Rawhiti Domain Tennis Club
PRK 2004 BLDG 020

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

35-37 Bowhill Road, New Brighton

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

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35-37 Bowhill Road, New Brighton

Christchurch City Council

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28th May 2013

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

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Background

This is a summary of the Quantitative report for the Rawhiti Domain Tennis Club building located at 35-37 Bowhill Road, New Brighton, 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 the 8th of May 2013, and seismic capacity calculations.

Key Damage Observed

Key damage observed includes:

- Minor cracking to external reinforced blockwork masonry walls; and
- Minor cracking to concrete floor under the veranda.

Critical Structural Weaknesses

No critical structural weaknesses have been identified when assessing the structure.

Indicative Building Strength (from IEP and CSW assessment)

Based on the information available, and using seismic capacity calculations, the original capacity of the building has been assessed to be in the order of 47% NBS and post-earthquake capacity also in the order of 47% NBS. As no critical structural weaknesses have been identified when assessing the building, the building's post-earthquake capacity is also in the order of 47% NBS.

The building has been assessed to have a seismic capacity in the order of 47% NBS and is therefore considered an Earthquake Risk building (Medium Risk).

Recommendations

It is recommended that:

- The current placard status of the building of 'Green' remain; and
- The building has not been assessed as being Earthquake Prone, and no critical structural weaknesses have been identified. As a result, general access to the single-storey reinforced blockwork masonry structure can continue.

1. Background

GHD has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the Rawhiti Domain Tennis Club.

This report is a Quantitative Assessment and is based in general on NZS 1170.5: 2004, NZS 4230:2004, the New Zealand Society for Earthquake Engineering (NZSEE) guidelines for the Assessment and Improvement of Unreinforced Concrete Masonry Buildings for Earthquake Resistance (02/2011) and the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (06/2006).

This quantitative assessment to the building comprises of an investigation of the in-plane and out-of-plane strengths of the unreinforced 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 the existing structural elements to resist the seismic forces applied to them. 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 block masonry walls.

At the time of this report, no finite element modelling of the building structure has been carried out.

2. Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

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

- The importance level and occupancy of the building;
- The placard status and amount of damage;
- The age and structural type of the building;
- Consideration of any critical structural weaknesses; and
- 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	Improvement of Structural Performance – 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).

Table 1: %NBS compared to relative risk of failure

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

4. Building Description

4.1 General

The building is a single-storey rectangular structure, located within Rawhiti Domain at 35-37 Bowhill Road, New Brighton. The date of construction is unknown; however it appears consistent with construction techniques from the late 1970's to early 1980's. The building is currently used for the New Brighton Lawn Tennis Club, including toilets, a kitchen and lunch and/or seminar space. No alterations to the original structure were obvious during the site inspection.

The building measures approximately 15.8 m long by 4.5 m wide by 4 m high. It is rectangular in plan, with a gross floor area of approximately 71 m^2. The veranda along the northwest side extends 1.7 m out from the building, as shown in Figure 2.

The nearby structures are all small single-storey park buildings, typically toilet blocks; the closest two being approximately 70 m north and 120 m east. The building is approximately 550 m northeast of the lower Avon River and 650 m west of the coastline.

No plans or drawings for the building were made available. This assessment is based on observations, measurements and reinforcing scans from the site inspection.

