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Woolston Park - Toilet Block
PRK 0919 BLDG 002 EQ2
Detailed Engineering Evaluation
Quantitative Report
Version FINAL

502 Ferry Road, Woolston

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

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502 Ferry Road, Woolston

Christchurch City Council

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Date
5 April 2013

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Quantitative Report Summary

Woolston Park - 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 Woolston Park Toilet Block at 502 Ferry Road, Woolston, 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 16th July 2012 and 15th October 2012.

Building Description

The overall structure comprises of a single toilet block with an independent roof structure. Roof and wall construction is consistent throughout. The roof is formed by curved lightweight metal cladding on steel tube purlins rigidly connected to trusses comprised of similar steel sections. Steel circular hollow columns extend from the roof structure to foundations. Walls extending from strip footings to eaves level are formed by reinforced fully filled 140mm concrete masonry units.

Key Damage Observed

Key damage observed includes:-

o Removal of freestanding privacy walls on both ends of toilet block post-earthquake with significant cracking and debris around base

Building Strength

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the building's baseline post-earthquake capacity (including critical structural weaknesses and earthquake damage) has been assessed to be the order of 50% NBS.

There were no critical structural weaknesses identified in the inspection; consequently there has been no reduction of the baseline %NBS. The building has been assessed to have a seismic capacity in the order of 50% NBS and is therefore considered to be a potential Earthquake Risk building.

Recommendations

The recent seismic activity in Christchurch has caused visible damage to the building and the attached freestanding walls. The building has achieved approximately 50% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends strengthening options should be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance with the NZSEE guidelines.

As no immediate collapse hazards or critical structural weaknesses have been identified and the building has achieved 50% NBS the building can remain occupied. It is recommended that the removed freestanding privacy walls should be reinstated to bring the building back to a pre-earthquake condition.

1. Background

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

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

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the concrete 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 block is located at 502 Ferry Road, Woolston Park in Woolston (refer Figure 3). The original construction date of the structure is unknown but based on site observation is estimated to be the early 1980's. The toilet block is not connected to any other structure in the park. The park site is bordered by residential properties in the southern and eastern directions. The Woolston School is located to the west of the park. The closest structure to the toilet block is a residential property on the opposite side of Ferry Road approximately 30m away.

