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Gatehouse Garage – Mona Vale
PRK 0555 BLDG 014

Detailed Engineering Evaluation

Qualitative Report

FINAL Version

Mona Vale, 63 Fendalton Road,
Riccarton, Christchurch

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

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Qualitative Report
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Mona Vale, 63 Fendalton Road,
Riccarton, Christchurch

Christchurch City Council

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Date
24/05/2013

Contents

Table Index

Figure Index

Appendices

Qualitative Report Summary

Gatehouse Garage – Mona Vale

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Detailed Engineering Evaluation

Qualitative Report - SUMMARY

FINAL Version

Mona Vale, 63 Fendalton Road, Riccarton, Christchurch

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 17 April 2012.

Building Description

The Gatehouse Garage at Mona Vale was constructed in 1978. The site is located on mostly flat ground in Mona Vale, which is situated alongside the Avon River in Riccarton, Christchurch. The sloped bank of the Avon River lies roughly 20m to the east of the Gatehouse Garage across flat land. Fendalton Road lies roughly 30m to the north. The surrounding land otherwise consists of walking paths and suburban homes.

The general structure of the Gatehouse Garage is a simple rectangular timber-framed garage with a duo-pitch roof. The roof consists of timber trusses with corrugated metal cladding. There is no sarking or ceiling structure attached to the underside of the trusses. The exterior timber framed walls contain 75x50 studs at 700mm on-centre and flat strap steel cross-bracing with weatherboard cladding. There are no interior walls. The Gatehouse Garage sits on a 100mm thick reinforced concrete slab with thickened edges. There is small window in the west wall; one entry door and one window in the north wall; and a large roller door in the east wall of the Garage.

The dimensions of the building are approximately 7.2m long, 4.0m wide, and 2.6m in height.

Key Damage Observed

No damage to the Gatehouse Garage was observed on site.

Critical Structural Weaknesses

No critical structural weaknesses were found for the building.

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the building's original capacity has been assessed to be greater than 100% NBS. The building's capacity excluding critical structural weaknesses is greater than 100% NBS. The building is neither potentially Earthquake Prone nor a potential Earthquake Risk.

Recommendations

No further assessment or strengthening is required.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of several buildings at Mona Vale in Riccarton, Christchurch. This report covers the Gatehouse Garage.

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. The building description is based on the visual inspection carried out on site and the building drawings made available.

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.

2.0 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

2.1 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.

2.1.1 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.

2.2 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.

2.3 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

4.0 General

The Gatehouse Garage at Mona Vale was constructed in 1978. The site is located on mostly flat ground in Mona Vale, which is situated alongside the Avon River in Riccarton, Christchurch. The sloped bank of the Avon River lies roughly 20m to the east of the Gatehouse Garage across flat land. Fendalton Road lies roughly 30m to the north. The surrounding land otherwise consists of walking paths and suburban homes.

The general structure of the Gatehouse Garage is a simple rectangular timber-framed garage with a duo-pitch roof. The roof consists of timber trusses with timber purlins and corrugated metal cladding. There is no sarking or ceiling structure attached to the underside of the trusses. The exterior timber framed walls contain 75x50 studs at 700mm on-centre and flat strap steel cross-bracing with weatherboard cladding. The external walls have no internal lining, and there are no interior walls. The Gatehouse Garage sits on a 100mm thick reinforced concrete slab with thickened edges. There is small window in the west wall; one entry door and one window in the north wall; and a large roller door in the east wall of the Garage.

The timber roof structure showed no evidence of cross-bracing and there is no roof cross-bracing shown on the available drawings. Flat-strap steel cross-bracing was evident in the two perpendicular walls, roughly as shown on the available drawings. Though an interior inspection of the Gatehouse Garage was not undertaken, the available drawings show diagonal cross-bracing around the roller door opening in the eastern wall. It is assumed that this cross-bracing in the walls is in place, as the rest of the structure appears to have been built in accordance with the drawings.

The dimensions of the building are approximately 7.2m long, 4.0m wide, and 2.6m in height.

