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Nunweek Park Toilets/Pavilion/Changing Rooms PRK 0305 BLDG 002 & PRK_0305_BLDG_003

Detailed Engineering Evaluation

Quantitative Report

Version 2 FINAL

240 Wooldridge Road

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240 Wooldridge Road

Christchurch City Council

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Reviewed By
David Lee

Date
15 August 2013

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Quantitative Report Summary

Nunweek Park Toilets/Pavilion/Changing Rooms

PRK_0305_BLDG_002 & PRK_3505_BLDG_003

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

240 Wooldridge Road

Background

This is a summary of the Quantitative report for the toilet block at 240 Wooldridge Road, and is based in part on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 19th July and 8th of September 2012.

Building Description

The overall structure comprises of two separate units (toilet and sports stores) with independent roof structures linked by a single wall. 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 columns extend from the roof structure to foundations. Walls extending from strip footings to eaves level are lightly reinforced partially grout filled 140mm concrete masonry units.

Key Damage Observed

- Minor cracking was observed in the blockwork walls
- Cracking was observed in the concrete floor slab

Building Strength

Based on the information available, and using the NZSEE Initial Evaluation Procedure, the baseline capacity (excluding critical structural weaknesses and earthquake damage) of the building has been assessed to be in the order of 46% NBS.

There was no damage nor critical structural weaknesses identified in our visual inspection; consequently the %NBS was not reduced. The building has been assessed to have a seismic capacity in the order of 46% NBS and is therefore classified as an Earthquake Risk building.

Recommendations

Following a detailed assessment, the pavilion has been assessed as achieving 46 **%**NBS. All the block walls have been assessed to be less than 67% NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) the building is considered to be an Earthquake Risk building. No critical structural weaknesses or collapse hazards have been identified in the building and as such the building can remain occupied.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Nunweek Park Toilets, Pavilion and Changing Rooms following on from a qualitative report issued in July 2012 in which the building was assessed to be in the order of 42% NBS.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004 and NZS 4230: 1990.

The quantitative assessment to the building comprises an investigation on in-plane and out-of-plane strength of the unreinforced masonry block walls providing lateral restraint to the roof structure. 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 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 Nunweek Park Toilets, Pavilion and changing rooms is located at 240 Wooldridge Road. The date of construction is estimated as late 1980's. The building's use is public toilets and sport club stores.

The overall structure comprises of two separate units (toilet and sports stores) with independent roof structures linked by a single wall. 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 columns extend from the roof structure to foundations. Walls extending from strip footings to eaves level are lightly reinforced partially grout filled 140mm concrete masonry units.

