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Cuthberts Green – Toilet/Pavilion
PRK 0893 BLDG 008 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

220 Pages Road, Wainoni

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

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220 Pages Road, Wainoni

Christchurch City Council

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Date
20 March 2013

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Cuthberts Green – Toilet/Pavilion

PRK_0893_BLDG_008 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

220 Pages Road, Wainoni

Background

This is a summary of the Quantitative Report for the Cuthberts Green Toilets and Pavilion at 220 Pages Road, Wainoni, 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 level building with four pavilion changing rooms with showers and a toilet block on the eastern end. The roof and wall construction is consistent throughout. The roof is formed by lightweight metal cladding underlayed with plywood supported by timber purlins and rafters. Timber ridge support beams are connected to the concrete masonry block walls with steel brackets. The southern side of the building facing the fields has a covered veranda. The ends of the veranda rafters are supported by timber beams and circular steel posts. Walls extending from strip footings to ridge level are formed by reinforced partially-filled 140mm concrete masonry blocks supported by concrete strip footings.

Key Damage Observed

Key damage observed includes:-

- Significant cracking, separation, and overturning of block walls on northern side of the building near the north-western corner.
- Settlement of the toilet section of the building with separation of the block walls where they connect to the pavilion end walls.

Building Strength

Based on the information available, and using the NZSEE guidelines for a Quantitative Assessment, the building's baseline post-earthquake capacity (including critical structural weaknesses and earthquake damage) has been assessed to be in the order of 20% NBS.

The significant liquefaction risk on site is considered to be a critical structural weakness that has been identified by the site inspection. The building has been assessed to have a seismic capacity in the order of 20% NBS and it is therefore considered to be Earthquake Prone.

Recommendations

The recent seismic activity in Christchurch has caused significant wall damage to the building. Walls have settled and separated resulting in crack damage. The building has achieved approximately 20% NBS following a Quantitative Detailed Engineering Evaluation Assessment. Further assessment is not required. GHD recommends ground stabilisation and wall strengthening options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance the NZSEE guidelines.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Cuthberts Green Toilet/Pavilion block at 220 Pages Road, Wainoni

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 in-plane and out-of-plane strength 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 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 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/pavilion block is located at Cuthberts Green, 220 Pages Road, Wainoni. The original construction date of the structure is unknown but based on site observation and is estimated to be the late 1970's or early 1980's. The toilet block and pavilion are a single level building in the park. The park site is bordered by residential properties in the northern, eastern, and western directions. The closest structure to the building is another park building approximately 2m away to the east

The pavilion building is approximately 1200m to the southeast of the Avon River.

