Awa-iti Domain Garage 007

PRK 3746 BLDG 007

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

4313 Christchurch Akaroa Road,
Little River.

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

Awa-iti Domain Garage 007
PRK 3746 BLDG 007

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

4313 Christchurch Akaroa Road,
Little River

Christchurch City Council

Prepared By
Peter O'Brien

Reviewed By
Hamish MacKinven

Date
26/9/13

Contents

Table Index

Figure Index

Appendices

Quantitative Report Summary

Awa-iti Domain Garage 007

PRK 3746 BLDG 007

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

4313 Christchurch Akaroa Road, Little River

Background

This is a summary of the Quantitative report for the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 26th June 2013 only.

Brief Description

The Awa-iti Domain Garage is located at 4313 Christchurch-Akaroa Road, Little River. The site consists of several buildings of various use, a car park and various sports facilities. The age of the building is unknown and there are no structural drawings for the building on record.

The building is a single storey concrete framed structure on a concrete slab with perimeter thickening. The roof is pitched up to a central longitudinal ridge and consists of lightweight metal cladding fixed to timber purlins. The purlins are fixed to the equal angle steel trusses which are supported by load bearing reinforced concrete columns. There is unreinforced concrete masonry infill panels between the reinforced concrete framing elements.

Key Damage Observed

No damage was noted to the structure

Building Capacity Assessment

Based on the Quantitative Analysis carried out on the structure using NZS 3101:2006 for Concrete Structures, NZS 3404:1997 for Steel Structures and referencing the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for unreinforced concrete masonry walls, the building has been assessed to be 85% NBS along the building and 37% NBS across. Based on this, the overall %NBS for the building is 37%.

Recommendations

The building has been assessed to have a seismic capacity of 37% NBS. As the building's capacity is assessed to be 37% NBS, it is not considered to be either an Earthquake Prone building. It is however, considered to be an Earthquake Risk building as it has been assessed as less than 67% NBS. GHD recommend that strengthening of the building is carried out to bring the capacity of the building up to 67% NBS in line with the NZSEE Guidelines.

In addition there are no immediate collapse hazards, or Critical Structural Weaknesses associated with the structure, therefore general occupancy of the building is permitted.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Awa-iti Domain Garage 007.

This report is a Quantitative Assessment of the building structure, and is based in general on NZS 1170.5:2004 Earthquake Actions, NZS 3404:1997 Steel Structure Standard, NZS 3101:2006 Concrete Structures Standard and the New Zealand Society for Earthquake Engineering (NZSEE) guidelines.

At the time of this report no intrusive site investigations have been carried out. A quantitative assessment of the structure involves the determination of the seismic and gravity loads that a building is subjected to, analysis of the distribution of these forces throughout the structure and an analysis of the capacity of the existing structural elements to resist the applied forces. The capacity of each of the existing structural elements is compared to the demand placed on the element in order to determine a %NBS.

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 assessment of the likely building strength in terms of percentage of new building standard (%NBS).

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 Awa-iti Domain Garage is located at 4313 Christchurch-Akaroa Road, Little River. The site consists of the several buildings of various use, a car park and various sports facilities. The age of the building is unknown and there are no structural drawings for the building on record.

The building is a single storey concrete framed structure on a concrete slab with perimeter thickening. The roof is pitched up to a central longitudinal ridge and consists of lightweight metal cladding fixed to timber purlins. The purlins are fixed to the equal angle steel trusses which are supported by load bearing reinforced concrete columns. There is unreinforced concrete masonry infill panels between the reinforced concrete framing elements.

The building dimensions are approximately 18.2 m long by 8.4 m wide with an approximate total floor area of 153 m^2. The overall height of the building is 4 m with masonry wall heights of 2.4 m.

The nearest building is approximately 1.5 m from the Garage building whilst the nearest waterway to the property is the Hukahuka Turoa Stream, located approximately 60 m to the east of the property.

A plan layout of the building is shown in Figure 2.