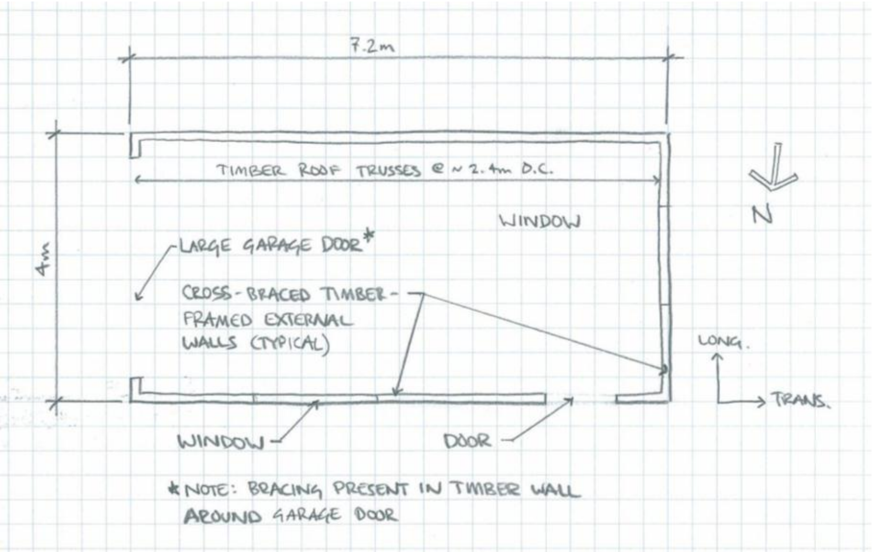


Figure 2 Plan Sketch Showing Key Structural Elements

4.1 Gravity Load Resisting System

The gravity loads in the structure are carried through the steel roof cladding to the timber roof trusses, out to the timber exterior wall framing, down through the walls to the thickened concrete pad and into the ground. The longitudinal load-bearing exterior walls carry the majority of the gravity loads as the roof trusses are oriented perpendicular to these walls. Gravity loads on the floor of the Gatehouse Garage are translated through the floor slab and into the ground.

4.2 Lateral Load Resisting System

In the longitudinal direction, lateral loads are translated through the corrugated steel roof cladding and timber roof purlins to the cross-braced exterior walls via shear in the roof connections, through the walls via diaphragm action, through the thickened concrete slab foundation and into the ground. In the transverse direction, lateral loads are translated through the corrugated steel roof cladding and timber roof trusses to the cross-braced exterior walls via shear in the roof connections, through the walls via diaphragm action, through the thickened concrete slab foundation and into the ground. In the front wall, which contains a roller door, lateral loads imposed on the wall are translated around the roller door via cross-bracing.

5. Assessment

A visual inspection of the exterior of the building was undertaken on 17 April 2012, with an interior inspection following on 31 May 2012. Since there are no interior walls, no internal lining to the external walls and no ceiling attached to the roof trusses, the structural elements of the building were able to be observed. There was no placard observed in place at the building. No detailed inspection of the foundation of the structure was able to be undertaken, though the concrete pad was visible.

The visual 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 is determined using the IEP procedure described by the NZSEE which is based on the information obtained from visual observation of the building.

6. Damage Assessment

6.0 Surrounding Buildings

The Gatehouse Garage at Mona Vale is located in a suburban area with residential homes and semi-open lands adjacent to the site. The site is located on mostly flat ground, and the sloped bank of the Avon River lies roughly 20m to the east of the Gatehouse Garage across flat land. Fendalton Road lies roughly 30m to the north. The surrounding land otherwise consists of walking paths and suburban homes. There was no apparent damage to any of the surrounding buildings.

6.1 Residual Displacements and General Observations

There were no residual displacements or damage noted in the garage building.

6.2 Ground Damage

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

7. Critical Structural Weakness

7.1 Short Columns

The building does not contain any short columns.

7.2 Lift Shaft

The building does not contain a lift shaft.

7.3 Roof

The timber roof structure does not contain any cross bracing. It is expected that the timber purlins and roof cladding will combine to form limited diaphragm action. The roof structure lacks cross-bracing, but this will not likely lead to a premature collapse of the building. Therefore, the roof does not contribute to a critical structural weakness.

7.4 Staircases

The building does not contain a staircase.

7.5 Plan Irregularity

The building does not contain any significant plan irregularity.

7.6 Liquefaction

No liquefaction was observed at the site. However, the geotechnical investigation has identified a high liquefaction potential for the site, as well as the potential for lateral spreading. It is anticipated that the concrete slab foundation would accommodate liquefaction without leading to premature collapse. Accordingly, the building does not exhibit a critical structural weakness in the form of site characteristics.

8. Geotechnical Consideration

This geotechnical study outlines the ground conditions, as indicated from sources quoted within.