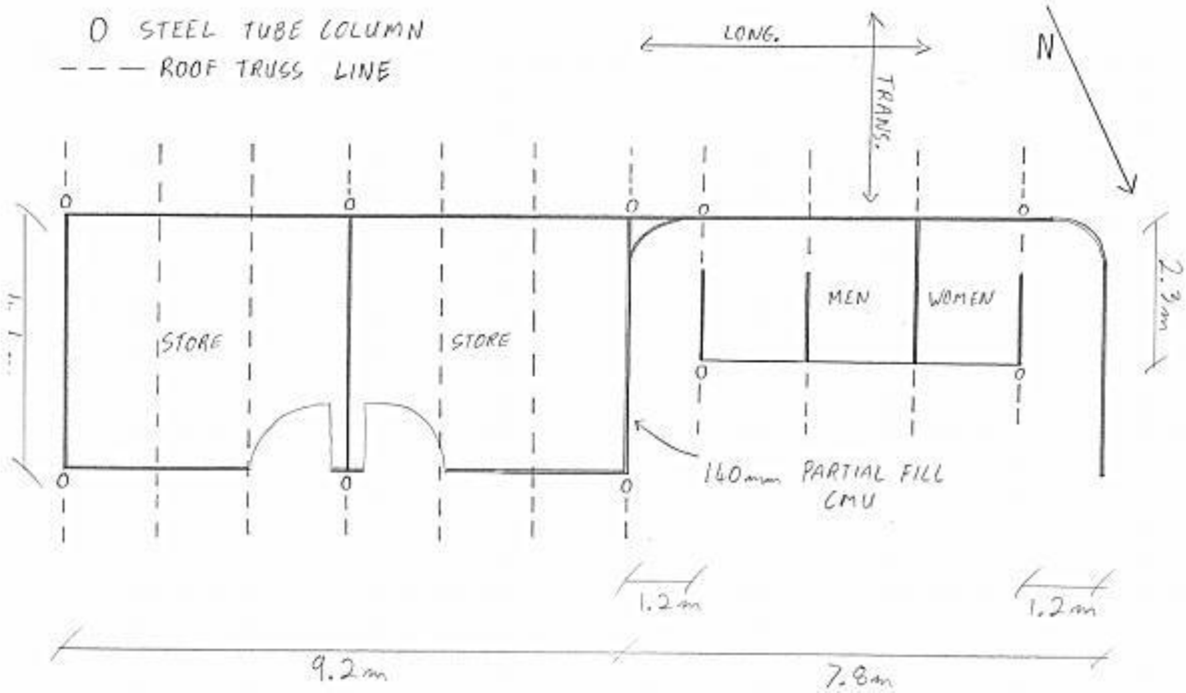

Figure 2 Plan Sketch Showing Key Structural Elements

The building is approximately 17m in length by 4m in width with a height of 3.35m. The building occupies an approximate area of $70m^2$. A residential building is the nearest structure, located over 100m to the east. The flat site is located approximately 6.5km east of Avon River.

No plans were available for the structure.

4.2 Gravity Load Resisting System

The gravity loads are supported by a load bearing frame system. The roof frame trusses transfer the roofs to load the steel column members which transfer the load directly to the foundations. The block walls do not support roof loads and transfer their own self weight through to their foundations which are assumed to consist of a strip footing. The floor is a concrete slab on grade which carries floor loads directly to the founding soil beneath.

4.3 Lateral Load Resisting System

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

The frames provide seismic load resistance to brace the roof and transfer the seismic roof loads to the masonry walls below through bolted shear connections on 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 loads and roof loads through to the foundation level. The walls provide in-plane panel action in shear and moment resistance. Lateral loads are transferred through the walls into the foundations. Loads through the foundations are taken directly into the ground. The masonry walls are cantilevered out-of-plane due to a lack of bolted connections to the roof at eave level and resist seismic loads through out of plane action. The loads are then transferred through to the strip foundation.

5. Damage Assessment

5.1 Surrounding Buildings

No damage was noted to surrounding buildings.

5.2 Residual Displacements and General Observations

No residual displacements of the structure were noticed during our inspection of the building.

A slight crack was noted in a vertical mortar joint, though damage was not sufficient to affect structural performance.

Cracking as observed in the concrete ground slab although it is not known whether this was caused due to the recent seismic activity or pre-existing.

5.3 Ground Damage

There was no evidence of ground damage on the property or surrounding neighbours land.

6. Geotechnical Consideration

6.1 Site Description

The site is situated within a recreational reserve, within the suburb of Harewood in northwest Christchurch. It is relatively flat at approximately 20m above mean sea level. It is approximately 20m north of a swale understood to be the head of the Styx River, 5km south of the Waimakariri River, and 20km west of the coast (Pegasus Bay).

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by Holocene alluvial soils of the Yaldhurst Member, sub-group of the Springston Formation, comprising alluvial sand and silt overbank deposits.

Figure 72 from Brown & Weeber indicates that groundwater is likely to be 5m below ground level (bgl), and the anticipated liquefaction susceptibility is low.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates the closest borehole with a lithographic log is located 220m west of the site (see Table 2). This borehole indicates the area typically comprises sand and gravel with an interbedded clay unit.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35-1494	15.8m	3.7m bgl	220m W

It should be noted that the boreholes were sunk for groundwater extraction and not for geotechnical purposes. Therefore, the amount of material recovered and available for interpretation and recording will have been variable at best and may not be representative. The logs have been written by the well driller and not a geotechnical professional or to a standard. In addition strength data is not recorded.

6.2.3 EQC Geotechnical Investigations

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

6.2.4 Land Zoning

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

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

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

The site is indicated to be within an area zoned TC2 (yellow)[2], meaning that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

However, residential properties to the southwest are zoned TC1 (grey).

