and 3.0m in height.

A plan sketch is provided in the following Figure 2 to illustrate the main structural members of the building.

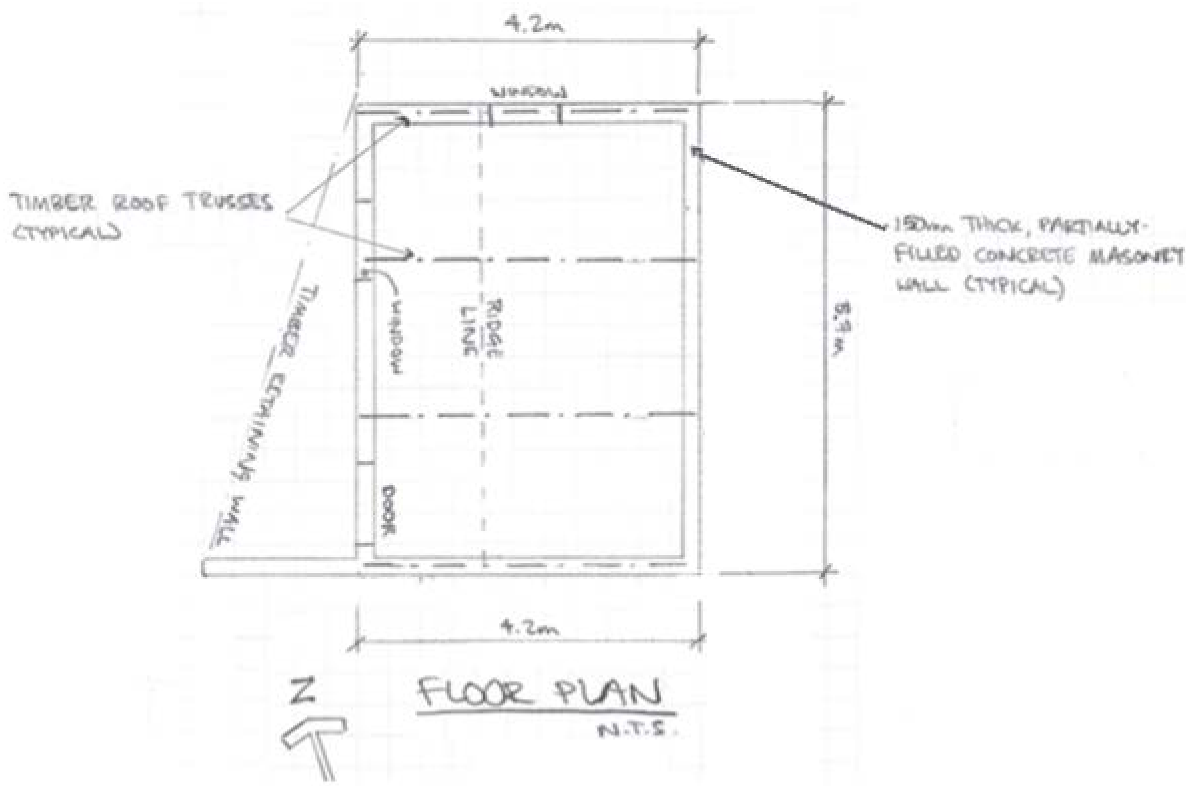

Figure 2 Plan of the building showing key structural elements

4.2 Gravity Load Resisting System

The gravity loads in the structure are carried through the steel roof cladding to the timber roof purlins, down through the timber trusses, out to the concrete masonry walls, down through the foundation walls to the concrete footings and into the ground. At the entrance of the building and the windows, gravity loads from the roof structure are carried across the openings by concrete masonry bond beams.

4.3 Lateral Load Resisting System

In both the longitudinal and transverse directions, the roof cladding, purlins, trusses and cross-bracing will combine to provide adequate diaphragm action to transfer lateral loads from the roof structure and into the longitudinal or transverse external walls. Out-of-plane seismic loads will be propped by the roof diaphragm and carried to other walls for in-plane loading to the building. Lateral loads will then be transferred through the external concrete masonry walls to the foundations, where the loads are dispersed into the founding soils. The longer transverse external wall and foundations on the south side of the building will provide additional lateral load resistance in the transverse direction.