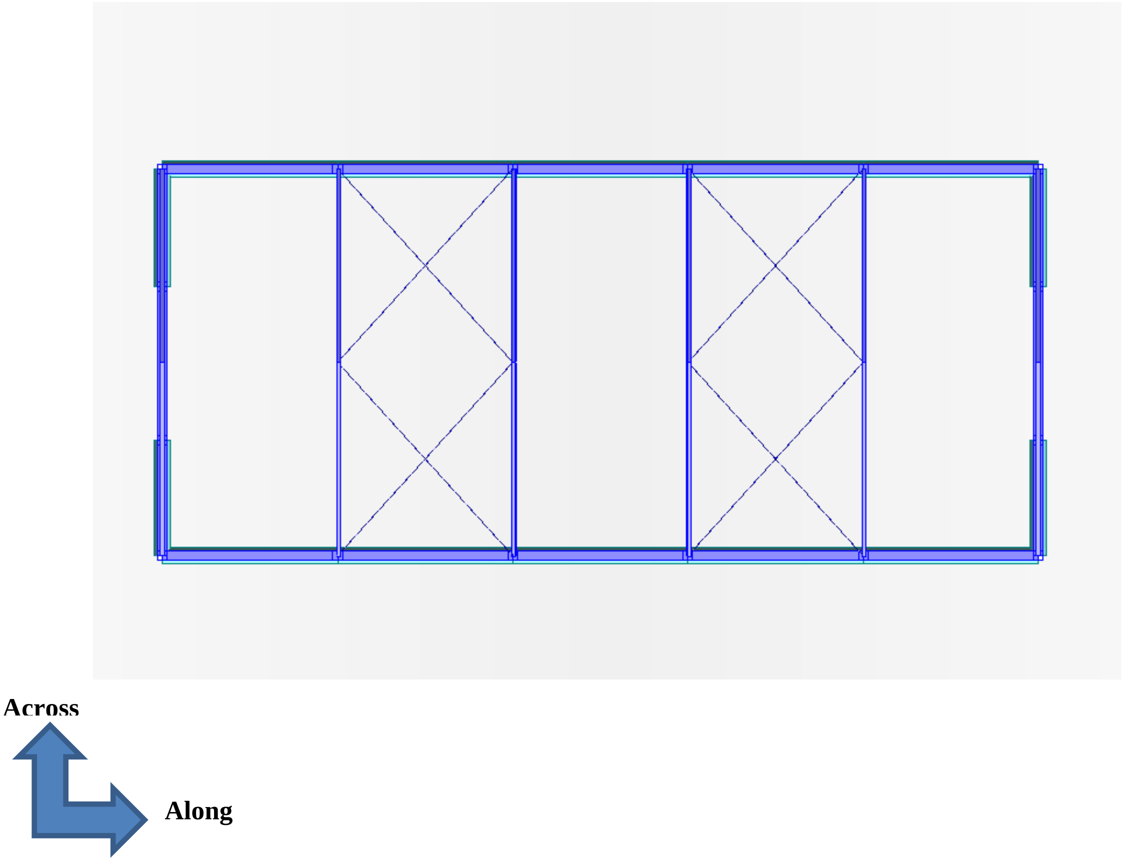

Figure 2 Plan sketch showing key structural elements

4.2 Gravity Load Resisting System

Gravity loads from the roof cladding are supported by timber purlins. These loads are then transferred from the purlins to the equal angle steel roof trusses. Gravity loads from the trusses are then transferred to the load bearing reinforced concrete framing and then to the concrete perimeter foundation walls. Self-weight of the concrete masonry walls is transferred down through the walls to the foundations below. Internal gravity loads are transferred through the concrete floor slab back to the thickened perimeter foundations and the soil below.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure in both the long and short directions of the building are resisted primarily by the reinforced concrete framing and masonry infill panels.