6.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 3**.**

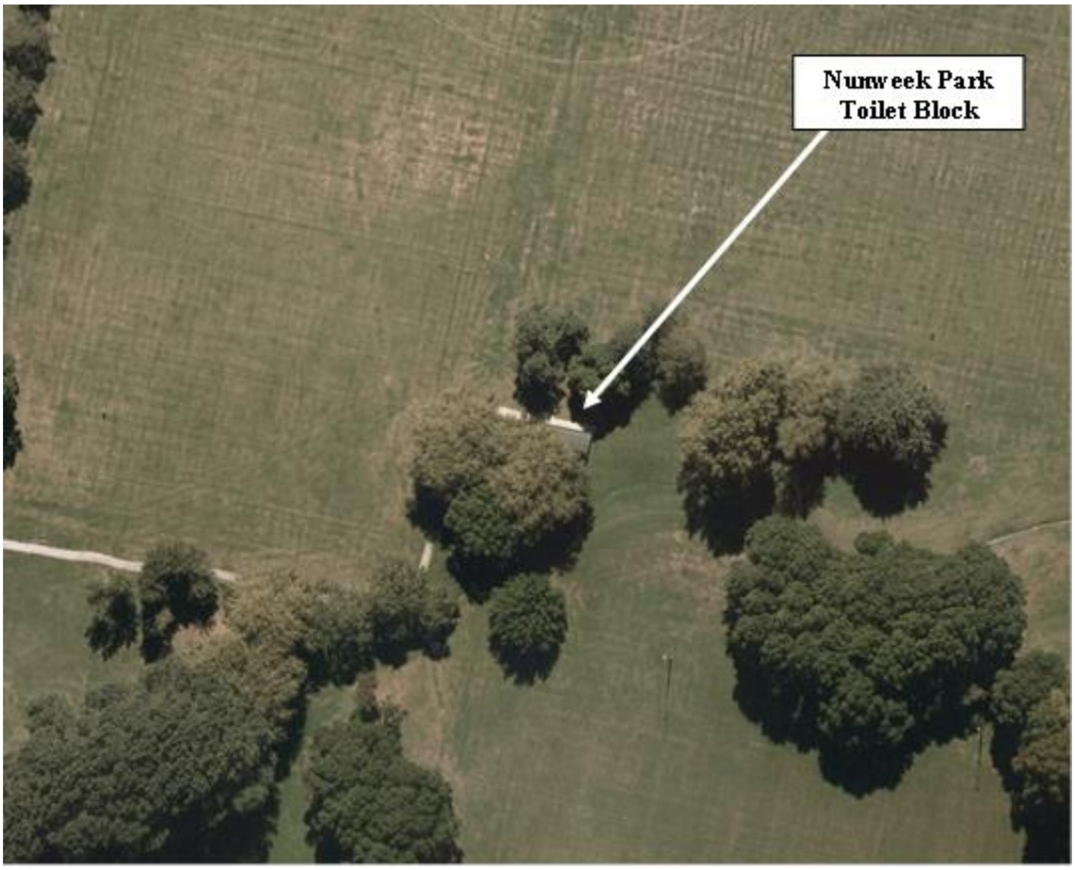

Figure 3 Post February 2011 Earthquake Aerial Photography [3]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise multiple strata of gravel, sandy gravel with varying amounts of silt and clay

[2] CERA Land check, http://cera.govt.nz/residential-green-zone-technical-categories

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

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 3.

Table 3 Summary of Known Active Faults[4][5]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale (2010) Fault	18 km	SW	7.1	~15,000 years
Hope Fault	100 km	NW	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	~150 years
Porters Pass Fault	55 km	NW	7.0	~1100 years

Recent earthquakes since 4 September 2010 have identified the presence of a previously unmapped active fault system underneath the Canterbury Plains, including Christchurch City, and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

6.3.2 Ground Shaking Hazard

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

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

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Harewood, a flat suburb in northwest Christchurch, 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.

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

[5] GNS Active Faults Database

6.5 Liquefaction Potential

The site is considered of minor susceptibility to liquefaction for the following reasons:

- Anticipation of gravel-dominated subsoils from shallow depths;
- Low susceptibility indicated by Brown & Weeber[1].
- No observed evidence of liquefaction during the site walkover; and,
- The site's technical category zoning at the boundary of TC1 and TC2.

