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Beckenham Park - Toilet Block
PRK 1077 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

18 Norwood Street, Beckenham

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

Beckenham Park – Toilet Block
PRK 1077 BLDG 001 EQ2

Detailed Engineering Evaluation
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Version FINAL

18 Norwood Street, Beckenham

Christchurch City Council

Prepared By
Dale Donovan

Reviewed By
David Lee

Date
20 December 2012

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Beckenham Park - Toilet Block

PRK_1077_BLDG_001 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

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Background

This is a summary of the Quantitative report for the Beckenham Park Toilet Block, 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 18th 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:-

- Minor cracking around the concrete base closest to river and ponding on slab, may indicate minor differential settlement

Building Strength

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

Recommendations

The recent seismic activity in Christchurch has caused only minor settlement damage to the building. The building has achieved approximately 50% NBS following a Quantitative Detailed Engineering Evaluation. Further assessment is not required. GHD recommends 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.

As no immediate collapse hazards or critical structural weaknesses have been identified and the building has achieved 50% NBS and there is no change to the normal occupancy.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Beckenham 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 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 elements to give the percentage of New Building Standard (%NBS) of each of the structural elements.

Electromagnetic scans have been carried out on site to ascertain the extent of the reinforcement in the 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 18 Norwood Street Beckenham. 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 northern and western directions. The Beckenham School is located to the southeastern end of the park. The closest structure to the toilet block is the Beckenham School Pool approximately 10m away.

The park has an eastern boundary on the Heathcote River and the toilet block is less than 25m from the river.