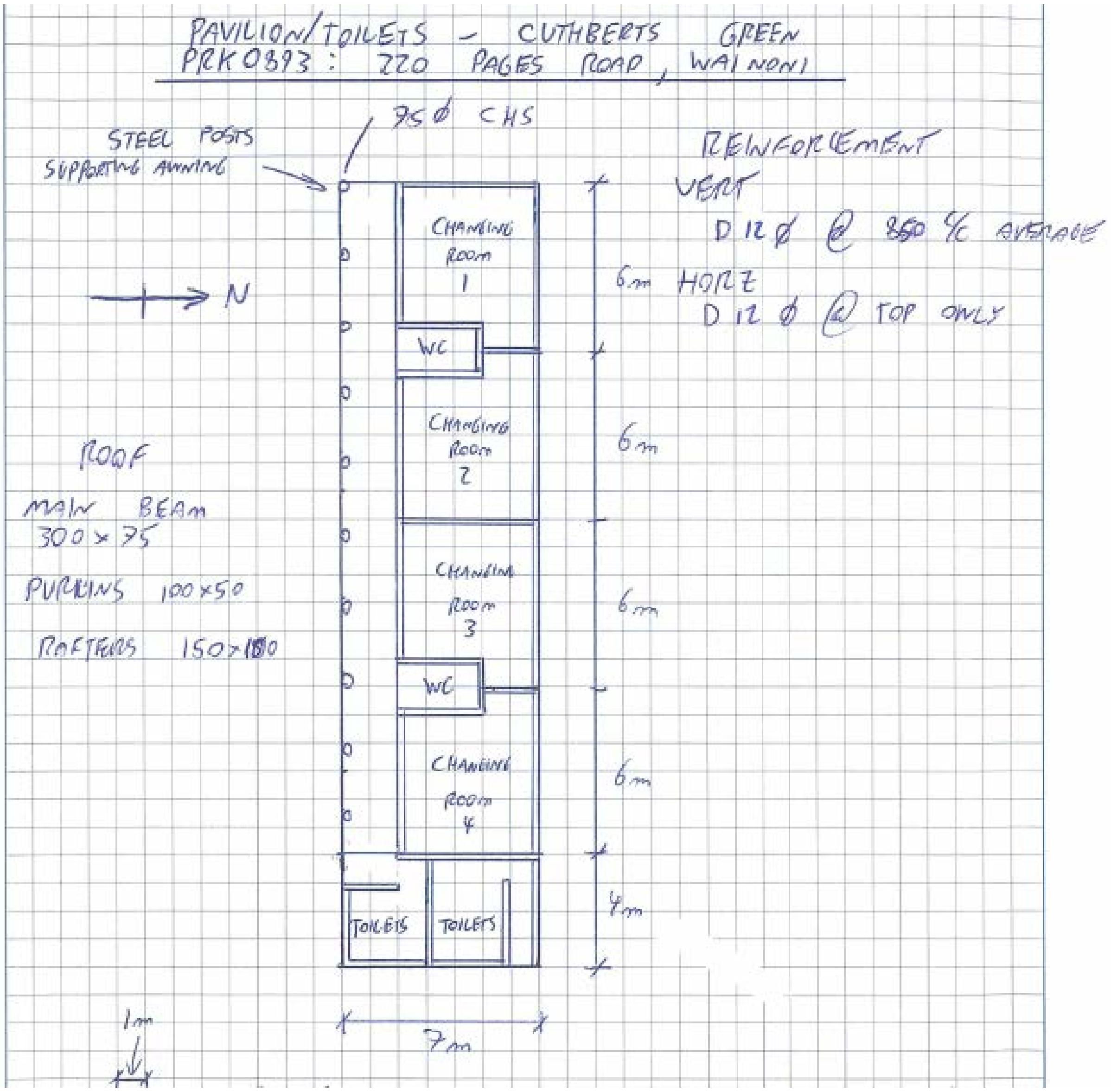


Figure 2 Plan Sketch Showing Key Structural Elements

The single level toilet/pavilion building has a concrete on grade floor slab. The building has partially filled 140mm series concrete masonry block walls with a timber framed roof structure. The roof and wall construction is consistent throughout. The roof is formed by timber purlins, rafters, and ridge beams supported by steel brackets bolted to the concrete masonry block walls.

The dimensions of the toilet/pavilion building are approximately 28m long by 7m wide and 3.6m in height at the apex, dropping to 2.6m at the eaves. The building has external concrete footpaths on eastern, western and southern sides.

From magnetic scanning of the masonry walls reinforcement was found to be vertical D12 bars at 850mm centres and a single horizontal D12 bars at the top of the wall.

No plans were available for the building.

4.2 Gravity Load Resisting System

The roof gravity loads in the structure are supported by timber ridge beams on the structure. The steel roof cladding is supported by timber purlins, rafters, ridge beams, and masonry walls. The roof loads are transferred from the timber rafters into the side walls and the timber ridge beams. From the ridge beams the loads are conveyed to the concrete masonry block walls. The walls then carry the loads into the concrete strip footings.

4.3 Lateral Load Resisting System

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 considered to be propped at the eaves level by the roof structure for out of plane loading. 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

Some damage to surrounding buildings or structures was observed. There was evidence of significant and widespread liquefaction in the area. Nearby numerous commercial and residential properties had been abandoned with significant damage to pavements in the surrounding road systems.

5.2 Residual Displacements and General Observations

The main settlement damage to the structure was settlement of the eastern end of the block. This appears to have caused the cracking and separation of the block walls between the toilet and pavilion sections of the building. The settlement damage and separation damage is visible in Photographs 4, 5, 6 and 7 in Appendix A.

In addition to the settlement damage to the eastern end of the building a section of the north wall appears to have separated from the internal walls and started to overturn. The section of the concrete slab next to the overturning wall has significant cracking as well (See photographs 8, 9, 10, and 11)

5.3 Ground Damage

There was evidence of ground movement and significant liquefaction in many areas of the park and properties adjacent to Cuthberts Green. The liquefaction on this site has mostly been cleared after significant quakes but some liquefaction is still evident in an unpaved area nearby.

6. Geotechnical Investigation

Site Description

The site is situated in the suburb of Wainoni, east of Christchurch City centre. It is relatively flat at approximately 4 m above mean sea level. It is approximately 1200 m southeast of the Avon River, 1.8 km northwest of the Avonhead Heathcote Estuary, and 3 km west of the coast (Pegasus Bay). The Bromley Oxidation ponds are located 200 m to the west of the site.

6.1 Published Information on Ground Conditions

6.1.1 Local Geology

Brown & Weeber (1992)[1] describes the site geology as:

- Marine deposits dominantly sand of fixed and semi-fixed dunes and beaches, Holocene in age
- Underlying sediments (younger than 6500 years) are surface marine sand
- The Riccarton gravels are located approximately 35 m below ground level (bgl)
- Groundwater is likely within 1 m of ground level.

6.1.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there are two boreholes located within 200 m of the site. The bore logs are shown in Table 2.

