CLIENTS | PEOPLE | PERFORMANCE

Malvern Park Toilets & Rugby Pavilion
PRK 0614 BLDG 003 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

180 Innes Road

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

Malvern Park Toilets & Rugby Pavilion
PRK 0614 BLDG 003 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

180 Innes Road

Christchurch City Council

Prepared By

Paul Clarke

Reviewed By

Stephen Lee

Date

9 April 2013

Contents

Table Index

Figure Index

Appendices

Quantitative Report Summary

Malvern Park Toilets & Rugby Pavilion

PRK 0614 BLDG 003 EQ2

Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

180 Innes Road, St Albans

Background

This is a summary of the Quantitative report for the building structure, and is based in general on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 12 December 2012 and available drawings itemised in 5.2.

Building Description

The single storey building is located at 180 Innes Road, St Albans. The overall structure consists of different sections of the building dating from approximately 1982 until 2009. The building's walls are formed from concrete masonry units which support either timber truss, flat timber, precast concrete slab or vaulted timber roof constructions. The building's uses are a public toilet, and meeting, changing and store rooms.

Key Damage Observed

The main damage observed was limited to the area where the sections of the building constructed at differing dates meet. A relatively large gap was formed between the original structure and the walls and floor of a newer section, most likely when the newer section moved away from the original structure. The original structure also sustained some cracking to the concrete masonry mortar joints however the crack and direction of masonry movement implies a minor pounding effect during this overall movement. Hairline cracks were also observed where precast concrete roof slabs rest on bedding mortar. Cracking in both cases were localised and as such are not expected to affect overall structural performance of the building.

Additional hairline cracks were noted where pre-existing doorways were blocked in to form windows. These cracks are not expected to be earthquake damage.

Building Capacity Assessment

Based on the results of the quantitative assessment the building scored 20% NBS. Therefore the building is Earthquake Prone.

Recommendations

The building has achieved a New Building Standard of 20%NBS, therefore strengthening schemes should be prepared to improve the seismic performance of the building to at least 67% NBS.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Toilets Marshland Reserve.

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

2. Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

2.2 Building Act

Several sections of the Building Act are relevant when considering structural requirements:

Section 112 – Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to any alteration. This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67% NBS however where practical achieving 100% NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67% NBS.

2.2.1 Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that other property could collapse or otherwise cause injury or death; or
- A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property. A moderate earthquake is defined by the building regulations as one that would generate ground shaking 33% of the shaking used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake of the 4th September 2010.

The 2010 amendment includes the following:

- A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
- A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
- A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
- Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

We anticipate that any building with a capacity of less than 33% NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% NBS of new building standard as recommended by the Policy.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code.
- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The building code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February Earthquake, on 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing.

3. Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement	Unacceptable	Unacceptable

Figure 1 NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE

Table 1 compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

Figure 2 %NBS compared to relative risk of failure

4. Building Description

4.1 General

The building is located at 180 Innes Road, St Albans. The overall structure consists of sections of the building constructed at different dates, identified by differing roof constructions. The original changing rooms(pitched timber frame roof), along with the sections where the roof consists of vaulted timber rafters or precast concrete slab, is estimated to have been constructed in 1982 based on conversations with staff. The existence of a joint separating the original changing rooms from the other aforementioned sections indicates that this is unlikely, however it is likely that all sections were constructed around the same period, with the changing rooms having been constructed first. These other aforementioned sections contain private toilets, a meeting room and some store rooms. The recent extension, circa '09, consisted of an additional changing room and facilities, public toilets and a store room and are distinguished from the original structure by a flat timber frame roof.

Four types of roof construction exist; timber frame consisting of corrugated sheets on timber rafters with plasterboard on ceiling joists below, precast concrete slabs with battens and plasterboard beneath, corrugated sheets on vaulted rafters with a ring beam at eaves level and a flat timber framed roof section with plasterboard lining beneath. 190mm concrete masonry forms the vast majority of walls, with the remainder being internal partitions formed with lightweight timber stud framing. The original structure consists of unfilled concrete masonry units and a ø12mm reinforced bond beam at the wall head. All other walls were found to have ø12mm reinforcement at 600mm centres horizontally and vertically. The 190mm thick by 200mm deep ring beam was found to consist of ø6mm reinforcement bars in each corner and ø6mm links at 150mm centres. Foundations are most likely concrete strip footings with the floor consisting of concrete slab on grade.

The building is approximately 26m in length by 8m in width with a maximum height of 4.5m. The building occupies an area of approximately 215m^2 and is located roughly 2m from the adjacent Scout Den. The flat site is 2km northeast of Avon River

No existing plans were made available.

