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Memorial Park – Cemetery - Toilets/Shelter
PRK 0880 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

31 Ruru Road, Bromley

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

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31 Ruru Road, Bromley

Christchurch City Council

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Date
09 June 2014

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Memorial Park – Cemetery – Toilets/Shelter

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

31 Ruru Road, Bromley

Background

This is a summary of the Quantitative report for the Memorial Park Cemetery Toilets, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011 and visual inspections on 17th July 2012 and 31st January 2013.

Building Description

The overall structure is a single level park facilities building and comprises of two toilets on the southern end and covered seating areas throughout the rest of the block. Roof and wall construction appears to be consistent throughout. The roof is formed by lightweight metal cladding supported by timber rafters, and purlins, with ceiling linings throughout. It is assumed that the rafters and roof beams are connected to the concrete bond beam at the top of the unreinforced double brick walls.

The floors are on-grade reinforced concrete slabs with strip footings located under the brick walls. The building consists of 180mm thick double brick internal and external walls. The concrete bond beam located at the top of the walls consists of a 200 x 250 bond beam extending around the perimeter of the structure and at the top of all full height brick masonry walls.

Key Damage Observed

Key damage observed includes:-

- Loss of mortar was observed in the brick masonry walls at several locations.
- Damage and differential settlement in concrete slab foundations.

Building Strength

Based on the information available, and using the NZSEE guidelines for a Quantitative Assessment, the building's baseline post-earthquake capacity (including critical structural weaknesses and earthquake damage) has been assessed to be the order of 16% NBS.

Geotechnical assessment of soil conditions indicate minor to moderate vertical land settlements however no liquefaction or land cracking has been observed on the site. However there is evidence of ground settlement with uneven and displaced floor slabs.

The building has been assessed to have a seismic capacity in the order of 16% NBS and is therefore considered to be Earthquake Prone.

Recommendations

The recent seismic activity in Christchurch has not caused any significant damage to the building. The uneven, settled floor slabs should be re-levelled or replaced. The building has achieved approximately 16% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends significant wall strengthening or full replacement options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance the NZSEE guidelines.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Memorial Park Cemetery Toilet at 31 Ruru Road, Bromley.

This report is a Quantitative Assessment and is based on NZS 1170.5: 2004 and NZS 4230: 2004.

The quantitative assessment of the building comprises an investigation of the in-plane and out-of-plane strength of the brick masonry 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 existing structural elements to resist the forces applied. 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 locate reinforcement in the brick 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 Toilet and Shelter Building is located in the Memorial Park Cemetery at 31 Ruru Road, Bromley. It is estimated that the building was constructed in 1970 and is used as a shelter, seating area, and toilet facility.

The structure is built on a flat site and is in the centre of the cemetery. The nearest building is approximately 110m away and water treatment ponds are approximately 800m to the east of the structure.

The building is a single storey public toilet and shelter. The building is approximately 11.2m long, 5.6m wide, and 2.8m high at the eaves and 3.2m at the apex. The overall footprint of the building is approximately 63m^2. The building is assumed to be constructed in 1970. It appears that no alterations have been made to the building since construction. Sketches of the plan view of the building are shown in Figures 2 and 3.

