Toilet Block - Cashmere Rd / Valley Rd Reserve
PRK_1795_BLDG_001 EQ2
Detailed Engineering Evaluation
Quantitative Report
Version Final

73 Cashmere Road, Cashmere

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

Cashmere Toilets

PRK_1795_BLDG_001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version Final

73 Cashmere Road, Cashmere

Christchurch City Council

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Date
05/02/13

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Cashmere Toilet Block

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

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Background

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

Building construction:

- Roof: timber truss cladded with corrugate metal sheets and plastic skylights;
- Wall: unreinforced masonry block;
- Floor: concrete slab-on-grade;
- Foundation: concrete strip footing;

Key Damage Observed

Key damage observed on site includes:

- Evidence of settlement in the pavement outside the toilet;
- Cracking that occurred in three different locations throughout the adjacent concrete masonry wall has been repaired by others.

Geotechnical Investigation

Due to the ground conditions on site it is possible that minor to moderate liquefaction will occur. However the effect liquefaction will have on the structure will not be a severe threat, therefore in terms of the IEP the site characteristics have been deemed to be significant.

Quantitative Detailed Engineering Evaluation Assessment

The quantitative assessment is based in general on the New Zealand Standard – NZS 1170: Structural Design Actions, 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). The assessment is limited to the toilet structure only. The building capacity has been assessed to be 5% of New Building Standard and in accordance with the aforementioned NZSEE guidelines it is deemed earthquake prone. The capacity of the boundary wall has been assessed to be over 100% of New Building Standard. However, horizontal cracking on the boundary wall indicates that the top two courses are not structurally tied to the rest of the wall and the top of the wall could potentially become a significant risk in future seismic activity.

Recommendations

As the building has been assessed to have a %NBS less than 34%NBS, it is deemed to be Earthquake Prone. It is recommended that strengthening options be explored and implemented to bring the %NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Prone buildings.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Cashmere Toilet Block.

This report is a Quantitative Assessment and is based in general on NZS 1170: 2002 Structural Design Actions, 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).

A quantitative assessment has been carried out to the toilet block (unreinforced block structure) and the adjacent boundary wall (reinforced masonry cantilever wall), and a separate percentage of New Building Standard (%NBS) to each structure is given in this report. Assessment of the retaining wall on the eastern side to the toilet block is outside the scope of this project.

The quantitative assessment to the toilet block 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 block walls.

The %NBS of the reinforced masonry wall was obtained via the comparison between the shear and the moment capacity of the wall and the design action.

Electromagnetic scans have been carried out on site at the northern wall of the toilet and the adjacent boundary wall. The result indicates that reinforcement is not evident in the northern wall of the toilet block but was present in the masonry boundary wall on the eastern side of the toilet block (Note: reinforcing was not detected over the top two courses of blocks)

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 34% 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 block is located at 73 Cashmere Road/Valley Road Reserve in Cashmere. The original construction date of the structure is unknown but based on site observation is estimated to be in the 1970’s. The toilet block is located at the rear of a local community playground. The roadside site is bordered by residential properties in the eastern and western directions with a local Bowls club located over a boundary wall on the southern side.

The site slopes gradually towards the playground entrance on Cashmere Road.

The building is a single story public toilet. The dimensions of the building are approximately 8 m long, 2.4 m wide (refer to sketch plan as shown below). The hipped roof is constructed using timber trusses, cladded with lightweight corrugate metal sheets and plastic skylights. Walls are 2.4 m high, constructed using 15 series unreinforced masonry blockwork. The walls are supported on concrete strip foundation and floor is concrete slab-on-grade.

Adjacent to the toilet block on the eastern side is a 15 series reinforced masonry wall. It is 2.2 m high and 24 m long in total. No reinforcing was evident in the top two courses of masonry blocks via the electromagnetic scan, so the top two courses not structurally tied to the bottom wall that is reinforced.

Note: Significant horizontal and vertical cracking was observed on the reinforced masonry wall during the initial inspection on 15 February 2012. Repairs to cracking were completed on 5 May 2012 by others.