This report is only specific to the gatehouse garage at Mona Vale Gatehouse Garage, 65 Fendalton Road, Fendalton, Christchurch. The property is bordered by the Main North Line Railway to the west, Wairarapa Stream (Avon River) to the east and Fendalton Road to the north. The property is owned and maintained by the Christchurch City Council.

8.0 Site Description

The site is situated on a relatively flat, low lying location at an elevation in the order of 10m above mean sea level (msl). It is approximately 10m west of Wairarapa Stream (Avon River), 10m east of Main North Line Railway, and 10 km west of the coast.

8.1 Published Information on Ground Conditions

8.1.1 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 gravel, sand and silt of historic river flood channels.

8.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates eight boreholes are located within a 150m radius of the site. However, none of these boreholes have logs that extend deeper than 5m, A number of these boreholes appear to have been drilled near to or adjacent to the Fendalton Road alignment. The shallow logs (<5m) indicate the ground beneath them to comprise predominately of sand or silt underlain by grey sand and gravel deposits.

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.

8.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing close to the site. Information pertaining to this investigation is included in Tonkin and Taylor Report[2]. Within 150m of the property there are two investigation points were undertaken, the results of which are detailed below in Table 1.

[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.

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

Table 2 EQC Geotechnical Investigation

Bore Name	Grid Reference	Log Summary
BH-FND-INC	2478263.58 mE	0-1.2m Fill
	5742918.99 mN	1.2-2m Sand
		2-3.75m Gravel
		3.7-7.5m Gravelly Silt/Sandy Silt/Silty Sand/Silt
		7.5-14m Gravel/Sand
		14- 17m Silt/Sandy Silt/Silty Sand
		17-20m Gravel
CPT-FND-28	2478234.85 mE	0-1.0m Clay
	5742749.09 mN	1.0-7m Sand/Silty sand/Sandy silt
		7-8m Sandy silt/silty clay
		8-9.5m Sand/Silty sand

Initial observations of the CPT results indicate that the soils on site are predominantly sand or silt with varying amount of clay, silt/sand mixtures and gravel. Ground water level is likely 1m of the ground surface. This would infer that liquefaction potential of the site is considered moderate to high due to the presence of sand and silt and shallow ground water level.

8.1.4 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 itself is a non-residential properties in urban area of Christchurch (but within the Green Zone), therefore it has not been given a CERA land zoning technical category.

However, a properties to the east and west of the subject structure have been given a technical category of TC3 (blue) – CERA indicates that this means that moderate to significant land damage from liquefaction is possible in future significant earthquakes, and that site-specific geotechnical investigation and specific engineering foundation design is required.

8.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of moderate to high liquefaction close to the site. Significant liquefaction is particulary observed at the Main North Line Railway approximately 10m west of the site, as shown in Figure 3**.**

Figure 3 Post February 2011 Earthquake Aerial Photography [3]

8.1.6 Summary of Ground Conditions

From the information presented, the soils on site are predominantly sand or silt with varying amount of clay, silt/sand mixtures or gravel. Ground water level is likely 1m of the ground surface.

8.2 Seismicity

8.2.1 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.

Table 3 Summary of Known Active Faults[4,5]

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

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

[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

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.

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

8.3 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.

In addition, the site is located adjacent to the Wairarapa Stream (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 2011 earthquake.

8.4 Liquefaction Potential

Due to the presence of sands and silts, and evidence from the post-earthquake aerial photography, the liquefaction potential of the site it is considered moderate to high, especially in locations where water-saturated sands and silts are present.

8.5 Recommendations

Given the anticipated ground conditions and observations following the February earthquake, we recommend that further investigation is undertaken. Specifically, one CPT investigation should be conducted to a target depth of 20m bgl.

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 22 February 2011.

The site appears to be situated on stratified alluvial deposits comprising sand and silt underlain by gravel. Associated with the site also has moderate to high liquefaction potential in locations where water-saturated sands and silts are present.

Ground shaking is expected to be moderate to high in anticipation of the stratified alluvial deposits and the site's 475-year PGA (peak ground acceleration) of ~0.4.

It is recommended that an intrusive investigation comprising of one piezocone CPT investigations be conducted. From this test, a more comprehensive liquefaction analysis can be undertaken.