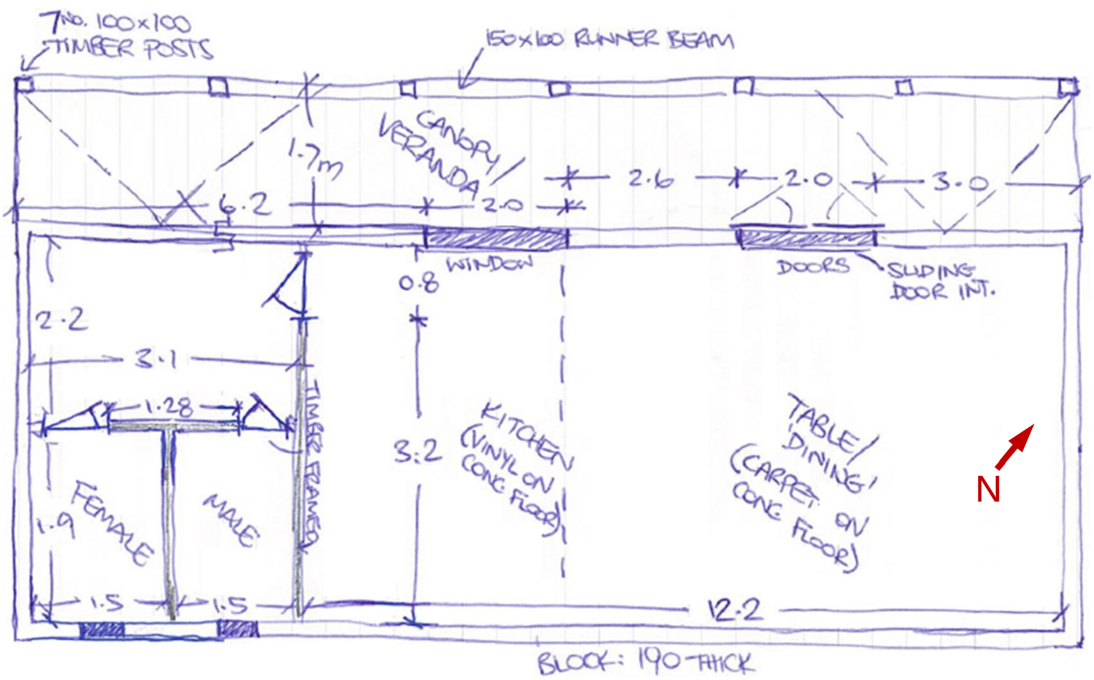

Figure 2: Plan Sketch Showing Key Structural Elements

The roof comprises roof trusses formed from 90x45 and 45x45 timber members spanning across the building at approximately 900 mm centres, clad with lightweight longrun iron roofing on 45x70 timber purlins at 700 mm centres typical. The ceiling is approximately 2.6 m high internally, and is attached to the roof trusses by 35x70 timber ceiling battens at 400 mm centres typical.

The external walls are 190 mm thick blockwork masonry, which form the perimeter and lateral load-resisting system of the building. There are openings in the northwest side wall for the doors and a window, each 2.0 m wide. Two openings are present in the southeast wall, each 400 mm wide by 800 mm high. Reinforcing scans indicate the walls contain 12 mm vertical steel bars at 600 mm centres. Scans showed starter bars to be the same as the steel in the walls. The walls have been assumed to be fully grouted.

The floor of the building is a concrete slab on grade, approximately 0.4 m above surrounding ground level. Reinforcing scans did not detect any steel in the floor slab. Perimeter foundations are inferred to be present underneath the blockwork perimeter walls; the thickness of these could not be confirmed. Reinforcing was detected in the perimeter foundation; this was deduced to be D12's, top and bottom.

The veranda structure is timber, with 45x70 purlins on 90x45 rafters at 900 mm centres typical. The rafters span between the blockwork wall and a 150x100 timber runner beam, supported by seven 100x100 timber posts. The posts are connected at the base using a single steel pin each.

4.2 Gravity Load Resisting System

Gravity loads are resisted by the reinforced blockwork walls around the building perimeter. The walls are supported on the reinforced concrete perimeter foundation.

Loads from the roofing are transferred through the timber roof trusses spanning across the building into the external reinforced blockwork walls. Loads are then transferred into the foundations and into the ground.

Internal loads acting on the concrete floor slab are transferred by the slab directly into the ground.

Foundations for the structure comprise a concrete slab on grade, with a perimeter thickening under the blockwork masonry walls. No reinforcement was detected in the floor slab. It is not known whether hardfill is present beneath the slab; however, drawings for other similar structures at the site indicate the presence of hardfill.

4.3 Lateral Load Resisting System

The lateral load resisting system for both the along and across directions is primarily formed by the reinforced blockwork masonry walls. These walls form a stiff perimeter, with a lightweight roof above. Internal walls have been assumed not to resist any of the lateral loads during an earthquake.

Trusses span across the building, which transfer lateral loads from the roof acting along the building to the side walls. These walls, acting as in-plane shear walls, carry the loads to the foundations and into the ground. As the ceiling spans only 4.1 m across the building, the plasterboard ceiling lining, along with the timber roof trusses, is assumed to act as a ceiling diaphragm. Out-of-plane action on the end walls is assumed to be simply supported between the two side walls, which resist these loads as shear walls.

Roof loads acting across the building are transferred through the trusses into the top of the side walls. As the ceiling spans over 15 m in this direction, the ceiling has been assumed to not act as a diaphragm. Hence, lateral loads from the roof act at the top of the reinforced blockwork walls. These walls act out-of-plane in cantilever resisting the loads by flexure, transferring them to the foundations, through the starter bars at the connection between the walls and footings.

5. Damage Assessment

An inspection of the building was undertaken on 8 May 2013. Both the interior and exterior of the building were inspected. The main structural components of the roof of the building were able to be viewed through the roof space access panel. Surveying of the roof space from the access panel was limited due to the location of the panel. Similarly the foundations were unable to be viewed due to inaccessibility.

The inspection 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.

No level or verticality surveys have been undertaken for this building at this stage as indicated by Christchurch City Council guidelines.

A Hilti PS 200 Ferroscan was used to determine the position, depth and diameter of the reinforcement in the blockwork masonry structure. This scanning equipment uses electro-magnetic fields to determine the size and depth of the reinforcing steel in the building. In the case of conflicting results, the most conservative bar diameter has been chosen for the capacity calculations.