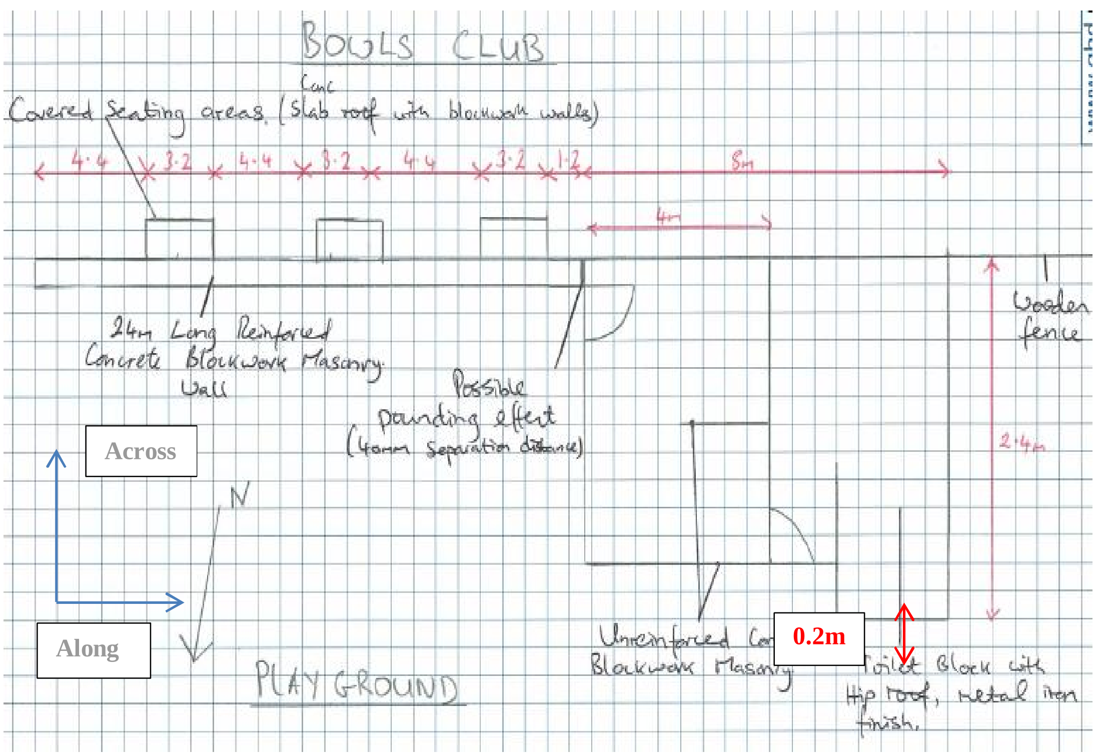


Figure 2 Plan Sketch Showing Key Structural Elements

4.2 Gravity Load Resisting System

The gravity support for the building is provided by the purlins, timber trusses, masonry wall and strip foundation. The roof cladding is supported by timber purlins spanning between the timber trusses. These trusses transfer the roof load to the unreinforced concrete block walls. The block walls then transfer the load to the strip foundation.

4.3 Lateral Load Resisting System

Lateral loads acting on the structure are resisted by concrete masonry walls in both of the longitudinal and the transverse directions of the building. These walls transfer the lateral seismic loading of the structure to the foundations.

5. Assessment

5.1 Qualitative Assessment

An initial visual inspection of the building was undertaken on 15 February 2012. Both the interior and exterior of the building were inspected. The main structural components of the building were all able to be viewed due to the exposed simple construction of the building.

Following an IEP assessment, the building was assessed as achieving 15% New Building Standard (NBS). Under the aforementioned NZSEE guidelines, the building was considered potentially Earthquake Prone as it achieves below 34% NBS. This score was adjusted when considering damage to the structure as all damage observed was relatively minor and considered unlikely to adversely affect the load carrying capacity of the structural systems. However, critical structural such as the potential of liquefaction could reduce the %NBS, excluding this it was initially 27%NBS. Significant cracking was observed on the boundary wall adjacent to the toilet.