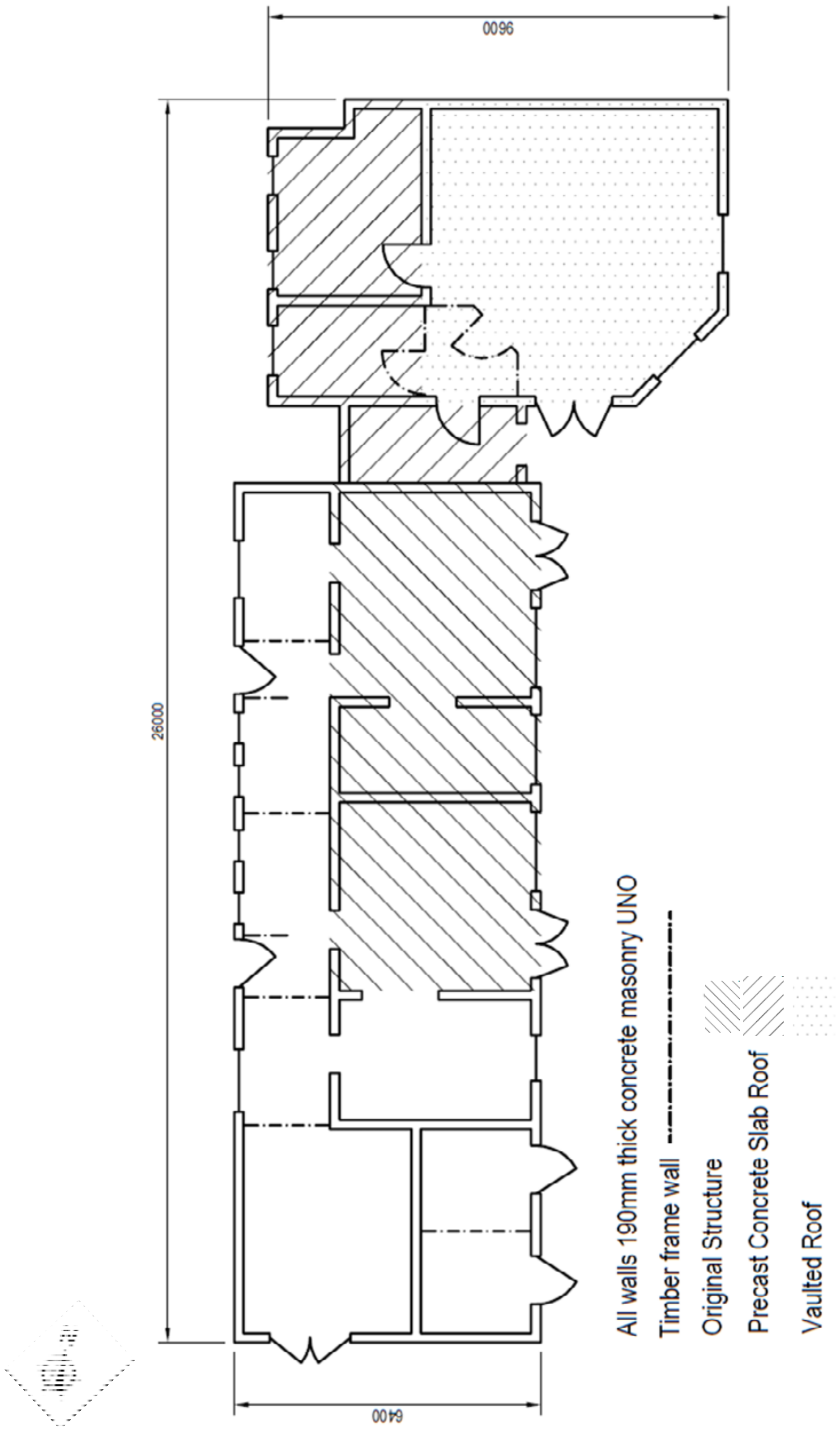


Figure 3 Plan of Structure

4.2 Gravity Load Resisting System

Gravity loads are resisted by load bearing concrete masonry walls. The precast concrete roof slabs span between walls to support self-weight and other gravity loads. The timber roof truss supports gravity loads and self-weight by the triangulation of forces through the timber members to the wall plate where the loads are then transferred to the wall beneath. Similar to the roof truss, the vaulted roof structure relies on the triangulation of forces to support self-weight and other gravity loads from the walls beneath. The concrete masonry walls transfer the gravity loads to the concrete strip footings and subsequently the ground beneath.