These indicate that the area is underlain by sand and gravel with few layers of clay and peat.

Groundwater was recorded at 0.5 m bgl in the ECan logs.

Bore Name	Log Depth	Groundwater	From Site	Log Summary
M35/1925	87.5 m	0.5 m bgl	120 m NW	0.0 – 29.8 m Sand
				29.8 – 37.4 m Clay
				37.4 – 39.6 m Sand
				39.6 – 46.3 m Gravel
M35/12522	1.83 m	Not recorded	160 m W	0.0 – 0.1 m Fill
				0.1 – 1.8 m Sand

Table 2 ECan Borehole Summary

It should be noted that the logs have been written by a well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

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

6.1.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken one geotechnical testing within 200 m of the site. This CPT terminated shallowly at 1.8 m and therefore does not provide additional data to this assessment.

6.1.4 CERA 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 is expected to perform in future earthquakes.

The site has been categorised as “N/A – Urban Non-residential”. Neighbouring residential properties located to north have been classified as TC3 (blue) zone. However, properties to northeast and southwest have been classified TC2 (yellow) zone.

TC2 means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

TC3 means that moderate to significant land damage from liquefaction is possible in future significant earthquakes

6.2 Post Earthquake Land Observations

6.2.1 Post-Earthquake Aerial Photography

Aerial photography taken following the 22 February 2011 and 4 September 2010 earthquakes shows no signs of liquefaction directly surrounding the structure. However, there are significant signs of liquefaction on the sports field adjacent to the site, as shown in Figure 3.

Figure 3 Post February 2011 Earthquake Aerial Photography[2]

6.2.2 Field Observations

During the site investigation the following observations were noted. The building has suffered cracking of the concrete slab and walls as a result of the Canterbury Earthquake sequence. Some damage including minor settlement has been observed at the eastern end of the building on the boundary of the changing room and toilet sections of the building.

Anecdotal evidence suggests the toilet section has been added on to the changing rooms at some time.

6.3 Seismicity

6.3.1 Nearby Faults

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

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

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

Greendale Fault (2010)	22km	W	7.1	~15,000 years
Hope Fault	130km	NW	7.2~7.5	120~200 years
Porter Pass Fault	70km	NW	7.0	1100 years
Port Hills Fault (2011)	4km	S	6.3	Not Estimated

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

The recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; these include the Greendale Fault and the Port Hills Fault. Research and published information on this system is in development and average recurrence intervals are yet to be established for the Port Hills Fault.

6.3.2 Ground Shaking

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 significant peak ground accelerations (PGA) across large parts of the city.

Conditional PGA's from the CGD indicate the PGA to be 0.2 g during the 4 September 2010 earthquake, 0.55 g on 22 February 2011, and 0.31 g on 13 June 2011.

6.4 Slope Failure and Rockfall Potential

Given the site's location in Bromley and the flat elevation, 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 Field Investigations

The geotechnical field investigation comprised a site walkover, one machine borehole (BH101) and two cone penetrometer tests (CPT201 and CPT202) located around the building. The investigation locations are shown in Figure 2 and the GPS locations of the tests are tabulated in Table 4.

Borehole Number	Depth (m bgl)	Northing	Easting
BH101	19.6m	5742761	2485508

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

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

CPT201	22.0m	5742757	2485511
CPT202	20.0m	5742746	2485503

Table 4 Investigation Locations

Machine drilled borehole and CPT's were undertaken by McMillan Specialist Drilling Services on 18 -19 October 2012.

Groundwater was measured at 1.2 m and 1.5 m bgl during the CPT tests.