5. Damage Assessment

5.1 Surrounding Buildings

The Lockup Shed at Birdsey Reserve is located in a rural area with open lands adjacent to the site. The nearest buildings are more than 100m away.

5.2 Residual Displacements and General Observations

Minor non-structural damage to the roof gutters on the west and east sides of the building were noted on-site.

Significant structural damage has occurred to the western and northern walls of the building. Displacement of the walls around the windows has occurred which constitutes a failure of these structural elements (Refer to photographs 12 and 13).

5.3 Ground Damage

No ground damage was observed during our inspection of the site.

6. Geotechnical Consideration

This desktop geotechnical study outlines the ground conditions, as indicated from sources quoted within. This is a desktop study report and no site visit has been undertaken by Geotechnical personnel.

This report is only specific to the existing structures adjacent to the car park, at Detailed Engineering Evaluations. The Reserve is located off Bridle Path Road and is bounded by farmland to the east and residential properties to the south. The Heathcote Domain is located opposite the site on Bridle Path Road. The property is owned and maintained by the Christchurch City Council.

6.1 Site Description

The site is situated within a recreational reserve, within the suburb of Heathcote in south-eastern Christchurch. It is located at the base of the Port hills at approximately 35m above mean sea level. The site is approximately 2km south of the Heathcote River, 4km west of New Brighton Spit, and 2km north of Lyttelton.

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is on or near the boundary of the following units:

- Valley fill and slope wash of loess-volcanic derived colluvium, Holocene in age; and,
- Dark grey to black, plagioclase-pyroxene-olivine phyric basalt through to grey-green trachyte interbedded with pyroclastic deposits, Miocene in age.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that no boreholes are located within a 200m radius of the site.

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 published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes.

The site is classified as Technical Category Not Applicable (TC N/A), being sites within non-residential properties in urban areas, properties in rural areas or beyond the extent of land damage mapping, and properties in the Port Hills have not been given a Technical Category.

[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.

6.2.5 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 3.

Figure 3 Post February 2011 Earthquake Aerial Photography [2]

6.2.6 Summary of Ground Conditions

From the information presented above, it is anticipated the geology comprises volcanic rock overlain by slope wash of loess-volcanic colluvium.

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 in Table 2.

[2] 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[3,4]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale (2010) Fault	27 km	W	7.1	~15,000 years
Hope Fault	120 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	~150 years
Porters Pass Fault	70 km	NW	7.0	~1100 years

Recent earthquakes since 22 February 2011 have identified the presence of a new active fault system / zone underneath 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.

In addition, with a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[3]), and volcanic bedrock being relatively shallow with a loess/volcanic colluvium cover at this location, ground shaking is not expected to be significantly amplified by the ground conditions.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location on the Port hills, and proximity to slopes that have undergone and are susceptible to rockfall, it is considered that the site has a moderate to high rockfall potential.

In addition any localised retaining structures should be further investigated to determine the site-specific slope instability potential.

6.5 Liquefaction Potential

Due to the anticipated presence of volcanic rock and loess-volcanic colluvium beneath the site, it is considered that site is of low susceptibility to liquefaction.

[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

6.6 Recommendations

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

If a more detailed assessment is required, intrusive investigation comprising one machine borehole to 15m bgl should be undertaken.

6.7 Conclusions & Summary

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

The site appears to be situated on volcanic and loess colluvium. Associated with this the site also has a low liquefaction potential.

The site has a moderate to high rockfall potential.

Should a more comprehensive ground condition assessment be required, it is recommended that an intrusive investigation be conducted comprising one machine drilled borehole, with SPT tests at 1.5m intervals.

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

7. Assessment

7.1 Quantitative Assessment

The quantitative assessment of the building comprised of an investigation on the in-plane and out-of-plane strength of the masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements was compared to the demand placed on the elements to give the %NBS of each of the structural elements. A full methodology of the calculation process is attached in Appendix C.