5.2 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. To investigate the reinforcement detail of the block walls, a further inspections were carried out on 9 July 2012. The cracking on the boundary, which was observed during the first visit, has been repaired and this has been undertaken by others on 5 May 2012 based on the council information.

5.2.1 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.

5.2.2 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 = 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, 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);

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

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.

5.2.3 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).

5.2.4 In-plane 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}.

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

5.2.5 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.

5.2.6 Shear capacity of the Reinforced Walls

The in-plane shear capacity of the reinforced filled masonry wall was provided by the vertical reinforcing and the strength was determined in accordance with NZS 4230:2004. The out-of-plane shear was generally not critical and the design shear stress should be less than the out-of-plane shear strength of the masonry alone. The strength reduction factor, ϕ, for shear was taken as 0.75 in accordance with Cl 3.4.7.

5.2.7 Moment capacity of the Reinforced Walls

The moment capacity of the reinforced masonry wall (i.e. in-plane and out-of-plane) was determined in accordance with NZS 4230:2004. The strength reduction factor, ϕ, for flexure with or without axial tension or compression was taken as 0.85 in accordance with Cl 3.4.7.

6. Damage Assessment

6.1 Surrounding Buildings

There are no buildings located immediately adjacent to the Cashmere toilet block. The nearest residential building is located approximately 20m to the west. Based on visual inspections from property boundaries there was no damage evident to these buildings. However the adjacent masonry boundary wall between the playground and the bowls club has suffered significant structural damage.

6.2 Residual Displacements and General Observations

- No residual displacements of the structure were noticed during the inspection of the building;
- No damage was evident to the exterior of the building;
- No damage was evident to the timber framed truss hip roof structure;
- Existing shrinkage cracks were noted in the slab on grade concrete floor at the entrance to the building;

With regards the adjacent reinforced concrete blockwork masonry wall significant damage was observed. A horizontal cracking along the second course of block was evident and significant vertical cracking was observed on the wall with the width up to 60 mm. Refer to Photographs 7 and 8 in Appendix A.

6.3 Ground Damage

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

7. Geotechnical Consideration

Given the size and significance of the public toilet (i.e. Importance Level 2), a decision was made not to undertake intrusive investigation. As a result, this section comprises the findings of the desktop study and interpretation thereof.

7.1 Site Description

The site is located in Cashmere, Christchurch and is accessed from 3 Valley Road and Cashmere Road via playground. The site is located at the base of the Port Hills to the South. The Heathcote River meanders in a west-northeast direction 150m to the north of the property. The subject area is low lying and topographically flat at approximately 10m above mean sea level.

7.2 Published Information on Ground Conditions

7.2.1 Published Geology

The geological map of the area[1] indicates that the site is close to the boundary of 2 material types:

- Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits; and
- Holocene colluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising valley fill and slope wash of loess-volcanic derived colluvium;

7.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates there are five boreholes in a 200m radius of the site. The boreholes are all less than 3m deep and all located on the northern side of the Heathcote River, therefore it is not considered likely to show indicative ground conditions at the site.

7.2.3 EQC Geotechnical Investigations

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

7.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has indicated the site is situated within the Green Zone, indicating that repair and rebuild may take place.

Land in the CERA green zone has been divided into three technical categories. These categories describe how the land in expected to perform in future earthquakes.

The site has been categorised as “N/A – Rural and Unmapped”. However, surrounding residential properties have been categorised TC2 (yellow). This means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

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

7.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake (Figure 3) shows evidence of liquefaction (in the form of sand boils) outside the building footprint and in the surrounding area.

Figure 3 Post February 2011 Earthquake Aerial Photography [2]

7.2.6 Summary of Ground Conditions

In the absence of site specific borehole logs are unavailable the underlying ground conditions are inferred to be colluvium derived material from the hills laid down in an alluvial environment.

