Mona Vale Implement Shed/Staff Rooms
PRK_0555_BLDG_009 EQ2

Detailed Engineering Evaluation

Qualitative Report

Version FINAL

40 Mona Vale Avenue, Fendalton

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

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40 Mona Vale Avenue, Fendalton

Christchurch City Council

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Date
1/10/13

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Mona Vale Implement Shed/Staff Rooms

PRK_1390_BLDG_001 EQ2

Detailed Engineering Evaluation

Qualitative Report - SUMMARY

Version FINAL

40 Mona Vale Avenue, Fendalton

Background

This is a summary of the Qualitative 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 11th April 2012 only.

Building Description

Mona Vale Implement Shed/Staff Rooms is located at 40 Mona Vale Avenue, Fendalton, Christchurch. The single story building is currently used as a garage for storing vehicles and equipment. There is a small staff room in the north-western corner of the building. The building was constructed in 2006. The gravity loads acting on the structure are resisted by timber trusses and timber framed load-bearing walls. Lateral loads acting on the structure of the building are resisted by braced timber framed walls on all four sides.

Key Damage Observed

No damage to the building was observed during our inspection.

Critical Structural Weaknesses

Liquefaction has a high probability of reoccurring on site, however due to the nature of the structure (timber framed, single storey) it is unlikely to cause premature collapse of the building. It should be noted that further liquefaction on site may cause differential settlement of the building and further damage, however in terms of the Detailed Engineering Evaluation it has been assessed as an “insignificant” site characteristic in accordance with NZSEE guidelines.

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the original capacity of the building has been assessed to be in the order of 85% NBS and post-earthquake capacity also in the order of 85% NBS. The building’s post-earthquake capacity excluding critical structural weaknesses is in the order of 85% NBS.

The building has been assessed to have a seismic capacity in the order of 85% NBS and is therefore not potentially Earthquake Risk or Earthquake Prone.

Recommendations

It is recommended that the building remains in use.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Mona Vale Implement Shed/Staff Rooms.

This report is a Qualitative 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 qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

At the time of this report, no intrusive site investigation, detailed analysis, or modelling of the building structure had been carried out. Construction drawings were not made available. The building description below is based on our visual inspections only.

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.

<table>
<tr><th rowspan="2">Description</th><th rowspan="2">Grade</th><th rowspan="2">Risk</th><th rowspan="2">%NBS</th><th rowspan="2">Existing Building Structural Performance</th><th colspan="2">Improvement of Structural Performance</th></tr>
<tr><th>Legal Requirement</th><th>NZSEE Recommendation</th></tr>
<tr><td>Low Risk Building</td><td>A or B</td><td>Low</td><td>Above 67</td><td>Acceptable (improvement may be desirable)</td><td rowspan="2">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.</td><td>100%NBS desirable. Improvement should achieve at least 67%NBS</td></tr>
<tr><td>Moderate Risk Building</td><td>B or C</td><td>Moderate</td><td>34 to 66</td><td>Acceptable legally. Improvement recommended</td><td>Not recommended. Acceptable only in exceptional circumstances</td></tr>
<tr><td>High Risk Building</td><td>D or E</td><td>High</td><td>33 or lower</td><td>Unacceptable (Improvement</td><td>Unacceptable</td><td>Unacceptable</td></tr>
</table>

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

Mona Vale Implement Shed/Staff Rooms is located at 40 Mona Vale Avenue, Fendalton, Christchurch. The single story building is currently used as a garage for storing vehicles and equipment. There is a small staff room in the north-western corner of the building. The building was constructed in 2006 based information observed on site.

Key structural details of the building are shown in Figure 2 below.