5.1 Surrounding Buildings

Surrounding structures are all related to the park and associated facilities. No obvious damage to these structures was noted during our inspection.

5.2 Residual Displacements and General Observations

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

Cracking was noted to the external blockwork walls in several locations around the building; see Photographs 5 and 6. These cracks are not considered to be significant.

Cracking was also observed in the concrete pad under the veranda canopy; see Photograph 8. These cracks are not considered to be significant.

5.3 Ground Damage

There was no evidence of ground damage within 100 m of the building.

6. Critical Structural Weakness

Short Columns

No short columns are present in the structure.

Lift Shaft

The building does not contain a lift shaft.

Roof

Roof bracing was not seen from the access point. Roof elements such as timber sheathing, purlins and trusses were clearly visible and are expected to provide bracing to the roof structure (see Photographs 9 and 10).

Diagonal steel strip bracing was visible underneath the veranda canopy.

Staircases

The building does not contain a staircase.

Site Characteristics

Lateral spreading is not considered a significant hazard for this site.

Following the geotechnical appraisal it was found that the site has a minor potential for liquefaction. For the purposes of this assessment, the effects of soil liquefaction on the performance of the building have been assessed as 'insignificant' in accordance with the NZSEE guidelines.

Plan Irregularity

The building is rectangular; no plan irregularity is present.

Vertical irregularity

The building is single-storey, with a constant ceiling height; no vertical irregularity is present.

Pounding effect

No adjacent buildings; pounding is not applicable.

Height difference offset

Not applicable.

7. Geotechnical Consideration

This desktop geotechnical study outlines the ground conditions, as indicated from sources quoted within, for inclusion in the subject structure's DEE Quantitative Assessment. This desktop study report includes observations from a site walkover undertaken by GHD Geotechnical personnel.

7.1 Site Description

The site is situated in the suburb of New Brighton, in eastern Christchurch. It is relatively flat at approximately 3m above mean sea level. It is approximately 550 m northeast of the Avon River, and 650 m west of the coast.

7.2 Published Information on Ground Conditions

7.2.1 Published Geology

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

- Dominantly sand of fixed and semi-fixed dunes and beaches, being marine soils of the Christchurch Formation, Holocene in age;
- Underlying sediments younger than 6,500 years are surface marine sands; those 10,000 to 6,500 years old are peat, or sand silt and clay;
- The Riccarton gravels are located approximately 40m bgl; and
- Groundwater is approximately 1m below ground level.

7.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that two boreholes with lithographic logs deeper than 3 m are located within 200m of the site; these are summarised in Table 2.

The borehole logs indicate the area is underlain by sand, with gravel at 39 m depth. One of the logs showed presence of sandy gravel at 15 m, underlain by peat at 30 m.

Table 2: ECan Borehole Summary

Bore Name	Log Depth	Groundwater*	From Site	Log Summary	
M35 / 2005	87 m	0.9 m	190 m SE	0 to 39 m	Sand
				39 m	Riccarton Gravel
M35 / 2388	85 m	0.5 m	150 m N	0 to 15 m	Sand and Clay
				15 to 30 m	Sand and Gravel
				30 to 39 m	Clay and Peat
				39 m	Riccarton Gravel

[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.

* Groundwater levels given are the calculated minimum groundwater levels from the borehole logs. The initial recorded levels are not considered accurate, given the date of drilling, and the site's elevation and proximity to the sea.

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

7.2.3 CGD Geotechnical Investigations

The nearest geotechnical investigation results available on the Canterbury Geotechnical Database[2] are more than 300 m from the site. As these are historical and only extend to a depth 3 m, they have not been considered in this assessment.

Median groundwater levels at the site are indicated to be ~2 m depth (GNS Science model)[3].

7.2.4 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. These categories describe how the land in expected to perform in future earthquakes.

The site has been categorised as "N/A" – Urban Non-residential" [4]. However, neighbouring residential properties have been categorised as TC2 (yellow), indicating minor to moderate land damage from liquefaction is possible in future significant earthquakes.

It is reasonable to assume the land at Rawhiti Domain has a similar ground performance to TC2-zoned land.

7.2.5 Post-Earthquake Land Observations

Aerial photography[5] taken following the 22 February 2011 earthquake shows no clear evidence of liquefaction near the building or on the site, as shown in Figure 3. All other aerial photography following the major earthquakes of the Canterbury earthquake sequence shows no clear evidence of liquefaction.

The Canterbury Geotechnical Database shows the nearest cracks to be at least 400 m west of the site[6]. All mapped cracks on the CGD are within land zoned TC3.