4.3 Lateral Load Resisting System

The lateral load resisting system differs with the varying construction type used through-out.

The original construction consists of unreinforced concrete masonry walls with a timber frame roof and a plasterboard lined ceiling. The plasterboard lining and timber frame roof will have sufficient diaphragm capacity to transfer lateral roof loads to walls in the plane of loading. The walls perpendicular to the seismic load will span vertically between the ground and the reinforced bond beam at the head of the wall, which will in-turn span horizontally between the orthogonal in-plane walls. These in-plane walls will resist the lateral loads from self-weight, the roof and the other aforementioned walls by the panel action of the concrete masonry to the foundations where they distribute into the ground beneath.

The precast concrete slab will form a roof diaphragm to transfer lateral roof loads to walls in the plane of loading. These in-plane walls will resist these lateral roof loads and lateral wall loads by the panel action of the concrete masonry to the foundation level where they will distribute into the ground beneath. The walls perpendicular to the loading will span vertically between the ground and the roof diaphragm, which again transfers the lateral loads to the in-plane wall.

The vaulted roof section is comprised of a reinforced concrete ring beam continuous around eaves level and inclined timber ‘beam’ members running from the ring beam corners up to the centrally located apex. Rafters in turn span between the inclined timber ‘beams’ and the ring beam. This construction effectively gives two inter-braced A-frames with the bases tied together by the ring beam. This ring beam and inclined beams form triangles which are inherently stable and transfers lateral roof loads to the ring beam by the triangulation of forces through members. The roof structure transfers the lateral roof loads to in-plane walls which resist them through the panel action of concrete masonry units and transfers these loads to the foundations where they are distributed into the ground beneath. The absence of a diaphragm to restrain the eaves of walls subject to perpendicular lateral loads results in these walls spanning horizontally between orthogonal return walls or cantilevering from the foundations. Wall panels of shorter lengths will span horizontally between orthogonal return walls, where-as the more efficient method for longer wall lengths will be cantilevering of the wall from the fixity provided at the foundations.

The recent addition's roof, predominately flat with a small pitched area, will have sufficient diaphragm capacity to transfer lateral roof loads to walls in the plane of loading. These in-plane walls will resist theses and lateral wall loads by the panel action of the concrete masonry to the foundations and the ground beneath. Walls subject to perpendicular lateral loads will again either span horizontally between orthogonal return walls or cantilever from the foundations.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 12th of December 2012. Both the interior and exterior of the building were inspected. The main structural components of the building were viewed where possible. The elements of the roof structure were inspected where elements were visible. It should be noted that inspection of the foundations of the structure was limited to the top of the external strips exposed above ground level.

The inspection consisted of observing the building to determine the structural systems and likely behaviours of the building during earthquake. The site was assessed for damage, including observing the ground condition, checking for damage areas where damage would be expected for the structure type observed and noting general damage observed throughout the building in both structural and non-structural elements.

A Hilti PS 200 Ferroscan was used to confirm the position, depth and diameter of the reinforcement in the fully-filled concrete masonry walls, the concrete ring beam and the original structure's bond beam. This scanning equipment using electro-magnetic fields allowed for the determination of the capacity of the various walls in the building. In the case of conflicting results, the most conservative bar diameter was chosen for the capacity calculations.

5.2 Available Drawings

No drawings were made available.

6. Damage Assessment

6.1 Surrounding Buildings

The adjacent Scout Den showed no signs of earthquake damage, though a perimeter fence prevented access for a full visual inspection.

6.2 Residual Displacements and General Observations

The main damage observed was limited to the area where the sections of the building constructed at differing dates meet. A relatively large gap was formed between the original structure and the walls and floor of a newer section, most likely when the newer section moved away from the original structure. The original structure also sustained some cracking to the concrete masonry mortar joints however the crack and direction of masonry movement implies a minor pounding effect during this overall movement. This unreinforced concrete masonry pier has not been considered for the overall %NBS score of the structure given the extent and nature of cracking (See Photograph 7) . The large diagonal crack continues through the entire width of the masonry pier reducing the in-plane shear capacity significantly. Hairline cracks were also observed where precast concrete roof slabs rest on bedding mortar. This cracking was localised and as such is not expected to affect overall structural performance of the building.

Additional hairline cracks were noted where pre-existing doorways were blocked in to form windows. These cracks are not expected to be earthquake damage.

6.3 Ground Damage

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

7. Structural Analysis

7.1 Seismic Parameters

Earthquake loads shall be calculated using New Zealand Code.

