CLIENTS | PEOPLE | PERFORMANCE

Macfarlane Park Pavilion
PRK_0663_BLDG_001 EQ2
Detailed Engineering Evaluation

Quantitative Report

Version FINAL

130 Skipton St

Mairehau

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

Macfarlane Park pavilion
PRK 0663 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

130 Skipton Street

Christchurch City Council

Prepared By
Kevan O Brien

Reviewed By
David Lee

Date
15 March 2013

Contents

Table Index

Figure Index

Appendices

Quantitative Report Summary

Macfarlane Park Pavilion

PRK 0663 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

135a Emmett Street, Mairehau

Background

This is a summary of the Quantitative report for the Macfarlane Park Pavilion building, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and inspections on the 19th of July and the 5th of September 2012.

Building Description

The building is located within Macfarlane Park, to the western side adjacent to Skipton Street, Mairehau. The date of construction is estimated to be in the 1970s and is used as a public changing room and storage shed. The building appears to be in its original configuration with no modifications apparent.

The roof consists of corrugated metal cladding supported on timber purlins and rafters which span onto the external walls. The internal and external walls are constructed of 200mm thick concrete blocks. Scanning was carried out on the block walls to establish the extent of reinforcement in the walls. No reinforcement was located from the scans carried out. The ceiling consists of plasterboard linings fixed to the underside of the roof structure. The walls are assumed to be supported by strip foundations and the floor is concrete slab on grade. No drawings are available to confirm the details of the foundations.

Key Damage Observed

Cracking was observed in a section of the floor slab and minor heave was observed. However this cracking is not believed to affect the structural system of the building, see Photo 5 in Appendix A.

Cracking was observed in the block work walls where they met perpendicular to each other. Wall junctions do not appear to be adequately keyed in together, see Photo 6 in Appendix A.

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the baseline capacity (excluding critical structural weaknesses and earthquake damage) of the building has been assessed to be 29% NBS.

There was no damage nor critical structural weaknesses identified in our visual inspection; consequently the %NBS has not been reduced. The building has been assessed to have a seismic capacity of 29% NBS and is therefore Earthquake Prone.

Recommendations

As the structure has been assessed to have a %NBS less than 33% NBS, it is deemed to be Earthquake Prone. It is recommended that block masonry wal strengthening options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance with Christchurch City Councils Earthquake Prone Buildings policy and the NZSEE guidelines.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Macfarlane Park Pavilion.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, NZS 4230: 1990, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Concrete Masonry Buildings for Earthquake Resistance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006).

This quantitative assessment to the building comprises of an investigation of the in-plane and out-of-plane strengths of the unreinforced masonry block walls. The investigation is based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of the existing structural elements to resist the seismic forces applied to them. The capacity of the existing structural elements is compared to the demand placed on the elements to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the block masonry walls.

At the time of this report, no finite element modelling of the building structure has 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 that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

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

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

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

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

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

2.4 Building Code

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

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

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

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

3. Earthquake Resistance Standards

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

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

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

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

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

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

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

Table 1 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The building is located within Macfarlane Park, to the western side adjacent to Skipton Street, Mairehau. The date of construction is estimated to be in the 1970s and is used as a public changing room and storage shed. The building appears to be in its original configuration with no modifications apparent.

The roof consists of corrugated metal cladding supported on timber purlins and rafters which span onto the external walls. The internal and external walls are constructed of 200mm thick concrete blocks. Scanning was carried out on the block walls to establish the extent of reinforcement in the walls. No reinforcement was located from the scans carried out. The ceiling consists of plasterboard linings fixed to the underside of the roof structure. The walls are assumed to be supported by strip foundations and the floor is concrete slab on grade. No drawings are available to confirm the details of the foundations.

