Bromley Park Pavilion and Toilets
PRK 0766 BLDG 001

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

170 Buckleys Road, Linwood

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

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170 Buckleys Road, Linwood

Christchurch City Council

Prepared By

Alex Baylis

Reviewed By

Stephen Lee

Date

22 May 2013

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Quantitative Report Summary

Bromley Park Pavilion and Toilets

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

170 Buckleys Road, Linwood

Background

This is a summary of the Quantitative report for the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 31 January 2013.

Building Description

The single storey building is located at 170 Buckleys Road, Linwood. The date of construction is estimated to be during the 1970s based on construction characteristics and site observation. An extension to the southern side of the building is estimated to have been constructed within the last 10 years.

The building is approximately 12m in length by 7.5m in width with a height of 3.6m and occupies a footprint of approximately $90m^2$.

The structure of the building consists of concrete masonry walls supporting a lightweight timber framed roof with a lightweight timber framed extension.

Key Damage Observed

The building was observed to generally be in good condition during the inspection. Minor cracking and chipping of the concrete slab was observed at the joint with the concrete apron. No residual displacements of the structure were observed during the inspection of the building.

Evidence of liquefaction was observed in Bromley Park during inspections. No ground damage was observed in the immediate vicinity of the building.

Building Capacity Assessment

The building has been assessed to have a seismic capacity in the order of 21% NBS and is therefore Earthquake Prone.

Recommendations

It is recommended that Christchurch City Council proceed with developing potential strengthening concepts for the building.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the pavilion and toilets in Bromley Park.

This report is a Quantitative 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 quantitative assessment involves a full site measure of the building which is used to determine the building's bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from the NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes. The demand for the building is determined and the percentage of New Building Standard (%NBS) is assessed.

At the time of this report, no intrusive site investigation or modelling of the building structure had been carried out.

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply ‘as near as is reasonably practicable’ with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Figure 1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Table 1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Table 1 %NBS compared to relative risk of failure

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

4. Building Descriptions

4.1 General

The single storey building is located at 170 Buckleys Road, Linwood. The date of construction is estimated to be during the 1970s based on construction characteristics and site observation. An extension to the southern side of the building is estimated to have been constructed within the last 10 years. The building is approximately 12m in length by 7.5m in width with a height of 3.6m and occupies a footprint of approximately $90m^2$.

Bromley Park is bordered by residential properties to the north, east, and west and Linwood Cemetery to the south. Buckleys Road is located on the northern side of the park. The closest building to the pavilion is a residential property approximately 75m to the east.

The structure of the building consists of concrete masonry walls supporting a lightweight timber framed roof with a lightweight timber framed extension.

The roof of the timber framed extension has been constructed over the southern half of the roof of the original section of the building. The original roof has been left intact. The roof structure of the original concrete masonry section of the building consists of a duo-pitch roof formed by corrugated sheet metal on plywood as shown in Photograph 4. The mono-pitch roof structure of the timber framed extension consists of corrugated sheet metal on lightweight timber framing with a plasterboard ceiling.

The walls in the original section of the building consist of 190mm thick partially filled concrete masonry units. The concrete masonry walls are unreinforced except for reinforcing bars around wall openings. The transverse walls have timber framed gable infill walls above the concrete masonry. The concrete masonry walls form the changing room and storage areas of the building.

The timber framed walls are lined internally with plasterboard and are connected to the original concrete masonry section of the building. The timber framed walls form the toilet and shower areas.

The foundations of the building are assumed to consist of a concrete slab-on-grade and concrete strip footings beneath the external and internal walls.

Figure 2 and Figure 3 below show the construction details.