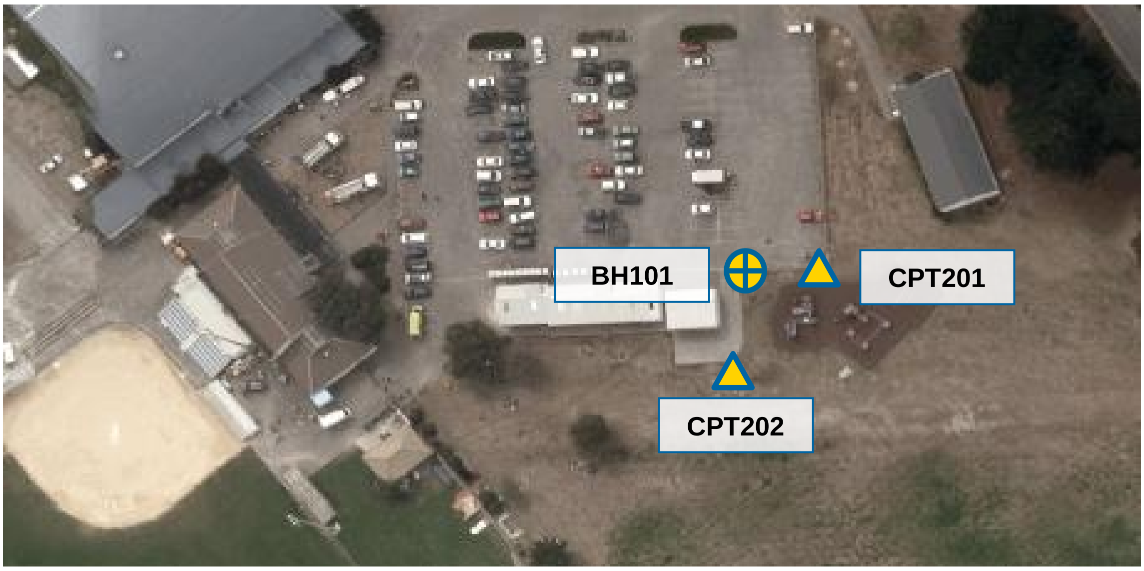


Figure 4 Investigation Location Plan

6.6 Ground Conditions Encountered

A summary of the ground conditions encountered in BH101 are shown in Table 5.

Depth (m)	Lithology	SPT-N Values[5]
0.0 – 1.4	Coreloss *inferred* silt underneath hardfill	-
1.4 – 1.9	Silty fine SAND, loose	4
1.9 – 14.2	Fine to medium SAND, medium dense	15 - 27
14.2 – 14.3	PEAT	-
14.3 – 19.6	Fine to medium SAND, dense	27 - 39
19.64	End of Borehole – Target Depth Achieved	

Table 5 Summary of Machine-drilled Boreholes

Detailed engineering borelogs can be found in Appendix D.

[5] Uncorrected SPT-N value see McMillan Specialist Drilling Services logs in Appendix D for further detail

A summary of the soil behaviour type determined from the CPT results is shown in Table 6. Groundwater was encountered at 1.2m and 1.5m bgl in the CPT's.

6.7 Liquefaction Assessment

Due to the anticipated presence of loose/soft alluvial soils a comprehensive liquefaction analysis has been undertaken.

6.7.1 Parameters used in Liquefaction Analysis

Assumptions made for the analysis process are as follows:

- Importance Level 2, 50-year design life, giving peak ground accelerations (PGA's) of:
  - → 0.35g for ULS,
  - → 0.13g for SLS;
- Earthquake Magnitude 7.5; and
- Groundwater levels at 1.2m bgl (form CPT)

Soil unit weights have been approximated using the tip resistance and sleeve friction from the CPT investigation data using formulae from Robertson & Cabal[6].

The liquefaction analysis process has been conducted using the methodology from Robertson & Wride[7], and from the NZGS Guidelines[8]. Settlements were estimated using the methodology from Zhang et al (2002)[9].

6.7.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 6, indicate that most of the soil is considered highly liquefiable.

Please refer to Appendix D for further detail.

Depth (m)	Soil Behaviour Type	Triggering Factor F_L	Liquefaction Susceptibility [10]
0.0 – 1.2	SAND	>1.5	Not liquefiable – above water table
1.2 – 2.3	SAND	0.5 – 1.5	Moderate

[6] Robertson P.K., & Cabal K.L. (2010): *Estimating soil unit weight from CPT*. Gregg Drilling & Testing Inc.: Signal Hill, CA USA.

[7] Robertson P.K. & Wride C.E. (1998): *Evaluating cyclic liquefaction potential using the cone penetration test.* Canadian Geotechnical Journal, 35: pp. 442-459.

[8] Cubrinovski M., McManus K.J., Pender M.J., McVerry G., Sinclair T., Matuschka T., Simpson K., Clayton P., & Jury R. (2010): *Geotechnical earthquake engineering practice: Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society.

[9] Zhang G., Robertson P.K., & Brachman R.W.I. (2002): *Estimating liquefaction-induced ground settlements from CPT for level ground*. Canadian Geotechnical Journal, Vol 39, pp. 1168-1180.

[10] Table 6.1, NZGS Guidelines Module 1 (2010)

2.3 – 3.7	SAND	1.0 – 2	Low
3.7 – 5.6	SAND Mixture/SAND	0.5 – 1.5	Moderate to High
5.6 – 7.7	SAND	>1.2	Low
7.7 – 12.6	SAND	0.5 – 1.5	High
12.6 – 13.6	SAND	>1.2	Low
13.6 – 19.5	SAND	0.5 – 1.5	Moderate to Low
19.5 – 22.0	SAND	>1.5	Low

Table 6 Summary of Liquefaction Susceptibility

Settlement estimates for the CPT locations are listed in Table 7.

CPT Number	SLS, Total	SLS Index Value	ULS, Total
CPT 201	8mm	8mm	165mm
CPT 202	41mm	38mm	225mm

Table 7 Estimated Liquefaction Induced Settlements

The SLS index value reflects the effects of vertical settlement of the shallow soils (<10m) for an SLS event.