◗ Site Classification	D
◗ Seismic Zone factor (Z)	
(Table 3.3, NZS 1170.5:2004 and NZBC Clause B1 Structure)	0.30 (Christchurch)
◗ Annual Probability of Exceedance	
(Table 3.3, NZS 1170.0:2002)	1/500 (ULS) Importance Level 2
◗ Return Period Factor (Ru)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)
◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_{μ})	1.14
◗ Performance Factor (Sp), based on NZS 3.1.0.1	0.925
◗ Gravitational Constant (g)	9.81 m/s^2

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing recommendations resulting in a reduced % NBS score.

7.2 Equivalent Static Method

Equivalent Static forces were calculated in accordance with NZS 1170.5:2004. A ductility factor of 1.25 has been assumed given the age and partially filled construction used. The structure is expected to have nominally ductile behavior given the lightly reinforced partially filled concrete masonry construction.

The elastic site hazard spectrum for horizontal loading:

$C(T_1)=C_h \cdot Z \cdot R \cdot N(T,D)$

$C_h=3.0$ – Value from 3.1 table for the period (T=0.4s)

Z=0.3 – Hazard factor determined from the table 3.3 (NZS 1170.5:2004)

R=1.0 – Return period factor determined from the table 3.5 (NZS 1170.5:2004)

N (T,D) = 1.0 – Near fault factor- clause 3.1.6. (NZS 1170.5:2004)

$C(T_1)= 3.0 \cdot 0.3 \cdot 1.0 \cdot 1.0 = 0.9$

The horizontal design action coefficient:

$$C_d(T_1)=\frac{C(T_1) \cdot S_p}{k_{\mu}}=\frac{0.90 \cdot 0.925}{1.14}=0.73$$

The structure is relatively simple, with direct load paths and no opportunity for redistribution of loads through the structure. Thus elements were considered individually, and subject to loads from seismic self-weight or those directly applied.

7.3 Dependable Capacity

7.3.1 Reinforced Masonry-Shear Capacity

The shear capacity of the reinforced concrete masonry walls was calculated using Sections 10.3 of NZS 4230:2004, and 11.3 of NZS 3101:2006.

Shear capacity comprises two components; that from the masonry, and that from the steel reinforcement. These are calculated separately, and added together.

This first involved calculating the shear capacity of the masonry, V_m, based on the following equations:

For reinforced masonry;

$$V_m = 0.8 d b_w v_m$$

$$v_m = (C_1 + C_2) v_{bm}$$

$$C_2 = 33 p_w \frac{f_y}{300}$$

$$p_w = A_s / b_w d$$

Where

C_1 = wall proportion factor = 1.0;

v_m = shear strength of masonry;

b_w = thickness of wall being considered;

d = effective depth of wall reduced further by 0.8 when in-plane shear is being considered,

A_s = area of reinforcement.

The shear capacity component from the reinforcing steel, V_S, was calculated using equation below;

$$V_S = A_V\, f_{yt} \frac{d}{s}$$

Where

A_V = area of transverse (horizontal) reinforcing at spacing s;

f_{yt} = characteristic yield strength of the transverse steel;

d = depth from compression end of wall to centroid of tension force.

7.3.2 Reinforced Masonry-Out-of-Plane Moment Capacity

The following method was used to calculate the out of plane moment capacity of the reinforced masonry walls.

$$\emptyset M_n = \emptyset \left(\frac{t}{2} - \frac{a}{2} \right) f_{yt} A_s$$

$$a = {A_s f_{yt}}/{\emptyset A_m f'_m}$$

Where

t = wall thickness

A_s = area steel

A_m = area of masonry

f'_m = masonry strength

A similar method is used to calculate the moment capacity of the reinforced concrete ring beam.

7.3.3 Unreinforced Masonry Capacity

The performance of unreinforced concrete masonry was calculated in accordance with NZSEE guidelines as detailed in ‘Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance’ and ‘Assessment and Improvement of the Structural Performance of Buildings in Earthquake’. The performance of unreinforced masonry panels for in-plane shear and bending, and out-of-plane bending were calculated for comparison to seismic demand

8. Geotechnical Consideration

8.1 Site Description

The site is situated within a recreational reserve, within the suburb of St Albans in central Christchurch. It is relatively flat at approximately 8m above mean sea level. It is approximately 2km north east of the Avon River,and 8.5km west of the coast (Pegasus Bay) at New Brighton.

8.2 Published Information on Ground Conditions

8.2.1 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.

8.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that 18 boreholes are located within a 200m radius of the site. Of these boreholes, three contain an adequate lithographic log (see Table 1), which identifies soils comprising silt, clay and sand.