In the transverse direction, lateral forces acting on the roof of the structure are transferred through the purlins to the supporting steel roof trusses and the supporting timber framing at each gable. Loads are then transferred in the plane of the trusses and gables to the supporting concrete framing. Loads at the gables are transferred down to the ground by the concrete framing and the unreinforced concrete masonry infill panels. Loads transferred to the concrete framing through the steel roof trusses are transferred down through the columns to the foundations beneath. Lateral load resistance for the unreinforced concrete masonry infill panels loaded laterally out of plane is provided by the capacity of the concrete masonry for out of plane flexure. The walls then transfer the lateral loads to the perimeter foundations

Lateral loads acting in the longitudinal direction are transferred from the roof through a combination of the steel roof cross bracing and the timber purlins to the steel roof trusses and into the supporting concrete framing. Loads are then transferred down through the concrete columns and unreinforced concrete masonry infill panels to the foundations below.

5. Damage Assessment

An inspection of the building was undertaken on the 26th of June 2013. 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.

5.1 Surrounding Buildings

Minor cracking was noted to the foundations of Shed 005. This damage is not believed to be earthquake related.

5.2 Residual Displacements and General Observations

No residual displacements of the garage structure were noted.

5.3 Ground Damage

There was no evidence of ground damage on the site.

6. Survey

No floor level survey or intrusive investigations have been carried out for the building.

7. Geotechnical Consideration

7.1 Site Description

The site is situated in Little River on Banks Peninsula. It is situated in the bottom on a valley at approximately 20 m above mean sea level. It is approximately 40 m west of the Hukahuka Turoa Stream, 2.5 km northeast of Lake Forsyth, and 14 km west of Akaroa.

7.2 Published Information on Ground Conditions

7.2.1 Published Geology

Forsyth *et. al* 2008[1] describes the site geology as:

- Grey river alluvium beneath plains or low-level terraces (Q1a), Holocene in age;

7.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that four boreholes with lithographic logs are located within 500 m of the site, the borehole logs for these wells are summarised in Table 2.

These indicate the area is underlain by loess colluvium and alluvium with occasional volcanic boulders to ~20 m bgl, underlain by volcanic rock to ~60 m bgl.

Groundwater was recorded between 1.85 m and 3.7 m bgl in the borehole logs.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary	
N36/0003	10.6 m	N/A	90 m E	0.0 – 7.6	Not logged
				7.6 – 9.1	Rock
				9.1 – 10.6	Blue clay
N36/0082	41.5 m	3.6 m bgl	400 m W	0.0 – 3.0	Sand and gravels
				3.0 – 7.0	Clay
				7.0 – 8.0	Clayey volcanic rock
				8.0 – 8.5	Volcanic rock
				8.5 – 24.0	Clay, silt and volcanics
				24.0 – 41.5	Hard volcanics
N36/0131	60.0 m	3.7 m bgl	465 m SW	0.0 – 0.5	Soil
				0.5 – 6.0	Sandy clay with gravels
				6.0 – 10.0	Volcanic gravels

[1] Forsyth, P.J., Barrell, D.J.A., Jongens, R. (2008) (compilers), Geology of the Christchurch Area, Institute of Geological and Nuclear Sciences 1:250 000 geological map 16. 1 sheet. Lower Hutt, New Zealand. GNS Science. ISBN 987-0-478-19649-8

Bore Name	Log Depth	Groundwater	From Site	Log Summary	
				10.0 – 13.0	Peaty silt/clay
				13.0 – 17.0	Gravelly clay
				17.0 – 24.0	Rock
				24.0 – 34.0	Rock and clay
				34.0 – 60.0	Rock
N36/0250	10.9 m	1.85 m bgl	475 m SW	0.0 – 0.3	Silty soils
				0.3 – 0.7	Silty clay
				0.7 – 3.0	Clay
				3.0 – 4.5	Rock and clay
				4.5 – 7.0	Clay and gravels
				7.0 – 10.0	Clay and rocks
				10.0 – 10.9	Volcanic rock

It should be noted that 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.

7.2.3 EQC Geotechnical Investigations

The Canterbury Geotechnical Database (CGD) shows no nearby geotechnical testing has been undertaken in the area by EQC and other independent investigations

7.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

The site has been categorised as “Port Hills & Banks Peninsula” because sites in the Port Hills and Banks Peninsula have not been given technical categories by CERA.

7.2.5 Aerial Photography

The site is not in coverage for aerial photography flown following major earthquakes of the Canterbury Earthquake sequence. An aerial photograph from March 2013 shows the sites location on the valley floor.