7.3 Seismicity

7.3.1 Nearby Faults

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

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

Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130	8.3	~300 years
Greendale (2010) Fault	30	7.1	~15,000 years
Hope Fault	110	7.2~7.5	120~200 years
Kelly Fault	115	7.2	~150 years
Porters Pass Fault	55	7.0	~1100 years

Table 2 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.

7.3.2 Ground Shaking Hazard

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

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

7.3.3 Slope Failure and/or Rockfall Potential

The site is located within Cashmere, a flat valley and hill suburb in Southern Christchurch. The structure itself is located in essentially flat terrain. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

7.3.4 Liquefaction Potential

It is considered that the site is prone to liquefaction from evidence of liquefaction and settlement occurring in past earthquake events. The true potential of liquefaction has not been established due to limited geotechnical and ground condition information on the area.

7.3.5 Lateral Spread Potential

The site is within 200m of the true right bank of the Heathcote River it is therefore considered possible that lateral spreading could occur. However, this is only considered minor as lateral spread mapping[5]

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

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

[5] Canterbury Geotechnical Database, https://canterburygeotechnicaldatabase.projectorbit.com

following the February and September earthquakes identifies cracks less than 10mm wide 130m from the side.

7.4 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 is inferred to be underlain by colluvium derived material from the hills laid down in an alluvial environment.

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

There is a moderate liquefaction potential with a low lateral spread potential.

In the event that the foundations are repaired or rebuilt, a shallow geotechnical investigation comprising hand augers and scala penetrometer’s should be undertaken at the commencement of the design phase. This testing should be in accordance with DBH and CERA guidelines. Specific advice can be provided upon request.

8. Survey

No level or verticality surveys have been undertaken for this building at this stage.

9. Detailed Capacity Assessment

9.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class assumed to be: C, NZS 1170.5:2004, Clause 3.1.3, Shallow 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.

9.2 Wall Investigation

Position of each URM walls is indicated in the plan below and each wall is named accordingly.

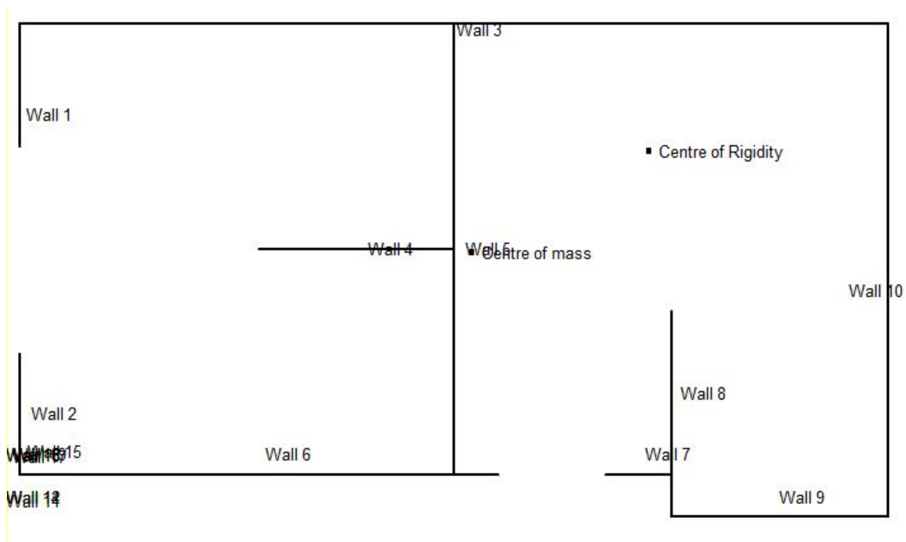


Figure 4 Plan Details and Wall Location

9.3 In-Plane Shear Demand

The in-plane shear demand of each wall is 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 is 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. The in-plane shear demand of each wall is shown as below:

Wall	In-Plane Shear Demand (kN)
1	5.43
2	5.43
3	69.01
4	4.20
5	45.30
6	49.14
7	0.91
8	6.72
9	11.30
10	34.84

Table 3 In-Plane Shear Demand of the URM Walls

9.4 In-Plane Shear Capacity

The in-plane shear capacity is analysed in accordance with NZSEE guidelines (02/2011). The in-plane nominal shear capacity of each wall is 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}).