6.6 Conclusions & Recommendations

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

The site appears to be situated on stratified alluvial deposits, comprising gravel and sand. Associated also with this the site is a low liquefaction potential, in particular where sand and/or silt is present.

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

Further geotechnical investigation is not considered necessary for this site.

7. Assessment

A visual inspection of the building was undertaken on the 19th of July 2012, a further inspection of the building was carried out on the 8th of September 2012. Both the interior and exterior of the building were inspected. No placard was evident during the inspection, however based on the inspection carried out it would be expected to have a green placard.

The inspection consisted of observing 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. No drawings were made available for the structure.

7.1 Quantitative Assessment

This 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. This was done by calculating the torsional effects on the building due to eccentricities developed between the buildings centre of mass and centre of rigidity. As the walls are not fully connected to the roof, they are treated as cantilever walls due to insufficient support at the top of the wall. From this the shear and moment demand for the walls was determined. The Hilti PS 200 Ferroscan was used to determine the level of reinforcement present in the walls, 12mm bars at 600mm vertical centres were detected. 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);

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

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

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

Where μ, 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 singly reinforced, filled concrete blocks.

The seismic weight coefficient was then calculated in accordance with Cl 5.2.1.1 of NZS 1170.5: 2011. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4s was taken for the structure. 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 Capacity of Reinforced Masonry Walls

7.3.1 Shear capacity of the Reinforced Walls

The shear capacity of the reinforced filled masonry wall was determined using NZS 4230: 2004. As there are no details as to the level of supervision during the construction stage, the Observation Type was classed in accordance with Table 3.1.

7.3.2 Moment capacity of the Reinforced Walls

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

7.4 Capacity of the Steel Members

The strength reduction factor Φ for steel elements was taken from Table 3.3 (1) of NZS 3404:1997. The capacities of the steel sections were determined using the following procedure.

7.4.1 Check the capacity of the steel posts

- Determine if the element needs be checked for combined action (bending and axial force), Cl 8.1.4 (a) (ii)
- Section bending capacity of the steel member;
- Shear capacity of the steel column, Cl 5.11.4.1;

$$\phi\, V_v = 0.6 f_y\, A_w \phi$$

8. Initial Capacity Assessment

8.1 Seismic Parameters

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

- Site soil class assumed to be: D, NZS 1170.5:2004, Clause 3.1.3, Soft Soil;
- Site hazard factor, Z = 0.3, NZBC, Clause B1 Structure, Amendment 11 effective from 1 August 2011;
- Return period factor R_u = 1.0, NZS 1170.5:2004, Table 3.5, Importance Level 2 structure with a 50 year design life.