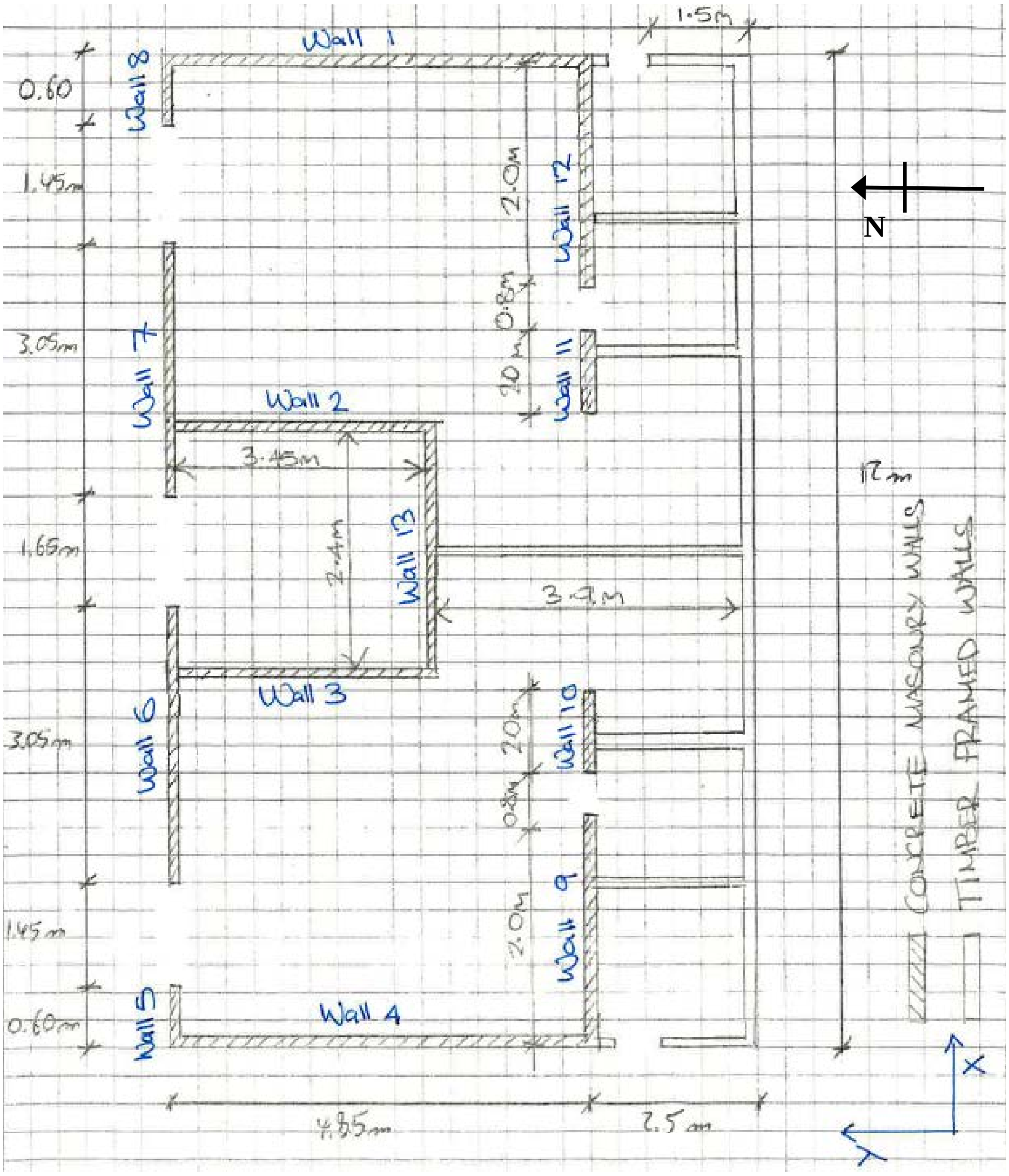


Figure 2 Floor plan of building

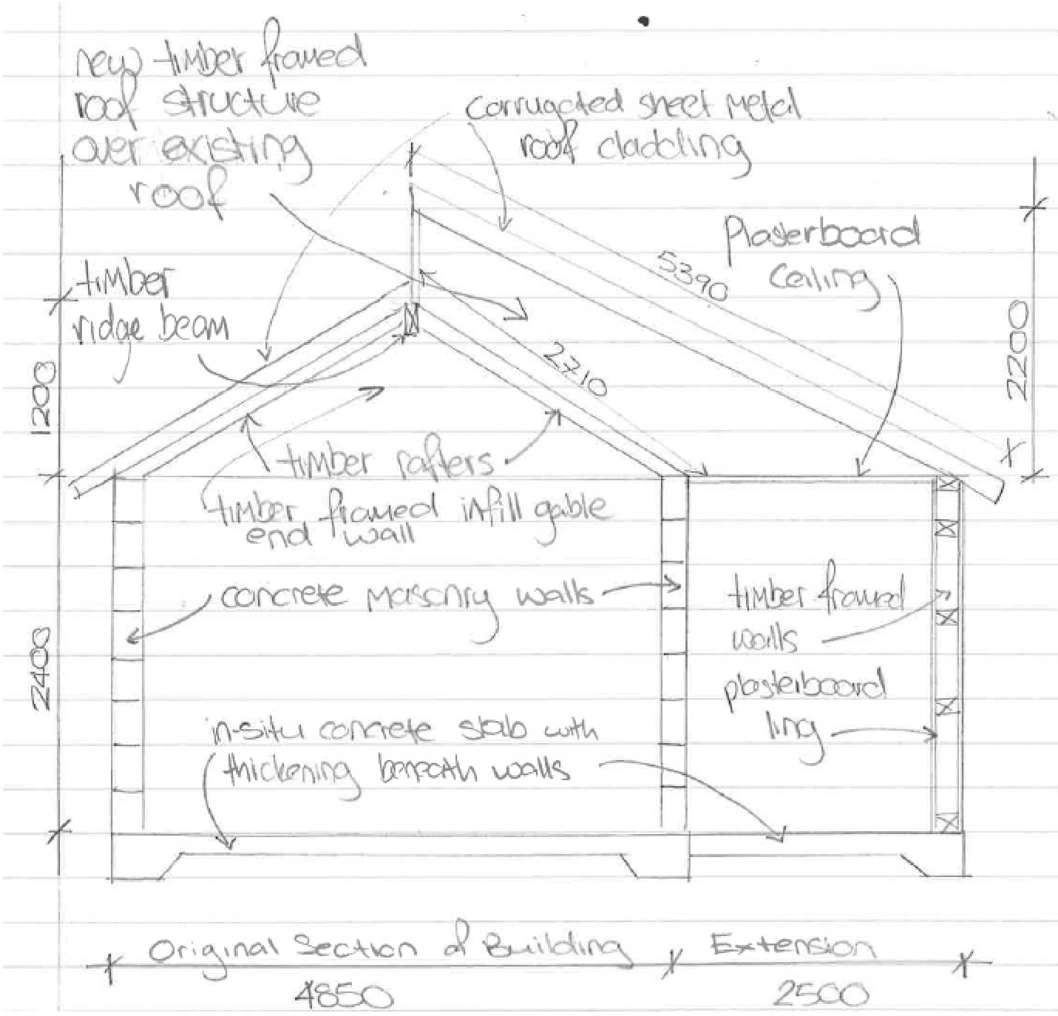


Figure 3 Building cross section

4.2 Gravity Load Resisting Systems

Gravity loads acting on the building are resisted by load bearing concrete masonry walls and timber framed walls. In the original section of the building, gravity loads from the corrugated steel roof are transferred via the timber rafters to the longitudinal concrete masonry walls. The gravity loads are transferred through the concrete masonry walls to the concrete strip footings where they are distributed into the ground.

In the extension, gravity loads from the corrugated steel roof are transferred via the timber roof framing to the load bearing timber framed walls. The gravity loads are transferred through the timber framed walls to the concrete strip footings where they are distributed into the ground.

Floor gravity loads are transferred through the reinforced concrete slab to the underlying ground.

4.3 Lateral Load Resisting Systems

In the original section of the building, the plywood panel ceiling lining provides a diaphragm to transfer seismic forces through the roof structure to the walls in the plane of loading. Lateral seismic loads in both the transverse and longitudinal direction are resisted by the concrete masonry walls in the plane of loading. The lateral forces are resisted by the panel action of concrete masonry units. Forces are transferred to the foundations through shear and bending of the concrete masonry walls.

In the timber framed extension to the building, the plasterboard ceiling lining provides a diaphragm to transfer seismic forces from the roof structure to the plasterboard lined timber framed walls in the plane of loading. The lateral seismic loads in both the longitudinal and transverse directions of the timber framed extension are resisted by the plasterboard lined timber framed walls which act as in-plane shear bracing panels.