Figure 3 Aerial Photography

7.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise loess colluvium and alluvium with occasional volcanic boulders to ~20 m bgl, underlain by volcanic rock to ~60 m bgl.

Groundwater is considered to vary between 1.85 m and 3.7 m bgl.

7.3 Seismicity

7.3.1 Nearby Faults

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

Table 3 Summary of Known Active Faults[2,3]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	150 km	NW	~8.3	~300 years
Greendale Fault (2010)	40 km	NW	7.1	~15,000 years
Hope Fault	130 km	N	7.2~7.5	120~200 years
Porters Pass Fault	90 km	NW	7.0	~1100 years
Port Hills Fault (2011)	23 km	NW	6.3	*Not Estimated*

The recent earthquake sequence since 4 September 2010 has identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; this includes the Greendale Fault and Port Hills Fault listed in Table 3 above. Research and published information on this system is in development and the average recurrence interval is yet to be established for the Port Hills Fault.

7.3.2 Ground Shaking Hazard

New Zealand Standard NZS 1170.5:2004 quantifies the Seismic Hazard factor for Christchurch as 0.30, being in a moderate to high earthquake zone. This value has been provisionally upgraded recently (from 0.22) to reflect the seismicity hazard observed in the earthquakes since 4 September 2010.

The Christchurch earthquake sequence has produced earthquakes with high peak ground accelerations (PGA) across large parts of the city. The CGD contains conditional peak ground accelerations during selected earthquakes of the Canterbury earthquake sequence.

Conditional PGA's from the CGD[4] are not available for the Banks Peninsula.

7.4 Global Land Movement

Given the site's location on flat land in the bottom of a valley, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

7.5 Listed Land Use Register

A search of the property address in the Environment Canterbury (ECan) Listed Land Use Register[5] shows the site has no listed land use.

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

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

[4] Canterbury Geotechnical Database (2012): "Conditional PGA for Liquefaction Assessment", Map Layer CGD5110 - 27 Sept 2012, retrieved 31/10/2012 from https://canterburygeotechnicaldatabase.projectorbit.com/

[5] http://llur.ecan.govt.nz/

7.6 Liquefaction Potential

The site is considered to have a minor susceptibly to liquefaction, because the site is situated on fine grained shallow soils (silts and clays) underlain by volcanic bedrock, which are unlikely to liquefy.

7.7 Summary & Recommendations

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

The site appears to be situated on loess colluvium and alluvium with occasional volcanic boulders to ~20 m bgl, underlain by volcanic rock to ~60 m bgl. The site is unlikely to liquefy.

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

Should soil parameters be required for foundation repair or design it is recommended that intrusive investigation be conducted.

8. Seismic Capacity Assessment

8.1 Quantitative Assessment

The quantitative assessment of the building comprised of an investigation on the in-plane and out-of-plane strength of the unreinforced masonry block infill panels and an assessment of the capacities of the structural steel and reinforced concrete members. 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.

8.1.1 Building demand

The demand forces for each wall and structural element was assessed by applying seismic loading to a finite element model of 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 structural elements. 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 as such has been applied in the analysis.

8.2 Seismic Loading

8.2.1 Seismic Weight Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1) of NZS 1170.5;

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

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

$$S_P = 1.33 - 0.3\mu$$

Where μ, the structural ductility factor, was taken as 1.5 in both the along and across directions.

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

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

8.3 Member Capacity Assessment

8.3.1 Steel Member Design Capacity

Assessment of the capacity of the individual structural steel members was carried out in accordance with NZS 3404:1997. Members were checked for flexure, shear, axial load and combined axial load and flexure in accordance with the relevant clauses of the standards.

8.3.2 Concrete Member Design Capacity

Assessment of the capacity of the individual reinforced concrete members was carried out in accordance with NZS 3101:2006. Members were checked for flexure, shear and axial load in accordance with the relevant clauses of the standards.

8.3.3 Masonry Wall Design Capacity

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).

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. Furthermore, the recent publication of “General Procedure for Seismic Assessment of Out-of-plane Loaded URM Wall” by Auckland University has been used in the analysis.