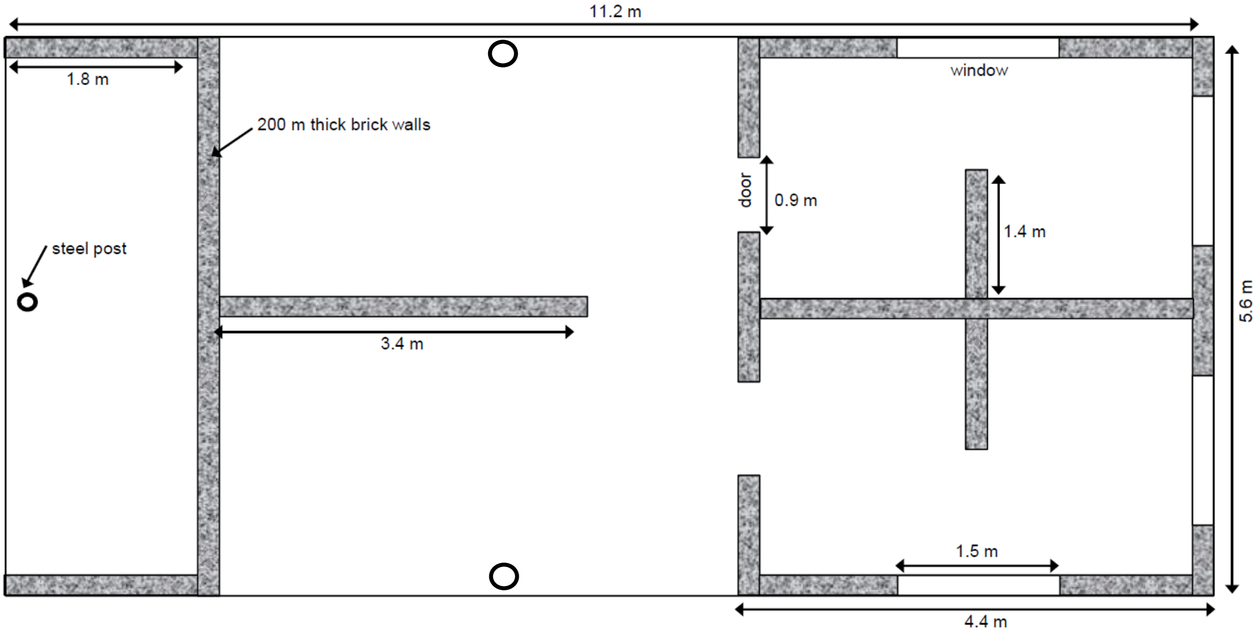

Figure 2 Plan Sketch Showing Load Bearing Walls

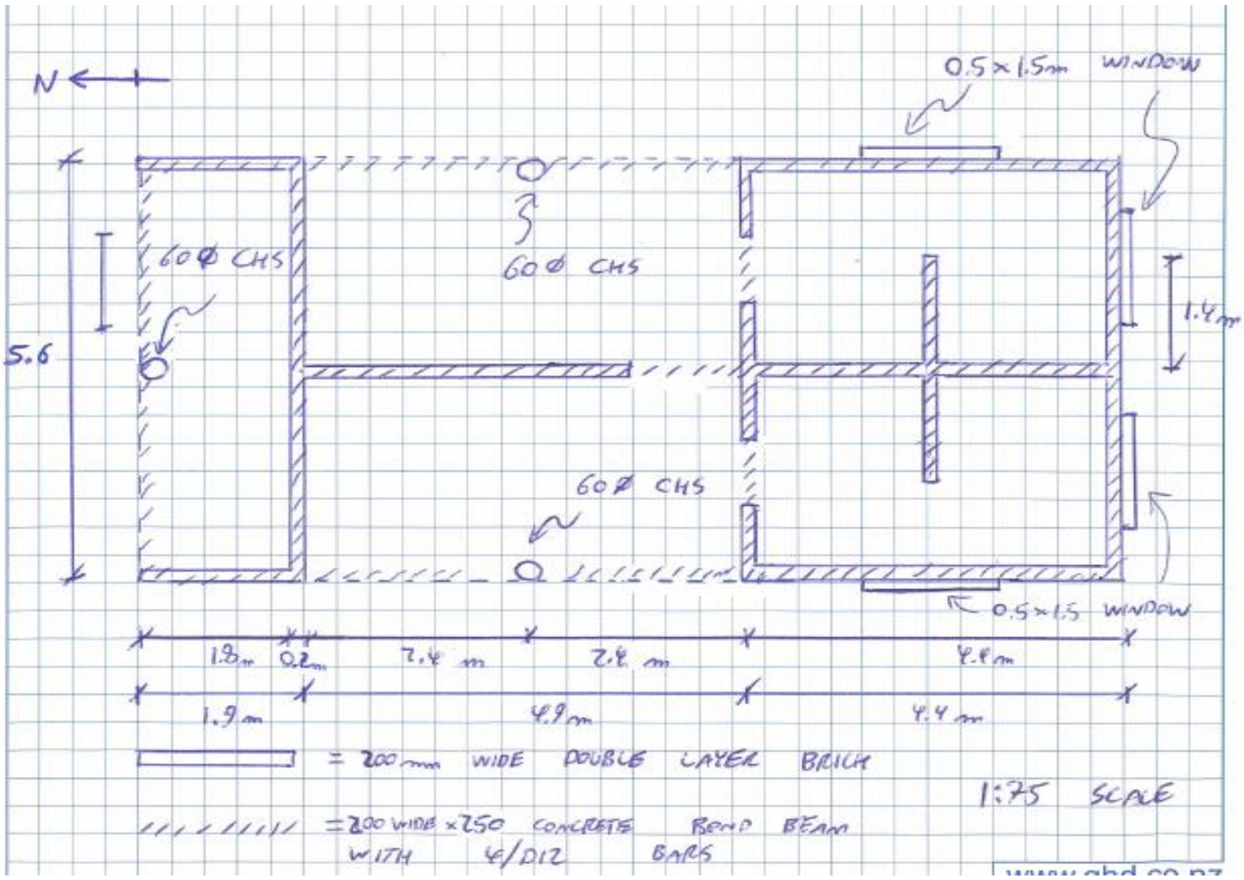

Figure 3 Plan Sketch Showing Walls and Support Posts

The building consists of 200mm thick double brick wall which forms the internal and external walls. The building does not have any wall linings.

The roof of the building consists of lightweight metal roof cladding fixed on lightweight timber purlins supported by timber roof trusses. The ceiling is fully lined with no areas of exposed roof structure. The timber roof trusses are supported by a concrete bond beam frame that rests on top of the brick masonry walls. The bond beam is supported by three 60mm dia. steel posts at the mid-point of the longest spans.

The building's foundation appears to consist of concrete strip footings connected to the concrete floor slab founded on hardfill.

No construction plans were made available. The results of the Hilti Ferroscans did not indicate any reinforcement present in the brick masonry walls. The scans did reveal horizontal reinforcement in the concrete bond beam at the top of the wall with four D12 horizontal bars..

4.2 Gravity Load Resisting System

The roof gravity loads in the structure are resisted by timber roof trusses to the structure. The roof cladding is supported by the timber purlins and trusses on the concrete bond beams. The concrete bond beams rest on the outer brick masonry walls. The roof loads are transferred from the bond beams into the brick masonry walls. The walls then carry the loads into the concrete slab and footings.

4.3 Lateral Load Resisting System

The brick masonry walls are the primary lateral load resistance system in this structure and serve to carry wall and roof seismic loads through to foundation level. The walls provide this function by in-plane panel action in shear and moment resistance. Upon reaching the foundations these lateral loads are dispersed into the founding soils via bearing and frictional resistance. The brick masonry walls are propped at the eaves level by the roof structure. The roof framing transfers lateral loads to the brick masonry walls. For face loading, the brick masonry walls are considered to be acting as simply supported walls connected to the foundations. In the absence of roof propping, there is a nominal level of horizontal spanning capability is present in the brick masonry, allowing lateral support from adjacent walls.