The concrete masonry walls are restrained at eaves level by the timber framed roof structure. Wall 13 (see Figure 2) is partial height and is not restrained by the timber roof diaphragm.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 31st of January 2013. Both the interior and exterior of the building was inspected. It should be noted that inspection of the foundations of the structure was limited to the top of the external strips exposed above ground level.

The inspection consisted of observing the building to determine the structural systems and likely behaviours of the building during earthquake. The site was assessed for damage, including observing the ground condition, checking for damage areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

5.2 Available Drawings

Drawings of the building were not available.

Sketches of the key structural features of the building are attached as Appendix B.

5.3 Damage Assessment

5.3.1 Surrounding Buildings

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

5.3.2 General Observations

The building was observed to generally be in good condition during the inspection. Minor cracking and chipping of the concrete slab was observed at the joint with the concrete apron as shown in Photograph 8.

No residual displacements of the structure were observed during the inspection of the building.

5.3.3 Ground Damage

Evidence of liquefaction was observed in Bromley Park during inspections. No ground damage was observed in the immediate vicinity of the building.

6. Geotechnical Consideration

6.1 Site Description

The site is situated in the suburb of Linwood, east of Christchurch City centre. The site is relatively flat at approximately 10m above mean sea level. It is approximately 1.1km south of Avon River, 2km north of the Heathcote River, and 4.5km west of the coast (Pegasus Bay).

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by:

- Marine deposits of Christchurch Formation, dominantly sand of fixed and semi-fixed dunes and beaches, Holocene in age.

Due to the low-lying location of the site, shallow ground water table is anticipated.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there are seven boreholes located within 200m of the site. Two boreholes with significant information are summarised in Table 2.