Please refer to Appendix D for further details.

6.7.3 Liquefaction Summary

The site is considered to have a moderate susceptibility to liquefaction based of the following:

- Observations of liquefaction in the surrounding area from post-earthquake aerial photography;
- Surrounding properties are classified TC2 and TC3;
- Estimated ULS settlements are greater than 100mm which categorise the site TC3, whereas SLS settlements are less than 50mm which categorise the site as TC2.
- Presence of several liquefiable layers identified in liquefaction assessments

6.8 "Sufficiently Tested at SLS"

Site observations of recent earthquake damage can be correlated to the likely performance of the site at serviceability limit state (SLS) by comparing the PGA observed with design values. This methodology is outlined in the MBIE guidance on Liquefaction Methodology.

Since the PGA for 22 February exceeds 170% of the SLS value, the site can be considered “sufficiently tested at SLS”. As a result, the ground damage during a future moderate earthquake (SLS) is likely to be similar or less than that observed in the 22 February 2011 earthquake

6.9 Interpretation

The site is considered to have moderate susceptibility to liquefaction. This is based on there being no obvious signs of liquefaction directly outside the structure’s footprint. Significant liquefaction was observed in the neighbouring field and carpark. The liquefaction analysis indicates highly liquefiable shallow soil from 1.2 m to 2.3 m bgl and 3.7 m to 5.6 m bgl. Ground settlement was observed onsite between the changing rooms and toilet sections of the building.

Anecdotal evidence suggests the toilet section has been added on to the changing rooms at some time. This would justify the significant damage to this section as it settled differentially to the changing rooms.

6.9.1 Summary and Recommendations

The subject structure has remained operational throughout the Canterbury earthquake sequence.

Based on the information presented above, we recommend the following for the subject site.

- A soil class of **D** (in accordance with NZS 1170.5:2004) should be adopted for this site;
- The site has a moderate susceptibility to liquefaction.
- While the nearby residential properties have a TC3 categorisation, the ground conditions at the subject site have behaved with TC2 type characteristics.

6.10 Scope and Limitations

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 by third parties.

7. Assessment

An inspection of the building was undertaken on the 16th July 2012. A further inspection of the building was carried out on 18th 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. No drawings were made available for the structure.

The inspections also consisted of scrutinising the building to determine the structural systems and likely behaviour of the building during earthquakes. 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 masonry 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 the following sections.

7.2 Seismic Coefficient

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

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

Where

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

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

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

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

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

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

Where μ is the structural ductility factor. A structural ductility factor of 1.25 has been taken for lateral loading across and along the building; this is due to the walls being constructed of reinforced, partially 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 walls 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 walls was calculated from Cl 10.3.2.1, Equation 10-4;

$$\mathrm{V_n} = \mathrm{v_n\, b_W\, d}\, \phi$$

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 walls 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 walls was calculated using the formula;

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

Where

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

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.

7.4 Calculation of %NBS

The shear and moment capacity of the concrete masonry walls, 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. It should be noted that due to the wall height, length and location walls number 9, 17, & 18 had minor loadings and therefore did not require evaluation.