Table 1 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/16958	3.3m	2.2m bgl	0m N/A
M35/13484	2.59m	N/A	~130m SE
M35/13847	2.74m	N/A	~88m S

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.

8.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Saint Albans[2]. Four investigation points were undertaken within 200m of the property, as summarised below in Table 2.

[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.

[2] Tonkin and Taylor, September 2011: Christchurch Earthquake Recovery, Geotechnical Factual Report, Saint Albans

Table 2 EQC Geotechnical Investigation Summary Table

Bore Name	Grid Reference	Depth (m bgl)	Log Summary
CPT – STA 24	2479913.5 mE 5744942.5 mN	21.76	0 – 1.0m Silt 1.0 – 3.0m Clay and silt 3.0 – 8.0m Silt and sand 8.0 – 21.76m Sand and gravel (GWL at 4.1m bgl)
CPT – STA 25	2480209.0 mE 5744837.7 mN	22.08	1 – 2.9m Clay and silt 2.9 – 7.0m Silt and sand 7.0 - 22.08m Sand and gravel (GWL at 1.0m bgl)
CPT – STA 27	2480135.0 mE 5744611.1 mN	22.51	0 – 0.7m Silt 0.7 – 2.6m Clay and silt 2.6 – 8.0m Silt and sand 8.0 – 12.2m Sand and gravel 12.2 – 22.51m Sand (GWL at 1.3m bgl)
CPT – STA 28	2479992.8 mE 5744515.1 mN	22.09	0 – 1.0m Silt 1.0 – 2.7m Silt and clay 2.7 – 8.0m Silt and sand 8.0 – 22.09m Sand and gravel (GWL at 1.0m bgl)

Initial observations of the CPT results indicate the soils are predominantly layers of loose to medium dense sand and silt. This would infer that liquefaction is possible and likely in a significant seismic event.

8.2.4 Land Zoning

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

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

The site is within an area classified as the technical category term N/A, which means either the site is “Rural & Unmapped” or “Urban Non residential”. However, the properties adjacent to the site have been classified as TC2 (yellow) zone[3]. This means that means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

8.2.5 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows evidence of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 4**.**

Figure 4 Post February 2011 Earthquake Aerial Photography [4]

8.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise layers of clay, silt, sand and gravel.

[3] CERA Landcheck website, http://cera.govt.nz/my-property

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

8.3 Seismicity

8.3.1 Nearby Faults

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

Table 3 Summary of Known Active Faults[5][6]

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	140 km	NW	~8.3	~300 years
Greendale (2010) Fault	22 km	SW	7.1	~15,000 years
Hope Fault	110 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	~150 years
Porters Pass Fault	70 km	NW	7.0	~1100 years

Recent earthquakes since 22 February 2011 have identified the presence of a previously unmapped active fault system underneath 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.

8.3.2 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.

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.

8.4 Slope Failure and/or Rockfall Potential

Given the site's location in St Albans, a flat suburb in central 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.

[5] 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.

[6] GNS Active Faults Database

8.5 Liquefaction Potential

Due to the anticipated presence of loose sand and silt, and evidence from the post-earthquake aerial photography it is considered possible and likely that liquefaction will occur where sands and silts are present.

Further investigation is recommended to better determine subsoil conditions. From this, a more comprehensive liquefaction assessment could be undertaken.

8.6 Conclusions & Summary

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 loose alluvial deposits, comprising sand, silt, gravel and clay and is considered to have a high liquefaction potential, due to the sands and/or silts are present on site.

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

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

9. Results of Analysis

The structure can be divided into reinforced and un-reinforced concrete masonry sections. All additions to the original structure were found to be fully-filled concrete masonry with both vertical and horizontal reinforcement.