5. Damage Assessment

5.1 Surrounding Buildings

No damage to the surrounding buildings was observed as the building is relatively isolated within the Memorial Park Cemetery. There was evidence of liquefaction in the surrounding area.

5.2 Residual Displacements and General Observations

Some differential settlement of the structure was noticed during the inspection of the building with sections of the floor slab sinking >5mm in relation to the surrounding concrete floor (See photograph 4 and 5).

Damage was evident in the mortar of the concrete brick masonry walls. Repairs to the mortar were evident in a number of areas throughout the structure (See photograph 6, 7, and 8)

5.3 Ground Damage

There was evidence of ground movement and liquefaction in areas of the park and properties adjacent to Memorial Park Cemetery. The Park has been designated as at moderate to highly susceptible to liquefaction. Aerial photography indicates signs of moderate liquefaction close to the site.

6. Geotechnical Investigation

6.1 Site Description

The site is situated in the suburb of Bromley, east of Christchurch City centre. The site is relatively flat at approximately 10m above mean sea level. It is approximately 1.4km south of Avon River, 2km northeast of the Heathcote River, and 4km 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:

- 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 thirteen boreholes located within 200m of the site. There are two boreholes with significant information summarised in the table below (see Table 2).

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

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35-10476-WC	5.5 m	Not indicated	190m N
M35-17298-WC	2 m	Not indicated	182m W

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 Bromley[2]. Two investigation points were undertaken within 300m of the site, as summarised below in Table 3.