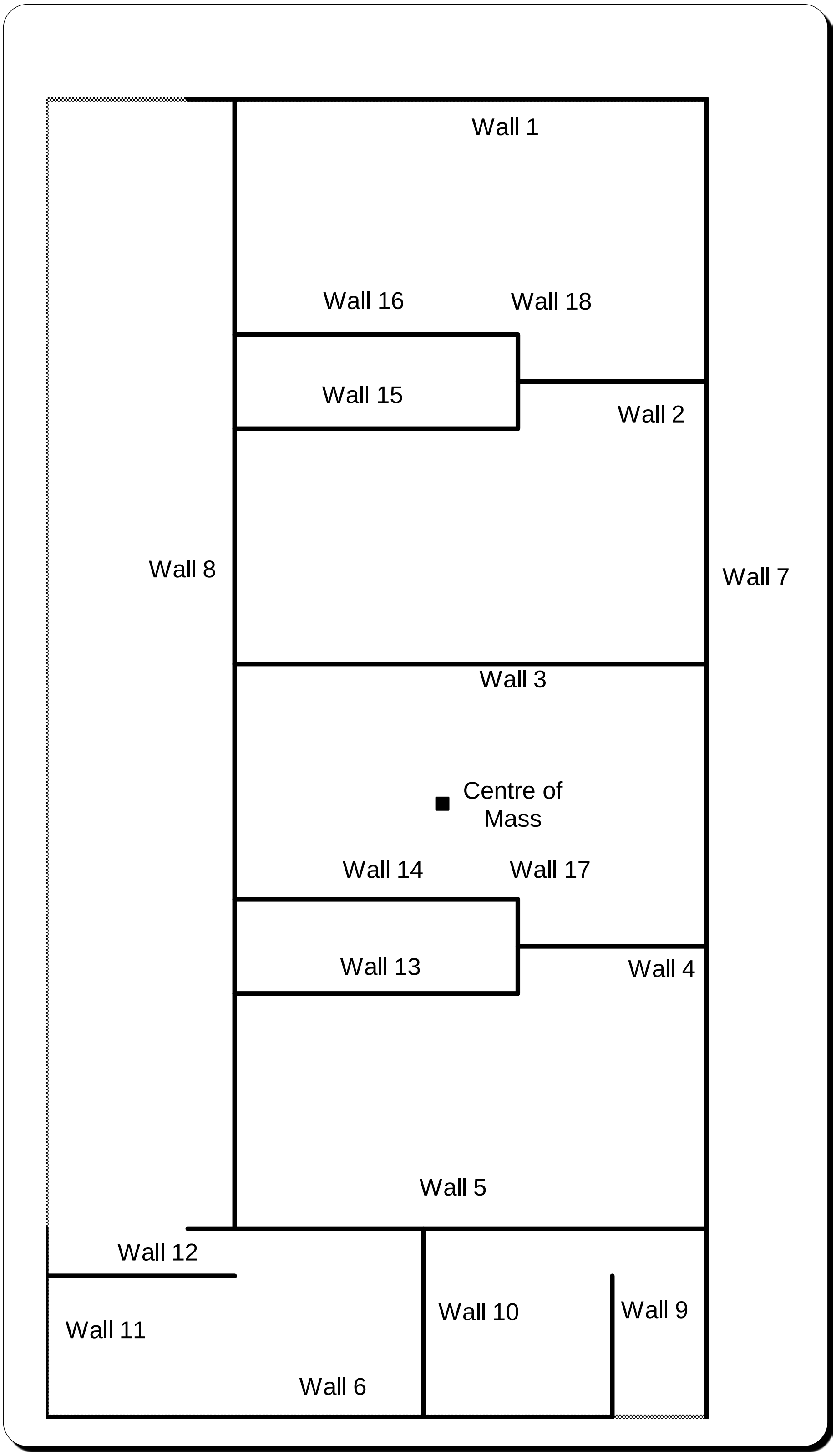


Figure 5 Plan Details: Roof Ouline and Wall Locations

8.3 Cuthberts Green Toilet/Pavilion Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 7. (Walls 9, 17, and 18 were not considered in the analysis.)

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	159.8	591.9	>100%	Not at Risk	511.3	4643.7	>100%	Not at Risk
2	19.6	97.1	>100%	Not at Risk	62.7	82.0	>100%	Not at Risk
3	112.9	591.9	>100%	Not at Risk	361.3	4643.7	>100%	Not at Risk
4	17.4	97.1	>100%	Not at Risk	55.7	82.0	>100%	Not at Risk
5	136.7	591.9	>100%	Not at Risk	437.4	4643.7	>100%	Not at Risk
6	166.7	349.8	>100%	Not at Risk	533.5	663.2	>100%	Not at Risk
7	553.0	1804.8	>100%	Not at Risk	1548.5	12838	>100%	Not at Risk
8	496.6	1546.9	>100%	Not at Risk	1390.4	9661	>100%	Not at Risk
9	N/A	N/A	N/A	N/A	15.8	N/A	N/A	N/A
10	10.8	145.3	>100%	Not at Risk	28.2	165.9	>100%	Not at Risk
11	18.9	145.3	>100%	Not at Risk	49.1	165.9	>100%	Not at Risk
12	22.4	99.5	>100%	Not at Risk	58.2	82.0	>100%	Not at Risk
13	45.0	145.3	>100%	Not at Risk	126.1	165.9	>100%	Not at Risk
14	44.5	145.3	>100%	Not at Risk	124.5	165.9	>100%	Not at Risk
15	49.7	145.3	>100%	Not at Risk	139.3	165.9	>100%	Not at Risk
16	51.1	145.3	>100%	Not at Risk	142.9	165.9	>100%	Not at Risk
17	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
18	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 8 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 8. (Walls 9, 17, and 18 were not considered in the analysis.)