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[2] Canterbury Geotechnical Database (2013): "Geotechnical Investigation Data", Map Layer CGD0010, retrieved from https://canterburygeotechnicaldatabase.projectorbit.com/

[3] Canterbury Geotechnical Database (2013): "GNS Science Median Groundwater Surface Elevations", Map Layer CGD5160 – 7 Mar 2013, retrieved from https://canterburygeotechnicaldatabase.projectorbit.com/

[4] CERA (2012): *Technical Categories Map*, http://cera.govt.nz/maps/technical-categories

[5] NZ Aerial Mapping (2011): *Christchurch Post-Earthquake Aerial Photos (24 Feb 2011)*. Sourced from Koordinates, http://koordinates.com/layer/3185-christchurch-post-earthquake-aerial-photos-24-feb-2011/

[6] Canterbury Geotechnical Database (2012): "Observed Ground Crack Locations", Map Layer CGD0400 - 23 July 2012, retrieved from https://canterburygeotechnicaldatabase.projectorbit.com/

Figure 3: Post February 2011 Earthquake Aerial Photography

No evidence of liquefaction-induced damage or ground cracking was observed during the site inspection on 8 May 2013.

7.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions at the site are anticipated to comprise sand of marine origin to ~40 m, underlain by the Riccarton Gravel. The presence of peat at ~30 m is possible. However, at this depth, the peat is not considered likely to affect the structure.

Groundwater is estimated to be 1 m to 2 m bgl.

7.3 Seismicity

7.3.1 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. [7,8]

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

[8] 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.

Table 3: Summary of Known Active Faults

Known Active Fault	Distance from Site[7]	Direction from Site[7]	Max Likely Magnitude[8]	Avg Recurrence Interval[8]
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale Fault (2010)	30 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Porters Pass Fault	65 km	NW	7.0	~1100 years
Port Hills Fault (2011)	4 km	S	6.3	*Not Estimated*

The recent earthquake sequence since 4 September 2010 has identified the presence of a previously unmapped active fault system underneath the Canterbury Plains; this includes the Greendale Fault and Port Hills Fault listed in Table 3 above. Research and published information on this system is in development and the average recurrence interval is yet to be established for the Port Hills Fault.

7.3.2 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 with significant peak ground accelerations (PGA) across large parts of the city. Conditional PGA's from the CGD[9] indicate the PGA at this site to be 0.19g during the 4 September 2010 earthquake, 0.54g on 22 February 2011, 0.24g on 13 June 2011, and 0.4g on 23 December 2011.

7.4 Global Land Issues

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

7.5 Liquefaction Potential

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

- No clear evidence of liquefaction in any post-earthquake aerial photography;
- Surrounding properties being classified TC2 (minor to moderate land damage); and,
- Underlying soils being medium dense marine sands.

[9] Canterbury Geotechnical Database (2012): "Conditional PGA for Liquefaction Assessment", Map Layer CGD5110 - 27 Sept 2012, retrieved 31/10/2012 from https://canterburygeotechnicaldatabase.projectorbit.com/

7.6 Sufficiently Tested at SLS

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

Since the PGA's for 22 February and 23 December exceed 170% of the SLS value, the site can be considered "sufficiently tested at SLS". As a result, the ground damage during a future moderate earthquake (SLS) is likely to be similar or less than that observed in these earthquakes.

7.7 Summary & 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 is situated on marine sand deposits. The site also has minor liquefaction potential.

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

If a more comprehensive assessment is required, intrusive investigation should be conducted.

8. Seismic Capacity Assessment

8.1 Seismic Parameters

The seismic design parameters based on current design requirements from NZS 1170:2002 and the NZBC clause B1 for this building are:

- Site soil class: D
  (NZS 1170.5:2004, Clause 3.1.3, "Deep or 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, 50 year design life).

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

8.1.1 Expected Structural Ductility Factor

A structural ductility factor, μ, of 1.25 has been assumed based on the reinforced masonry structural system observed and the likely date of construction.

8.1.2 Fundamental Period of Building

A fundamental period of oscillation, T_1, of 0.4 seconds has been adopted for this assessment. This is based on the stiff, rigid nature of the reinforced blockwork masonry construction.

8.2 Quantitative Assessment Procedure

The seismic capacity was calculated in accordance with NZS 4230:2004 and the NZSEE guidelines[10] and based on the information obtained from visual observation and site measurements of the building. The demand for the structure was calculated in accordance with NZS 1170.5:2004, and the percentage of New Building Standard capacity (%NBS) was assessed.

The building was modelled as in-plane and out-of-plane shear walls. For further details on the assessment methodology, please refer to Appendix D.

8.2.1 %NBS

The shear capacity of the walls and the out of plane moment capacities were then compared to their respective demands to assess which was the most critical and thus determine the overall %NBS for the structure as follows:

$$\%NBS = \frac{\phi S_n \text{ (Capacity)}}{S^* \text{ (Demand)}} \times 100\%$$

[10] New Zealand Society for Earthquake Engineering (2006): *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*. Recommendations of a NZSEE Study Group on Earthquake Risk Buildings, June 2006. NZSEE

8.3 % NBS Assessment

As part of the Quantitative assessment, a more detailed seismic capacity assessment was carried out for the shear capacity of the building, with the results summarised below.