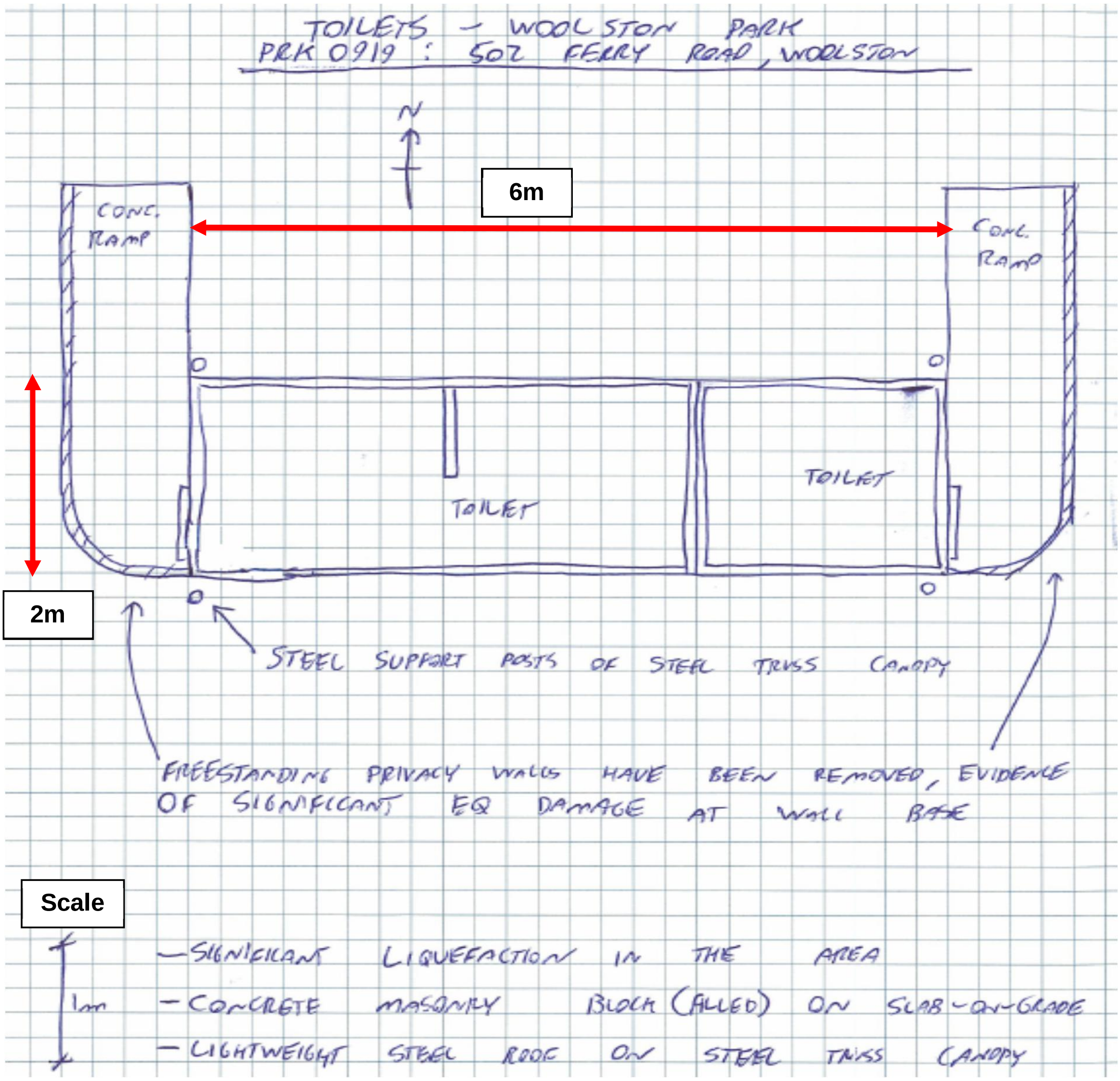

Figure 2   Plan Sketch Showing Key Structural Elements

The park has its eastern boundary adjacent to the Heathcote River but the toilet block is over 200m from the river.

The single storey toilet block has a concrete slab on grade floor. The building has filled concrete masonry block walls with an independent roof structure. Roof and wall construction is consistent throughout. The roof is formed by curved lightweight metal cladding supported by steel tube purlins rigidly connected to similar trusses. Steel circular hollow support columns extend from the roof structure to foundations.

The dimensions of the main toilet block are approximately 6m long by 2m wide and 3.2m in height. Concrete ramps lead to the entrances on both sides of the toilet block.

Adjoining both ramps were 4m long unsupported concrete masonry block walls providing privacy to toilet block users. The freestanding walls on both sides have been recently removed, and the damage to the base indicates potential earthquake damage. The bases of the freestanding walls show significant cracks, exposed reinforcement, and debris.

No plans were available for the structure.

4.2 Gravity Load Resisting System

The roof gravity loads in the structure are supported by steel trusses across the structure. The steel roof cladding is supported by a welded group of steel trusses and bracing. The roof trusses are independently supported by four steel posts and are not connected to the concrete masonry block walls. The roof loads are then transferred from the steel posts to concrete pad footings, separate from the slab, and from there into the ground. The masonry wall loads are supported by the concrete floor slab and strip footings.

4.3 Lateral Load Resisting System

The roof consists of a steel frame constructed of circular hollow sections fully welded at their connections.

The moment frame provides what appears to be adequate seismic load resistance to brace the roof and transfer that load to the masonry walls below through welded shear connections to the top of the masonry walls at each tube post.

The 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 masonry walls are not propped at the eaves level by the roof structure. The masonry walls are considered to be acting as vertical cantilever walls connected to the foundations. In the absence of propping, there is a nominal level of horizontal spanning capability is present in the masonry, allowing lateral support from adjacent walls. However this action has been treated as negligible and disregarded as a support mechanism.

5. Damage Assessment

5.1 Surrounding Buildings

Other structures around Woolston Park have sustained significant damage. The school nearest the toilet block has experienced significant liquefaction and differential settlement. Another building in the park partially collapsed and is now fenced off.

5.2 Residual Displacements and General Observations

Significant cracks were noted at the base of the removed freestanding walls at the side entrances to the toilet block next to the access ramps. The extent of cracking, debris and damage indicates that removal was likely prompted by earthquake damage. The cracking, damage and minor debris are clearly visible in Photographs 4, 5 and 6 in Appendix A.

No damage was evident to the steel truss roof structure and no damage was evident to the exterior of the building.

5.3 Ground Damage

There was evidence of ground movement and liquefaction in many areas of the park in addition to ground damage on the property and surrounding neighbours land. The liquefaction on site has been cleared since the significant aftershocks. There is minor overturning evident in the low boundary fences along Ferry Road. The toilet block appears to be level with no signs of building settlement or foundation damage.