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	41.3	1008	>100%	Not at Risk	63.3	12.7	20%	Prone
2	32.9	48.7	>100%	Not at Risk	23.0	4.6	20%	Prone
3	41.3	1008	>100%	Not at Risk	63.3	12.7	20%	Prone
4	32.9	48.7	>100%	Not at Risk	23.0	4.6	20%	Prone
5	41.3	1008	>100%	Not at Risk	63.3	12.7	20%	Prone
6	42.5	59.4	>100%	Not at Risk	69.1	13.8	20%	Prone
7	49.4	67.1	>100%	Not at Risk	246.8	64.7	26%	Prone
8	49.4	67.1	>100%	Not at Risk	211.5	55.4	26%	Prone
9	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

10	32.8	48.6	>100%	Not at Risk	26.4	6.9	26%	Prone
11	32.8	48.6	>100%	Not at Risk	26.4	6.9	26%	Prone
12	32.9	48.7	>100%	Not at Risk	15.2	4.6	30%	Prone
13	32.8	48.6	>100%	Not at Risk	26.4	6.9	26%	Prone
14	32.8	48.6	>100%	Not at Risk	26.4	6.9	26%	Prone
15	32.8	48.6	>100%	Not at Risk	26.4	6.9	26%	Prone
16	32.8	48.6	>100%	Not at Risk	26.4	6.9	26%	Prone
17	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
18	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Table 9 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 have resulted in a reduction in the level of compliance of existing buildings relative to new buildings despite the capacity of existing buildings not changing.

Following a detailed assessment, the toilet/pavilion block has been assessed as achieving 20 **%**NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines Cuthberts Green Toilet/Pavilion is considered to be Earthquake Prone. Significant liquefaction on site is a critical structural weakness. The separation and overturning of walls is also a potential collapse hazard that has been identified in the building. The wall requires urgent securing or propping to remove a public hazard.

9. Recommendations

The recent seismic activity in Christchurch has caused significant damage to the building. Walls have separated from the building and have resulted in damage. The building has achieved approximately 20% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends ground stabilisation and wall strengthening options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance the NZSEE guidelines.

Walls identified as a potential collapse hazard shall be temporarily secured until strengthening works are undertaken.

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.

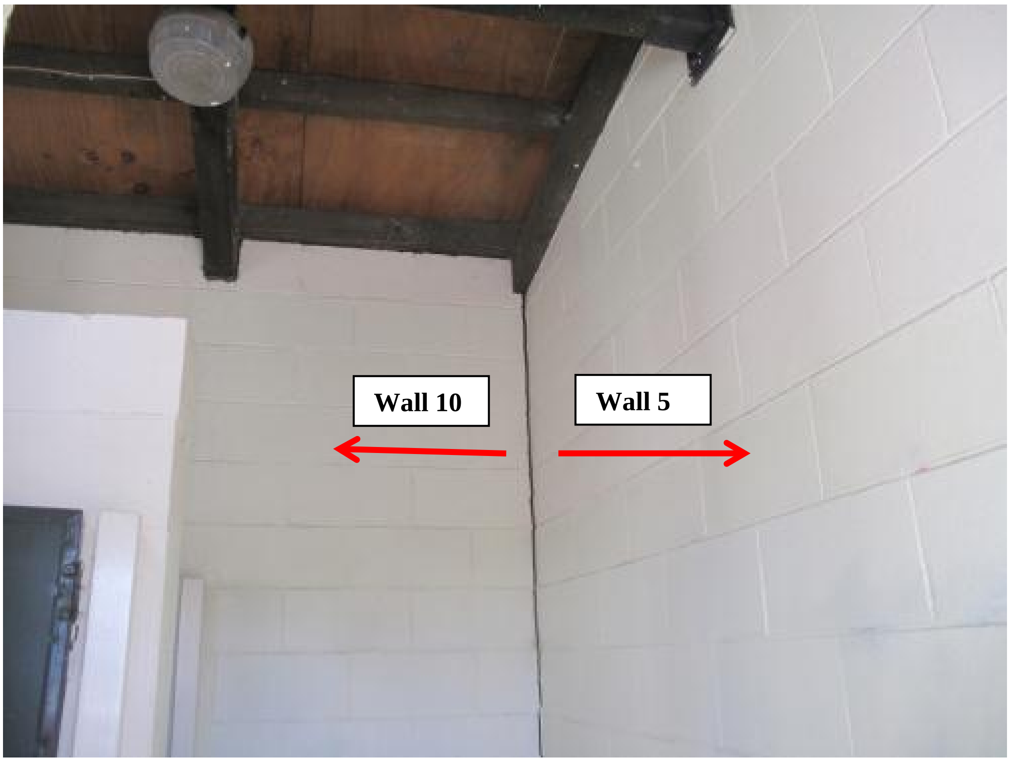
Appendix A

Photographs

Photograph 1 South elevation.

Photograph 2 View of the toilet block from the northwest.

Photograph 3 View of the toilet block from the southwest.



Photograph 4 Gap between Wall 10 and Wall 5, due to settlement of toilet section.