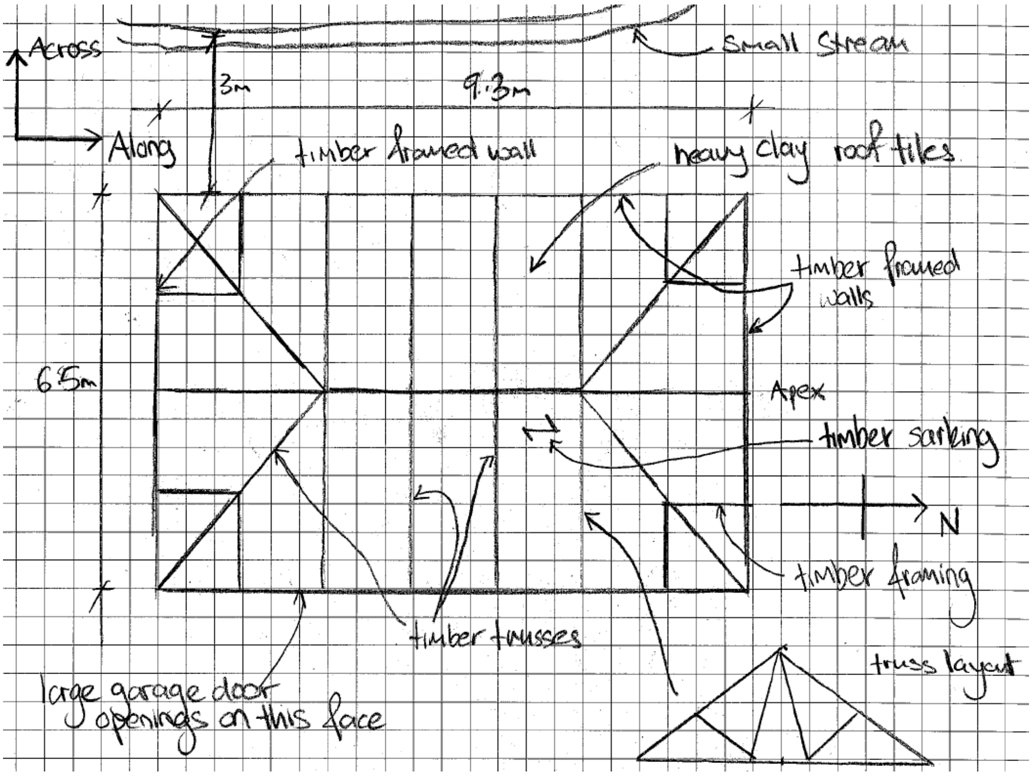


Figure 2 Plan sketch showing key structural elements

The dimensions of the building are approximately 6.5 m wide by 9.3 m long and 6 m tall. The overall footprint of the building is approximately 60 m^2.

4.2 Gravity Load Resisting System

The gravity loads acting on the structure are resisted by timber trusses and timber framed load-bearing walls.

Gravity loads from the heavy clay tile roof are supported by timber sarking over timber trusses spanning across the building (trusses shown in Photographs 3, 5 and 8). The truss members are connected by steel nail plates as shown in Photograph 5. The timber trusses are connected to the timber top plates of the timber framed walls on all four sides of the building. This is shown in Photograph 7. Gravity loads are transferred through the load-bearing timber walls and into the foundations. The foundations are likely to consist of a reinforced concrete slab on grade with reinforced concrete strip footings beneath the timber framed walls. It should be noted that intrusive investigations of the foundations for the building were not carried out.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure the building are resisted by braced timber framed walls on all four sides of the structure. Lateral forces are distributed from the roof structure to the timber framed walls through diaphragm action of the timber sarking over the roof trusses.

Diagonal steel strap bracing is present in the northern, southern and western walls of the building. A section of the steel strap bracing on the southern wall is shown in Photograph 4. The eastern wall has two large openings to allow vehicle access to the garage. The openings can be seen in Photograph 1. The sections of wall between the openings on this side are braced on the internal face with plywood. As the plywood bracing is reasonably stiff it is expected the eastern wall will have a similar stiffness to the other walls despite the relatively small sections of braced walls. As a result, plan irregularity has been considered insignificant for the building. All bracing details appear to be consistent with the design standard NZS 3604:1999.

5. Assessment

An inspection of the building was undertaken on the 11th of April 2012. Both the interior and exterior of the building was inspected. No inspection of the foundations of the structure was able to be undertaken.

The inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including observing the ground conditions, checking for damage in 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.

The %NBS score determined for the building has been based the IEP procedure described by the NZSEE based on the information obtained from visual observation of the building only.

6. Damage Assessment

6.1 Surrounding Buildings

No damage to surrounding buildings was observed during our inspection of the site.

6.2 Residual Displacements and General Observations

No damage to the building and no residual displacements of the structure were observed during our inspection of the building.

6.3 Ground Damage

Some cracking in the asphalt paving to the east and north-east of the structure was observed. It is unlikely that these cracks have occurred during seismic activity as they are filled with dirt and do not appear recent.

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

7. Critical Structural Weakness

7.1 Short Columns

No short columns were observed in the building.