Wall	V_{dt}	V_s	V_r	V_{tc}	In-Plane Shear Capacity (kN)	Failure Mode
1	3.45	7.89	0.38	0.40	0.38	Rocking
2	3.45	7.89	0.38	0.40	0.38	Rocking
3	29.60	103.88	63.86	67.14	29.60	Diagonal Tension
4	8.57	23.43	3.27	3.44	3.27	Rocking
5	8.96	27.72	4.13	4.36	4.13	Rocking
6	16.65	57.29	19.58	20.58	16.65	Diagonal Tension
7	3.59	8.02	0.41	0.43	0.41	Rocking
8	5.10	12.6	0.85	0.90	0.85	Rocking
9	9.26	26.12	4.10	4.31	4.10	Rocking
10	10.13	30.91	5.52	5.80	5.52	Rocking

Table 4 In-Plane Shear Capacity of the URM Walls

Table 4 indicates the in-plane shear capacity of the wall and the corresponding critical failure mode. The results reveal that the URM walls in this building could fail in either the diagonal tension failure or the rocking failure. Diagonal tension failure occurs when the maximum principal tensile stress in the wall exceeds the diagonal tensile strength of the masonry and it is often characterised by diagonal cracking on the wall. Rocking failure generally results in overturning of a URM wall when subjected to in-plane earthquake loading and it can be identified by large flexural cracks at the top and bottom of the wall.

As indicated in Table 4, Wall 3 and Wall 6 could potentially fail in diagonal tension failure. This failure is a force controlled failure mode, as a result of diagonal cracking, a rapid deterioration of strength could lead to ultimate collapse even at small deformation.

On the other hand, rocking failure was found to be critical for the rest of the walls and the in-plane shear capacity ranges from 0.38 kN (for Wall 1 and 2) to 5.52 kN (for Wall 10). As rocking failure is drift controlled failure mode, large deflection but within the drift limit as per Section C8.5 in NZSEE guidelines (02/2011) can be acceptable with gradual degradation of strength.

9.5 Percentage of New Building Standard (%NBS)

10.5.1 In-Plane %NBS

The percentage of New Building Standard (NBS) is the ratio of the shear demand over the shear capacity and the value of In-plane %NBS of each wall is shown in Table 5. Wall 1 is found to have the lowest value of in-plane %NBS of 5%.

10.5.2 Out-of-Plane %NBS

The out-of-plane flexural capacities of all the walls are calculated using the process outlined in the NZSEE Guidelines Section 10.3. The value of out-of-plane %NBS is presented in Table 5 and the critical value of out-of-plane %NBS is 35%.

9.6 Discussion of Results

Wall	In-Plane %NBS	Out-of-Plane %NBS	%NBS
1	5	35	5
2	5	35	5
3	32	35	32
4	58	35	35
5	7	38	7
6	25	35	25
7	34	35	34
8	10	38	10
9	27	35	27
10	12	35	12

Table 5 %NBS of URM Walls

Following a detailed assessment, the building has been assessed as achieving 5**%**NBS (in-plane strength governs). Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered an Earthquake Prone building as it does not achieve above 33% NBS.

9.7 Assessment to Boundary Wall

Assessment to reinforced masonry wall comprises the investigation on the shear and the moment capacity of the wall (refer to Appendix C Calculation Methodology).

Importance level 1 and 50 years of design life are adapted to work out the earthquake loading to the reinforced concrete boundary wall.

The analysis has revealed that over 100% NBS for the boundary wall hence the strength of the boundary wall is adequate in terms of the current building standard. However, a horizontal crack along the second course of block from the top was evident. This crack indicates that the top two block courses have not been structurally tied to the bottom wall that is reinforced, so the wall may become a significant risk in future seismic activity.