6. Geotechnical Consideration

6.1 Site Description

The site is situated in the suburb of Woolston, east of Christchurch City centre. The site is relatively flat at approximately 10m above mean sea level. It is approximately 200m northwest of Heathcote River, 270m northeast of the Main South Line Railway, and 3km west of the coast (Pegasus Bay).

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area1 indicates that the site is underlain by:

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

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

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there are six boreholes located within 100m of the site. Four boreholes with significant information are summarised in Table 2.

These indicate that the area is underlain by layers of sand and silt mixtures with lenses of clay.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35-14081-WC	2.52 m	Not indicated	40m SE
M35-14082-WC	2.52 m	Not indicated	51m SE
M35-14083-WC	2.44 m	Not indicated	84m SE
M35-14084-WC	2.74 m	Not indicated	84m SE

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

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 Woolston2. One investigation points were undertaken within 200m of the site, as summarised below in Table 3.

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

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Log Summary
CPT-WSW-17	100m W	0-2 2-6.2 6.2-12 12-15.5	Very loose silty sand to soft sandy silt Soft sandy silt to silty clay Medium dense to dense sand to silty sand Dense to very dense gravelly sand to dense sand/Sand to silty sand (GWT 1.7 bgl)

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

6.2.4 CERA Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has classified 502 Ferry Road, Woolston as “Green Zone – N/A – Urban Nonresidential” category. Land in this zone is generally considered suitable for residential construction, though some areas may require stronger foundations or design where rebuilding or repairs are required. “Not Applicable – Urban Nonresidential” technical category is the classification given for nonresidential properties in urban area beyond the extent of land damage mapping.

However, properties to the north of the site where are classified as “Green Zone, Technical Category 2 – yellow.” Land in this zone is generally considered suitable for residential construction, though some areas may require stronger foundations or design where rebuilding or repairs are required. Technical Category 2, yellow means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of minor to moderate signs of liquefaction at road corridors and nearby properties, as shown in Figure 3.

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

Figure 3 Post February 2011 Earthquake Aerial Photography[3]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise multiple strata of gravel, sand and silt with varying amounts of clay. Ground water table is found to be approximately 2m below ground level.

6.3 Seismicity

6.3.1 Nearby Faults

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years

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

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

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

Greendale (2010) Fault	25 km	W	7.1	~15,000 years
Hope Fault	110 km	NW	7.2~7.5	120~200 years
Kelly Fault	110 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 Woolston, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

6.5 Liquefaction Potential

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

- Signs of minor to moderate liquefaction at road corridors and properties near the site (evidence from the post-earthquake aerial photograph);
- Properties to the north of the site are classified by CERA as "Green Zone, Technical Category 2, yellow";
- Anticipated presence of saturated sand and silt layers beneath the site; and,
- Anticipated shallow ground water table within 2 m of ground level.

6.6 Conclusions & Recommendations

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

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

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

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

7. Assessment

An inspection of the building was undertaken on the 16 July 2012. A further inspection of the building was carried out on 15th October 2012. No placard 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. The main structural components of the building were all able to be viewed due to the exposed simple construction of the building.

Electro-magnetic scanning to the reinforced concrete was undertaken to confirm the presence, size, and spacing of the reinforcement in the block walls. During the electro-magnetic scanning it was determined that the concrete masonry walls contained vertical D12 reinforcement rods at 550 centers, and horizontal D12 reinforcement rods at the top, middle, and bottom of the walls. No drawings were made available for the structure.

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 masonry block walls. The investigation was based on the analysis of the seismic loads that the structure is subjected to, distribution of these forces throughout the structure and the analysis of the capacity of existing structural elements to resist the forces applied. A Hilti PS 200 Ferroscan was used to determine the level of reinforcement present 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 C.

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

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

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

$$S_P = 1.3 - 0.3\mu$$

Where μ is the structural ductility factor. A structural ductility factor of 1.25 has been taken for lateral loading across and along the building; this is due to the walls being constructed of reinforced, filled concrete blocks.

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

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

7.3 Bracing capacity of Reinforced Masonry Walls

7.3.1 Shear Capacity

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

7.3.2 In-Plane Moment Capacity

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;

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

Where

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

N_n = the axial load due to the self-weight of the wall

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

7.3.3 Building Demand

The out-of-plane effects on the individual walls have been checked by analysing the wall as cantilever sections. The walls self-weight was modelled as a uniformly distributed load and multiplied by the elastic response factor, $C_d(T_1)$ per metre width. Structural analysis then determined the critical shear and moment demand.