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[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, Bromley.*

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary	
CPT-BRY-04	290m NW	0-1	Silty Sand to Sandy Silt	
		1-4	Sand to Silty Sand	
		4-5.5	Silty Sand to Sandy Silt	
		5.5-12.28	Sand to Silty Sand	(GWL 8.7m bgl)
CPT-BRY-06	182m SW	0-4	Sand to Silty Sand	
		4-5	Silty Sand to Sandy Silt	
		5-12.16	Sand to Silty Sand	(GWL 2.7m bgl)

Initial observations of the CPT result indicate the site is underlain by sand and sand mixtures.

6.2.4 CERA Land Zoning

Adjacent properties to the south and east are classified as Green Zone, Technical Category 2 – yellow. Technical Category 2 – yellow means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

Adjacent properties to the north are classified as Green Zone, Technical Category 3 – blue. Technical Category 3 (TC3, blue) means that moderate to significant land damage from liquefaction is possible in future significant earthquakes. Site-specific geotechnical investigation and specific engineering foundation design is required.

6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of moderate liquefaction close to the site, as shown in Figure 4.

Table 4 Post February 2011 Earthquake Aerial Photography[1]

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, silty sand and sandy silt.

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 5 Summary of Known Active Faults[3,4]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	25 km	W	7.1	~15,000 years

[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

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	NW	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 Bromley, 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 moderately to highly susceptible to liquefaction, due to the following reasons:

- Signs of moderate liquefaction close to the site (evidence from the post-earthquake aerial photograph);
- Anticipated presence of sand and deposits beneath the site; and,
- Anticipated shallow ground water table.

Due to the limited subsoil information, further investigation is recommended to better determine subsoil conditions. From this, a more comprehensive liquefaction assessment could be undertaken.

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 sand and silt 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. Assessment

An inspection of the building was undertaken on the 17th July 2012. Further inspection of the building was carried out on 31st January 2013. No placard sign was evident during the inspection, however based on the inspection carried out it would be expected to have a green placard. Both the interior and exterior of the building were inspected. Most of the main structural components of the building were able to be viewed due to the exposed simple construction of the building.

Electro-magnetic scanning of the brick walls was undertaken and no reinforcement was located in the walls. Four 12mm diameter horizontal bars at were found in the bond beams at the top of the brick masonry walls. Construction plans were not made available.

The inspection also consisted of scrutinising 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.

7.1 Quantitative Assessment

The quantitative assessment of the building includes the investigation of in-plane and out-of-plane strength of the unreinforced brick masonry 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. A Hilti PS 200 Ferroscan was used to check for reinforcement in the walls. 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 D.

7.2 Seismic Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1) of NZS 1170:2004

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

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 = 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

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

Where µ is the structural ductility factor. A structural ductility factor of 1.00 has been taken for lateral loading across and along the building; this is due to the walls being constructed of unreinforced bricks.

For T1 < 0.7s and soil class D, the seismic weight coefficient was determined in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T1, of 0.4 was assumed for the in-plane masonry walls. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

7.2.1 In-Plane Capacity of the Unreinforced Unfilled Walls

The in-plane capacity of the unreinforced brick 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).

7.2.2 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}.

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

7.2.3 Out-of-Plane Capacity of the Unreinforced Walls

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

7.3 Calculation of %NBS

The shear and moment capacity of the brick masonry walls, the axial, bending and shear capacity of the concrete masonry as well as the bracing capacity of the walls both in the along and across directions were then compared to their respective demands to assess which were the most critical and thus determine the overall %NBS for the building.

8. Initial Capacity Assessment

8.1 % NBS Assessment of Walls

A summary of the results of the calculations detailed in this section of the report can be seen below in Tables 6 and 7. The %NBS for each unreinforced brick masonry wall was calculated by comparing the demand placed upon the individual element with the capacity of that element as derived from the current design code.

The position of each wall is indicated on Figure 4 below and each wall is numbered accordingly.