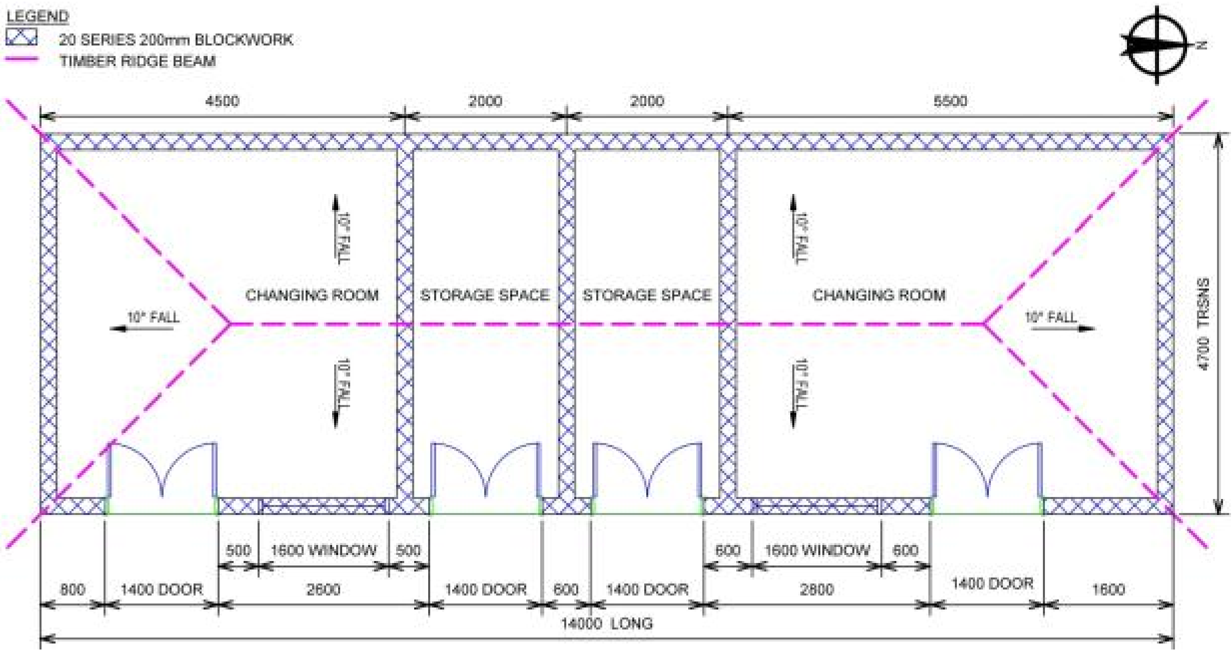

Figure 2 Plan Sketch Showing Key Structural Elements

The building is approximately 14.0m in length by 4.7m in width with an overall height of 3.4m. The building occupies a footprint of approximately 65.8m^2 and is 3m from the nearest structure, the neighbouring public toilets adjacent to the northern side of the building.

4.2 Gravity Load Resisting System

The gravity loads in the building are transferred from the roof through the timber rafters to the external blockwork walls. The loads are transferred through the walls to the supporting strip foundations. Floor loads are transferred directly through the concrete slab in to the underlying ground.

4.3 Lateral Load Resisting System

Lateral loads in the structure are resisted by the unreinforced block walls by in plane moment and shear resistance. In both the longitudinal and transverse directions of the building, the loads are transferred from the roof level through diaphragm action of the plasterboard lined ceiling to the block walls. The loads are then resisted by the walls which transfer the loads directly in to the concrete strip foundations.

5. Damage Assessment

5.1 Surrounding Buildings

The building is located next to an isolated toilet block to the northern side of the building, with approximately 3m of separation between the two structures. Nearby residential buildings have had their brick chimneys removed and patched with iron.

5.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

Cracking was observed in a section of the floor slab and minor heave was observed. However this cracking is not believed to affect the structural system of the building, see Photo 5 in Appendix A.

Cracking was observed in the block walls where they meet perpendicular to each other. Wall junctions do not appear to be adequately keyed in together, see Photo 6 in Appendix A.

5.3 Ground Damage

There was no evidence of ground damage in the park area.

6. Geotechnical Consideration

6.1 Site Description

The site is situated in the suburb of Mairehau, north of Christchurch City centre. It is relatively flat at approximately 10m above mean sea level. It is approximately 350m west of Shirley Stream, 6km north of Avon River, and 6km 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:

- Yaldhurst member of the Springston Formation, dominantly alluvial sand and silt overbank deposits, Holocene in age.

Figure 72 from Brown & Weeber indicates that groundwater is approximately 1m below ground level and and liquefaction susceptibility is low to moderate.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that six boreholes with lithographic logs are located within 200m of the site (see Table 2).

The soil condition in that location comprises of gravel, sand and silt layers, with some lenses of clay with ground water table at approximately 2m bgl.