7.2 Plan Irregularity

Lateral forces are distributed from the roof structure to the timber framed walls through diaphragm action of the timber sarking over the roof trusses.

Diagonal steel strap bracing is present in the northern, southern and western walls of the building. The eastern face of the building has two large garage doors to allow vehicles to park inside the garage. The timber framed walls between the openings are braced with plywood. The detailing of the internal plywood lining bracing appears to be in accordance with NZS 3604:1999. As a result, it is expected that the eastern face of the building is adequately braced. The building is therefore regularly braced and is not considered to have plan irregularity.

7.3 Roof

No critical structural weaknesses were identified in the roof structure. The timber roof structure is regular and symmetrical. Timber sarking over the timber roof trusses provides some diaphragm action, allowing lateral forces in the roof structure to be distributed to the timber framed walls.

7.4 Staircases

The building does not contain a staircase.

7.5 Liquefaction Potential

Liquefaction has a high probability of reoccurring on site, however due to the nature of the structure (timber framed, single storey) it is unlikely to cause premature collapse of the building. It should be noted that further liquefaction on site may cause differential settlement of the building and further damage, however in terms of the Detailed Engineering Evaluation it has been assessed as an "insignificant" site characteristic in accordance with NZSEE guidelines.

8. Geotechnical Consideration

8.1 Site Description

The site is situated within a recreational reserve, within the inner suburb of Fendalton, Christchurch. It is relatively flat at approximately 10m above mean sea level. It is approximately 60m west and 100m south of the Avon River, and 12km west of the coast (Pegasus Bay).

8.2 Published Information on Ground Conditions

8.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates numerous boreholes are located within a 200m radius of the site. Of these boreholes, four were considered to have adequate lithographic logs (see Table 2). These logs indicate the site geology to be a surface layer of clays and sands underlain by stratified gravel and sand deposits.

The logs also indicate the presence of organic layers (Peat) to be present between 17 and 40m bgl.

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/1985	~99m	~1.8m bgl	~120m SE
M35/2064	~59.4m	~0.9m bgl	~230m NW
M35/2096	~55.1m	~1.4m bgl	~130m SW
M35/2504	~55.1m	~0.9m bgl	~160m NE

Table 2 ECan Borehole Summary

It should be noted that the purpose of the boreholes the well logs are associated with, were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

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

8.2.3 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 Fendalton[2]. One investigation point was undertaken approximately 250m east of the site, as summarised below in Table 3.

Bore Name	Grid Reference	Log Summary	
CPT-FND-01	2478670 mE	0 – 1	Surface Soil
	5742558 mN	1 – 2.5	SAND, with minor silt; medium dense to dense
		2.5 – 8	Silty CLAY and clayey SILT
		8 – 9	SAND, with minor silt and clay; medium dense
		9 – 10	CLAY
		10 – 17.7	Silty SAND and SAND; medium dense to dense
			(WT at 1.5m bgl)

Table 3 EQC Geotechnical Investigation Summary Table

Initial observations of the CPT results indicate the soils are layers of sand, silt and clay, of varying density and strength.

In order to get better understanding of soil conditions, one CPT was undertaken on 02 April 2012. The testing results are summarised in Table 4 below:

Bore Name	Grid Reference	Depth (m)	Lithology
CPT-29	2478451 mE	0 – 1.5	Peat/Clay/Silty Clay
	5742533 mN	1.5 – 5.0	Sandy silt / Silty sand
		5.0 – 12.5	Clay/Sensitive fine grained soil
		12.5 –18.0	Sand/Silty Sand

Table 4 Summary of CPT undertaken 02 April 2012 – Inferred Lithology

8.2.4 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.

[2] Tonkin and Taylor . September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Fendalton

The site is classified as Technical Category Not Applicable (TC N/A), being non-residential properties in urban areas, properties in rural areas or those beyond the extent of land damage mapping.

8.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of moderate liquefaction outside the building footprint, and significant liquefaction in the Christchurch Girls' High School grounds adjacent to the site, as shown in Figure 3.

Figure 3 Post February 2011 Earthquake Aerial Photography [3]

8.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions have been found to comprise strata of sand, silt and clay to ~18m bgl, underlain by layers of sand and gravel. Intermittent bands of organic soils (peat) are also present between 17 and 40m bgl.

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

8.3 Seismicity

8.3.1 Nearby Faults

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	20 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	150 years
Porters Pass Fault	60 km	NW	7.0	1100 years

Table 5 Summary of Known Active Faults[4][5]

Recent earthquakes since 22 February 2011 have identified the presence of a previously unmapped active fault system 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.

