Barn - 004
Styx River Reserve #2
PRK 2546 BLDG 004

Detailed Engineering Evaluation
Qualitative Report
FINAL Version

Styx River Reserve, 303 Radcliffe Road,
Belfast, Christchurch

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Christchurch

Christchurch City Council

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Date
22/5/13

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Detailed Engineering Evaluation

Qualitative Report - SUMMARY

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303 Radcliffe Road, Belfast, 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 16 April 2012.

Building Description

The Barn (004) at Styx River Reserve #2 is assumed to have been constructed around 1970. It is located among similar sheds and barns on flat ground alongside the Styx River and alongside Radcliffe Road in Belfast, Christchurch.

The general structure is a simple rectangular timber barn with a duo-pitch roof. The roof consists of corrugated steel cladding atop timber sarking which is attached to timber purlins, which are supported by timber trusses. Timber cross-bracing members span across the four corners of the roof, in plane with and attached to the underside of the bottom chords of the trusses. The load-bearing exterior walls are timber-framed and cross-braced, and clad on the exterior with corrugated steel. The external walls generally have no interior cladding, and there are no internal walls. The front of the building is mainly an open entry to the barn, with a timber lintel spanning the opening, which is supported by posts at each end and at mid-span. There is no floor to the barn, only soil on-grade. The external walls sit atop concrete strip footings, with short foundation walls as required due to the slope. It is assumed that the lintel post at mid-span is supported by an isolated pad.

The dimensions of the building are approximately 11.4m long by 6.2m wide and 3m in height.

Key Damage Observed

Some damage was observed on-site, but it is believed that the damage is due to poor workmanship and not caused by seismic action. Key damage observed includes partial separation of the external corrugated steel cladding from the timber framing along the north transverse wall.

Critical Structural Weaknesses

The building exhibits the following critical structural weakness:

- Site Characteristics (liquefaction and lateral spread potential) 30% reduction

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 in the order of 37% NBS. The building's capacity excluding critical structural weaknesses is in the order of 53% NBS. Therefore the building is not potentially Earthquake Prone but is a potential Earthquake Risk.

Recommendations

It is recommended that a detailed seismic assessment and geotechnical investigation be carried out for the building, as the building is a potential Earthquake Risk. There are no collapse hazards and the building is not potentially Earthquake Prone; therefore, building occupancy should not be restricted.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of several buildings at the Styx River Reserve #2. This report covers the Barn (004).

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.

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 timber Barn (004) at Styx River Reserve #2 is assumed to have been constructed in 1970. The site is located on mostly flat ground alongside the Styx River and alongside Radcliffe Road in Belfast, Christchurch. The surrounding area consists of rural lands with occasional barns and other structures.

The site is generally flat with some variation in the topography due to the river and some short, gentle hills to the north. The shed is located alongside a man-made creek which feeds into the Styx River roughly 5m downstream from the southeastern corner of the building. There is less than 1m of ground between the foundations and the edge of the creek, and the river runs roughly parallel with the eastern longitudinal (rear) edge of the barn, roughly 5m away, with the ground sloping to the bank of the river.

The general structure is a simple rectangular timber barn with a duo-pitch roof. The roof consists of corrugated steel cladding atop timber sarking which is attached to timber purlins. The purlins run in the longitudinal direction of the building and are supported by shallow timber trusses oriented in the transverse direction of the building. Timber cross-bracing members span across the four corners of the roof, in plane with and attached to the underside of the bottom chords of the trusses. The load-bearing exterior walls are timber-framed and cross-braced with timber, and clad on the exterior with corrugated steel. The north transverse wall is only partially braced. There appears to have been an opening in the original design of the north transverse wall that has since been framed in and clad on the interior and exterior with the same corrugated steel that is used on the rest of the exterior walls. The other walls have no interior cladding, and there are no internal walls. The front of the building is mainly an open entry to the barn, with a 250mm deep timber lintel spanning the opening. The lintel is supported by timber posts at each end, and another at mid-span. There is no floor to the barn, only soil on-grade. The northern external wall sits atop a concrete strip footing. Due to the sloping ground toward the river, the eastern and southern walls sit atop a short retaining foundation wall, which is supported by concrete strip footings. It is assumed that the lintel post at mid-span is supported by an isolated pad, and that the posts at either end are supported by strip footings that continue to the transverse wall foundations.