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35-10325-WC	116.1 m	Not indicated	140m W
M35-16360-WC	3 m	Not indicated	96m W
M35-1712-WC	73.7 m	Not indicated	75m W
M35-1977-WC	93.6 m	3.12 m bgl	140m W
M35-2157-WC	82.9 m	2.7 m bgl	140m W
M35-2160-WC	116.1 m	2.7 m bgl	140m W
M35-2416-WC	142.6 m	2.7 m bgl	140m W

Table 2 ECan Borehole Summary

It should be noted the quality of soil logging descriptions included on the boreholes is unknown and were likely written by the well driller and not a geotechnical professional or to a recognised geotechnical standard. In addition strength data is not recorded.

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

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has not undertaken geotechnical testing within 200m of the site.

6.2.4 Land Zoning

Land adjacent to the site is classified as “Green Zone, Technical Category 2 – yellow” category. Land in the green zone is generally considered suitable for residential construction. Houses in some areas will need more robust foundations or site foundation design where foundation repairs or rebuilding are required. A “Technical Category 2 – yellow” means that minor to moderate 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 minor to moderate signs of liquefaction close to the site, as shown in Figure 3.

Figure 3 Post February 2011 Earthquake Aerial Photography[2]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise strata of gravel, sand and silt with lenses of clay with ground water table at approximately 2m bgl.

[2] 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.

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	24 km	W	7.1	~15,000 years
Hope Fault	106 km	NW	7.2~7.5	120~200 years
Kelly Fault	106 km	NW	7.2	~150 years
Porters Pass Fault	60 km	W	7.0	~1100 years

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

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, 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.

6.5 Liquefaction Potential

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

[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, June 2002, pp. 1878-1903.

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

- Signs of minor to moderate liquefaction close to the site (evidence from the post-earthquake aerial photography);
- Adjacent properties are classified by CERA as Green Zone, TC2- yellow;
- Anticipated presence of alluvial sand and silts deposits beneath the site; and,
- Shallow ground water level at approximately 2m bgl.

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 alluvial deposits, comprising sand and silt. Associated with this the site also has a minor to moderate liquefaction potential, in particular where sands and/or silts are present.

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

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

7. Assessment

A visual inspection of the building was undertaken on the 19th of July 2012, a further inspection of the building was carried out on the 5th of September 2012. Both the interior and exterior of the building were inspected. No placard was evident during the inspection, however based on the inspection carried out it would be expected to have a green placard.

The inspection consisted of observing the building to determine the structural systems and likely behaviour of the building during an earthquake. The site was assessed for damage, including examination of the ground conditions, checking for damage in areas where damage would be expected for the type of structure and noting general damage observed throughout the building in both structural and non-structural elements. No drawings were made available for the structure.

7.1 Quantitative Assessment

The quantitative assessment to the building comprised of an investigation of in-plane and out-of-plane strength of the masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The Hilti PS 200 Ferroscan was used to determine the level of reinforcement present in the walls, however no reinforcement was detected. The capacity of the existing structural elements was compared to the demand placed on the elements to give the %NBS of each of the structural elements. A full methodology of the calculation process is attached in Appendix B.

7.2 Seismic Coefficient

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

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

Where

$C_h(T)$ = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = 1.0, the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

The structural performance factor, S_P, was calculated in accordance with CL 4.4.2

$$S_P = 1.3 - 0.3\mu$$

Where µ, the structural ductility factor. A structural ductility factor of 1.0 has been taken for lateral loading across and along the building, this is due to the walls being constructed of unreinforced, unfilled concrete blocks.

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4s was taken for the structure. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.3 Bracing capacity of Un-reinforced Masonry Walls

7.3.1 In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

7.3.2 In-plane Wall Shear Capacity of the Unreinforced Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

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

7.3.3 Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes Section 10.3.

8. Capacity Assessment

8.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2004 and the NZBC clause B1 for this building are:

- Site soil class assumed to be: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil;
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011;
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

8.2 Wall Investigation

The position of each wall is indicated in the plans below and each wall is named accordingly.