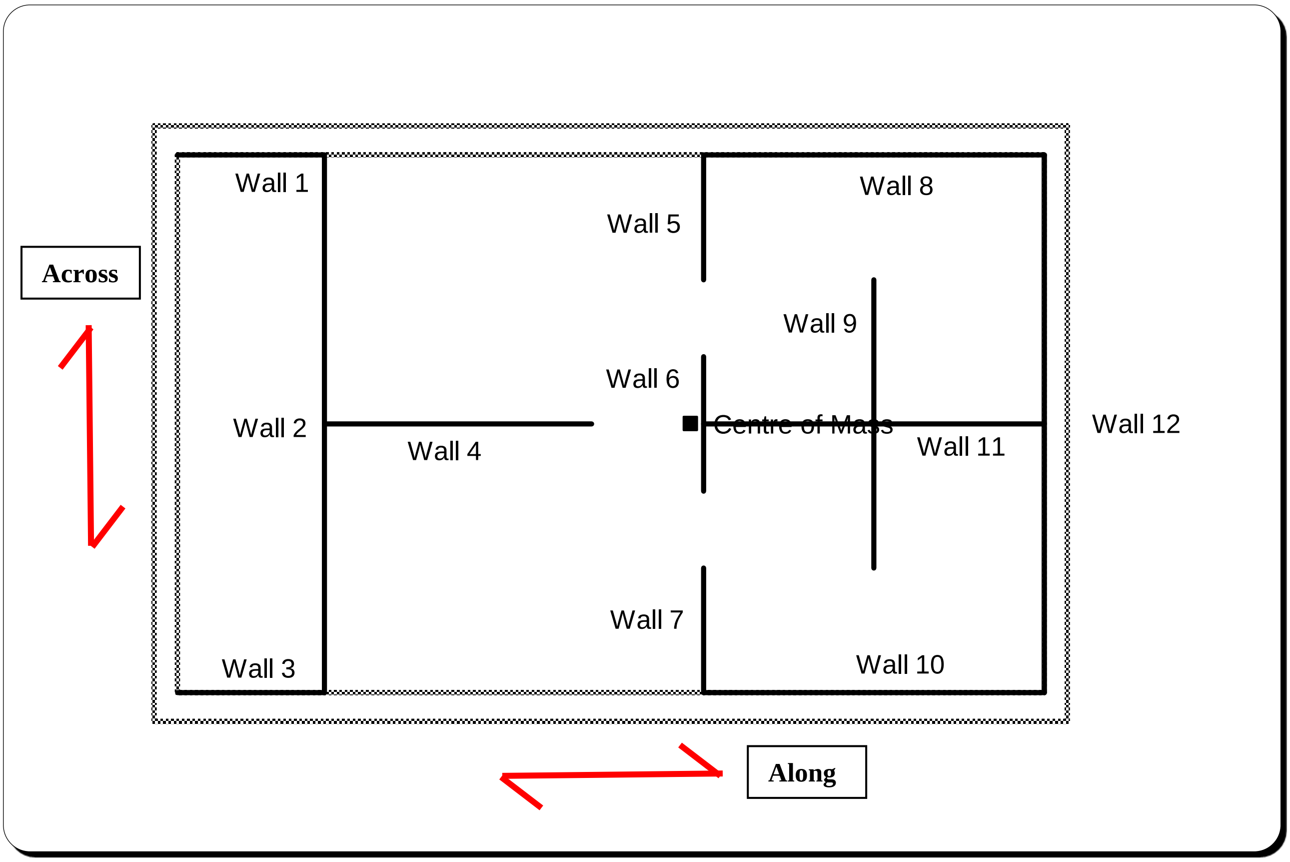

Figure 4 Plan Centres of Mass and Wall Numbering

8.2 Memorial Park Cemetery Toilet Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 6.

Wall number	ϕV_n kN	V* kN	%NBS	Earthquake Status	ϕM_n kNm	M* kNm	%NBS	Earthquake Status
1	5.8	7.8	21%	**Prone**	18.3	77.7	24%	**Prone**
2	39.9	53.2	16%	**Prone**	159.0	716.0	22%	**Prone**
3	5.8	7.8	21%	**Prone**	18.3	77.7	24%	**Prone**
4	19.2	25.6	21%	**Prone**	60.4	253.8	24%	**Prone**
5	2.7	3.6	27%	**Prone**	8.6	27.9	31%	**Prone**
6	3.2	4.2	27%	**Prone**	9.9	32.3	31%	**Prone**
7	2.7	3.6	27%	**Prone**	8.6	27.9	31%	**Prone**
8	31.2	41.6	21%	**Prone**	98.2	416.5	24%	**Prone**
9	31.2	41.6	21%	**Prone**	98.2	412.9	24%	**Prone**
10	31.2	41.6	21%	**Prone**	98.2	416.5	24%	**Prone**
11	14.5	19.4	24%	**Prone**	45.6	166.2	27%	**Prone**
12	39.9	53.2	17%	**Prone**	159.0	644.8	25%	**Prone**