These indicate that the area is underlain by layers of sand and gravel with varying amount of clay at intermediate layers.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35-1928	138.6	2.4m bgl	110m S
M35-2442	120	2.5m bgl	122m SW

It should be noted that the boreholes 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.

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Linwood[2]. One investigation points were undertaken within 200m of the site, as summarised in Table 3.

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

[2] Tonkin & Taylor Ltd., 2011: Christchurch Earthquake Recovery, *Geotechnical Factual Report, Linwood.*

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary
CPT-LWD-11	170m N	0 – 1.0	Pre-drilled
		1.0– 2.5	Sand
		2.5-5	Gravelly Sand
		5-8	Sand
		8-20	Sand/Clayey Sand
			(WT at 4.1m bgl)

Initial observations of the CPT result indicate the site is underlain by sand and gravels with varying amount of gravel and clay.

6.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has classified 170 Buckleys Road, Linwood as “Green Zone – N/A – Urban Non-residential” category. Land in this zone is generally considered suitable for residential construction, though some areas may require stronger foundations or design where rebuilding or repairs are required. “Not Applicable – Urban Non-residential” technical category is the classification given for non-residential properties in urban area beyond the extent of land damage mapping.

However, properties to the east of the site are classified as “Green Zone, Technical Category 3 – blue.” Land in this zone is generally considered suitable for residential construction, though some areas may require stronger foundations or design where rebuilding or repairs are required. Technical Category 3, blue means that moderate to significant land damage from liquefaction is possible in future significant earthquakes.

6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of moderate to significant liquefaction at road corridors and nearby properties, as shown in Figure 4.

Figure 4 Post February 2011 Earthquake Aerial Photography[3]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise multiple strata of sand and gravel with varying amounts of clay.

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

6.3 Seismicity

6.3.1 Nearby Faults

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	125 km	NW	~8.3	~300 years
Greendale (2010) Fault	25 km	W	7.1	~15,000 years
Hope Fault	105 km	NW	7.2~7.5	120~200 years
Kelly Fault	105 km	NW	7.2	150 years
Porter Pass Fault	65 km	W	7.0	1100 years

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains, including Christchurch City, and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

6.3.2 Ground Shaking Hazard

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 recent 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.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Linwood, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

[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, June 2002, pp. 1878-1903.

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

6.5 Liquefaction Potential

The site is considered to be moderately to significantly susceptible to liquefaction, due to the following reasons:

- Signs of severe liquefaction at road corridors and properties near the site (evidence from the post-earthquake aerial photograph);
- Anticipated presence of saturated sand layers beneath the site; and,
- Anticipated shallow ground water table.

6.6 Conclusions & Recommendations

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

The site appears to be situated on marine deposits. Associated with this the site also has a moderate to high liquefaction potential, in particular where sands and/or silts are present.

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

Should a more comprehensive liquefaction and/or ground condition assessment be required, it is recommended that intrusive investigation be conducted.

7. Structural Analysis

7.1 Seismic Parameters

Seismic loading on the structure has been determined using New Zealand Standard 1170.5:2004.

◗ Site Classification	D
◗ Seismic Zone factor (Z)	
(Table 3.3, NZS 1170.5:2004 and NZBC Clause B1 Structure)	0.30 (Christchurch)
◗ Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/500 (ULS) Importance Level 2
◗ Return Period Factor (R_u)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)

Concrete Masonry

◗ Ductility Factor (μ)	1.0
◗ Ductility Scaling Factor (k_μ)	1.0
◗ Performance Factor (S_p)	1.0

Timber Framing

◗ Ductility Factor (μ)	3.0
◗ Ductility Scaling Factor (k_μ)	2.14
◗ Performance Factor (S_p)	0.7

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing.

The structural performance factor, S_P, was calculated in accordance with Clause 4.4.2 NZS 1170.5.

$$S_P = 1.3 - 0.3\mu \geq 0.7$$

The seismic weight coefficient was then calculated in accordance with Clause 5.2.1.1 of NZS 1170.5:2004. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4 was assumed for both directions of the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. The distribution of lateral forces in the timber framed section of the building follows the bracing design procedure discussed in Section 5 of NZS 3604:2011. The seismic bracing demand in each direction was resolved into Bracing Units (BUs) and compared to the bracing capacity of the timber walls. In both the longitudinal and transverse directions, a ductility factor of 3.0 has been assumed based on the relatively flexible, lightweight timber framed walls resisting lateral seismic loads

The lateral seismic forces have been distributed to the concrete masonry walls assuming that the roof structure over the original section of the building behaves as a rigid diaphragm and that the lateral load resisted by each wall is proportional to the stiffness of each wall. An accidental eccentricity of 10% has been assumed in each direction.