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's capacity was assessed using the Initial Evaluation Procedure based on the available information. The building's capacity excluding critical structural weaknesses is expressed as a percentage of new building standard (%NBS) and is in the order of that shown below in Table 4. This capacity is subject to confirmation by a more detailed quantitative analysis. As no critical structural weaknesses were found at the building, no capacities for critical structural weaknesses have been listed in Table 4 below.

Item	%NBS
Building Capacity excluding CSW's	>100%

Table 4 Indicative Capacity based on the NZSEE Initial Evaluation Procedure

Following an IEP assessment, the building has been assessed as achieving greater than 100% New Building Standard (NBS). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is not considered potentially Earthquake Prone as it achieves above 33% NBS. Likewise, the building is not considered to be a potential Earthquake Risk as it achieves greater than 67% NBS. The overall score of the building has not been adjusted to reflect critical structural weaknesses, as none have been found. The building's score has not been adjusted when considering damage to the structure as no damage to the building was observed.

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 = 0.5, NZS 1170.5:2004, Table 3.5, Importance Level 1 structure with a 50 year design life.

Several key seismic parameters that have influenced the %NBS score obtained from the IEP assessment. The building has been assessed as an Importance Level 1 building. An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations.

10.3 Expected Structural Ductility Factor

A structural ductility factor of 2.0 has been assumed based on the timber frame structure and date of construction.

10.4 Discussion of Results

The results obtained from the initial IEP assessment are consistent with those expected for an Importance Level 1 building of this age and construction type founded on Class D soils. This building would have been designed to the standards at the time, namely NZS4203:1976. The design loads used in this code will have been significantly less than those required by the current loading standard, with lower detailing requirements for ductile seismic behaviour than those that are present in the current standards.

10.5 Occupancy

The building does not pose an immediate risk to users and occupants as no collapse hazards or critical structural weaknesses have been identified. The building's capacity exceeds 34% NBS and is therefore not potentially Earthquake Prone. Occupancy of the building should not be restricted.

11. Initial Conclusions

The building has been assessed to have a seismic capacity greater than 100% NBS and is therefore not potentially earthquake prone. As a result, occupancy of the building is allowed. There is no need for further investigation as this building is not a potential Earthquake Risk.

12. Recommendations

The building has not suffered any apparent damage during recent seismic activity in Christchurch and has no collapse hazards. In addition, the building has achieved a seismic capacity of greater than 100% NBS following an initial IEP assessment of the building. Therefore, the building is not potentially Earthquake Prone, nor is it a potential Earthquake Risk, and no further assessment of the building is required. Occupancy of the building is permitted.

No placard status was observed at the site. If a placard is attached to the building, it should be green.

13. Limitations

13.0 General

This report has been prepared subject to the following limitations:

- No inspection of the bracing in in the timber framed walls could be undertaken.
- 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.
- 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.

13.1 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and 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.

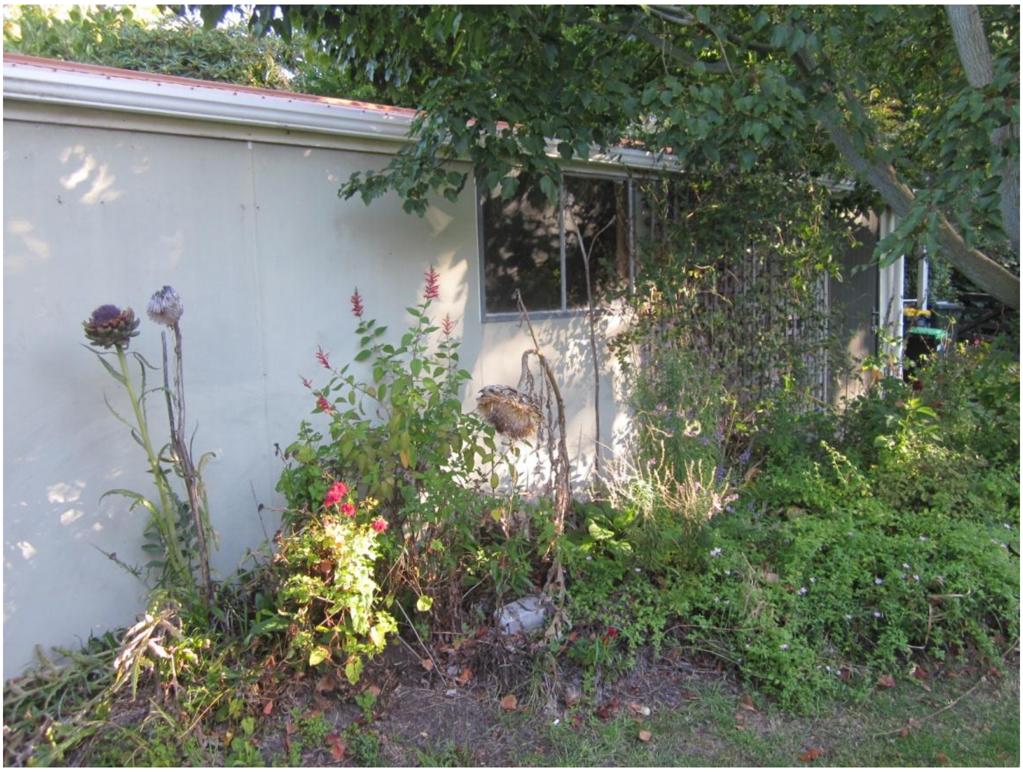
An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in 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.