8.2 Wall Investigation

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

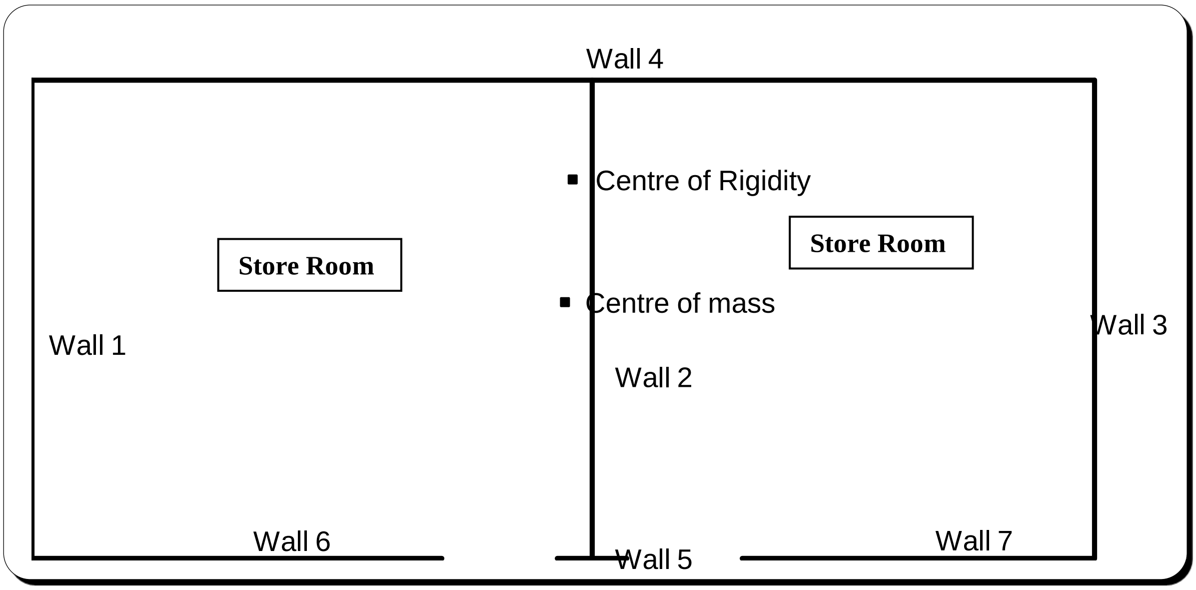

Figure 4 Pavilion wall layout

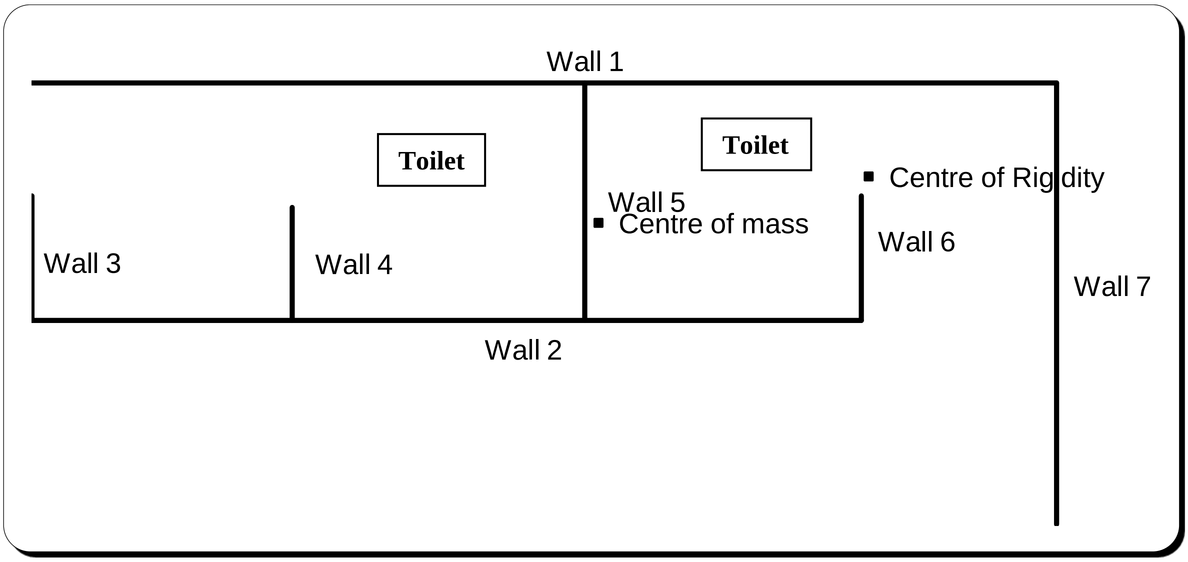

Figure 5 Toilet wall layout

8.3 Analysis Results

The results of the in plane analysis and subsequent earthquake designation under the NZSEE guidelines are listed below in Table 4 and Table 5.

Wall number	Minimum %NBS	Earthquake status
1	48%	Risk
2	49%	Risk
3	50%	Risk
4	54%	Risk
5	52%	Risk
6	50%	Risk
7	49%	Risk

Table 5

Wall number	Minimum %NBS	Earthquake status
1	49%	Risk
2	49%	Risk
3	49%	Risk
4	48%	Risk
5	46%	Risk
6	47%	Risk
7	53%	Risk

Table 4 %NBS Results of the Pavilion block

Wall number	Minimum %NBS	Earthquake status
1	48%	Risk
2	49%	Risk
3	50%	Risk
4	54%	Risk
5	52%	Risk
6	50%	Risk
7	49%	Risk

Table 5 Out Of Plane Analysis Results

8.4 Discussion of Results

Following a detailed assessment, the pavilion has been assessed as achieving 46 **%**NBS. All the block walls have been assessed to be less than 67% NBS. Under the New Zealand Society for Earthquake Engineering (NZSEE) the building is considered to be an Earthquake Risk building. No critical structural weaknesses or collapse hazards have been identified in the building and as such the building can remain occupied as per the Christchurch City Councils policy regarding Earthquake Prone buildings.