The structure is considered to be brittle. As a result, 30% loading from the other orthogonal direction has been included when determining the loading on the masonry walls for an earthquake in a particular direction as per NZS 1170.5:2004 requirements.

A ductility factor of 1.0 has been assumed in both the longitudinal and transverse direction based on the unreinforced concrete masonry walls that resist lateral seismic loading.

7.3 Capacity of Structural Elements

7.3.1 Timber Framed Wall Bracing Capacity

The bracing capacity of the timber framed walls in both the longitudinal and transverse directions was calculated in accordance with NZS 3604:2011 and the NZSEE guidelines. The demand for each building was calculated in accordance with NZS 1170.5:2004 and resolved into Bracing Units (BUs) for comparison.

There is no reliable information available regarding the bracing capacities of the plasterboard lining to the timber framed walls. Assumptions regarding the likely bracing capacity of the plasterboard lined timber walls have been made in accordance with Table 11.1 of the in NZSEE guidelines. A bracing capacity value of 3 kN/m (60 BU/m) and a strength reduction factor of 0.7 have been used in calculations.

Section 11.4 of the NZSEE guidelines suggests that shear panels may utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. For aspect ratios greater than 2:1 and up to 3.5:1 a limiting factor may be applied in accordance with the NEHRP Recommended Provisions (BSSC, 2000) as follows;

$$\text{Aspect Ratio Factor} = \frac{2 \times \text{Width}}{\text{Height}}$$

Any sections of wall with an aspect ratio greater than 3.5:1 were not included in the bracing calculations.

The buildings were also checked against the current requirements in NZS 3604:2011 for spacing of bracing lines, minimum bracing line values, diaphragm spans and the bracing capacities of walls supporting diaphragms.

7.3.2 Unreinforced Masonry In-Plane Shear Capacity

The in-plane shear capacity of the unreinforced concrete masonry walls was determined using Section 8.4 of the NZSEE guidelines "Assessment & Improvement of Unreinforced Masonry Building for Earthquake Performance (2011)". The strength reduction factor, ϕ, for shear and shear friction was taken as 0.85 in accordance with NZSEE guidelines. The overall shear capacity of each wall was evaluated considering four shear failure modes. These are diagonal tension failure, rocking failure, bed-joint sliding failure and toe crushing failure. The in-plane shear capacity of each wall is,

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

7.3.3 Unreinforced Masonry In-Plane Moment Capacity

The in-plane flexural capacity of the unreinforced concrete masonry walls was calculated as,

$$M_n = N_b \left[Z - \frac{1}{2} \times \frac{N_b}{0.85 f'_m t_w} \right]$$

$$Z = {}^{L_w}/_{2}$$

Where

N_b = normal force acting at wall base

f'_m = compressive strength of masonry

t_w = wall thickness

L_w = wall length

7.3.4 Unreinforced Masonry Out-of-Plane Capacity

The out-of-plane flexural capacity of the unreinforced concrete masonry walls was determined using Section 10.3 of the NZSEE guidelines "Assessment & Improvement of the Structural Performance of Buildings in Earthquakes (2006)". The overall out-of-plane capacity of each wall was evaluated by comparing the likely displacement of the wall during an earthquake and the displacement that would cause instability of the wall. The out-of-plane capacity of each wall is,

$$\%NBS = 0.72 \frac{\Delta_i}{D_{ph}}$$

Where

Δ_i = out-of-plane deflection that would cause instability

D_{ph} = out-of-plane displacement response demand for a wall panel

7.3.5 %NBS

The shear and bending moment capacities of the concrete masonry walls and timber framed walls were compared to their respective demands to determine the overall %NBS.

$$\%\text{NBS} = \frac{V_n}{V^*} \text{ x } 100$$

$$\%\text{NBS} = \frac{M_n}{M^*} \text{ x } 100$$

8. Results

The New Zealand Society for Earthquake Engineering (NZSEE) publication ‘Assessment & Improvement of Structural Performance of Buildings (2006)’ and the relevant New Zealand material standards were used to provide a framework and method for the analysis. Our analysis applied live loads, imposed dead loads and seismic loads to the structure. The elements were then assessed against their respective load capacities.

Our calculations show that the structure achieves **21% NBS** and is therefore **Earthquake Prone.**

The structural analysis results are discussed in the following sections.

8.1.1 Unreinforced Concrete Masonry Walls

In-Plane Shear

The unreinforced concrete masonry walls achieve **21% NBS** under in-plane shear seismic loading.

In-Plane Moment

The unreinforced concrete masonry walls achieve **26% NBS** when considering in-plane bending of the walls.