From the seismic capacity calculations, a strength summary of the different seismic actions assessed is presented in Table 4 below.

Table 4: Indicative Building Seismic Capacities based on NZS 1170.5:2004 and NZS 4230:2004

Element / Direction Assessed	%NBS Achieved
Along direction – in-plane shear	>100
Across direction – in-plane shear	>100
Along direction – out-of-plane	47
Across direction – out-of-plane	78
Critical %NBS for building	**47**

The overall seismic capacity for the Rawhiti Domain Tennis Club building assessed in accordance with NZSEE guidelines is **47%** NBS.

This score has not been adjusted when considering damage to the structure as all damage observed was relatively minor and considered unlikely to adversely affect the load carrying capacity of the structural systems.

8.4 Discussion of Results

Under the New Zealand Society for Earthquake Engineering (NZSEE) guidelines the building is considered Earthquake Risk as it achieves greater than 33% but less than 67% NBS. The results obtained from the seismic capacity assessment are consistent with those expected for a building of this age and construction type, and combined with the increase in the hazard factor for Christchurch to 0.3, it would be expected that the building would not achieve 100% NBS. Due to the lack of any Critical Structural Weaknesses and the presence of vertical reinforcing steel in the blockwork masonry walls, it is reasonable to expect the building to be classified as Earthquake Risk.

8.5 Occupancy

The building does not pose an immediate risk to users and occupants as no critical structural weaknesses have been identified. The building has not been assessed as being Earthquake Prone. As a result, the tennis club building can remain occupied.

9. Conclusions and Recommendations

The building has been assessed to have a seismic capacity in the order of **47%** NBS and is therefore deemed to be an Earthquake Risk building.

It is recommended that the current placard status of the building of 'Green' remain.

The recent seismic activity in Christchurch has only caused minor damage to the building, with minor cracking in the concrete blockwork masonry walls the only damage noted. As the building suffered insignificant damage that does not compromise the load resisting capacity of the existing structural systems, and has achieved between 33% and 67% NBS in seismic capacity calculations, no further assessment is required.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- No intrusive structural investigations have been undertaken;
- No intrusive geotechnical investigations have been undertaken;
- No inspection of the floor slab or foundations could be undertaken due to floor coverings;
- Visual inspections of the roof space were limited to the vicinity of the access hatch and due to its non-central location, the entirety of the roof space could not be inspected visually;
- No level or verticality surveys have been undertaken;
- No material testing has been undertaken; and
- No modelling of the building for structural analysis purposes has been performed.

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 reportrite a specific limitations section.

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 (front)

Photograph 2: East elevation

Photograph 3: View of the tennis club from the west

Photograph 4: Southwest perspective of tennis club

Photograph 5: Typical cracking in blockwork walls between concrete blocks

Photograph 6: Close up of crack through bottom concrete block

Photograph 7: Underside of veranda canopy showing structural members

Photograph 8: Close up of crack veranda concrete slab

Photograph 9: Inside main club room, looking northeast

Photograph 10: Inside main club room, looking southwest

Photograph 11: Roof space, showing timber roof trusses and purlins

Photograph 12: Roof space, showing end blockwork wall and end timber roof truss

Appendix B

Supporting Geotechnical Information

Bore or Well No: M35/2005
Well Name:
Owner: GOLF LINKS

Environment Canterbury
Your regional council

Street of Well:	SHAW AVE	**File No:**	
Locality:	NEW BRIGHTON	**Allocation Zone:**	Christchurch/West Melton
NZGM Grid Reference:	M35:877-447 QAR 4		
NZGM X-Y:	2487700 - 5744700		
Location Description:		**Uses:**	
ECan Monitoring:			
Well Status:	Not Used		

Drill Date:	02 Oct 1936	**Water Level Count:**	0
Well Depth:	87.10m -GL	**Strata Layers:**	12
Initial Water Depth:	5.30m -MP	**Aquifer Tests:**	0
Diameter:	51mm	**Isotope Data:**	0
		Yield/Drawdown Tests:	0
Measuring Point Ait:	2.20m MSD QAR 3	**Highest GW Level:**	
GL Around Well:	0.00m -MP	**Lowest GW Level:**	
MP Description:		**First Reading:**	
		Last Reading:	
Driller:	J W Horne (& Co)	**Calc. Min. GWL:**	0.90m -MP
Drilling Method:	Unknown	**Last Updated:**	18 Nov 1994
Casing Material:		**Last Field Check:**	
Pump Type:	Unknown		
Yield:		**Screens:**	
Drawdown:		**Screen Type:**	
Specific Capacity:		**Top GL:**	
		Bottom GL:	
Aquifer Type:	Flowing Artesian		
Aquifer Name:	Linwood Gravel		