7.1.1 Demand

The in-plane shear demand of each wall was assessed by completing a torsion analysis to the building. NZS 1170.5:2004 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of 10% of the building dimension which is perpendicular to the force applied. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to the direct shear. As each wall was made of the same material and with the same properties, the direct shear and the force induced in each wall are proportional to the length squared. Cl 5.3.1.2 of NZS 1170.5: 2004 states that for nominally ductile and brittle structures an action set of 100% of the earthquake actions in one direction and 30% in the orthogonal direction must be applied when calculating the demand for any structural member and has such been applied in the analysis.

7.1.2 Seismic Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1);

$$\mathrm{C(T) = C_h\ Z\ R\ N(T.D)}$$

Where

$C_h(T)$ = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/100 for an Importance Level 1 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

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

$$\mathrm{S_P} = 1.33 - 0.3\mu$$

The structural ductility factor, μ, was taken as 1.00 for the out of plane assessment and 2.00 for the in plane analysis of walls as suggested by the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (02/2011).

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 0.1 was assumed for 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

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 \text{ For out of plane assessment}$$

$$k_\mu = 1.2 \text{ For in plane assessment}$$

7.1.3 In-Plane Capacity of Unreinforced Walls

The in-plane capacity of unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (02/2011). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (02/2011).

7.1.4 In-plane Wall Shear Capacity of Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

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

7.1.5 Out-of-Plane Capacity of Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006) Section 10.3.

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8. Initial Capacity Assessment

8.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class assumed to be: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil;
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011;
- Return period factor R_u = 0.5, NZS 1170.5:2004, Table 3.5, Importance Level 1 structure with a 50 year design life.

8.2 Wall Investigation

The position of each wall is indicated in the plans below and each wall is named accordingly.

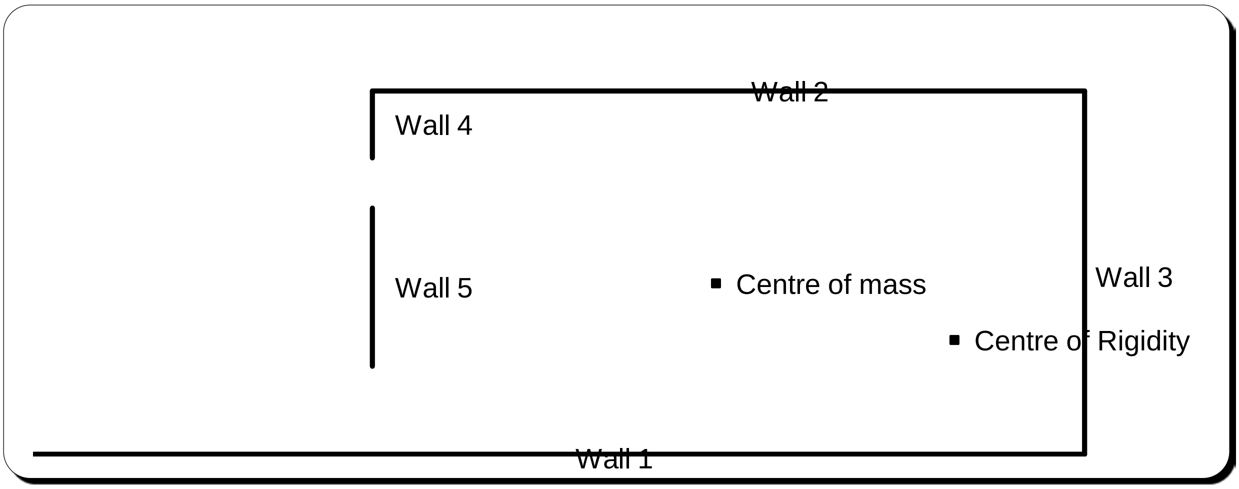

Figure 4 Plan Details and Wall Locations of Lockup Shed

8.3 Lockup Shed Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 3 .

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	15.1	14.5	95.9%	Not Risk or Prone	36.3	59.8	165%	Not Risk or Prone
2	12.4	9.5	76.62%	Not Risk or Prone	29.8	27.4	92%	Not Risk or Prone
3	17.1	14.9	86.71%	Not Risk or Prone	41.1	62.7	152%	Not Risk or Prone
4	1.0	0.7	74.81%	Not Risk or Prone	2.4	2.1	90%	Not Risk or Prone
5	5.5	4.1	74.81%	Not Risk or Prone	13.2	11.9	90%	Not Risk or Prone

Table 3 In Plane Analysis Results

The results of the out of plane displacement response capability analysis and subsequent earthquake designation under the NZSEE guidelines are listed in Table 4.