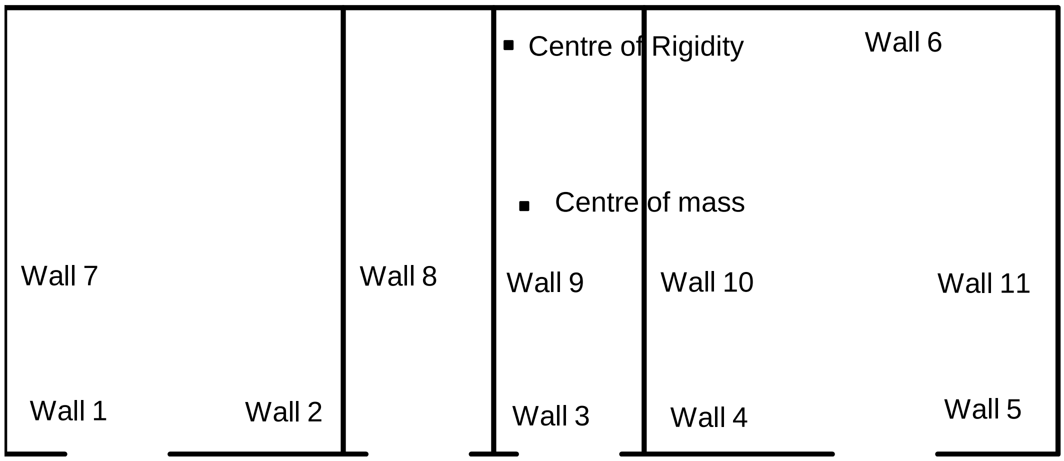

Figure 4 Plan Details and Wall Locations

8.3 Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 4. Walls 1-5 fail due to rocking failure while the remainder of the walls fail through diagonal tension.

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	0.8	0.5	63%	**Risk**	2.0	1.5	75%	**Not Risk or Prone**
2	8.9	5.6	63%	**Risk**	21.4	16.1	75%	**Not Risk or Prone**
3	0.5	0.3	63%	**Risk**	1.1	0.9	75%	**Not Risk or Prone**
4	10.4	6.5	63%	**Risk**	24.9	18.7	75%	**Not Risk or Prone**
5	3.4	2.1	63%	**Risk**	8.1	6.1	75%	**Not Risk or Prone**
6	133.3	47.4	36%	**Risk**	319.9	466.9	146%	**Not Risk or Prone**
7	44.8	15.7	35%	**Risk**	107.4	52.6	49%	**Risk**
8	34.7	14.9	43%	**Risk**	83.4	46.6	56%	**Risk**
9	30.3	14.9	49%	**Risk**	72.6	46.6	64%	**Risk**
10	34.8	14.9	43%	**Risk**	83.4	46.6	56%	**Risk**
11	49.8	15.7	31%	**Prone**	119.6	52.6	44%	**Risk**

Table 4 In Plane Analysis Results

The results of the out of plane displacement response capability analysis and subsequent earthquake designation under the NZSEE guidelines are listed in Table 5.

Wall number	Δ_i	D_{ph} kN	%NBS	Earthquake Status
1	0.092	0.227	29%	**Prone**
2	0.092	0.227	29%	**Prone**
3	0.092	0.227	29%	**Prone**
4	0.092	0.227	29%	**Prone**
5	0.092	0.227	29%	**Prone**
6	0.092	0.227	29%	**Prone**
7	0.092	0.227	29%	**Prone**
8	0.093	0.221	30%	**Prone**
9	0.093	0.221	30%	**Prone**
10	0.093	0.221	30%	**Prone**
11	0.092	0.227	29%	**Prone**

Table 5 Out Of Plane Analysis Results

8.4 Discussion of Results

Following a detailed assessment, the pavilion has been assessed as achieving 29 **%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered an Earthquake Prone. No critical structural weaknesses or collapse hazards have been identified in the building.

9. Recommendations

As the structure has been assessed to have a %NBS less than 33% NBS, it is deemed to be Earthquake Prone. It is recommended that masonry wall strengthening options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance with Christchurch City Councils earthquake Prone buildings policy and the NZSEE guidelines.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were unavailable. As a result the information contained in this report has been inferred from visual inspections of the building and site only.
- No intrusive structural investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 5 have been carried out on the structure.

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 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 Pavilion East elevation.

Photograph 2 Pavilion South elevation.

Photograph 3 Pavilion roof detail.

Photograph 4 Pavilion perspective view.

Photograph 5 Cracking to slab.

Photograph 6 Cracking to block wall at connection.