The wall's out-of-plane capacity has been determined using the methodology for a singly-reinforced wall, as outlined in Sections 7.3.1 and 7.3.2 above, and then checked against the demand.

7.1 Calculation of %NBS

The shear and moment capacity of the concrete 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 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: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil;
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011;
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

8.2 Wall Investigation

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

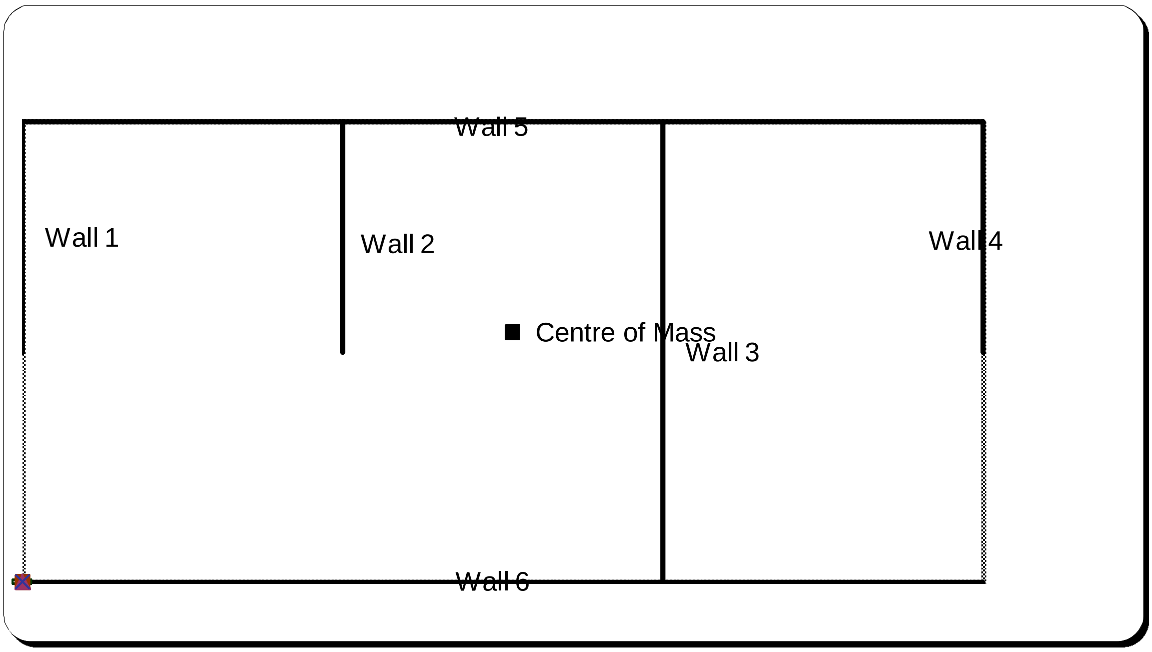

Figure 4 Plan Details and Wall Locations

8.3 Woolston Toilet Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 5

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	27.3	88.4	>100%	**Not at Risk**	33.97	46.8	>100%	**Not at Risk**
2	23.9	88.4	>100%	**Not at Risk**	33.97	46.8	>100%	**Not at Risk**
3	62.7	176.4	>100%	**Not at Risk**	109.3	89.6	82%	**Not at Risk**
4	22.8	88.4	>100%	**Not at Risk**	59.3	63.5	>100%	**Not at Risk**
5	78.2	655.0	>100%	**Not at Risk**	136.2	1323.8	>100%	**Not at Risk**
6	78.2	655.0	>100%	**Not at Risk**	136.2	1323.8	>100%	**Not at Risk**

Table 5 In Plane Analysis Results

The results of the out of plane displacement response capability analysis and subsequent earthquake designation under the NZSEE guidelines are listed in Table 6.

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	27.3	109.5	>100%	**Not at Risk**	7.6	3.8	50%	**Risk**
2	23.9	109.5	>100%	**Not at Risk**	7.6	3.8	50%	**Risk**
3	87.2	218.9	>100%	**Not at Risk**	15.2	7.7	50%	**Risk**
4	22.8	109.5	>100%	**Not at Risk**	7.6	3.8	50%	**Risk**
5	108.7	828.0	>100%	**Not at Risk**	45.6	23.0	50%	**Risk**
6	98.8	828.0	>100%	**Not at Risk**	45.6	23.0	50%	**Risk**

Table 6 Out Of Plane Analysis Results

8.4 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 block has been assessed as achieving 50 **%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines while Woolston Park Toilet is not considered to be Earthquake Prone but it is considered to be a potential Earthquake Risk building.