Table 6 In Plane Analysis Results for URM Walls

The results of the out of plane displacement response capability analysis and subsequent earthquake designation under the NZSEE guidelines are listed in Table 7

Wall number	D_{ph} kN	Δ_m kN	%NBS	Earthquake Status
1	0.281	**0.052**	22%	**Prone**
2	0.281	**0.052**	22%	**Prone**
3	0.281	**0.052**	22%	**Prone**
4	0.281	**0.052**	22%	**Prone**
5	0.281	**0.052**	22%	**Prone**
6	0.281	**0.052**	22%	**Prone**
7	0.281	**0.052**	22%	**Prone**
8	0.281	**0.052**	22%	**Prone**
9	0.281	**0.052**	22%	**Prone**
10	0.281	**0.052**	22%	**Prone**
11	0.281	**0.052**	22%	**Prone**
12	0.281	**0.052**	22%	**Prone**

Table 7 Out Of Plane Analysis Results for URM Walls

8.3 Discussion of Results

The loading standards following the Christchurch earthquakes have been modified with increased seismic requirements. The additional requirements 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.

Following a detailed assessment, the toilet/shelter block has been assessed as achieving 16 **%**NBS for the unreinforced brick masonry walls. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines Memorial Park Cemetry Toilet is considered to be Earthquake Prone. The buildings brick masonry walls are unreinforced, but there are no other critical structural weaknesses or potential collapse hazards identified in the building.

9. Recommendations

The recent seismic activity in Christchurch has caused loss or damage to the mortar in multiple areas on the building, as apparent from the repairs. There is also evidence of ground settlement and the uneven floor slabs should be re-levelled or replaced. The park building has achieved approximately 16% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends that continuing occupancy may be permitted. However, significant wall strengthening or wall replacement options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance 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. Electro-magnetic scanning of the walls was conducted to determine the levels of steel reinforcement present.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 8 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: has been prepared by GHD for Christchurch City Council and may only be used and relied on by Christchurch City Council for the purpose agreed between GHD and the Christchurch City Council as set out in section 1.1 of this report.

GHD otherwise disclaims responsibility to any person other than Christchurch City Council arising in connection with this report. GHD also excludes implied warranties and conditions, to the extent legally permissible.

The services undertaken by GHD in connection with preparing this report were limited to those specifically detailed in the report and are subject to the scope limitations set out in the report.

The opinions, conclusions and any recommendations in this report are based on conditions encountered and information reviewed at the date of preparation of the report. GHD has no responsibility or obligation to update this report to account for events or changes occurring subsequent to the date that the report was prepared.

The opinions, conclusions and any recommendations in this report are based on assumptions made by GHD described in this report. GHD disclaims liability arising from any of the assumptions being incorrect.

The opinions, conclusions and any recommendations in this report are based on information obtained from, and testing undertaken at or in connection with, specific sample points. Site conditions at other parts of the site may be different from the site conditions found at the specific sample points.

Investigations undertaken in respect of this report are constrained by the particular site conditions, such as the location of buildings, services and vegetation. As a result, not all relevant site features and conditions may have been identified in this report.

Site conditions (including the presence of hazardous substances and/or site contamination) may change after the date of this Report. GHD does not accept responsibility arising from, or in connection with, any change to the site conditions. GHD is also not responsible for updating this report if the site conditions change.

Appendix A
Photographs

Photograph 1 East elevation.

Photograph 2 View of the toilet/shelter block end from the northeast corner.

Photograph 3 View of the toilet section from the south.

Photograph 4 Differential settlement of the concrete foundation slab.

Photograph 5 Differential settlement of the concrete foundation slab.

Photograph 6 Repairs to damaged mortar in the brick masonry walls.

Photograph 7 Repairs to damaged mortar in the brick masonry walls.

Photograph 8 Repairs to damaged mortar in the brick masonry walls.

Photograph 9 Concrete bond beam at the top of the wall and perimeter of the roof..

Photograph 10 Minor spalling damage to concrete window sill.

Appendix B

Existing Drawings

No existing drawings were available for the building.