Appendix B

Calculation Methodology

a. Quantitative Assessment

The quantitative assessment to the building comprised an investigation on in-plane and out-of-plane strength of the unreinforced masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, the analysis of the distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. The capacity of the existing structural elements was compared to the demand placed on the element to give the %NBS of each of the structural elements.

b. Shear Demand

The in-plane shear demand of each wall was assessed by completing a torsion analysis to the building. NZS 1170.5:2004 makes allowance for accidental eccentricity and requires that the earthquake action be applied at an eccentricity of 10% of the building dimension which is perpendicular to the force applied. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to the direct shear. As each wall was made of the same material and with the same properties, the direct shear and the force induced in each wall are proportional to the length squared.

c. Seismic Coefficient

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

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

Where

C_h(T) = the spectral shape factor determined from CL 3.1.2

Z = the hazard factor from CL 3.1.4 and the subsequent amendments which increased the hazard factor to 0.3 for Christchurch

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/500 for an Importance Level 2 building

N(T,D) = the near-fault scaling facto from CL 3.1.6

The structural performance factor, S_P, was calculated in accordance with CL 4.4.2

$$\mathrm{S_P = 1.3 - 0.3\mu}$$

Where µ, the structural ductility factor, was taken as 1.00.

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.1 was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

For $T_1 < 0.7$s and soil class A, B, C and D.

d. In-Plane Capacity of Unreinforced Masonry Walls

The in-plane capacity of the concrete masonry wall was determined using the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance (06/2006). The NZSEE guidelines recommend checks for 4 different in-plane response modes.

- Diagonal tension failure mode
- Bed-sliding failure mode
- Toe crushing failure mode
- Rocking failure mode

An analysis of each wall was carried out using the methods set out in Section 8 – In-Plane Wall Response, of the NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance (06/2006).

e. In-plane Wall Properties of Unreinforced Masonry Walls

Properties of in-plane loaded URM walls, piers or spandrels for use in the calculation of nominal in-plane shear capacity were as follows:

- **Unit Weight of Masonry**

2.1 kN/m^2 was adapted for the unit weight of 20 series concrete hollow block masonry with standard aggregate (see Table A2 from NZS 1170.1:2002).

- **Weight of Wall**

The weight of the wall, W_w, was calculated in accordance with the equation.

$$W_w = 2.1 \times l_w \times h$$

Where: Values for wall length, l_w, and wall height, h.

- **Normal Force at Base of Wall**

The normal force acting on the cross section of the base of the wall, N_b, was calculated in accordance with the equation.

$$N_b = W_w + N_t$$

Where: Values for weight of the wall, W_w, and axial load above the wall, N_t.

- **Diagonal Tension Strength**

The diagonal tension strength of masonry, f_{dt}, was calculated in accordance with the equation below for walls, piers and spandrels.

$$f_{dt} = \frac{1}{2}\left(c + \frac{N_t}{A_w} 0.8\mu_f\right)$$

Where: Values for cohesion, c, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Distance to Centre of Inertia of Wall**

Distance to the centre of inertia of the wall from the compression toe, a_i, was calculated in accordance with the equation for walls with no flanges:

$$a_i = 0.5 \times l_w$$

- **Average Compressive Stress**

Average compressive stress acting on the wall, σ_{ave}, was calculated in accordance with the equation

$$\sigma_{avg} = \frac{N_t}{l_w . b_w}$$

Where: Value for width of the block shell, b_w which was equivalent to half of the block width.

Note: According to Design of Reinforced Masonry Structures NZS 4230:2004, flue area of a 190 mm thick block was 120 x150 = 18,000 m^2, while the area of the block was 190 x 190 = 36,100 m^2. This implied that the width of the block shell was half of the block width for a 190 thick block.

f. Solid In-plane Wall Nominal Shear Capacity of Unreinforced Masonry Walls

The in-plane nominal shear capacity of a wall, pier or spandrel was taken as the minimum of the nominal capacity in the diagonal tension failure mode, V_{dt}, the rocking failure mode, V_r, the bed-joint sliding failure mode, V_s, and the toe crushing failure mode, V_{tc}.