Wall number	Δ_i	D_{ph} kN	%NBS	Earthquake Status
1	0.068	0.113	43.9%	Risk
2	0.068	0.113	43.9%	Risk
3	0.068	0.114	42.9%	Risk
4	0.068	0.114	42.9%	Risk
5	0.068	0.114	42.9%	Risk

Table 4 The Lockup Shed Out Of Plane Analysis Results

8.4 Percentage of New Building Standard (%NBS)

The critical % NBS for each wall is listed below in Table 5.

Wall number	Minimum %NBS	Earthquake status
1	43.9%	Risk
2	43.9%	Risk
3	42.9%	Risk
4	42.9%	Risk
5	42.9%	Risk

Table 5 %NBS Results

8.5 Discussion of Results

Following a detailed assessment, the Lockup Shed has been assessed as achieving 43 **%**NBS excluding any structural damage. Significant damage to the walls of the north-west corner is considered to constitute a failure of these structural elements and as such they have achieved 0% NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the lockup shed is considered to be an Earthquake Prone building.

9. Strengthening

As the %NBS of the Lockup Shed has been assessed at 0%, additional strengthening or replacement works are required to increase the capacity of the walls to achieve the minimum 67% as recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

10. Recommendations

As the Lockup Shed has been assessed to have a %NBS less than 33% NBS, it is deemed to be Earthquake Prone. It is recommended that wall strengthening or replacement options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were unavailable. As a result the information contained in this report has been inferred from visual inspections of the building and site only.
- No intrusive structural investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 7 have been carried out on the structure.

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 Limited accepts no responsibility for any other party or person who relies on the information contained in this report.

11.2 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: Western elevation.

Photograph 2: Southern Elevation.

Photograph 3: Southeast corner elevation.

Photograph 4: Northern wall damaged by tree roots.

Photograph 5: Small concrete pad and extended southern external wall at western wall entry door.

Photograph 6: Leftover concrete pad and southern external wall.

Photograph 7: Leftover external concrete pad adjacent to southern external wall.

Photograph 8: Concrete pad at eastern side of structure used for timber storage.

Photograph 9: Interior view of timber roof trusses and external concrete masonry walls.

Photograph 10: Interior view of timber trusses and external concrete masonry walls.

Photograph 11: Typical timber roof truss attachment to top plate on external concrete masonry walls.

Photograph 12: Interior view of displacement to western wall by window adjacent to the north western corner of the building.

Photograph 13: Interior view of damage to north eastern wall.

Appendix B

Existing Drawings

Note: no existing drawings for this building were able to be located.

Appendix C

Calculation Methodology

a. Quantitative Assessment

The quantitative assessment to the building comprised an investigation of in-plane and out-of-plane strength of the masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements was compared to the demand placed on the element to give the %NBS of each of the structural elements.

b. Demand

The in-plane shear demand of each wall was assessed by completing a torsion analysis to the building. NZS 1170.5:2004 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of 10% of the building dimension which is perpendicular to the force applied. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to the direct shear. As each wall was made of the same material and with the same properties, the direct shear and the force induced in each wall are proportional to the length squared. Cl 5.3.1.2 of NZS 1170.5: 2004 states that for nominally ductile and brittle structures an action set of 100% of the earthquake actions in one direction and 30% in the orthogonal direction must be applied when calculating the demand for any structural member and has such been applied in the analysis.

c. Seismic Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1);

$$\mathrm{C(T) = C_h\, Z\, R\, N(T.D)}$$

Where

$C_h(T)$ = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

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

$$\mathrm{S_P = 1.33 - 0.3\mu}$$

Where μ, the structural ductility factor, was taken as 1.00.