The dimensions of the building are approximately 11.4m long by 6.2m wide and 2.6m in height.

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

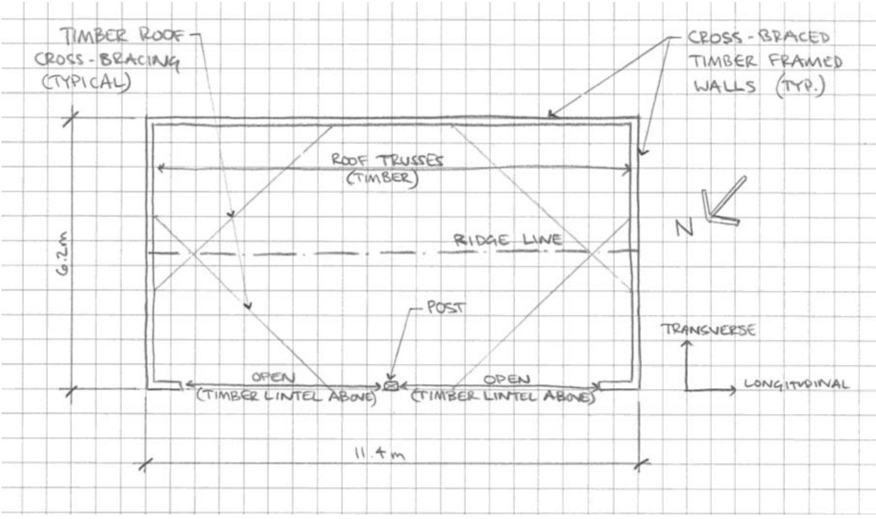

Figure 2 Plan Sketch Showing Key Structural Elements

4.1 Gravity Load Resisting System

The gravity loads in the structure are carried through the steel roof cladding to the timber sarking and roof purlins and down through the trusses via direct bearing in the connections. Gravity loads are then transferred through the trusses via truss action and out to the timber exterior wall framing and the lintel at the building entry via direct bearing on these members. Gravity loads are transferred through the exterior bearing wall as axial loads through the braced studs, and through the lintel via shear into the posts as axial loads. The gravity loads are transferred from the posts and wall studs into the concrete foundations and into the ground via direct bearing.

4.2 Lateral Load Resisting System

Cross-bracing to the bottom of the timber trusses will combine with the roof cladding, sarking, purlins and trusses to form diaphragm action which will carry the lateral loads across the roof and into the timber external walls. The cross-braced and steel-clad timber external rear wall will carry lateral loads in-plane through the wall and into its foundation. The short concrete foundation wall at the rear of the building provides additional shear resistance to lateral loads in the longitudinal direction. The lintel and posts which comprise the majority of the front of the building will not carry lateral loads, but the diagonal timber cross-bracing at the undersides of the timber roof trusses will help translate lateral loads from the front of the building to the rear. The short wall sections at either side of the openings are cross-braced and will therefore carry some lateral load. The transverse side walls will help carry torsional loads that are generated in the building due to the large entry opening, as lateral loads are transferred to these walls via diaphragm action in the roof.

Similar to the longitudinal direction, cross-bracing to the bottom of the timber trusses will combine with the roof cladding, sarking, purlins and trusses to form diaphragm action which will carry the lateral loads across the roof and into the timber external side walls. The cross-braced and steel-clad timber external side walls will transfer lateral loads in-plane to their foundations via shear in the anchor bolted connections. The northern wall is not as well-braced as the southern wall, but diaphragm action in the roof will help translate excess lateral load to the rear wall and other side wall. The short foundation wall under the southern exterior wall will provide additional shear resistance to lateral loads in the transverse direction.