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

Nominal capacity of each failure mode was derived as following:

- **Capacity in Diagonal Tension Failure Mode, V_{dt}**

Nominal shear capacity corresponding to diagonal tension failure, V_{dt}, was calculated in accordance with the equation below for walls where no perpendicular flanges are present

$$V_{dt} = 0.54 . b_w . l_w . \zeta . f_{dt} . \sqrt{\left(1 + \frac{\sigma_{avg}}{f_{dt}}\right)}$$

Where: ζ was a factor to correct for nonlinear stress distribution (See Table 2) Linear interpolation may be used for values of h/l_w:

	ζ
Slender walls, where $h/l_w > 2$	1.5
Stout walls, where $h/l_w < 0.5$	1.0

Table 6 Shear stress factor for inclusion in diagonal tension failure mode equation

- **Capacity in Rocking Failure Mode, V_r**

Nominal shear capacity corresponding to the rocking failure mode, V_r, was calculated in accordance with the equation

$$V_r = \frac{N_b}{h}.\left[a_i - \frac{l_{er}}{3}\right]$$

Where: l_{er} was the effective length of the wall in rocking, taken as 0.1 x l_w.

- **Capacity in Bed-joint Sliding Failure Mode, V_s**

Bed-joint sliding failure was not an expected behaviour of URM walls subjected to seismic loading. The bed-joint sliding capacity of an in-plane loaded wall needed only be assessed when conditions suited the initiation of bed-joint sliding, specifically, when either or both the brick compressive strength and mortar compressive strength fell in the bounds of “soft”.

Ultimate shear capacity corresponding to bed-joint sliding failure, V_s, was calculated in accordance with the equation

$$V_s = l_w.b_w.c + 0.8.\mu_f.N_t$$

Where: Values for cohesion, *c*, and coefficient of friction, μ_f, were given in Section 2.5.5 of NZSEE guidelines for the Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Performance. The factor of 0.8 is to account for vertical accelerations and other dynamic effects.

- **Capacity in Toe Crushing Failure Mode, V_{tc}**

Nominal shear capacity corresponding to toe crushing failure, V_{tc}, was calculated in accordance with the below equation for walls where perpendicular flanges were present:

$$V_{tc} = \frac{N_b}{h}.\left[\frac{1}{2}.l_w - \frac{1}{3}.l_{etc}\right]$$

where effect length of wall was calculated as below:

$$l_{etc} = \frac{2.N_b}{1.3.f'_m.b_w}$$

g. Out-of-Plane Capacity of Unreinforced Masonry Walls

The % NBS for out-of-plane flexure of the concrete masonry walls was determined using the methods set out in NZSEE guidelines for the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes Section 10.3. The following steps were those required to assess the displacement response capability and the displacement demand, from which the adequacy of the walls can be determined.

The wall panel was assumed to form hinge lines at the points where effective horizontal restraint was assumed to be applied. The centre of compression on each of these hinge lines was assumed to form a pivot point. The height between these pivot points was the effective panel height *h*. At mid-height between these pivots, a third pivot point is assumed to form.

Step 1

The wall panel was divided into two parts, a top part bounded by the upper pivot and the mid-height between the top and bottom pivots, and a bottom part bounded by the mid-height pivot and the bottom pivot.

Step 2

The weight of the wall parts, W_b of the bottom part and W_t of the top part, and the weight acting at the top of the storey, *P* were calculated.

Step 3

From the nominal thickness of the wall, t_{nom}, the effective thickness, *t* was calculated as follows:

$$t = t_{nom}\left(0.975 - 0.025\frac{P}{W}\right)$$

Step 4

The eccentricity values e_p, e_b, e_t and e_o were calculated . Usually, the eccentricities e_b and e_p will each vary between 0 and *t/2* (where *t* is the effective thickness of the wall). Exceptionally they may be negative.

Where,

e_p = eccentricity of the P measured from the centroid of W_t

e_t = eccentricity of the mid-height pivot measure from the centroid of W_t

e_b = eccentricity of the pivot at the bottom of the panel measured from the centroid of W_b

e_o = eccentricity of the mid-height pivot measured from the centroid of W_b

Step 5

The mid-height deflection, Δ_i was calculated, which would cause instability under static conditions. The following formula was used to calculate this deflection.

$$\Delta_i = \frac{b\mathrm{h}}{2a}$$

Where

$$b = W_b e_b + W_t(e_0 + e_b + e_t) + P(e_0 + e_b + e_t + e_p) - \Psi(W_b y_b + W_t y_t)$$

And

$$a = W_b y_b + W_t\left(\frac{h}{2} + y_t\right) + Ph$$

And

$$\Psi = Initial\ slope\ of\ wall$$

Step 6

The maximum usable deflection, Δ_m was calculated as 0.6 Δ_i.