9. Building Analysis and Results

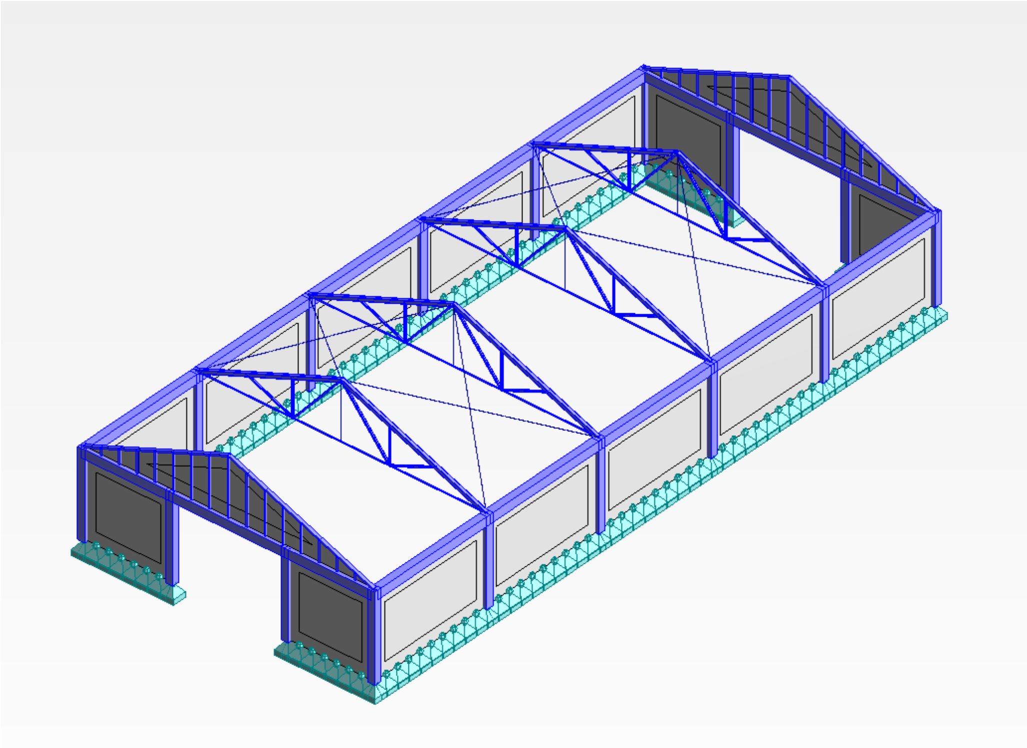

Figure 4 Key structural elements modelled

9.1 Longitudinal Assessment

In the longitudinal direction, the unreinforced concrete masonry panels provide the resistance to lateral loading. The largest base shear force applied to any of the infill panels has been assessed as 21.46 kN by the finite element model. The capacity of the individual unreinforced panels for in-plane shear is 18.4 kN based on the shear capacity calculations outlined in the NZSEE guidelines. Comparing the demand and the capacity of the walls, they are assessed as achieving 85% NBS. The in-plane moment capacity of the walls has been assessed to be >100% NBS.

The capacities of the concrete columns and steel roof trusses in the longitudinal direction have been assessed as being >100% NBS.

In the longitudinal direction, the building has been assessed to have a capacity of 85% NBS limited by the shear capacity of the unreinforced concrete masonry infill panels.

9.2 Transverse Assessment

In the transverse direction, the gable end walls resist a tributary amount of the lateral loading. The gable walls has been assessed as having a capacity of 37% NBS for in-plane shear. The unreinforced concrete masonry infill panels have been assessed as having an out-of-plane capacity of 85% NBS.

The steel roof trusses have been assessed as achieving >100% NBS.

The concrete frame beams have been assessed to have a capacity greater than >100% NBS. The concrete columns have been assessed to have a capacity of 52% NBS limited by their flexural capacity .

In the transverse direction, the building has been assessed to have a capacity of 37% NBS limited by the in-plane shear of the gable end wall unreinforced concrete masonry infill panels.