10. Recommendations

The building capacity has been assessed to be 5% of New Building Standard and in accordance with the aforementioned NZSEE guidelines it is deemed earthquake prone. As the building has been assessed to have a %NBS less than 34%NBS, it is deemed to be Earthquake Prone. It is recommended that strengthening options be explored and implemented to bring the %NBS of the building up to the required 67% in order to comply with Christchurch City Council policy regarding the strengthening of potentially Earthquake Prone buildings.

11. Limitations

11.1 General

The assessment is limited to the public toilet and the adjacent boundary wall. Assessment of the retaining wall on the eastern side to the toilet block is outside the scope of this project.

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 geotechnical investigations have been undertaken.
- No level or verticality surveys have been undertaken.
- No calculations, other than the wall bracing calculations, shear and moment capacity checks included in this report 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.

11.2 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

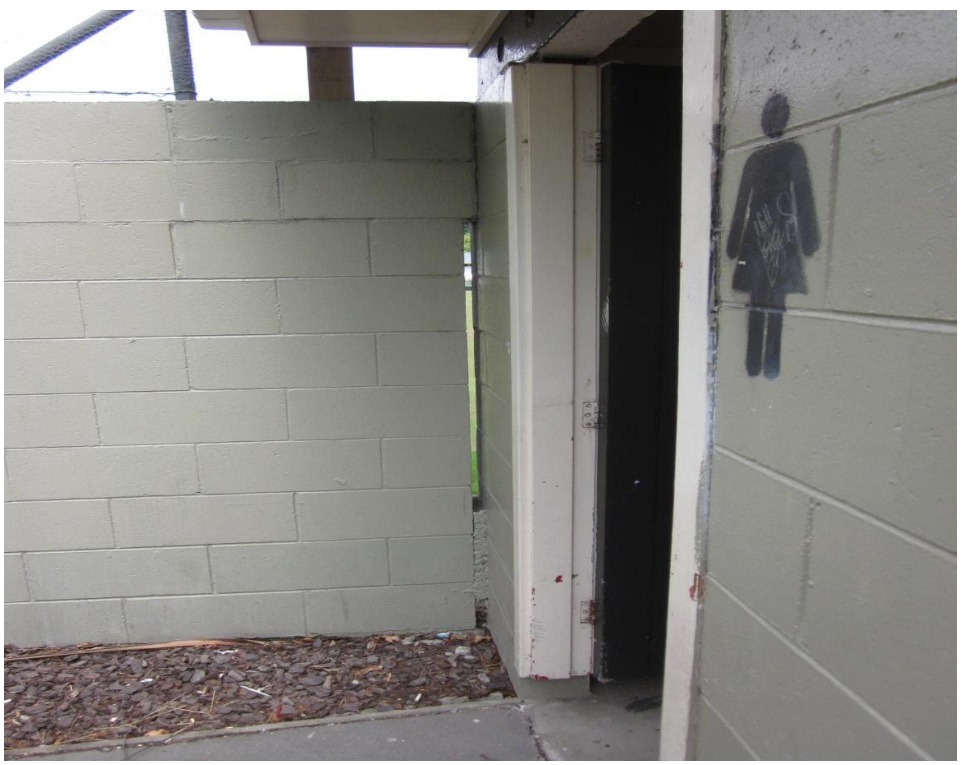
Photograph 1: Cashmere toilet block located at rear of Cashmere playground

Photograph 2: Front northern facing elevation

Photograph 3: Reinforced masonry boundary wall located to the eastern side of the toilet block with covered seating situated on the bowls club side

Photograph 4: Front northern and side western facing elevations

Photograph 5: Timber truss roof with skylight and permiter lintel beam



Photograph 6: Potential pounding effect with evident building separation

Photograph 7: Cracking to the masonry boundary wall found during the initial inspection on 15 February 2012.

Photograph 8: As above

Appendix B

Existing Drawings

No existing drawings were available for the building.

Appendix C

Calculation Methodology

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);

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

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

$$S_P = 1.3 - 0.3\mu$$

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);

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

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**

1.82 kN/m^2 was adapted for the unit weight of 15 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.