Appendix C
CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Memorial Park Cemetery - Toilets/Shelter			Reviewer:	David Lee
	Unit	No:	Street	CPEng No:	112052
Building Address:	Ruru Road, Bromley	31		Company:	GHD
Legal Description:	PRK_0880_BLDG_001			Company project number:	513090248
				Company phone number:	
	Degrees	Min	Sec		
GPS south:				Date of submission:	09-06-14
GPS east:				Inspection Date:	31-01-13
				Revision:	
Building Unique Identifier (CCC):	PRK_0880_BLDG_001			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	3.2
Soil type:	silty sand	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	5	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:	mat slab		if Foundation type is other, describe:	Slab on grade
Building height (m):	3.20		height from ground to level of uppermost seismic mass (for IEP only) (m):	2.8
Floor footprint area (approx):	63			
Age of Building (years):	43		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 truss	truss depth, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:			
Columns:			
Walls:	load bearing brick	#N/A	

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - brick	**Note: Define along and across in**	note wall thickness and cavity	0.18
Ductility assumed, μ:	1.00	**detailed report!**		0
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	unreinforced masonry bearing wall - brick		note wall thickness and cavity	0.18

Ductility assumed, μ:	1.00			0
Period across:	0.40	0.00	estimate or calculation?	
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
Roof Cladding:	Metal	describe	Light corrugated steel
Glazing:	timber frames		
Ceilings:	fibrous 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	partial	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	0-25mm	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:	slight	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	Strengthen to 67%
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	16%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Quantitative Assessment
	Assessed %NBS after:	16%		
Across	Assessed %NBS before:	16%	##### %NBS from IEP below	
	Assessed %NBS after:	16%		

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 1965-1976

h_n from above: 2.8m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

	along	across
Period (from above):	0.4	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**:

	along	across
2.5 Ductility Scaling Factor 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, $(NBS\%)_b = (\%NBS)_{nom}$ x A x B x C x D x 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

	Along	Across
3.6. Other factors, Factor F 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

	Along	Across
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!**

Appendix D

Assessment Methodology

A. Seismic Coefficient

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

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

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 = 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 factor from CL 3.1.6

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

$$S_P = 1.30 - 0.3\mu$$

A structural ductility factor μ of 1.25 has been taken for lateral loading both across and along the building.

For T_1 < 0.7s and soil class D, the seismic weight coefficient was determined 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 0.4 was assumed for the in-plane masonry walls. The coefficient was then calculated using Equation 5.2(1);

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

Where

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

Horizontal Design Actions on Parts

Horizontal Design Actions on Parts $F_{ph,}$ was determined using Equation 8.5(1):

$$F_{ph} = C_p(T_p)C_{ph}R_pW_p \leq 3.6W_p$$

Where

$C_p(T_p)$ = the horizontal design coefficient of the part, determined from Clause 8.2

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

Where

$C(0) = C_h(T)ZRN(T.D)$

$C_h(T)$ =1.12 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) = 1.0 the near-fault scaling factor from CL 3.1.6

C_{hi} = (1 + h_i/6) from Equation 8.3(1) CL8.3 the floor acceleration coefficient at level i

Where

h_i = height of the attachment of the part

h_n = height from the base of the structure to the uppermost seismic weight or mass

$C_i(T_p)$ = 2.0 for $T_p \leq 0.75$s

C_{ph} = the part horizontal response factor determined from Clause 8.6

R_p = the part risk factor as given by Table 8.1

W_p = the weight of the part

The mean value of the seismic coefficient was calculated using horizontal coefficient for the top and bottom of the wall.

B. In-Plane Capacity of the Unreinforced Walls

The in-plane capacity of the unreinforced brick 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).

C. In-plane Wall Properties of the Unreinforced 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**

2.20 kN/m^2 was adapted for the unit weight of the brick 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.

$$W_w = 1.82 \times l_w \times h$$

Where: l_w is the total wall length and h is the wall height.

- **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:

$$a_i = 0.5 \times l_w$$

D. 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}.

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

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 3)

	ζ
Slender walls, where $h/l_w > 2$	1.5
Stout walls, where $h/l_w < 0.5$	1.0
Linear interpolation may be used for values of h/l_w	

Table 3 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 the effective length of wall was calculated as:

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

E. Out-of-Plane Capacity of the Unreinforced Walls

The % NBS for out-of-plane flexure of the brick 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

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

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

F. Calculation of %NBS

The capacities from the brick masonry walls were compared to their respective demands to assess which were the most critical and thus determine the overall %NBS for the building.

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Dale Donovan	David Lee	DLee	Hamish MacKinven		10/06/2013
Final V2	H Erentz	David Lee	DLee	F Watt		09/06/2014