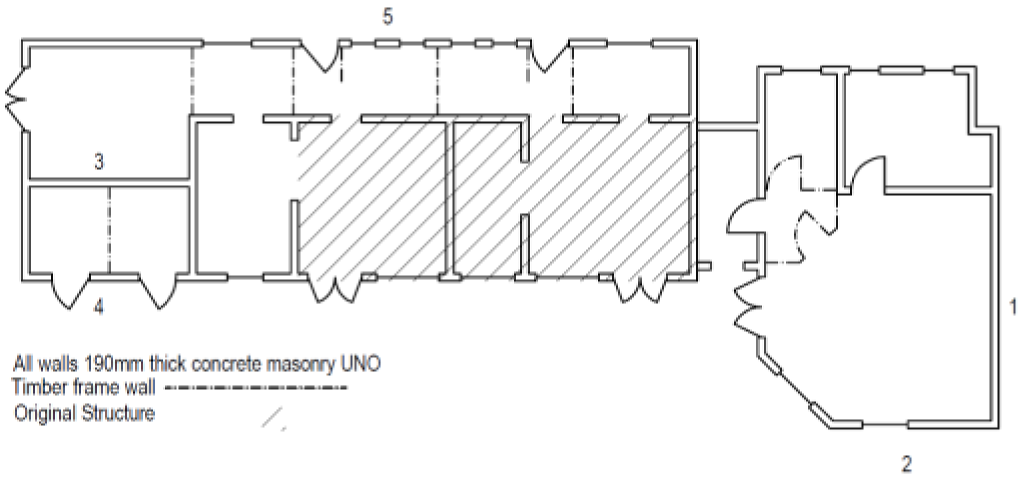


Figure 5 Plan Identifing Reinforced Concrete Masonry Walls

All reinforced concrete masonry walls were found to achieve >100% NBS for lateral seismic in-plane. Walls subject to perpendicular lateral loads were found to perform less satisfactorily.

The reinforced concrete masonry walls which achieved <100% NBS are identified in Figure 9.1 and the performance of those walls are quantified in Table 4. Walls found to achieve a <100% NBS did not have a reliable roof diaphragm to allow the panels to span vertically onto when out-of-plane seismic loads are applied. The absence of a roof diaphragm required the wall panels to span horizontally between orthogonal in-plane walls or to cantilever from the foundation level. It is assumed sufficient fixity exists from the reinforced concrete floor slab and the reinforced concrete strip foundation to support these cantilever demands.

Table 4 Out-of-Plane %NBS for Reinforced Concrete Masonry Walls

Wall	% NBS
1	49%
2	60%
3	79%
4	52%
All other Reinforced Walls	>100%

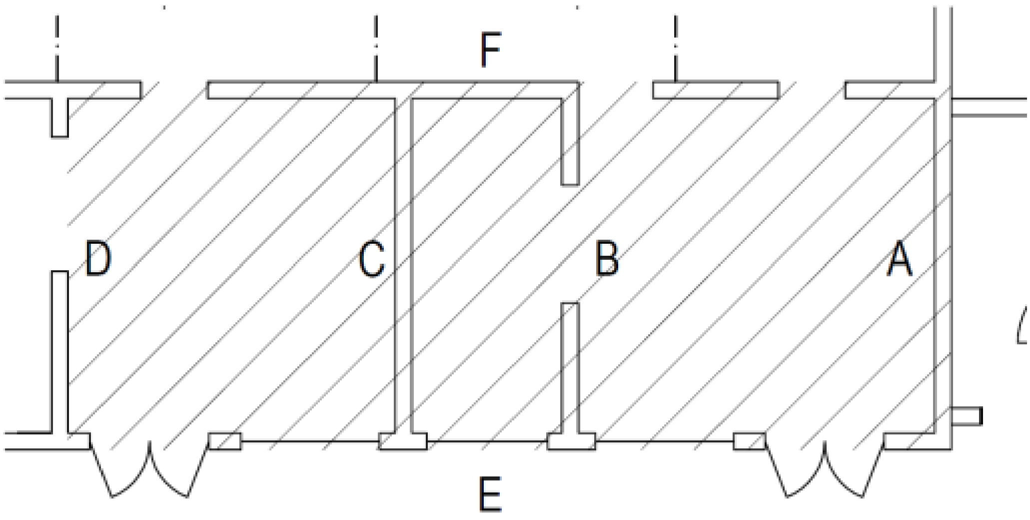


Figure 6 Plan Identifying Original Structure's Wall Lines.

All unreinforced concrete masonry walls were found to achieve 50% NBS for lateral seismic loads applied perpendicular to the walls. In all instances the out-of-plane loading was resisted by the wall panels spanning vertically from the ground to the reinforced concrete bond beam at the wall head. The bond beam was found to have sufficient capacity for the loads applied.

The original building was segmented into wall lines as shown in Figure 6 with the %NBS for in-plane loading quantified in Table 5. It can be seen that wall lines with higher demands and a large amount of openings i.e. shorter masonry pier lengths, were the poorest performers. It was found that the controlling value for the structure was 20% NBS from Wall Line B & F.

Table 5 In-Plane %NBS of Original Structure Wall Lines

Wall	% NBS
A	>100%
B	20%
C	70%
D	52%
E	20%
F	83%

9.1 Discussion of Results

The results obtained from the analysis are generally consistent with those expected for a building of this size, age and varying construction types, founded on Class D soils.