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.

8.3.2 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.

In addition, the geology is anticipated to be stratified alluvial deposits of varying density, and the site has a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[4]). Bedrock is anticipated to be in excess of 500m deep, and hence ground shaking is expected to be moderate to high.

8.4 Slope Failure and / or Rockfall Potential

The site is located in the typically flat suburb of Fendalton. Global slope instability is considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

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

[5] GNS Active Faults Database

In addition, the site is located adjacent to the Avon River. Should significant liquefaction occur in the area, it is considered possible and likely that lateral spreading will occur along the river. This is reinforced by evidence of lateral spreading in nearby areas following the February earthquake.

8.5 Liquefaction Potential

Due to the presence of sands and silts, and evidence from the post-earthquake aerial photography, it is considered highly possible that liquefaction will occur in layers and locations where sands and silts are present.

8.6 Conclusions & Summary

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010, as well as on the results based on the piezo CPT site test undertaken on 02 April 2012.

The site appears to be situated on stratified alluvial deposits, comprising sand and silt. Associated with this the site also has a moderate to severe 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.

9. Survey

No level or verticality surveys have been undertaken for this building at this stage as indicated by Christchurch City Council guidelines.

10. Initial Capacity Assessment

10.1 % NBS Assessment

The building has had its capacity assessed using the Initial Evaluation Procedure based on the information available. The buildings capacity excluding critical structural weaknesses and the capacity of any identified weaknesses are expressed as a percentage of new building standard (%NBS) and are in the order of that shown below in Table 6. These capacities are subject to confirmation by a more detailed quantitative analysis.

Item	%NBS
Building excluding CSW's	85

Table 6 Indicative Building and Critical Structural Weaknesses Capacities based on the NZSEE Initial Evaluation Procedure

Following an IEP assessment, the building has been assessed as achieving 85% New Building Standard (NBS). The building is therefore not considered potentially Earthquake Risk or Earthquake Prone as it achieves greater than 67% NBS. This score has not been adjusted when considering damage to the structure as no damage was observed during our inspection.

10.2 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: 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 = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing resulting in a reduced % NBS score.

10.3 Expected Structural Ductility Factor

A structural ductility factor of 3.0 has been assumed both along and across the building based on the timber frame system observed and the date of construction. The building was likely built to the design standard NZS 3604:1999 which uses a structural ductility factor of 3.0.

10.4 Discussion of Results

The results obtained from the initial IEP assessment are consistent with those expected for a building of this age and construction type founded on Class D soils. The building was constructed in 2006 based on information observed on site during our inspection. The increase in the hazard factor for Christchurch to 0.3 further reduces the %NBS score.

Liquefaction has a high probability of reoccurring on site, however due to the nature of the structure (timber framed, single storey) it is unlikely to cause premature collapse of the building. In terms of the IEP, The overall %NBS score for the building has not been reduced as liquefaction potential has been assessed as an “insignificant” site characteristic in accordance with NZSEE guidelines.

10.5 Occupancy

The building has not been assessed as potentially Earthquake Risk or Earthquake Prone and no damage to the structure was observed. As a result it is recommended that the building remain in use.

11. Initial Conclusions

The building has been assessed to have a seismic capacity in the order of 85% NBS and is therefore not potentially Earthquake Risk or Earthquake Prone.

12. Recommendations

The damage to the building during recent seismic activity in Christchurch has not caused any visible damage to the structure. No critical structural weaknesses have been observed. Based on the construction date, the soils on which the building is founded and construction type of the building, it has achieved over 67% NBS following an initial IEP assessment and is therefore not considered Earthquake Risk or Earthquake Prone.

13. Limitations

13.1 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.
- The foundations of the building were unable to be inspected.
- 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 included as part of the IEP in the CERA Building Evaluation Report, have been undertaken. No modelling of the building for structural analysis purposes has been performed.

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.

13.2 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 and a CPT undertaken on 02 April 2012. 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.

Given the enclosed information we would recommend a series of additional location specific geotechnical assessments, including testing and investigation, be completed.