Out-of-Plane Moment

The unreinforced concrete masonry walls achieve **24% NBS** when considering out-of-plane bending of the walls.

8.1.2 Timber Framed Walls

The plasterboard lined timber framed walls achieve **100% NBS** under in-plane shear seismic loading.

The bracing demand was determined by evaluating the seismic weight of the extension and multiplying this value by the horizontal design action coefficient corresponding to the timber framed walls. The demand was then resolved into bracing units (BUs) for comparison with bracing capacities of timber framed walls.

The total bracing capacity of the extension was evaluated by determining the lengths of plasterboard lined timber framed walls available that satisfy the aspect ratio limit of 3.5:1 suggested in the NZSEE guidelines.

The timber framed wall bracing system for the extension satisfies current NZS 3604:2011 requirements for minimum bracing line capacities, maximum spacing of bracing lines and ceiling diaphragm requirements.

8.1.3 Timber Diaphragm

No information is available regarding the detailing of the timber plywood diaphragm. For the purposes of this assessment it has been assumed that fixing of the roof diaphragm is in accordance with the minimum requirements of NZS4229:1999.

Therefore, based on NZS 4229:1999, it has been assumed that the plywood sheets are fixed with 2.5mm diameter flathead nails spaced at 150mm centres. The wall plates are assumed to be fixed to the concrete masonry walls by centrally placed single M12 anchors at 1200mm centres.

The timber diaphragm achieves **23% NBS.** The critical aspect of the seismic performance of the timber diaphragm is the connection between the concrete masonry walls and the timber wall plate.

8.2 Summary

Element	Seismic Action	%NBS
Longitudinal Direction		
Concrete Masonry Walls	In-Plane Shear	21
	In-Plane Bending	26
	Out-of-Plane Bending	24
Timber Diaphragm	In-Plane Shear	84
	In-Plane Bending	100
	Diaphragm Connections	23
Timber Framed Walls	In-Plane Shear	100
Transverse Direction		
Concrete Masonry Walls	In-Plane Shear	30
	In-Plane Bending	45
	Out-of-Plane Bending	71
Timber Diaphragm	In-Plane Shear	32
	In-Plane Bending	100
	Diaphragm Connections	56
Timber Framed Walls	In-Plane Shear	100

Table 5 Summary of %NBS scores

8.3 Discussion of Results

The results obtained from the analysis are generally consistent with those expected for a building of this age and construction type.

The building is assumed to have been designed in the 1970s and was likely designed in accordance with the loading standard, NZS 4203:1976. The design loads used are likely to have been less than those required by the current loading standard.

The timber framed section of the building performs well in both the transverse and longitudinal directions. The timber framed walls achieve 100% NBS and meet minimum spacing and bracing values required by NZS3604:2011.

The concrete masonry walls in the original section of the building are unreinforced and as a result, there is a significant risk of wall failure during a seismic event. It is therefore reasonable to expect the detailed assessment of the structure to indicate that the building is Earthquake Prone.

9. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of 21% NBS and is therefore Earthquake Prone. It is recommended that Christchurch City Council proceed with developing potential strengthening concepts for the building.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No material testing has been undertaken.

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.

10.2 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 Ministry of Education 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 View of the building from the north-east