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

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

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

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 ($\mu_p = 1$) 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

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

h. Shear capacity of Reinforced Walls

The shear capacity of the reinforced filled masonry wall was determined using NZS 4230:2004. As there are no details as to the level of supervision during the construction stage, the Observation Type was classed in accordance with Table 3.1. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.75 in accordance with Cl 3.4.7. The overall shear capacity of the wall was calculated from Cl 10.3.2.1, Equation 10-4;

$$V_n = v_n b_W d \phi$$

Where

v_n = the total shear stress which consists of the contribution of the masonry, v_m, the axial load, v_p and the contribution of the shear reinforcement, v_s.

b_w = the thickness of the wall

d = 0.8 times the length of the wall

i. Moment capacity of Reinforced Walls

The moment capacity of the reinforced filled masonry wall was determined using NZS 4230:2004 and the user's guide to NZS 4230:2004. The strength reduction factor, ϕ, for flexure with or without axial tension or compression was taken as 0.85 in accordance with Cl 3.4.7. The overall moment capacity of the wall was calculated using the formula;

$$M_n = \left(N_n + A_s f_y\right) x \left(\frac{t - a}{2}\right) x \phi$$

Where

$$a = \frac{N_n + A_s f_y}{0.85 f'_m 1.0}$$

N_n = the axial load acting on the top of the wall including the self weight of the wall above the point where maximum of moment occurs

A_s = the area of steel reinforcement

f_y = the strength of steel as specified by the NZSEE guidelines

f'_m = specified compressive strength of masonry from Table 10.1

t = thickness of the masonry wall

Appendix D

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Toilets Cashmere Rd/Valley Rd Reserve	Reviewer:	Derek Chinn
	Unit / No: / Street	CPEng No:	177243
Building Address:	73 Cashmere Road, Cashmere	Company:	GHD
Legal Description:		Company project number:	513059634
		Company phone number:	
	Degrees / Min / Sec		
GPS south:		Date of submission:	
GPS east:		Inspection Date:	15/02/2012
		Revision:	
Building Unique Identifier (CCC):	PRK_1795_BLDG_001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	slope < 1in 10	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	C		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	Concrete slab on grade
Building height (m):	3.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):				
Age of Building (years):			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	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:			
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	

Lateral load resisting structure

Lateral system along:	other (note)		Note: Define along and across in detailed report!	describe system	Unreinforced unfilled masonry block
Ductility assumed, μ:	1.00				
Period along:	0.10	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	
Lateral system across:	other (note)			describe system	Unreinforced unfilled masonry block
Ductility assumed, μ:	1.00				
Period across:	0.10	0.00		estimate or calculation?	estimated
Total deflection (ULS) (mm):				estimate or calculation?	
maximum interstorey deflection (ULS) (mm):				estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Metal	describe	
Glazing:	timber frames		
Ceilings:	strapped or direct fixed		
Services(list):			

Available documentation

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

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	0-25mm	notes (if applicable):	Possible settlement occurred near adjancent masonry wal
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:
	Describe (summary):	Significant shear cracking in adjacent wall	
Across	Damage ratio:	0%	
	Describe (summary):	Significant shear cracking in adjacent wall	

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

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

Recommendations

	Level of repair/strengthening required:			Describe:
	Building Consent required:			Describe:
	Interim occupancy recommendations:	do not occupy		Describe:
Along	Assessed %NBS before:	5%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: quantitative DEE assessment
	Assessed %NBS after:	5%		
Across	Assessed %NBS before:	5%	##### %NBS from IEP below	
	Assessed %NBS after:	5%		

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):	0.1	0.1
(%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 =$k\mu$, 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:		1
3.2. Vertical irregularity, Factor B:		1
3.3. Short columns, Factor C:		1
3.4. Pounding potential	Pounding effect D1, from Table to right	
	Height Difference effect D2, from Table to right	
	Therefore, Factor D:	0
3.5. Site Characteristics		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!**

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Draft	Eddie He	Kevan O'Brien		Derek Chinn		10/08/12
Final	Eddie He	Kevan O'Brien		Derek Chinn		05/02/13