As expected the more modern sections constructed with reinforced concrete masonry performed best, with those sections containing reliable roof diaphragms scoring >100% NBS. The original Malvern Park Toilets and Rugby Pavilion structure was designed in 1982 approximately and was likely designed in accordance with the loading standard current at the time, NZS 4203:1976. The design loads used are likely to have been less than those required by the current loading standard. In addition the unreinforced concrete masonry used for the construction of the original structure has been recognised as not performing well seismically. This recognition has been substantiated with the structure achieving a New Building Standard in the order of 20% NBS.

Two wall lines in the original structure were found to achieve a New Building Standard in the order of 20% for in-plane seismic loads. These wall lines were found to have proportionally higher seismic demands compared to the length of unreinforced concrete masonry piers.

10. Conclusions and Recommendations

The building overall has been assessed as having a seismic capacity of 20% NBS and is therefore classified as being ‘Earthquake Prone’.

The building has achieved a New Building Standard of 20%NBS, therefore strengthening schemes should be prepared to improve the seismic performance of the building to at least 67% NBS.

11. Limitations

11.1 General

This report has been prepared subject to the following limitations:

- Available drawings itemised in 5.2 was used in the assessment.
- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- No level or verticality surveys have been undertaken.
- No material testing has been undertaken.

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

11.2 Geotechnical Limitations

The data and advice provided herein relate only to the project and structures described herein and must be reviewed by a competent geotechnical professional before being used for any other purpose. GHD Limited (GHD) accepts no responsibility for other use of the data by third parties.

Where drill hole or test pit logs, cone tests, laboratory tests, geophysical tests and similar work have been performed and recorded by others under a separate commission, the data is included and used in the form provided by others. The responsibility for the accuracy of such data remains with the issuing authority, not with GHD.

The advice tendered in this report is based on information obtained from the desk study investigation location test points and sample points. It is not warranted in respect to the conditions that may be encountered across the site other than at these locations. It is emphasised that the actual characteristics of the subsurface materials may vary significantly between adjacent test points, sample intervals and at locations other than where observations, explorations and investigations have been made. Subsurface conditions, including groundwater levels and contaminant concentrations can change in a limited time. This should be borne in mind when assessing the data.

It should be noted that because of the inherent uncertainties in subsurface evaluations, changed or unanticipated subsurface conditions may occur that could affect total project cost and/or execution. GHD does not accept responsibility for the consequences of significant variances in the conditions and the requirements for execution of the work.

The subsurface and surface earthworks, excavations and foundations should be examined by a suitably qualified and experienced Engineer who shall judge whether the revealed conditions accord with both the assumptions in this report and/or the design of the works. If they do not accord, the Engineer shall modify advice in this report and/or design of the works to accord with the circumstances that are revealed.

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 building from the Northeast.

Photograph 2 View of the main Southwest wall.

Photograph 3 Gap between two sections of structure, cracking to mortar top left.

Photograph 4 Gap between building sections in back right corner.