Photograph 2 View of the building from the north-west

Photograph 3 **Eastern elevation of the building**

Photograph 4 **Timber framed infill wall**

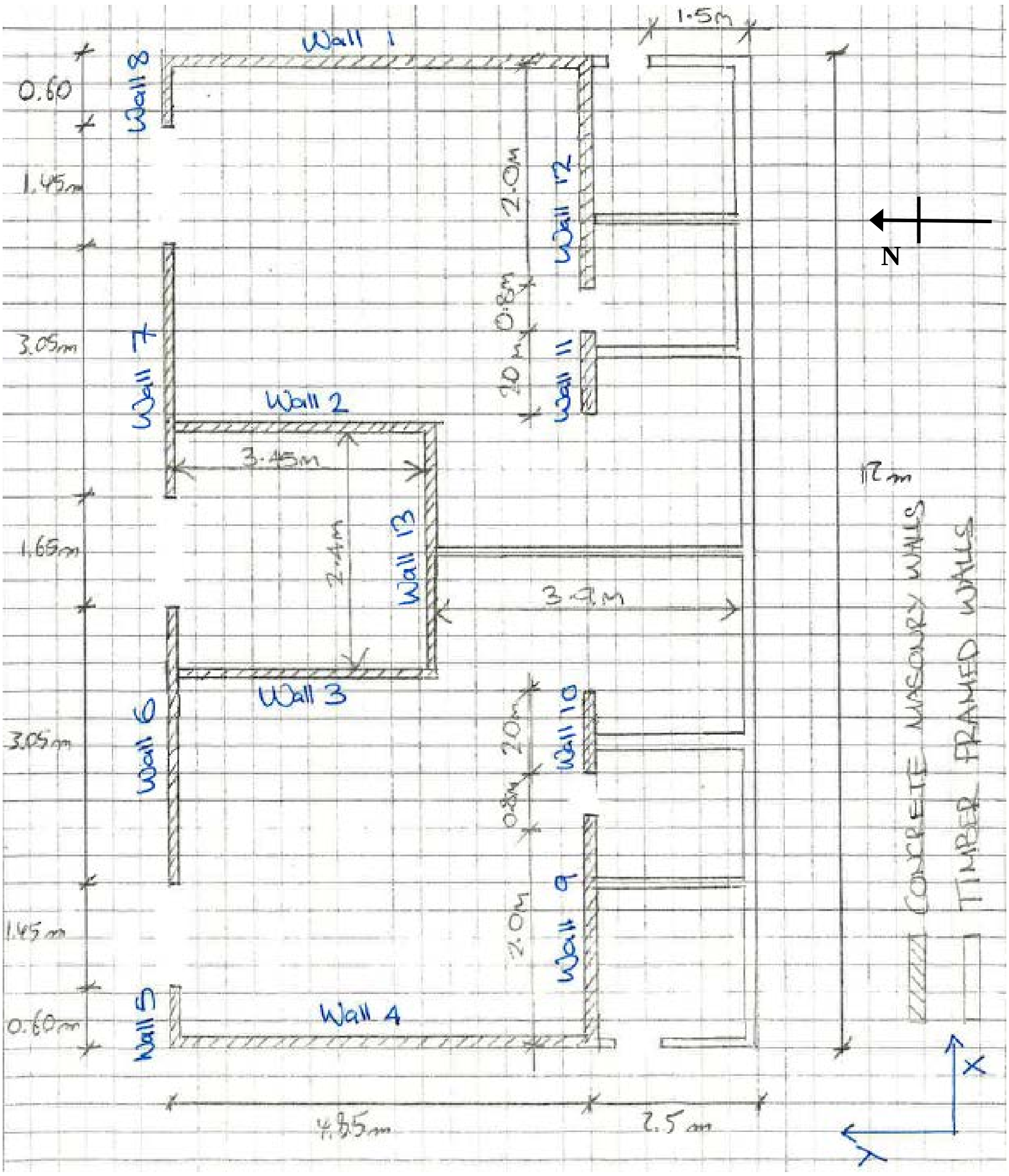
Photograph 5 Concrete masonry lintel over openings

Photograph 6 Plywood lined timber roof structure

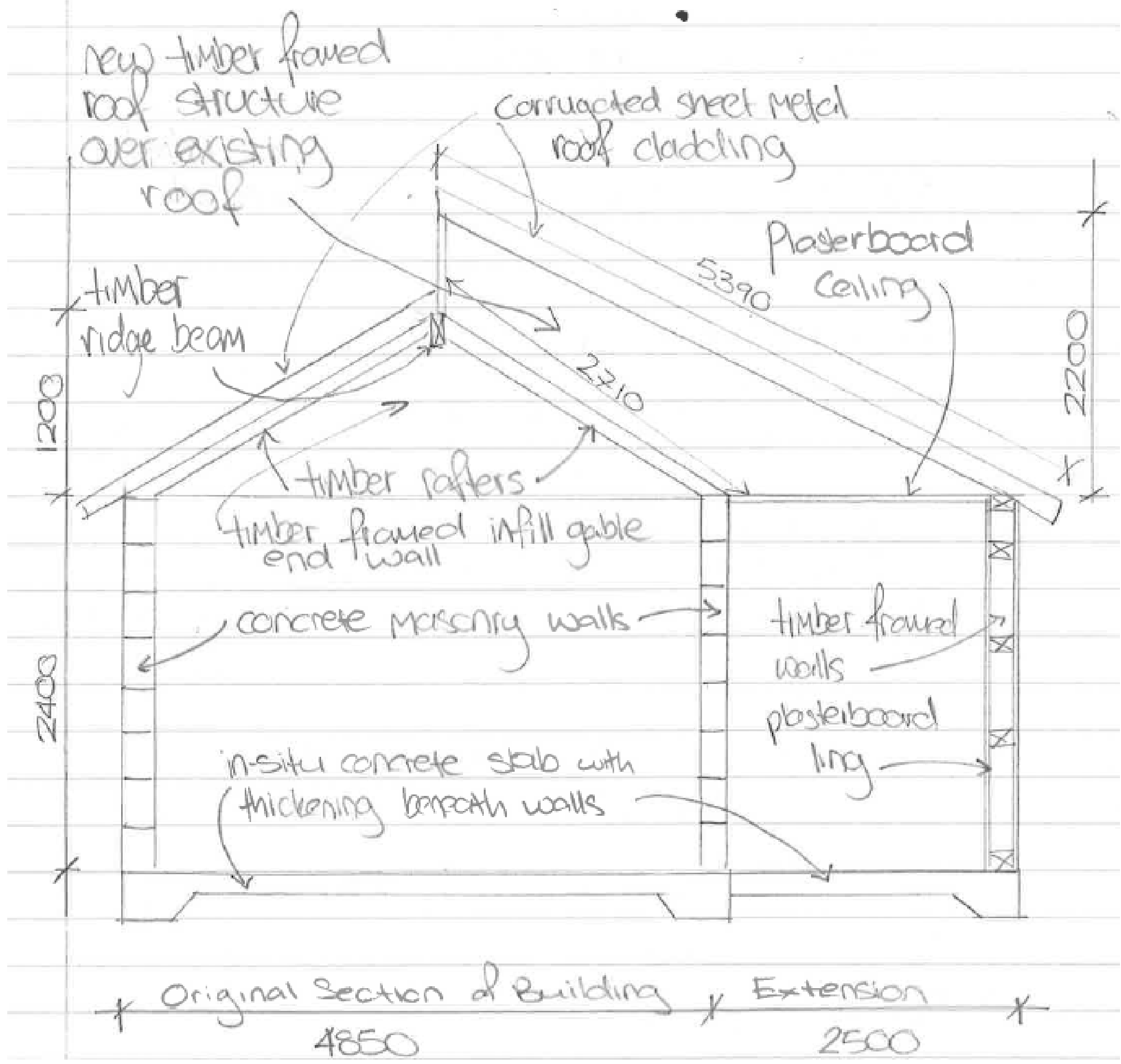
Photograph 7 Cracking of concrete slab at joint with concrete apron