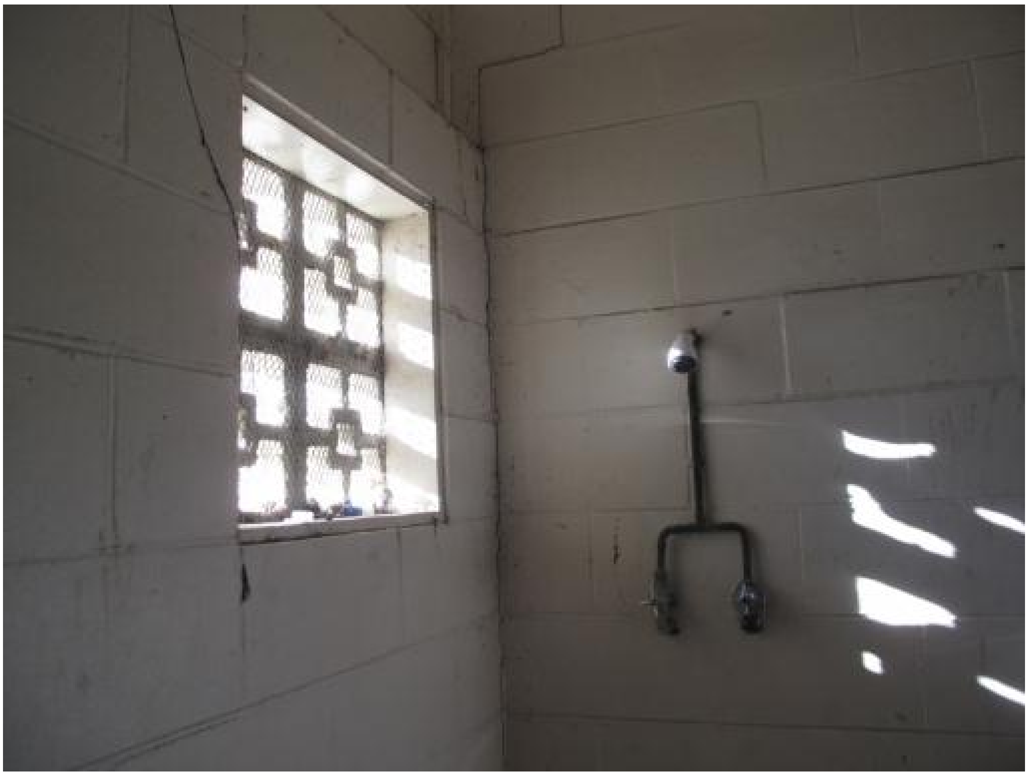
Photograph 5 Gap between Wall 10 and Wall 5, from the south side.

Photograph 6 Eastern end of building where settlement has occurred.

Photograph 7 Southern edge of toilet area. Note differential settlement in slab.

Photograph 8 Step cracks in Wall 2 due to lean in external Wall 7.

Photograph 9 Step cracking extends above the level of external wall 7.



Photograph 10 Crack propagates through full height of wall.

Photograph 11 Cracking in slab at base of section where Wall 2 and Wall 7 intersect.

Photograph 12 Bolted brackets supporting timber ridge beam.

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:	Cuthbert Green Pavilion/Toilet			Reviewer:	David Lee
		Unit No:	Street	CPEng No:	112052
Building Address:	Pages Road, Wainoni	220		Company:	GHD
Legal Description:	PRK_0893_BLDG_008 EQ2			Company project number:	513090250
				Company phone number:	33780900
	Degrees	Min	Sec		
GPS south:				Date of submission:	1/15/2013
GPS east:				Inspection Date:	7/17/2012
				Revision:	
Building Unique Identifier (CCC):	PRK_0893_BLDG_008 EQ2			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	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):			height from ground to level of uppermost seismic mass (for IEP only) (m):	2.4
Floor footprint area (approx):	196			
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:	timber framed	rafter type, purlin type and cladding	
Floors:	concrete flat slab	slab thickness (mm)	
Beams:			
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	140

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in**	note total length of wall at ground (m):	28
Ductility assumed, μ:	1.25	**detailed report!**	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):	7

Ductility assumed, μ:	1.25		wall thickness (m):	0.14
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:	Good	Describe damage:	
Settlement:	0-25mm	notes (if applicable):	Settlement of foundations.
Differential settlement:	1:350-1:250	notes (if applicable):	Settlement of foundations.
Liquefaction:	2-5 m²/100m³	notes (if applicable):	Significant liquefaction visible 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:	widespread to major (in in 3 to most)	notes (if applicable):	Entire area has widespread damage

Building:

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

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening	Describe:	Building has significant damage
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	do not occupy	Describe:	Building should not be used

Along	Assessed %NBS before:	20%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative Assessment
	Assessed %NBS after:	20%			
Across	Assessed %NBS before:	20%	##### %NBS from IEP below		
	Assessed %NBS after:	20%			

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: 2.4m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building

not required for this age of building

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

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

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

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

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

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

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

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	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, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$

	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:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	1

3.4. Pounding potential

Pounding effect D1, from Table to right

Height Difference effect D2, from Table to right

Therefore, Factor D: 0

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	insignificant	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

	Along	Across
3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
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