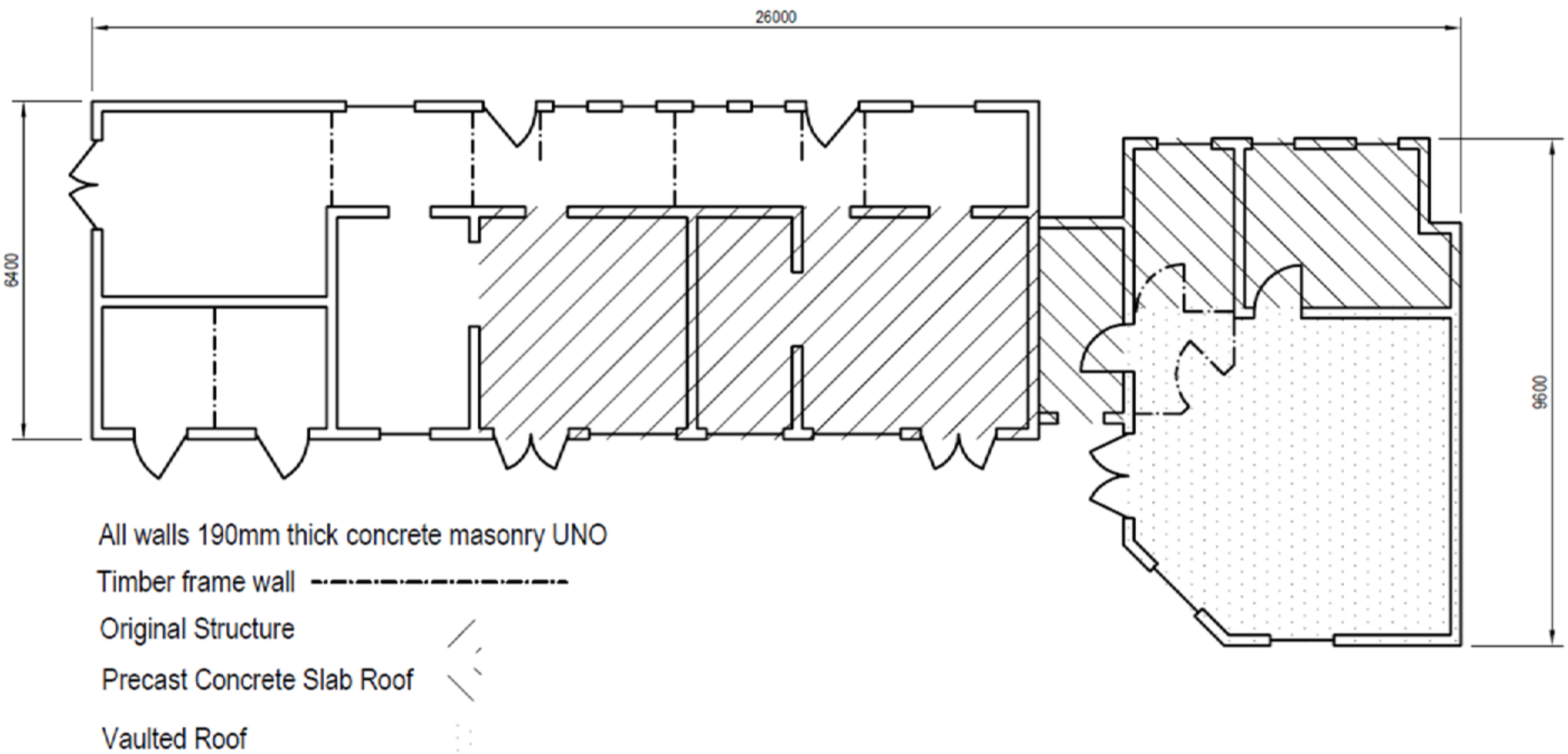
Photograph 5 Crack along bedding mortar of precast concrete roof slabs.

Photograph 6 Internal view of vaulted roof.

Photograph 7 Masonry pier containing diagonal crack.

Appendix B

Sketch


26000
6400
9600
All walls 190mm thick concrete masonry UNO
Timber frame wall
Original Structure
Precast Concrete Slab Roof
Vaulted Roof

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Malver Park Toilets & Rugby Pavillion	Reviewer:	Stephen Lee
	Unit / No: / Street	CPEng No:	1006840
Building Address:	180 Innes Road	Company:	GHD
Legal Description:		Company project number:	513090207
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:		Date of submission:	22-Jan-13
GPS east:		Inspection Date:	12-Dec-12
		Revision:	
Building Unique Identifier (CCC):	PRK_0614_BLDG_003 EQ2	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):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	no		Ground floor elevation above ground (m):	
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	4.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	4.5
Floor footprint area (approx):	215			
Age of Building (years):	30		Date of design:	1976-1992
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):	Public Toilet & Sports Rooms			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	other (note)	describe system	Vaulted rafters, precast slab & timber framing
Floors:	other (note)	describe sytem	Slab on grade
Beams:			
Columns:			
Walls:	load bearing concrete	#N/A	190

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?	estimated
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?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Metal	describe	Corrugated sheets
Glazing:			
Ceilings:	plaster, fixed		Plasterboard
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):	0		
Across	Damage ratio:	0%		
	Describe (summary):	0		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	

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

Recommendations

	Level of repair/strengthening required:		Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	20%	#### %NBS from IEP below	If IEP not used, please detail assessment methodology: detailed calculations
	Assessed %NBS after e'quakes:	20%		
Across	Assessed %NBS before e'quakes:	20%	#### %NBS from IEP below	
	Assessed %NBS after e'quakes:	20%		

IEP **Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.**

Period of design of building (from above): 1976-1992

h_n from above: 4.5m

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:	0.0%	0.0%

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	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

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

2.2 Near Fault Scaling Factor

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

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

2.3 Hazard Scaling Factor

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

Z_{1992}, from NZS4203:1992

Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above): 2

Return Period Scaling factor from Table 3.1, **Factor C**:

2.5 Ductility Scaling Factor

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

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.925	0.925
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.081081081

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ **$\%NBS_b$:** **#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	significant	0.7

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

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

3.6. Other factors, Factor F

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

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

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

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!**

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

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
Final	Paul Clarke	Stephen Lee		Nick Waddington		9/4/2013