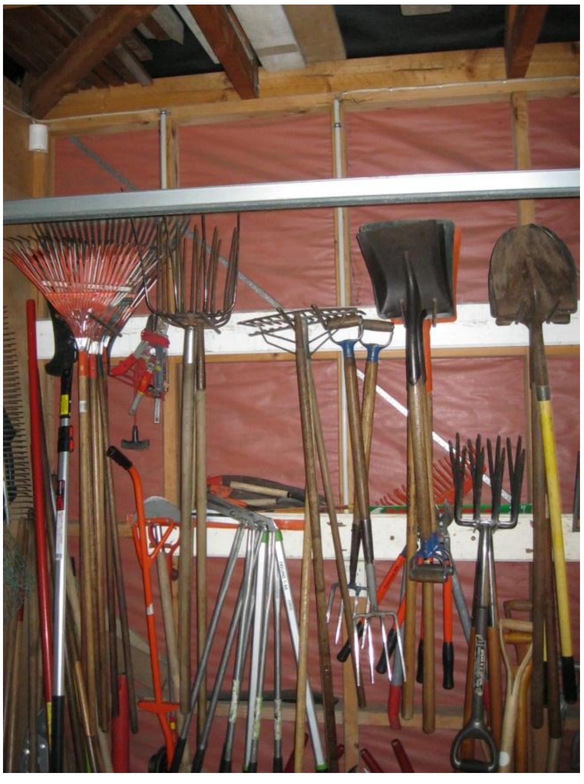
Appendix A

Photographs

Photograph 1 View of the eastern face of the building.

Photograph 2 View of the western face of the building.

Photograph 3 Internal roof trusses.



Photograph 4 Internal wall bracing.

Photograph 5 Roof truss nail plate connections.

Photograph 6 Internal roof trusses connection.

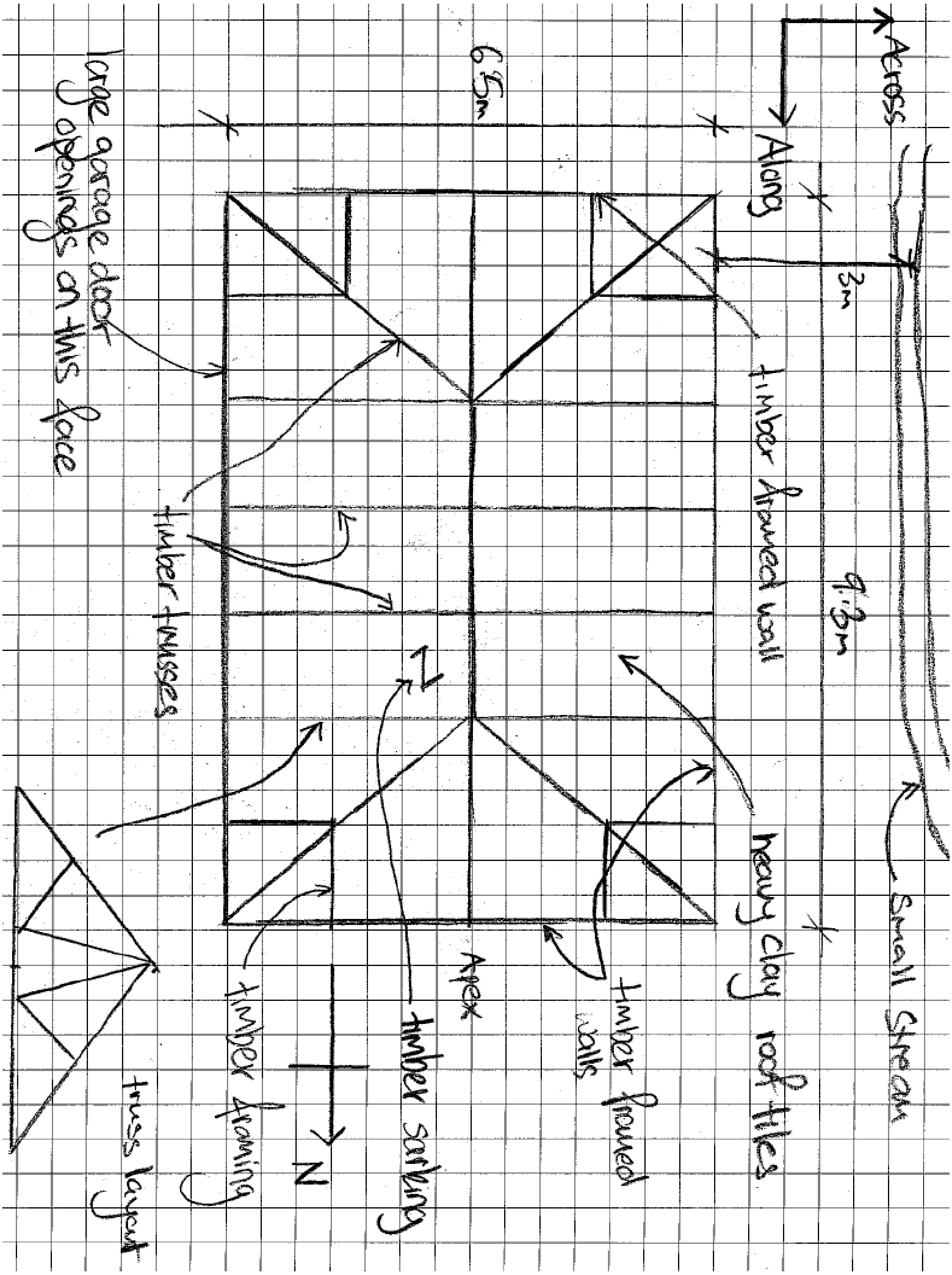
Photograph 7 Connection between roof trusses and timber wall.