5. Assessment

A visual inspection of the building was undertaken on 16 April 2012. Both the interior and exterior of the building were inspected. There was no placard observed in place at the building. Most of the main structural components of the building were able to be viewed due to the exposed nature of the structure. No detailed inspection of the foundation of the structure was able to be undertaken, though the top of the concrete footings and part of the concrete foundation walls were 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. A critical structural weakness in adverse site characteristics was observed, which reduced the overall %NBS.

6. Damage Assessment

6.0 Surrounding Buildings

The Barn (004) at Styx River Reserve is located in a rural area with open lands adjacent to the site. There are a few other small sheds and barns on the same property. Most of the surrounding buildings showed little sign of seismic damage, but one of the nearby concrete sheds showed severe signs of damage.

6.1 Residual Displacements and General Observations

Apart from some of the wall cladding separating from the framing, no residual displacements were observed. Some of the edges of the corrugated steel cladding panels on the northern exterior wall had separated from the timber wall framing. This is likely due to poor craftsmanship in the post-construction works to fill the opening in this wall, and not due to seismic damage. In addition, there was one hole in the exterior cladding roughly 100mm in diameter noted near the bottom of the wall on the western wall next to the barn entry. The rest of the exterior wall cladding on the northern and other exterior walls appeared to be in reasonable condition and still well-attached to the framing.

A pre-existing concrete pad was found adjacent to and abutting the building to the north. Though the pad has deteriorated and cracked into several pieces, the pad does not appear to be part of the main barn structure and was not considered in this assessment. Due to the irregular shape of the pad in plan, it appears that the pad may have been poured concurrently with the foundations with excess concrete as an afterthought.

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 significant short columns.

7.2 Lift Shaft

The building does not contain a lift shaft.

7.3 Roof

The roof structure features simple timber trusses, with continuous purlins running atop the trusses, and sarking underneath the corrugated steel cladding. The timber trusses have timber cross-bracing attached to the underside of the bottom chords. Adequate diaphragm action can be expected from the roof structure, and the roof structure would not therefore contribute to plan irregularity of the building.

7.4 Staircases

The building does not contain a staircase.

7.5 Plan Irregularity

The building shape in plan is a simple rectangle. The rear longitudinal side of the building is stiffer than the front of the building in terms of lateral load resistance. However, diaphragm action in the roof structure should be adequate to transfer lateral load from the front of the building to the rear with the assistance of the braced side walls. The building therefore does not exhibit any irregularity in plan that would constitute a critical structural weakness.

7.6 Liquefaction and Lateral Spreading

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. The southeast corner of the shed is situated on the bank of a creek which feeds the nearby Styx River, and the east side of the building runs roughly parallel to the creek and is especially susceptible to lateral spreading along the river's edge. Lateral spreading along the river could possibly undermine the building's foundations and lead to a premature collapse of the structure. Accordingly, the site characteristics have been noted as "significant" in the IEP assessment.

8. Geotechnical Consideration

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

This report is only specific to the south-western section of the property at 303 Radcliffe Rd, Harewood. The property is located off Radcliffe road. It is bounded by residential and agricultural properties. The property is owned and maintained by the Christchurch City Council.

8.0 Site Description

The site is situated within a large property on the banks of the Styx River. The site is relatively flat at approximately 5m above mean sea level. It is located approximately 5km south of the Waimakariri River, and 5km west of the coast (Pegasus Bay). Running through the property there is what appears to be an old river channel, which may also have been altered mechanically in size and shape. A section of this appears to be used as a small lake, perhaps for irrigation.

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

8.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates three boreholes are located within a 200m of the site. However, these boreholes do not have records of the strata encountered. One borehole approximately 250m from the site indicates the ground to comprise layers of gravelly sand, sand and clay, clay-bound gravels and sand.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/8937	~30m	~0.72m bgl	250m NE

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] Forsyth P.J., Barrell D.J.A., & Jongens R. 2008: Geology of the Christchurch Area. Institute of Geological and Nuclear Sciences 1:250,000 Geological Map 16. Lower Hutt. Institute of Geological and Nuclear Sciences Limited.