Borelog for well M35/2005

Gridref: M35:877-447 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 2.2 +MSD
Driller : J W Horne (& Co)
Drill Method : Unknown
Drill Depth : -87.09m Drill Date : 2/10/1936

Environment Canterbury Regional Council

Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
	Artesian		Black sand	
-10				
-20		- 21.3m		ch
		- 24.3m	Black sand	ch
		- 27.4m	Black sand	ch
-30				
		- 39.0m	Black sand	ch
-40				
		- 49.3m	Black shingle, flow at 42.6 - 45.7m & best flow at 48.7m, rose 0.9m	ri
-50		- 50.2m	Brown sand	br
		- 51.8m	Black sand	br
-60				
		- 69.4m	Black sand	br
-70				
		- 77.7m	Brown shingle, flow at 73.1m rises 2.1m	li-1
-80		- 81.6m	Black clay & sand	li-2
		- 83.5m	Yellow clay	li-2
		- 87.1m	Brown shingle, flow at 84.1m to 85.9m & 87.1m	li-2

Bore or Well No: M35/2388

Well Name:

Owner: GOLF LINKS

Environment Canterbury
Your regional council

Street of Well: SHAW AVE

File No:

Locality: NEW BRIGHTON

Allocation Zone: Christchurch/West Melton

NZGM Grid Reference: M35:876-450 QAR 4

NZGM X-Y: 2487600 - 5745000

Location Description:

Uses:

ECan Monitoring:

Well Status: Not Used

Drill Date: 19 Mar 1930

Water Level Count: 0

Well Depth: 84.70m -GL

Strata Layers: 12

Initial Water Depth: 5.20m -MP

Aquifer Tests: 0

Diameter: 51mm

Isotope Data: 0

Yield/Drawdown Tests: 0

Measuring Point Ait: 3.80m MSD QAR 3

Highest GW Level:

GL Around Well: 0.00m -MP

Lowest GW Level:

MP Description:

First Reading:

Last Reading:

Driller: Job Osborne (& Co/Ltd)

Calc. Min. GWL: 0.50m -MP

Drilling Method: Hydraulic/Percussion

Last Updated: 21 Sep 2006

Casing Material:

Last Field Check:

Pump Type: Unknown

Yield: 0 l/s

Screens:

Drawdown: 0 m

Screen Type:

Specific Capacity:

Top GL:

Bottom GL:

Aquifer Type: Flowing Artesian

Aquifer Name: Linwood Gravel

Borelog for well M35/2388

Gridref: M35:876-450 Accuracy : 4 (1=high, 5=low)
Ground Level Altitude : 3.8 +MSD
Driller : Job Osborne (& Co/Ltd)
Drill Method : Hydraulic/Percussion
Drill Depth : -84.69m Drill Date : 19/03/1930

Environment Canterbury Regional Council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
- 15.2m	Sand & clay	ch
- 30.4m	Sand & gravel	ch
- 39.3m	Clay & peat	ch
- 44.5m	Blue shingle	ri
- 49.9m	Brown shingle (Water rises +0.9m)	ri
- 51.8m	Brown sand & gravel	ri
- 55.7m	Yellow clay	br
- 67.0m	Yellow sand & gravel	br
- 69.2m	Yellow clay	br
- 73.2m	Brown shingle (Water rises 3.05m)	li-1
- 80.8m	Clay	li-2
- 84.7m	Brown shingle	li-2

Scale(m): -10, -20, -30, -40, -50, -60, -70, -80

Appendix C

CERA Building Evaluation Form

Detailed Engineering Evaluation Summary Data V1.14

Location

Building Name:	Rawhiti Domain Tennis Club			Reviewer:	Hamish Mackinven
	Unit	No:	Street	CPEng No:	1003941
Building Address:		35-37	Bowhill Road	Company:	GHD Limited
Legal Description:	Pt Res 1579, and Pt Res 1616			Company project number:	51 31526 11
				Company phone number:	03 3780900
	Degrees	Min	Sec		
GPS south:	43	30	12.00	Date of submission:	31/05/2013
GPS east:	172	43	22.00	Inspection Date:	8/05/2013
				Revision:	FINAL
Building Unique Identifier (CCC):	PRK 2004 BLDG 020			Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	sand to >30m
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):	3.00

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):	0.40
Storeys below ground	0			
Foundation type:	mat slab		if Foundation type is other, describe:	
Building height (m):	4.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	3
Floor footprint area (approx):	71			
Age of Building (years):	35		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):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	90x45 framing, 1.2m deep, corrugated iron cladding
Floors:	concrete flat slab	slab thickness (mm)	100 (assumed)
Beams:			
Columns:			
Walls:	fully filled concrete masonry	#N/A	