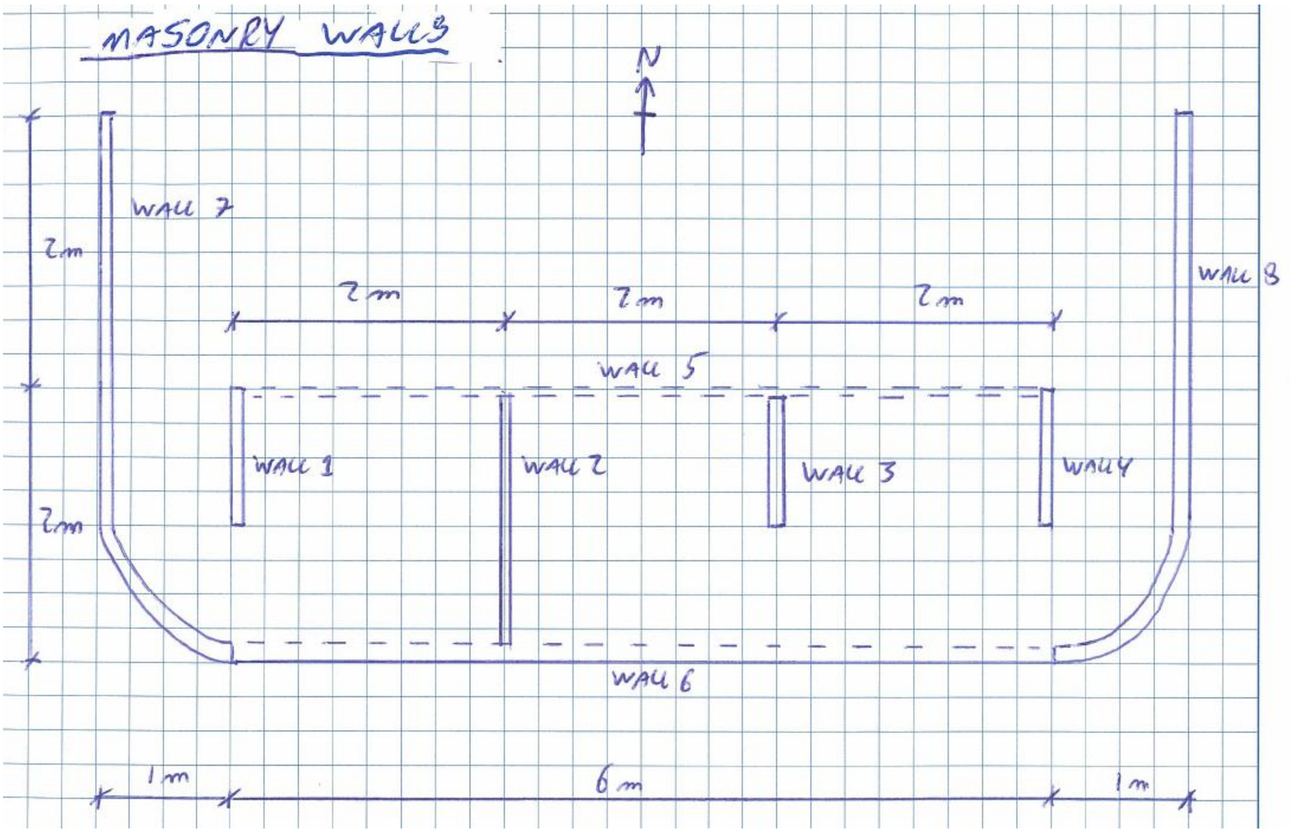

Figure 2 Plan Sketch Showing Key Structural Elements

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 block.

Adjoining both ramps are 4m long concrete masonry block walls providing privacy to toilet block users. These freestanding walls on both sides do not have any visible signs of damage.

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. Return walls can provide restraint to out-of-plane face loading to the masonry walls, but this action has been treated as negligible and disregarded as a support mechanism.

5. Damage Assessment

5.1 Surrounding Buildings

No damage to surrounding buildings or structures was observed. There was lateral spreading evident on the adjacent road surface.

5.2 Residual Displacements and General Observations

The only visible damage to the structure was minor settlement at the southeast corner of the toilet block and minor cracking in the adjacent strip footing. The minor damage and ponding is visible in Photographs 4, 5 and 6 in Appendix A.

No damage was evident in the steel truss roof structure or the exterior walls of the building.

5.3 Ground Damage

There was evidence of ground movement, liquefaction, and lateral spreading in some areas of the park and road adjacent to the Heathcote River. The liquefaction on site has mostly been cleared since the significant aftershocks but some liquefaction is evident in a small pond nearby.

6. Geotechnical Investigation

6.1 Site Description

The site is situated in the suburb of Beckenham, south of Christchurch City centre. The site is gently sloping eastwards towards Eastern Terrace and the Heathcote River. The site elevation is estimated at 6m above mean sea level and is situated approximately 25m south of a pond, 30m west of Heathcote River and 4.5km west of the Avon-Heathcote Estuary.

6.2 Published Information on Ground Conditions

6.2.1 Local Geology

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

- Yaldhurst member of the Springston Formation, dominantly alluvial sand and silt overbank deposits, Holocene in age.
- Underlying sediments (younger than 6500 years) are surface alluvial silt and sand, subsurface marine sand and alluvial silt and sand and some peat with no interbedded gravel;
- The Riccarton Gravel horizon is estimated to be 10m below mean sea level; and
- Groundwater is likely within 1m of ground level.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there are nine boreholes located within 200m of the site. Only two of these boreholes have good lithographic information and these are summarised in Table 2.

These indicate that the area is underlain by sand and clay to 12.4 to 13.1 m, underlain by gravels encountered with the groundwater table recorded at 2m bgl.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	From Site	Log Summary
M36/0987	58.5m	3.2m bgl	50m NW	0.0 to 12.4m Sand and Clay 12.4 to 20.7m Shingle
M36/1115	26.2m	1.8m bgl	50m NW	0.0 to 13.1m Sand and Clay 13.1 to 26.2m Shingle

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.

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site and the information is reported in the Tonkin & Taylor Report for Beckenham-Cashmere Stage 2 Land Report[2]. Eight Cone Penetrometer Tests (CPTs) were undertaken within 100 m of the site. Two of these CPT investigations, conducted by Tonkin & Taylor within 50 m east of the site, had a shallow refusal at 3 m bgl on dense gravelly sand. Six CPT investigations were conducted at Beckenham School by Pro-Drill Ltd on 10 October 2012. The results of these investigations have been obtained through Canterbury Geotechnical Database[3].The results for two of these CPT investigations are summarised below in Table 3.