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 0.1 was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

d. In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

e. In-plane Wall Properties of the Unreinforced Walls

Properties of in-plane loaded URM walls, piers or spandrels for use in the calculation of nominal in-plane shear capacity were as follows:

- **Unit Weight of Masonry**

2.10 kN/m^2 was adapted for the unit weight of 20-series concrete hollow block masonry with standard aggregate (see Table A2 from NZS 1170.1:2002).

- **Weight of Wall**

The weight of the wall, W_w, was calculated in accordance with the equation.

$$W_w = 1.82 \times l_w \times h$$

Where: l_w is the total wall length and h is the wall height.

- **Normal Force at Base of Wall**

The normal force acting on the cross section of the base of the wall, N_b, was calculated in accordance with the equation.

$$\mathrm{N_b = W_w + N_t}$$

Where: Values for weight of the wall, W_w, and axial load above the wall, N_t.

- **Diagonal Tension Strength**

The diagonal tension strength of masonry, f_{dt}, was calculated in accordance with the equation below for walls, piers and spandrels.

$$\mathrm{f_{dt}} = \frac{1}{2}\left(\mathrm{c} + \frac{\mathrm{N_t}}{\mathrm{A_w}} 0.8\mu_\mathrm{f}\right)$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Distance to Centre of Inertia of Wall**

Distance to the centre of inertia of the wall from the compression toe, a_i, was calculated in accordance with the equation for walls with no flanges:

$$a_i = 0.5 \times l_w$$

- **Average Compressive Stress**

Average compressive stress acting on the wall, σ_{ave}, was calculated in accordance with the equation

$$\sigma_{avg} = \frac{N_t}{l_w . b_w}$$

Where: Value for width of the block shell, b_w which was equivalent to 0.45 of the block width. This reduced value of b_w was calculated by multiplying the actual width by a modification factor based on the difference between the unit density of the block compared to the unit density of concrete.

$$b_w = \text{width x } \frac{\rho_{block}}{\rho_{concrete}}$$

f. In-plane Wall Shear Capacity of the Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

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

Nominal capacity of each failure mode was derived as following:

- **Capacity in Diagonal Tension Failure Mode, V_{dt}**

Nominal shear capacity corresponding to diagonal tension failure, V_{dt}, was calculated in accordance with the equation below for walls where no perpendicular flanges are present

$$V_{dt} = 0.54 . b_w . l_w . \zeta . f_{dt} . \sqrt{\left(1 + \frac{\sigma_{avg}}{f_{dt}}\right)}$$

Where: ζ was a factor to correct for nonlinear stress distribution (See Table 6)

	ζ
Slender walls, where $h/l_w > 2$	1.5
Stout walls, where $h/l_w < 0.5$	1.0
Linear interpolation may be used for values of h/l_w	

Table 6 Shear stress factor for inclusion in diagonal tension failure mode equation

- **Capacity in Rocking Failure Mode, V_r**

Nominal shear capacity corresponding to the rocking failure mode, V_r, was calculated in accordance with the equation;

$$V_r = \frac{N_b}{h} . \left[a_i - \frac{l_{er}}{3}\right]$$

Where: l_{er} was the effective length of the wall in rocking, taken as 0.1 x l_w.

- **Capacity in Bed-joint Sliding Failure Mode, V_s**

Bed-joint sliding failure was not an expected behaviour of URM walls subjected to seismic loading. The bed-joint sliding capacity of an in-plane loaded wall needed only be assessed when conditions suited the initiation of bed-joint sliding, specifically, when either or both the brick compressive strength and mortar compressive strength fell in the bounds of “soft”.

Ultimate shear capacity corresponding to bed-joint sliding failure, V_s, was calculated in accordance with the equation

$$V_s = l_w . b_w . c + 0.8. \mu_f . N_t$$

Where: Values for cohesion, *c*, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Capacity in Toe Crushing Failure Mode, V_{tc}**

Nominal shear capacity corresponding to toe crushing failure, V_{tc}, was calculated in accordance with the below equation for walls where perpendicular flanges were present:

$$V_{tc} = \frac{N_b}{h} . \left[\frac{1}{2} . l_w - \frac{1}{3} . l_{etc} \right]$$

Where the effective length of wall was calculated as:

$$l_{etc} = \frac{2. N_b}{1.3. f'_m . b_w}$$

g. Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes Section 10.3. The following steps were those required to assess the displacement response capability and the displacement demand, from which the adequacy of the walls can be determined.