8.1.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing in the area of the subject site.

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 is classified as Technical Category Not Applicable (TC N/A). This means that non-residential properties in urban areas, properties in rural areas or beyond the extent of land damage mapping have not been given a Technical Category.

8.1.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows a large amount of material on site (and properties adjacent) that maybe liquefaction as shown in Figure 3, although this material could also be evident on the surface due to a dry summer. However, lateral spreading is visible to the south along the Styx River. Material in the base of the drainage channel appears to be sand ejected from liquefaction.

Figure 3 Post February 2011 Earthquake Aerial Photography [2]

8.1.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise layers of gravelly sand, sand and clay, clay-bound gravels, and sand.

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

8.2 Seismicity

8.2.1 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[3,4]

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	26 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

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.

8.2.2 Ground Shaking Hazard

This seismic activity has produced earthquakes of Magnitude-6.3 with peak ground accelerations (PGA) up to twice the acceleration due to gravity (2g) in some parts of the city. This has resulted in widespread liquefaction throughout Christchurch.

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

In addition, anticipation of recent alluvial deposits, a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 20023), and bedrock anticipated to be in excess of 500m deep, ground shaking is expected to be moderate to high.

8.3 Slope Failure and/or Rockfall Potential

The topography surrounding the site is typically flat, and hence rockfalls are not considered to be a hazard at this site. However, given the site's proximity to the Styx River, it is considered possible that lateral spreading may occur in the area.

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

[4] GNS Active Faults Database

In addition, any localised retaining structures should be investigated to better establish the site-specific slope instability.

8.4 Liquefaction Potential

Due to the presence of alluvial deposits, and evidence from the post-earthquake aerial photography it is possible and likely that liquefaction will occur where sands and silts are present.

This may result in lateral spreading along and towards the Styx River, which was evident following 4 September and 22 February earthquakes, south of the subject site, or along the old river channel.

Further investigation is recommended to better determine subsoil conditions. An intrusive investigation comprising one piezocone CPT test to 20m bgl is recommended to allow a more comprehensive liquefaction assessment to be undertaken.

8.5 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.

The site appears to be situated on stratified alluvial deposits, comprising gravel, sand and silt. Associated with this the site also has a moderate to high liquefaction potential, in particular where sands and/or silts are present. There is the potential for liquefaction to manifest as lateral spreading at this site.

To allow a more comprehensive liquefaction and/or ground condition assessment to be undertaken, it is recommended that an intrusive investigation comprising of at least one piezocone CPT be conducted.

A soil class of **D/E** (in accordance with NZS 1170.5:2004) should be adopted for the site. Further refinement can only be made by reviewing intrusive investigation data.

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 information available. The building's 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 4. These capacities are subject to confirmation by a more detailed quantitative analysis.

Item	%NBS
Building Capacity excluding CSW's	53%
Building Capacity including:	
Site Characteristics (Liquefaction/Lateral Spread; 30% Reduction)	37%

Table 4 Indicative Capacities based on the NZSEE Initial Evaluation Procedure

Following an IEP assessment, the building has been assessed as achieving 37% 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 greater than 33% NBS. The overall %NBS has been reduced by 30% for significant Site Characteristics, owing to the liquefaction and lateral spread potential for the soil underneath.

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: E, NZS 1170.5:2004, Clause 3.1.3, Very 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.

Some key seismic parameters 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 1.25 has been assumed based on the timber frame structure (without internal linings) and date of construction.

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 potentially Class E soils. This building would have been designed to the standards at the time, namely NZS1900:1965. The design loads used in this code would have been significantly less than those required by the current loading standard. When combined with the increase in the hazard factor for Christchurch to 0.3 and the potential for liquefaction and lateral spreading of the soil underneath, it is reasonable to expect the building to be classified as a potential Earthquake Risk.