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Orientation from Site	Depth (m bgl)	Soil Behaviour Type Summary
Beckenham School_CPT 01	24 m S	0.0 – 0.4	CLAYS,
		0.4 – 0.8	SAND mix,
		0.8 – 1.4	Sensitive fine grained,
		1.4 – 2.7	CLAYS,
		2.7 – 4.2	SANDS,
		4.2 – 4.9	Gravelly SAND,
		4.9 – 8.0	SANDS,
		8.0 – 9.2	SAND mix,
		9.2 – 10.1	CLAYS,
		10.1 – 10.5	SAND mix, loose
		10.5 – 11.0	SANDS, dense
			(GWT 1.5m bgl)
Beckenham School_CPT 01	36 m SW	0.0 – 0.4	Pre-drilled
		0.4 – 1.2	SILT mix, stiff
		1.2 – 3.5	SAND mix, very loose
		3.5 – 4.7	SILT mix, stiff
		4.7 – 5.2	SAND mix, loose
		5.2 – 6.4	CLAYS, stiff
		6.4 – 7.1	SILT mix, stiff
		7.1 – 8.4	SANDS, medium dense
		8.4 – 12.7	SILT mix, firm
			(GWT 1.5m bgl)

The CPT result indicates the soils to 16.5m bgl are interbedded sand, silt and clays with the groundwater table 1.5m bgl. Gravels as indicated by the ECan boreholes were not encountered.

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

[3] Canterbury Geotechnical Database (2012) "Geotechnical Investigation Data", Map Layer CGD0010 - 1 June 2012, retrieved [date] from https://canterburygeotechnicaldatabase.projectorbit.com/

6.2.4 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 (TS). These categories describe how the land is expected to perform in future earthquakes.

The site has been categorised as “N/A – Urban Non-residential”. However, the closest neighbouring residential properties immediately to the south have been classified as Technical Category 2 (TC2). The residential areas to the east, on the other side of the Heathcote River meander are classified as Technical Category 3 (TC3).

- TC2 classification indicates that minor to moderate land damage from liquefaction is possible in future significant earthquakes; and
- TC3 classification indicates moderate to significant land damage from liquefaction is possible in future significant earthquakes.

6.2.5 Post Earthquake Observations

Aerial Photography

Aerial photography[4] taken following the 22 February 2011 earthquake shows signs of liquefaction along Eastern Terrace; and in the adjacent school grounds to the west, however the land immediately adjacent to the toilet block structure is obscured by trees. Refer to Figure 3, taken from the Canterbury Geotechnical Database (CGD).

No aerial photography was provided after the 4 September 2010 earthquake covering the site. Aerial photography taken following the 13 June 2011 and the 23 December 2011 earthquake show no further signs of liquefaction.

Site Observations

There was no obvious evidence of liquefaction or significant ground settlement visible in the area of the toilet block during the investigation on 24 October 2012. However some settlement was observed on the river side of the subject structure as detailed below.

Slight cracking and movement around the concrete skirting was visible in the southeast corner of the building where the underground services access the structure.

Lateral Spread

The site is located within 100m of the Heathcote River and is therefore within the potential zone of lateral spread. However, no obvious evidence of global lateral movement was observed or recorded[4]. A localised slump on the riverbank was recorded[4] on the lateral spread cracking map.

[4] Canterbury Geotechnical Database (2012) "Aerial Photography", Map Layer CGD0100 - 1 June 2012, retrieved [date] from https://canterburygeotechnicaldatabase.projectorbit.com/

Figure 3 Post February 2011 Earthquake Aerial Photography[5]

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 in Table 4.

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130km	NW	~8.3	~300 years
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 4 Summary of Known Active Faults[6,7]

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

[6] 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.

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

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 Hazard

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 and has resulted in widespread liquefaction throughout Christchurch.

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

Conditional peak ground accelerations (PGAs) from the Canterbury Geotechnical Database[4] (CGD) indicate the PGA to be 0.24g during the 4 September 2010 earthquake. 0.54g on 22 February 2011, and 0.24g on 13 June 2011.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Beckenham, global slope instability is considered negligible. Any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

Lateral spreading associated with the proximity to the adjacent Heathcote River has already been discussed in Section 6.2.5

6.5 Field Investigations

The geotechnical field investigation comprised a site walkover and one CPT test. The investigation layout is shown in Figure 4. GPS coordinates of CPT01 are E2481529 N5738285. The intrusive investigation was undertaken by McMillan Drilling Ltd on 24 October 2012.