Step 7

The period of the wall, T_p, was four times the duration for the wall to return from a displaced position measured by Δ_m to the vertical. The period was calculated from the following equation:

$$T_p = 6.27\sqrt{\frac{J}{a}}$$

Where J was the rotational inertia of the masses associated with W_b, W_t and P and any ancillary masses, and was given by the following equation.

$$J = J_{bo} + J_{to} + \frac{1}{g}\left\{W_b[{e_b}^2 + {y_b}^2] + W_t[(e_0 + e_b + e_t)^2 + {y_t}^2] + P\left[\left(e_0 + e_b + e_t + e_p\right)^2\right]\right\} + J_{ancillary}$$

Where

$$J_{bo} = J_{to} = \frac{\left\{\left(\frac{W}{h}\right)[h^2 + 16t^2] + 4Pt^2\right\}}{g}$$

Where y_t was the distant from the top of the wall to the centroid of the top wall and y_b was the distant from the bottom of the wall to the centroid of the bottom wall.

Step 8

The seismic coefficient ($C_p(T_p)$) for an elastically responding part (μ_p = 1) with this period (T_p), was calculated as follows:

$$C_p(T_p) = C(0)C_{Hi}C_i(T_p)$$

Where

C(0) = the site hazard coefficient for T = 0 determined from NZS 1170.5 Section 3.1, using the values for the modal response spectrum method and numerical integration time history methods

C_{Hi} = the floor height coefficient for level I, from NZS 1170.5 Section 8.3.

$C_i(T_p)$ = the part spectral shape factor at level I, from NZS 1170.5 Section 8.4

Step 9

The participation factor, γ for the rocking system was taken as:

$$\gamma = \frac{(W_b y_b + W_t y_t)h}{2Jg}$$

Step 10

From $C_p(T_p)$, T_p, R_p *and* γ, the displacement response, D_{ph} was obtained from;

$$D_{ph} = \gamma\left(\frac{T_p}{2\pi}\right)^2 \times C_p(T_p) \times R_p \times g$$

Where R_p was from NZS 1170.5 Table 8.1

Step 11

The % NBS was obtained from

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

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

	Unit	No:	Street		
Building Name:	Macfarlane Park Pavilion			Reviewer:	Stephen Lee
				CPEng No:	1006840
Building Address:		130	Skipton	Company:	GHD
Legal Description:				Company project number:	513090243
				Company phone number:	04 472 0799
	Degrees	Min	Sec		
GPS south:				Date of submission:	
GPS east:				Inspection Date:	19-Jul-12
				Revision:	
Building Unique Identifier (CCC):	PRK_0663_BLDG_001 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	10.00

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:	3.4
Building height (m):	3.40		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	66			
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):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	Timber roof joists, Corrugated metal cladding
Floors:	concrete flat slab	slab thickness (mm)	0.1m slab on grade
Beams:	None	overall depth x width (mm x mm)	n/a
Columns:	None	typical dimensions (mm x mm)	None
Walls:	partially filled concrete masonry	thickness (mm)	20 Series CMU

Lateral load resisting structure

Lateral system along:	partially filled CMU	Note: **Define along and across in detailed report!**	note total length of wall at ground (m):	
Ductility assumed, μ:	1.25		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:	1.25		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:			n/a
Wall cladding:	Exposed structure	describe	Concrete masonry walls
Roof Cladding:	Metal	describe	Corrugated iron
Glazing:	other (specify)		Blockwork openings
Ceilings:	strapped or direct fixed		particle board
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 apparent	notes (if applicable):	
Differential settlement:	none apparent	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:	No placard in place	
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):	No damage observed	
Across	Damage ratio:	0%	
	Describe (summary):	No 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:	do not occupy		Describe:
Along	Assessed %NBS before:	29%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after:	29%		
Across	Assessed %NBS before:	29%	##### %NBS from IEP below	
	Assessed %NBS after:	29%		

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: m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

not required for this age of building

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

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0

Note 2: for RC buildings designed between 1976-1984, use 1.2

Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)

	along	across
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

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

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
Z_{1992}, from NZS4203:1992	
Hazard scaling factor, **Factor B:**	#DIV/0!

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

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

$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

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

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

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
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)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

GHD

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

Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Kevan O Brien	David Lee		Nick Waddington		15/3/13