The wall panel was assumed to form hinge lines at the points where effective horizontal restraint was assumed to be applied. The centre of compression on each of these hinge lines was assumed to form a pivot point. The height between these pivot points was the effective panel height *h*. At mid-height between these pivots, a third pivot point is assumed to form.

Step 1

The wall panel was divided into two parts, a top part bounded by the upper pivot and the mid-height between the top and bottom pivots, and a bottom part bounded by the mid-height pivot and the bottom pivot.

Step 2

The weight of the wall parts, W_b of the bottom part and W_t of the top part, and the weight acting at the top of the storey, *P* were calculated.

Step 3

From the nominal thickness of the wall, t_{nom}, the effective thickness, *t* was calculated as follows:

$$t = t_{nom} \left(0.975 - 0.025 \frac{P}{W} \right)$$

Step 4

The eccentricity values e_p, e_b, e_t and e_o were calculated. Usually, the eccentricities e_b and e_p will each vary between 0 and *t/2* (where *t* is the effective thickness of the wall). Exceptionally they may be negative.

Where,

e_p = eccentricity of the P measured from the centroid of W_t

e_t = eccentricity of the mid-height pivot measure from the centroid of W_t

e_b = eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b

e_o = eccentricity of the mid-height pivot measured from the centroid of W_b

Step 5

The mid-height deflection, Δ_i was calculated, which would cause instability under static conditions. The following formula was used to calculate this deflection.

$$\Delta_i = \frac{b\mathrm{h}}{2a}$$

Where

$$b = W_b e_b + W_t(e_0 + e_b + e_t) + P(e_0 + e_b + e_t + e_p) - \Psi(W_b y_b + W_t y_t)$$

And

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph$$

And

$$\Psi = Initial\ slope\ of\ wall$$

Step 6

The maximum usable deflection, Δ_m was calculated as 0.6 Δ_i.

Step 7

The period of the wall, T_p, was four times the duration for the wall to return from a displaced position measured by Δ_m to the vertical. The period was calculated from the following equation:

$$T_p = 6.27\sqrt{\frac{J}{a}}$$

Where J was the rotational inertia of the masses associated with W_b, W_t and *P* and any ancillary masses, and was given by the following equation.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b[{e_b}^2 + {y_b}^2] + W_t[(e_0 + e_b + e_t)^2 + {y_t}^2] + P\left[(e_0 + e_b + e_t + e_p)^2\right]\right\} + J_{ancillary}$$

Where;

$$J_{bo} = J_{to} = \frac{\left\{\left(\frac{W}{h}\right)[h^2 + 16t^2] + 4Pt^2\right\}}{g}$$

Where y_t was the distant from the top of the wall to the centroid of the top wall and y_b was the distant from the bottom of the wall to the centroid of the bottom wall.

Step 8

The seismic coefficient ($C_p(T_p)$) for an elastically responding part ($\mu_p = 1$) with this period (T_p), was calculated as follows:

$$C_p(T_p) = C(0)C_{Hi}C_i(T_p)$$

Where

C(0) = the site hazard coefficient for T = 0 determined from NZS 1170.5 Section 3.1, using the values for the modal response spectrum method and numerical integration time history methods

C_{Hi} = the floor height coefficient for level I, from NZS 1170.5 Section 8.3.

$C_i(T_p)$ = the part spectral shape factor at level I, from NZS 1170.5 Section 8.4

Step 9

The participation factor, γ for the rocking system was taken as:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg}$$

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ, the displacement response, D_{ph} was obtained from;

$$D_{ph} = \gamma \left(\frac{T_p}{2\pi}\right)^2 \times C_p(T_p) \times R_p \times g$$

Where R_p was from NZS 1170.5 Table 8.1

Appendix D

CERA Building Evaluation Form

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
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