9. Recommendations

As the structure has been assessed to have a %NBS greater than 33% NBS but less than 67% NBS, it is deemed to be an Earthquake Risk building. All the block walls have been assessed to be less than 67% NBS. It is recommended that wall strengthening options be explored and implemented to bring the %NBS of the building up to a minimum of 67% NBS in accordance with 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.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.
- No calculations, other than those detailed in Section 5 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 View of store rooms from the east.

Photograph 2 View of the toilet from the west.

Photograph 3 North toilet elevation.

Photograph 4 Roof wall bolt connection

Photograph 5 Crack in mortar joint.

Photograph 6 Roof truss with rigid purlin connections.

Appendix B

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Nunweek Park Toilets/Pavilion/Changing Rooms	Reviewer:	David Lee
	Unit / No: / Street	CPEng No:	112052
Building Address:	240 Wooldridge Road	Company:	GHD
Legal Description:	Lot 2 DP 22949 5.6656	Company project number:	513090214
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:	43 / 29 / 14.00	Date of submission:	15/08/2013
GPS east:	172 / 34 / 5.00	Inspection Date:	8/9/2012
		Revision:	Version 2 Final
Building Unique Identifier (CCC):	PRK_0305_BLDG_002 & PRK_0305_BLDG_003	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	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):	8.00

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	8.00
Ground floor split?	no	Ground floor elevation above ground (m):	8.00
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	3.35	height from ground to level of uppermost seismic mass (for IEP only) (m):	3.35
Floor footprint area (approx):	70		
Age of Building (years):	25	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 framed	rafter type, purlin type and cladding	Steel tube with metal cladding
Floors:	other (note)	describe sytem	Ground slab on grade
Beams:			
Columns:	structural steel	typical dimensions (mm x mm)	
Walls:	partially filled concrete masonry	thickness (mm)	140

Lateral load resisting structure

Lateral system along:	partially filled CMU	**Note: Define along and across in detailed report!**	
Ductility assumed, μ:	1.25	note total length of wall at ground (m):	
Period along:	0.40 #### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):		estimate or calculation?	
maximum interstorey deflection (ULS) (mm):		estimate or calculation?	
Lateral system across:	partially filled CMU		
Ductility assumed, μ:	1.25	note total length of wall at ground (m):	
Period across:	0.40 #### enter height above at H31	estimate or calculation?	
Total deflection (ULS) (mm):		estimate or calculation?	
maximum interstorey deflection (ULS) (mm):		estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Metal	describe	
Glazing:			
Ceilings:			
Services(list):			

Available documentation

Architectural		original designer name/date	
Structural		original designer name/date	
Mechanical		original designer name/date	
Electrical		original designer name/date	
Geotech report		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:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
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?:	yes	Describe:	Minor cracking in some masory joints

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

Recommendations

	Level of repair/strengthening required:	none	Describe:	
	Building Consent required:	no	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before e'quakes:	46% #### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Calculations detailed in report
	Assessed %NBS after e'quakes:	46%		
Across	Assessed %NBS before e'quakes:	46% #### %NBS from IEP below		
	Assessed %NBS after e'quakes:	46%		

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: 3.35m

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: 1.00

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

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3: 0.30
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** 3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**: 1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or $=k_\mu$, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D:**	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
S_p:		
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: 1.0
Height Difference effect D2, from Table to right: 1.0

Therefore, Factor D: 1

3.5. Site Characteristics insignificant | 1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

		Along	Across
3.6. Other factors, Factor F	For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	0.7	0.7
	Rationale for choice of F factor, if not 1	wall edges unrestrained	wall edges unrestrained

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)		**0.70**	**0.70**
4.3 PAR x (%NBS)b:	PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)			**#DIV/0!**

Official Use only:

Accepted By:

Date:

GHD

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		Name	Signature	Name	Signature	Date
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