Appendix B

Sketches



Wall 1
1.5m
0.60
Wall 8
1.45m
2.0m
Wall 12
N
0.8m
3.05m
Wall 7
2.0m
Wall 11
Wall 2
3.45m
12m
1.65m
2.4m
Wall 13
3.9m
CONCRETE MASONRY WALLS
TIMBER FRAMED WALLS
Wall 3
Wall 10
2.0m
3.05m
Wall 6
0.8m
Wall 9
2.0m
1.45m
Wall 5
Wall 4
0.60m
4.85m
2.5m
X
Y


new timber framed roof structure over existing roof
corrugated sheet metal roof cladding
Plasterboard Ceiling
5390
timber ridge beam
2710
2200
1200
timber rafters
timber framed infill gable end wall
concrete masonry walls
timber framed walls
plasterboard lining
2400
in-situ concrete slab with thickening beneath walls
Original Section of Building
Extension
4850
2500

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Bromley Park Pavilion and Toilets	Reviewer:	Stephen Lee
	Unit / No: 170 / Street: Buckleys Road, Linwood	CPEng No:	1006840
Building Address:	170 Buckleys Road, Linwood	Company:	GHD
Legal Description:		Company project number:	51/30902/46
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	1/31/2013
		Revision:	
Building Unique Identifier (CCC):	PRK_0766_BLDG_001 EQ2	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):		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:	mat slab	if Foundation type is other, describe:	Slab on grade
Building height (m):	3.60	height from ground to level of uppermost seismic mass (for IEP only) (m):	2.4
Floor footprint area (approx):	90		
Age of Building (years):	40	Date of design:	1976-1992
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):	Pavilion and Toilets		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:			
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	190

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	partially filled CMU		note total length of wall at ground (m):	
Ductility assumed, μ:	1.00		wall thickness (m):	
Period along:	0.40	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	partially filled CMU		note total length of wall at ground (m):	
Ductility assumed, μ:	1.00		wall thickness (m):	
Period across:	0.40	##### enter height above at H31	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	exposed structure	describe	Painted Block Walls and full plaster
Roof Cladding:	Heavy tiles	describe	Light corrugated steel
Glazing:	aluminium frames		
Ceilings:	plaster, fixed		
Services(list):			

Available documentation

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

Damage

Site: (refer DEE Table 4-2)

Site performance:	Good	Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Minor damage observed
	Describe (summary):	Minor damage observed		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage observed		
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:		Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before:	21% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Detailed Assessment
	Assessed %NBS after:	21%		
Across	Assessed %NBS before:	30% ##### %NBS from IEP below		
	Assessed %NBS after:	30%		

GHD

Level 11, Guardian Trust House
15 Willeston street, Wellington 6011
T: 64 4 472 0799   F: 64 4 472 0833   E: wgtnmail@ghd.com

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Alex Baylis	Stephen Lee		Nick Waddington		22/5/13