Figure 4 Investigation location

Interpretation of output graphs[8] showing Cone Tip Resistance (qc), Friction Ratio (Fr), Soil Behaviour Type and Inferred Liquefaction Potential are presented in Table 5 and Table 6.

6.6 Ground Conditions Encountered

A summary of the CPT inferred lithology is presented in Table 5.

Depth (m)	Soil Behaviour Type[9]	Cone Tip Resistance q_c (MPa)	Friction Ratio Fr (%)
0.0 – 0.5	SAND Mixture; loose	2.5 – 5.0	1.5 – 2.5
0.5 – 1.2	SILT Mixture; firm	0.5 – 2.0	0.7 – 1.2
1.2 – 1.7	CLAY; very soft	0.1 – 0.2	3.0 – 5.5
1.7 – 1.8	SANDS; loose	5.0 – 7.0	0.1 – 0.2
1.8 – 2.8	SANDS; medium dense	7.5 – 15.0	0.3 – 1.0
2.8 – 9.5	SAND to Gravely SAND; very dense	20 - 50	0.3 – 0.6
9.5 – 10.0	SILT Mixture; stiff	2.0	0.8 – 3.0
10.0 – 10.45	SANDS; medium dense	10 - 15	0.5 – 1.5
10.45	*Gravel?*	Effective Refusal	

Table 5 Summary of CPT Inferred Lithology

6.6.1 Groundwater

Groundwater was encountered at 0.8m bgl during the site investigation.

[8] McMillans Drilling CPT data plots, Appendix C.

[9] Robertson 2010

6.6.2 Summary of Ground Conditions Encountered

In summary there is a good correlation of the ground conditions between the Brown & Weeber information; ECan borelogs; and the on-site investigations. Sands and silts are the dominant lithology to depths of at least 11m bgl with gravel below.

Groundwater levels are anticipated to vary seasonally. For analysis and design purposes, groundwater has been assumed to be 0.8m bgl.

6.7 Liquefaction Analysis

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 Ultimate Limit State (ULS),
  - 0.13g for Serviceability Limit State (SLS);
- Earthquake Magnitude 7.5; and
- Groundwater levels at 0.8m bgl.

Soil unit weights have been approximated using the tip resistance and sleeve friction from the CPT investigation data using formulae from Robertson & Cabal[10].

The liquefaction analysis process has been conducted using the methodology from Robertson & Wride[11], and from the NZGS Guidelines[12]. Settlements were estimated using the methodology from Zhang et al (2002)[13].

6.7.2 Results of Liquefaction Analysis

The results of the liquefaction analysis, as outlined in Table 6, indicate there are narrow bands of interbedded liquefiable silt and loose sand beneath the site. These are specifically at depths between 0.7m to 1.2m, 1.7m to 1.8m and 9.5m to 10.0m bgl.

Depth (m)	Soil Behaviour Type	Triggering Factor F_L	Liquefaction Susceptibility [14]
0.0 – 0.5	SAND Mixture; loose	-	*Non Liquefiable*
0.5 – 0.8	SILT Mixture; firm	-	*Non Liquefiable*

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

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

[12] 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.

[13] 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.

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

Depth (m)	Soil Behaviour Type	Triggering Factor F_L	Liquefaction Susceptibility [14]
0.8 – 1.2	SILT Mixture; firm	0.3 – 0.5	Moderate
1.2 – 1.7	CLAY; very soft	-	*Non Liquefiable*
1.7 – 1.8	SAND; loose	0.3 – 1.0	Low
1.8 – 9.5	SANDS to Gravely SAND	-	*Non Liquefiable*
9.5 – 10.45	SILT Mixture	0.5 – 1.0	Low

Table 6 Summary of Liquefaction Susceptibility

6.7.3 Liquefaction Induced Vertical Settlement

	At SLS	SLS index	At ULS
CPT01	16 mm	16 mm	28 mm

Both the SLS and ULS settlements assessed from the CPT data, are controlled by the silt mixtures and loose sands specifically at depths between 0.7m to 1.2m, 1.7m to 1.8m and 9.5m to 10.0m bgl .