There are no critical structural weaknesses or collapse hazards identified in this building. The walls have been determined to be weak for out-of-plane bending. The toilet walls have a NBS rating of 50% in out-of-plane bending. In all other load actions to the building is either fully compliant with current NBS levels or at least above 67% NBS. The building has no significant damage and has no collapse hazards.

9. Recommendations

The recent seismic activity in Christchurch has damaged the freestanding privacy walls which have been removed but caused no visible damage to the building. Because the building has no Critical Structural Weaknesses or collapse hazards the building can remain occupied. As the building has achieved between 34% NBS and 67% NBS no further work is required under NZSEE guidelines. The council may wish to consider seismic strengthening at some point in the future. It is recommended that the existing masonry walls be strengthened to at least 67% NBS. The removed freestanding privacy walls should also be reinstated to bring the building back to a pre-earthquake condition.

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 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 North elevation.

Photograph 2 View of the toilet block from the south.

Photograph 3 View of the toilet block from the east.

Photograph 4 Significant damage to the base where the freestanding wall was removed.

Photograph 5 Cracking, damage, and exposed reinforcement at base of removed wall.

Photograph 6 Base of freestanding wall on eastern side shows fewer signs of cracking

Photograph 7 Steel trusses and welded structure.

Photograph 8 Edge of roof structure with steel truss.

Photograph 9 Roof structure is largely supported by four steel posts.

Photograph 10 Area of rear wall where reinforcement checks have been done.

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:	Woolston Park Toilets	Reviewer:	David Lee
	Unit / No: / Street	CPEng No:	112052
Building Address:	Ferry Road, Woolston / 502 /	Company:	GHD
Legal Description:	PRK_0919_BLDG_002 EQ2	Company project number:	
	Degrees / Min / Sec	Company phone number:	
GPS south:		Date of submission:	27/11/2012
GPS east:		Inspection Date:	16/07/2012
		Revision:	
Building Unique Identifier (CCC):	PRK_0919_BLDG_002 EQ2	Is there a full report with this summary?	yes

Site

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

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	Slab on grade
Building height (m):	3.20		height from ground to level of uppermost seismic mass (for IEP only) (m):	1.5
Floor footprint area (approx):	12			
Age of Building (years):			Date of design:	1976-1992
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	steel truss	truss depth, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:			
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	200

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	2
Ductility assumed, μ:	1.25		wall thickness (m):	0.2
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	partially filled CMU		note total length of wall at ground (m):	6

	Ductility assumed, μ:	1.25		wall thickness (m):	0.2
	Period across:	0.40	0.40 from parameters in sheet	estimate or calculation?	estimated
	Total deflection (ULS) (mm):			estimate or calculation?	
	maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Separations:	north (mm):		leave blank if not relevant		
	east (mm):				
	south (mm):				
	west (mm):				
Non-structural elements	Stairs:				
	Wall cladding:	exposed structure		describe	Painted Block Walls
	Roof Cladding:	Metal		describe	Light corrugated steel
	Glazing:	steel frames			
	Ceilings:	none			
	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:	average	Describe damage:	Significant liquefaction and settlement in park
	Settlement:	none observed	notes (if applicable):	
	Differential settlement:	0-1:350	notes (if applicable):	Settlement in park
	Liquefaction:	2-5 $m^3/100m^2$	notes (if applicable):	Significant liquefaction in park
	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:	moderate to substantial (1 in 5)	notes (if applicable):	Visible damage to other structures
Building:	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):	Freestanding side walls have been removed post quake.		
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:	minor non-structural	Describe:	Replace freestanding, missing privacy walls
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before:	50%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	
	Assessed %NBS after:	50%			
Across	Assessed %NBS before:	50%	##### %NBS from IEP below		
	Assessed %NBS after:	50%			

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): 1976-1992

h_n from above: 1.5m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

not required for this age of building

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

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

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

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

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

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6: 1.00

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	
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:**	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
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

	along	across
$\%NBS_b$:	**#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 1.0

Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 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

3.5. Site Characteristics 1

		Along	Across
3.6. Other factors, Factor F	For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.0	1.0
	Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

		Along	Across
3.7. Overall Performance Achievement ratio (PAR)		1.00	1.00
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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