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: East elevation.



Photograph 2: Northern elevation.

Photograph 3: Southern elevation.

Photograph 4: Exterior door at northwest corner elevation.

Photograph 5: Interior view of the garage showing roller door and wall bracing on longitudinal walls.

Photograph 6: Interior view of wall bracing at longitudinal wall.

Photograph 7: Interior view of roof structure, which has no cross-bracing.

Appendix B

Existing Drawings

Note: No existing drawings were available for this building

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Mona Vale Gatehouse Garage			Reviewer:	Stephen Lee
	Unit	No:	Street	CPEng No:	1006840
Building Address:		63	Fendalton Road, Riccarton	Company:	GHD
Legal Description:	Lot 9 DP 7787			Company project number:	513059653
				Company phone number:	6433780900
	Degrees	Min	Sec		
GPS south:	43	31	24.00	Date of submission:	24/205/2013
GPS east:	172	36	32.00	Inspection Date:	17/4/2012
				Revision:	FINAL
Building Unique Identifier (CCC):	PRK 0555 BLDG 014			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	50	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):	50		
Proximity to cliff base (m,if <100m):	50	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):	2.60		height from ground to level of uppermost seismic mass (for IEP only) (m):	2
Floor footprint area (approx):				
Age of Building (years):	44		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:	Storage
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL1			

<u>Gravity Structure</u>

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:	Timber Framed	thickness (mm)	

Lateral load resisting structure

Lateral system along:	lightweight timber framed walls		**Note: Define along and across in detailed report!**	note typical wall length (m)	3
Ductility assumed, μ:	2.00				
Period along:	0.12	0.00		estimate or calculation?	calculated
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)	2.4
Ductility assumed, μ:	2.00				
Period across:	0.12	0.00		estimate or calculation?	calculated
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:	other light	describe	Timber weatherboard
Roof Cladding:	Metal	describe	
Glazing:	timber frames		
Ceilings:	none		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Unknown
Structural	partial	original designer name/date	Unknown
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:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	Liquefaction Potential
Lateral Spread:	none apparent	notes (if applicable):	Lateral Spread Potential
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 (summary):

Describe how damage ratio arrived at:

Across — Damage ratio: 0%
Describe (summary):

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

Diaphragms	Damage?: no	Describe:	
CSWs:	Damage?: no	Describe:	
Pounding:	Damage?: no	Describe:	
Non-structural:	Damage?: no	Describe:	

Recommendations

Level of repair/strengthening required:	none	Describe:	
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along
Assessed %NBS before: 110% — 110% %NBS from IEP below
Assessed %NBS after: 110%

If IEP not used, please detail assessment methodology:

Across
Assessed %NBS before: 110% — 110% %NBS from IEP below
Assessed %NBS after: 110%

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): 1976-1992

h_n from above: 2m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.12	0.12
(%NBS)nom from Fig 3.3:	16.5%	16.5%

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 $(\%NBS)_{nom}$:	**17%**	**17%**

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:**	3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above):	1
Return Period Scaling factor from Table 3.1, **Factor C**:	2.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	2.00	2.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.57	1.57
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, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

$\%NBS_b$:	**157%**	**157%**

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

3.1. Plan Irregularity, factor A:	significant	0.7
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

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

3.5. Site Characteristics	insignificant	1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =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)	**0.70**	**0.70**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**110%**	**110%**

4.4 Percentage New Building Standard (%NBS), (before) **110%**

GHD

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
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