6.7.4 "Sufficiently Tested at SLS"

Since the PGA for 22 February 2011 exceeds 170% of the magnitude-corrected 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 to that observed in the 22 February 2011 earthquake.

6.8 Interpretation

The site is considered to be of low susceptibility to liquefaction. This is based on there being no obvious signs of liquefaction outside the structure's footprint or immediately adjacent to the structure as a result of the Christchurch earthquake sequence and on the results of the CPT investigation.

The liquefaction susceptibility of the silt mixture below 9.5 m is likely to be overestimated.

6.8.1 Summary and Recommendations

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 low susceptibility to liquefaction.
- While the nearby residential properties have a TC2 categorisation, the ground conditions at the subject site are different and have behaved with TC1 type characteristics.
- The site has a low risk of damage from lateral spread.

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 reinforcement in the block 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.

Magnetic scanning indicates vertical reinforcement to be D12 bars at 500mm centers and D20 horizontal bars at top, mid-height, and bottom of the block masonry

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 D.

7.2 Seismic Coefficient

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

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

Where

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

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

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

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

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

$$S_P = 1.3 - 0.3\mu$$

Where μ 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;

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

Where

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

N_n = the axial load 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, 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.

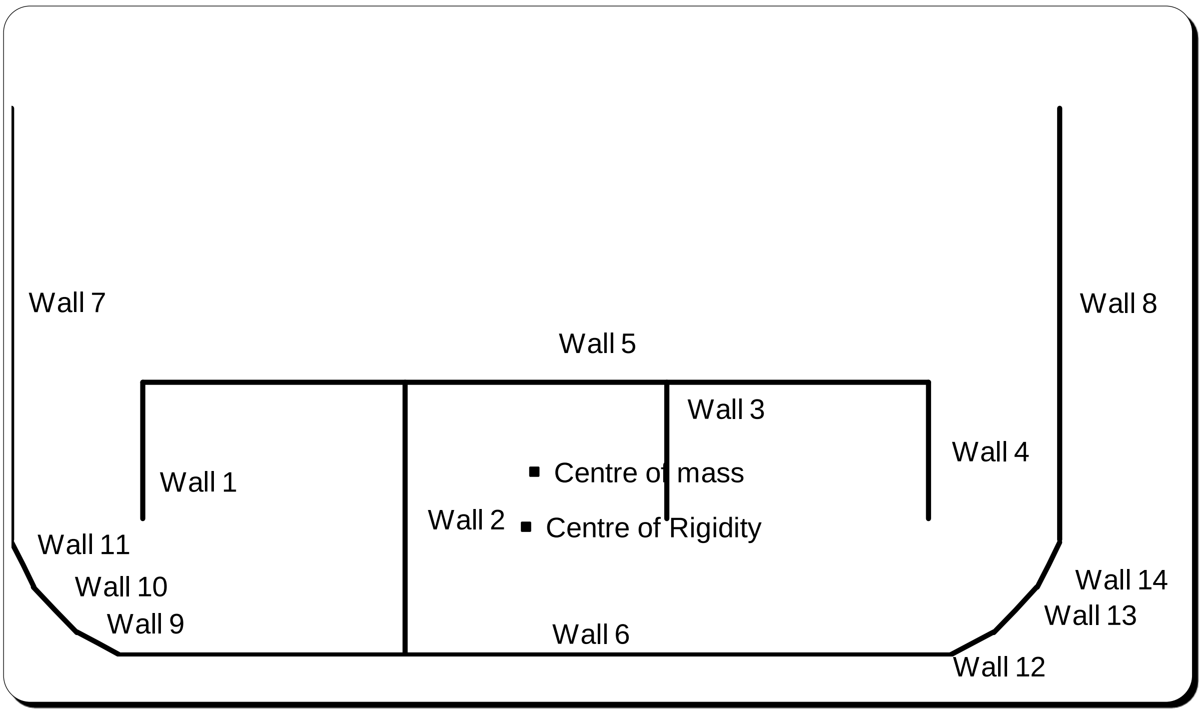

Figure 5 Plan Details and Wall Locations

8.3 Beckenham Park Toilet Block Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 7.

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	6.5	91.4	>100%	Not at Risk	17.0	40.0	>100%	Not at Risk
2	23.5	265.4	>100%	Not at Risk	61.2	134.2	>100%	Not at Risk
3	6.0	91.4	>100%	Not at Risk	15.6	40.0	>100%	Not at Risk
4	6.7	91.4	>100%	Not at Risk	17.5	40.0	>100%	Not at Risk
5	83.5	754.5	>100%	Not at Risk	217.2	1120.5	>100%	Not at Risk
6	80.6	754.5	>100%	Not at Risk	209.5	1120.5	>100%	Not at Risk
7	69.1	487.3	>100%	Not at Risk	179.7	571.2	>100%	Not at Risk
8	71.5	487.3	>100%	Not at Risk	186.0	571.2	>100%	Not at Risk

Table 7 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.

Wall number	V* kN	ϕV_n kN	%NBS	Earthquake Status	M* kNm	ϕM_n kNm	%NBS	Earthquake Status
1	32.93	57.12	>100%	Not at Risk	7.6	3.75	50%	Risk
2	32.93	62.20	>100%	Not at Risk	15.2	7.60	50%	Risk
3	32.93	57.12	>100%	Not at Risk	7.6	3.75	50%	Risk
4	32.93	57.12	>100%	Not at Risk	7.6	3.75	50%	Risk
5	32.93	56.1	>100%	Not at Risk	47.9	24.2	50%	Risk
6	32.93	56.1	>100%	Not at Risk	47.9	24.2	50%	Risk
7	32.93	57.11	>100%	Not at Risk	30.4	15.29	50%	Risk
8	32.93	57.11	>100%	Not at Risk	30.4	15.29	50%	Risk