Lateral load resisting structure

Lateral system along:	fully filled CMU	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	15.8
Ductility assumed, μ:	1.25			
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:	fully filled CMU		note total length of wall at ground (m):	4.5
Ductility assumed, μ:	1.25			
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:	exposed structure	describe	blockwork masonry
Roof Cladding:	Metal	describe	longrun corrugated iron
Glazing:	timber frames		
Ceilings:	plaster, fixed		plasterboard on ceiling battens
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	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Good	Describe damage:	None observed
Settlement:		notes (if applicable):	
Differential settlement:		notes (if applicable):	
Liquefaction:		notes (if applicable):	
Lateral Spread:		notes (if applicable):	
Differential lateral spread:		notes (if applicable):	
Ground cracks:		notes (if applicable):	
Damage to area:		notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	IEP, as per equation
	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 to blockwork

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

Recommendations

Level of repair/strengthening required:	minor non-structural	Describe:	
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before e'quakes:	47%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Seismic capacity calculations, NZS 4230
	Assessed %NBS after e'quakes:	47%		
Across	Assessed %NBS before e'quakes:	47%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	47%		

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: 3m

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	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:	0.30
Z_{1992}, from NZS4203:1992	0.8
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 =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, (NBS%)$_b$ = (%NBS)$_{nom}$ x A x B x C x D x E

%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		
	Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

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

Appendix D

Quantitative Assessment Methodology

C1. Building Seismic Demand

The demand on the structure was determined in accordance with NZS 1170.5:2004, which uses the equivalent static method. The structure is located in Christchurch, on class D soils.

An Importance Level of 3 was used for the calculations, in line with the client's request. This results in the Return Period Factor, as given by Table 3.5 of NZS 1170.5: 2004 and as prescribed by Table 3.3 of AS/NZS 1170.0:2002, for the structure as 1.3.

C2. Seismic Weight Coefficient

The elastic site hazard spectrum for horizontal loading, C(T), for the building was derived from Equation 3.1(1), NZS 1170.5:2004;

$$\mathbf{C(T) = C_h(T)\, Z\, R\, N(T, D)}$$

Where

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

Z = the hazard factor from Clause 3.1.4, and subsequent amendments issued by DBH, which increased the hazard factor to 0.30 for Christchurch;

R = the return period factor from Table 3.5 for an annual probability of exceedance of 1/1000 (earthquake action for an Importance Level 3 building); and,

N(T,D) = the near-fault scaling factor from Clause 3.1.6.

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

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

Where the structural ductility factor (μ) was taken as 1.25.

The seismic weight coefficient, $C_d(T_1)$, was then calculated in accordance with Clause 5.2.1.1 of NZS 1170.5:2004. For the purposes of calculating the seismic weight coefficient a period, T_1, of 0.4s was assumed for the building. The coefficient was then calculated using Equation 5.2(1);

$$\mathbf{C_d(T_1) = \frac{C(T)\, S_P}{k_\mu}}$$

Where

$$\mathbf{k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1 \qquad for\ T_1 < 0.7s}$$

C3. Induced Shear Forces to Walls

The lateral forces induced on the walls of the building in a seismic event include the direct seismic shear and any torsional forces caused by the centre of mass and centre of rigidity of the building being offset.

NZS 1170.5 makes allowance for accidental eccentricity of ± 0.1 times b, the plan dimension of the structure at right angles to the direction of loading. That is, the force is applied at ±0.1b from the centre of mass. This results in a torsional action about the centre of resistance of the building, and induces forces in the lateral force resisting (in-plane) walls in addition to direct shear.

Clause 5.3.1.2 of NZS 1170.5 also requires that for brittle and nominally ductile structures, the forces are to be applied in such a way that 100% of the force is applied in one direction while 30% of that force is applied simultaneously in the orthogonal direction.

The induced shear force plus the direct shear is what must be designed for and the magnitude of the forces distributed into the walls is relative to their in-plane stiffness.

Moment demands were calculated by multiplying the shear forces by the effective seismic mass height. This effective height comes from the weighted average of the heights of all seismic weights. This is typically approximately half the structure's height for a single-storey structure.

C4. 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, and considered to be Type B. The overall shear capacity of the wall was calculated from Clause 10.3.2.1, Equation 10-4;

$$\emptyset \mathbf{V_n} = \emptyset\, \mathbf{v_n b_w\, d}$$

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 (zero as no horizontal reinforcing detected);

b_w = the thickness of the wall;

d = depth from compression end of wall to centre of reinforcing, approximated as 0.8 times the wall length (NZS 4230:2004); and

$\emptyset$ = strength reduction factor, 0.75 for concrete masonry in shear (Clause 3.4.7).

C5. In-Plane Moment Capacity

The moment capacity of the columns and shear walls has been calculated using NZS 4230:2004, in conjunction with first principles for strain compatibility and force equilibrium. The maximum allowable compressive strain is 0.003. Type B masonry construction was assumed for moment capacity.