10.5 Occupancy

The building does not pose an immediate risk to users and occupants as no extant collapse hazards due to seismic damage have been identified. A critical structural weakness due to adverse site characteristics was identified in the assessment process. However, the building has scored greater than 33% 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 in the order of 37% NBS and is therefore a potential Earthquake Risk but not potentially Earthquake Prone. As a result, occupancy of the building is allowed.

12. Recommendations

The building has achieved greater than 33% NBS but less than 67% NBS capacity according to an initial IEP assessment, which classifies the building as a potential Earthquake Risk. The building does not present any collapse hazard, though it does exhibit a critical structural weakness. The building should be subjected to further detailed seismic assessment, and occupancy of the building should not be prohibited.

13. Limitations

13.1 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.

13.2 Geotechnical Limitations

This report presents the results of a geotechnical appraisal prepared for the purpose of this commission, and for prepared solely for the use of Christchurch City Council and their advisors. The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical engineer before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data.

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that Geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

Appendix A

Photographs

Photograph 1: Northwest corner elevation.

Photograph 2: Southwest corner elevation. Note proximity to the creek at the corner of the building.

Photograph 3: Interior view at entry of cross-braced walls and truss roof members

Photograph 4: Interior view at entry of cross-braced walls and truss roof members, including mid-span lintel post.

Photograph 5: Interior view of cross-braced rear wall and foundation wall below, and timber trusses above.

Photograph 6: Interior view of cross-braced rear wall and foundation wall, timber trusses above, and part of northern wall.

Photograph 7: Interior view of southern wall and foundations below, trusses above, and partial view of entry.

Photograph 8: Interior view of the northern wall, trusses above, partial view of entry and partial view of rear wall and its foundations.

Photograph 9: Exterior wall cladding at northern wall.

Photograph 10: Proximity of river at southeast corner. Note proximity to the creek.

Appendix B

Existing Drawings

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

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Barn 004 - Styx River Reserve #2	Reviewer:	Stephen Lee
	Unit / No: / Street	CPEng No:	1006840
Building Address:	303 Radcliffe Rd, Belfast, Christchurch	Company:	GHD
Legal Description:	Lot 3 DP 313448	Company project number:	513059654
		Company phone number:	6433780900
	Degrees / Min / Sec		
GPS south:	43 / 27 / 55.19	Date of submission:	22/05/2013
GPS east:	172 / 39 / 1.25	Inspection Date:	16/4/2012
		Revision:	Final
Building Unique Identifier (CCC):	PRK 2456 BLDG 004	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):	E		
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:	strip footings	if Foundation type is other, describe:	
Building height (m):	3.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	2.6
Floor footprint area (approx):			
Age of Building (years):	42	Date of design:	1965-1976
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):	Hay barn		
Importance level (to NZS1170.5):	IL1		

Gravity Structure

Gravity System:	frame system		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:			Floor on-grade
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)	11.4
Ductility assumed, μ:	1.25			
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)	6.2
Ductility assumed, μ:	1.25			
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:	profiled metal	describe	Corrugated steel
Roof Cladding:	Metal	describe	Corrugated steel
Glazing:			
Ceilings:	none		Diagonal cross-bracing
Services(list):			

Available documentation

Architectural	partial	original designer name/date	Adjacent property records
Structural	none	original designer name/date	
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		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):	High 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:		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):		
Across	Damage ratio:	0%	
	Describe (summary):		
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:	37%	37% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	37%		
Across	Assessed %NBS before:	37%	37% %NBS from IEP below	
	Assessed %NBS after:	37%		

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): 1965-1976

h_n from above: 2.6m

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:	6.4%	6.4%

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 $(\%NBS)_{nom}$:	**6%**	**6%**

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	
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)	1.25	1.25
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.14	1.14
Ductiity Scaling Factor, **Factor D**:	1.14	1.14

2.6 Structural Performance Scaling Factor:

Sp:	0.925	0.925
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.081081081

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

$\%NBS_b$:	**53%**	**53%**

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	significant	0.7

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 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:	**37%**	**37%**
4.4 Percentage New Building Standard (%NBS), (before)		**37%**

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Document Status

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