Table 8 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 for both along and across the building. Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines while the Beckenham Park Toilet Block is not considered Earthquake Prone it is considered a potential Earthquake Risk building. No critical structural weaknesses or collapse hazards have been identified in the building.

9. Recommendations

The recent seismic activity in Christchurch has caused minor settlement and cracking in the strip foundations but caused no visible damage to the building. Because the building has no Critical Structural Weaknesses or collapse hazards the building can remain occupied. GHD recommends strengthening options be explored and implemented to bring the 50 %NBS of the building up to a minimum of 67% NBS in accordance the NZSEE guidelines.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- Drawings of the building were unavailable. As a result the information contained in this report has been inferred from visual inspections of the building and site only.
- No intrusive structural investigations have been undertaken. Electro-magnetic scanning of the walls was conducted to determine the levels of steel reinforcement present.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 8 have been carried out on the structure.

It is noted that this report has been prepared at the request of Christchurch City Council and is intended to be used for their purposes only. GHD accepts no responsibility for any other party or person who relies on the information contained in this report.

10.2 Geotechnical Limitations

This report 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 northeast.

Photograph 4 Minor damage to the base where the strip footings have settled.

Photograph 5 Minor damage to the base where the strip footings have settled.

Photograph 6 Ponding on concrete entrance area possibly due to minor settlement.

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:	Beckenham Park Toilets	Reviewer:	David Lee
	Unit / No: / Street	CPEng No:	112052
Building Address:	Norwood Street, Beckenham / 18 /	Company:	GHD
Legal Description:	PRK_1077_BLDG_001 EQ2	Company project number:	513090256
		Company phone number:	33780900
	Degrees / Min / Sec		
GPS south:		Date of submission:	10/12/2012
GPS east:		Inspection Date:	16/07/2012
		Revision:	
Building Unique Identifier (CCC):	PRK_1077_BLDG_001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	slope < 1in 10	Max retaining height (m):	
Soil type:	sandy silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	25	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):	8			
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:		Describe damage:	
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	slight	notes (if applicable):	

Building:

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

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

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	50%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Quantitative Assessment
	Assessed %NBS after:	50%		
Across	Assessed %NBS before:	50%	##### %NBS from IEP below	
	Assessed %NBS after:	50%		

GHD

Level 11, Guardian Trust House
15 Willeston street, Wellington 6011
T: 64 4 472 0799   F: 64 4 472 0833   E: wgtnmail@ghd.com

Document Status

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