9.3 Discussion of Results

The date of construction of the building is unknown but it is estimated to have been constructed pre-1970's and therefore it was most likely designed to the loading standard current at the time, NZS 1900:1965. The results obtained from the quantitative assessment are consistent with those expected for a building of this age and construction type. The design loads used in this code are less than those required by the current loading standard, especially when taking in to account the Department of Building and Housings (DBH) increase in the hazard factor for Christchurch District to 0.3. As a result, it would be expected that the building would not achieve 100% NBS.

9.4 Occupancy

The building has not been assessed as potentially Earthquake Prone and there was no observed damage to the lateral load resisting system. In addition the building does not pose an immediate risk to users and occupants as no collapse hazards have been identified, therefore it is recommended that general occupancy of the building continue.

10. Recommendations and Conclusions

The building has been assessed to have a seismic capacity of 37**%** NBS. As the building's capacity is assessed to be 37% NBS, it is not considered to be either an Earthquake Prone building. It is however, considered to be an Earthquake Risk building as it has been assessed as less than 67% NBS. GHD recommend that strengthening of the building is carried out to bring the capacity of the building up to 67% NBS in line with the NZSEE Guidelines

In addition there are no immediate collapse hazards, or Critical Structural Weaknesses associated with the structure, therefore general occupancy of the building is permitted.

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.
- The foundations of the building were only able to be inspected where they were above ground level.
- No intrusive structural investigations have been undertaken.
- No intrusive geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken. Material properties have been assumed based on the recommendations from the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes

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.

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 by GHD. 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 Transverse gable wall

Photograph 2 Longitudinal wall

Photograph 3 Internal view showing eastern gable

Photograph 4 Internal view showing western gable

Photograph 5 Typical of roof truss connections

Photograph 6 Steel roof cross bracing visible in roof space

Appendix B

Existing Drawings / Sketches

No structural or architectural drawings have been made available for this building.

Appendix C

CERA Standardized Report Form

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Garage 007			Reviewer:	Hamish MacKinven
	Unit	No:	Street	CPEng No:	1003941
Building Address:	7	4313	Christchurch Akaroa Road	Company:	GHD Ltd
Legal Description:	Lot 1 DP 423920			Company project number:	513152627
				Company phone number:	03 3780900
	Degrees	Min	Sec		
GPS south:	43	45	51.64	Date of submission:	28-08-13
GPS east:	172	47	47.28	Inspection Date:	26-06-13
				Revision:	FINAL
Building Unique Identifier (CCC):	PRK 3746 BLDG 007			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	mixed	Soil Profile (if available):	N/A
Site Class (to NZS1170.5):	C		
Proximity to waterway (m, if <100m):	60	If Ground improvement on site, describe:	N/A
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):	0.00
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	4.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	2.2
Floor footprint area (approx):	150			
Age of Building (years):			Date of design:	1976-1992
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			
Importance level (to NZS1170.5):	IL1			

Gravity Structure

Gravity System:	frame system		
Roof:	steel truss	truss depth, purlin type and cladding	1.6m, Timber, Profiled Metal
Floors:	concrete flat slab	slab thickness (mm)	125
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	250 x 200
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	200 x 200
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	concrete frame with infill	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	40
Ductility assumed, μ:	1.00			
Period along:	0.05	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	concrete frame with infill		note total length of wall at ground (m):	16
Ductility assumed, μ:	1.50			
Period across:	0.10	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Metal	describe	Corrugated Steel
Glazing:			
Ceilings:			
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	N/A
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):	No damage		
Across	Damage ratio:	0%		
	Describe (summary):	No damage		
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:	minor structural	Describe:	Strengthening to 67% NBS
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	85%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after e'quakes:	85%		
Across	Assessed %NBS before e'quakes:	37%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	37%		

GHD

GHD Building
226 Antigua Street, Christchurch 8013
T: 64 3 378 0900   F: 64 3 377 8575   E: chcmail@ghd.com

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
DRAFT	Peter O'Brien	Hamish MacKinven		Donna Bridgman		28/08/2013
FINAL	Peter O'Brien	Hamish MacKinven		Donna Bridgman		26/09/2013