It was assumed that all steel within the wall yields at the wall's ultimate capacity.

The forces involved in the shear wall can be summarised by equating the net internal compressive force to the net external compressive force:

$$\mathbf{C_m + C_S - T_S = N_n}$$

Where

C_m = strength of masonry in compression;

C_S = strength of steel reinforcement in compression;

T_S = strength of steel reinforcement in tension; and,

N_n = axial compressive load acting on the shear wall.

The reinforcing steel is considered to have forces calculated as the cross-sectional area multiplied by the characteristic yield strength, in line with the "all steel yields" assumption.

The capacity of concrete in compression can be calculated using the following equation:

$$\mathbf{C_m = \alpha\beta f'_m\, t\, c}$$

Where

α = stress block adjustment factor, 0.85 (Clause 7.4.2.7(c), NZS 3101:2006);

β = neutral axis depth factor, 0.85 (Clause 7.4.2.7(d));

f'_m = the compressive strength of blockwork masonry;

t = thickness of the reinforced masonry wall; and,

c = depth from compression edge to neutral axis of the wall.

Combining these equations gives the following expression:

$$\mathbf{\alpha\beta\, f'_m\, t\, c - \sum_{T-C} A_S f_y = N_n}$$

Where

$\boldsymbol{\Sigma\, A_S f_y}$ = sum of tension steel strength *less* sum of compression steel strength

This equation involves an iterative process, where the number of bars in each tension and compression is initially estimated, and the force equilibrium solved for the neutral axis depth, ‘c’. Once ‘c’ is between the last compression steel and first tension steel, then the equation is solved.

Internal moments for the masonry and steel force components are then taken about the neutral axis to calculate the moment capacity of the wall, as follows:

$$\mathbf{\emptyset M_n = \emptyset\left[C_m\left(1-\frac{\beta}{2}\right)c + \sum A_S f_y(l - c) + N_n\left(\frac{L_w}{2} - c\right)\right]}$$

Where

l = distance of each steel reinforcing element from the compression edge

∅ = strength reduction factor = 0.85 for flexure (NZS 3101:2006)

C6. Out-of-Plane Capacity

Due to the structural systems observed in the building (chipboard roof acting as a diaphragm, and shear walls resisting loads in both the along and across directions), out-of-plane moments have been assumed to be transferred to elements perpendicular which in turn resist moments as in-plane shear walls.

However, the out-of-plane flexural strength of the walls has been checked by analysing the wall using the models in Figure 4 (cantilever for side walls, simply supported for end walls). The out-of-plane flexural strength and demand was determined per unit length of wall (or height for end walls).

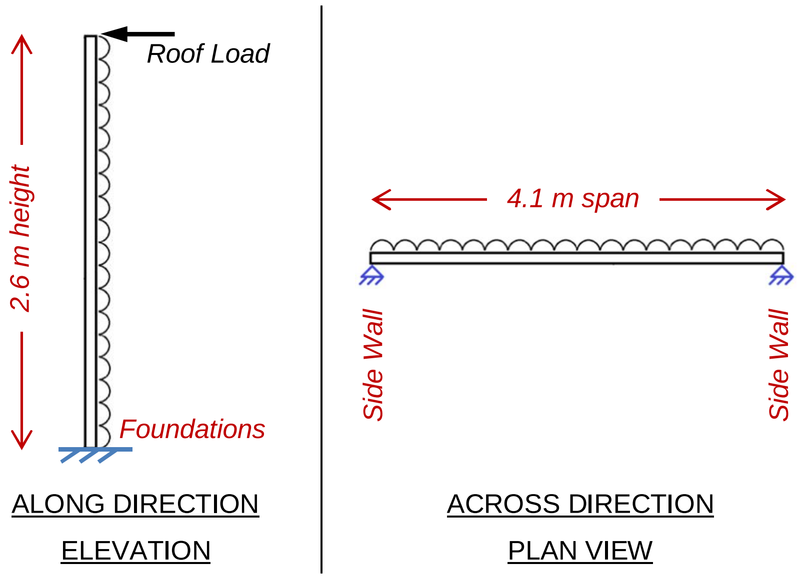

Figure 4: Out-of-Plane Assessment Model, Along and Across

The walls were analysed per metre width using the models in Figure 4 above, and the critical bending moments and shear forces calculated. The uniformly-distributed load (UDL) was modelled as the wall's self-weight multiplied by the elastic response factor, $C_d(T_1)$. For the along direction, the lateral load of the roof was applied at the top as no diaphragm is present and trusses transfer loads to these side walls.

The wall's out-of-plane capacity has been determined using the methodology for a singly-reinforced beam, as outlined in the *In-plane Moment Capacity* section in this Appendix, and then checked against the demand.

GHD

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