Photograph 8 View of timber roof structure.

Appendix B

Existing Drawings / Sketches

No structural or architectural drawings have been made available for this building. Shown below is a marked up plan of the building showing key structural elements.



Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Mona Vale Implement Shed/Staff Rooms			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:		40	Mona Vale Avenue	Company:	GHD
Legal Description:				Company project number:	51/30596/23
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	11/4/2012
				Revision:	0
Building Unique Identifier (CCC):				Is there a full report with this summary?	yes

Site

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

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	raft slab		if Foundation type is other, describe:	
Building height (m):	6.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	6
Floor footprint area (approx):	30			
Age of Building (years):	6		Date of design:	2004-
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):	other (specify)			
Use notes (if required):	Storage Garage			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:	timber	type	
Columns:	timber	typical dimensions (mm x mm)	
Walls:			

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls		**Note: Define along and across in detailed report!**	note typical wall length (m)	
Ductility assumed, μ:	3.00				
Period along:	0.40	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	
Lateral system across:	lightweight timber framed walls			note typical wall length (m)	
Ductility assumed, μ:	3.00				
Period across:	0.40	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	plaster system	describe	
Roof Cladding:	Heavy tiles	describe	
Glazing:	aluminium frames		
Ceilings:	none		
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:	No ground damage observed.	Describe damage:	
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:	green	
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):	Minor damage. Less than 5% if at all.	
Across	Damage ratio:	0%	
	Describe (summary):	Minor damage. Less than 5% if at all.	
Diaphragms	Damage?:	no	Describe:
CSWs:	Damage?:	no	Describe:
Pounding:	Damage?:	no	Describe:
Non-structural:	Damage?:	no	Describe:

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

Recommendations

	Level of repair/strengthening required:	none		Describe:
	Building Consent required:	no		Describe:
	Interim occupancy recommendations:	full occupancy		Describe:
Along	Assessed %NBS before:	85%	85% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	85%		
Across	Assessed %NBS before:	85%	85% %NBS from IEP below	
	Assessed %NBS after:	85%		

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 2004-

h_n from above: 6m

Seismic Zone, if designed between 1965 and 1992: B

Design Soil type from NZS1170.5:2004, cl 3.1.3:
not required for this age of building

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:	22.3%	22.3%

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final $(\%NBS)_{nom}$:	**22%**	**22%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	0.30
Z_{1992}, from NZS4203:1992	0.8
Hazard scaling factor, **Factor B:**	2.666666667

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	3.00	3.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.00	1.00
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

Sp:	0.700	0.700
Structural Performance Scaling Factor **Factor E**:	1.428571429	1.428571429

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x E

%NBS$_b$:	**85%**	**85%**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1
3.4. Pounding potential — Pounding effect D1, from Table to right		1.0
Height Difference effect D2, from Table to right		1.0
Therefore, Factor D:		1
3.5. Site Characteristics	insignificant	1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max value =1.5, no minimum	1.0	1.0
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

3.7. Overall Performance Achievement ratio (PAR)	**1.00**	**1.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**85%**	**85%**
4.4 Percentage New Building Standard (%NBS), (before)		**85%**

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
DRAFT	Alex Baylis	Stephen Lee		Nick Waddington		31/10/12
FINAL	Alex Baylis	Stephen Lee		Donna Bridgman		01/10/13