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Wycola Park Hockey Pavilion

PRK 1557 BLDG 005

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

34 Manurere Street, Hei Hei

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

Wycola Park Hockey Pavilion
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Detailed Engineering Evaluation
Quantitative Report
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34 Manurere Street, Hei Hei

Christchurch City Council

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Date
27 May 2013

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Quantitative Report Summary

Wycola Park Hockey Pavilion

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

34 Manurere Street, Hei Hei

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 13 December 2012 and available drawings itemised in 5.2.

Building Description

The single storey structure consists of a partial fill concrete masonry external wall supporting a timber frame roof with a lightweight metal cladding. Internal storage areas are formed by partial height lightweight timber frame partitions. The foundations are formed by a reinforced concrete strip footing with the ground floor being formed by a concrete slab on grade.

Key Damage Observed

No damage was observed in the structure.

Building Capacity Assessment

Based on the results of the quantitative assessment the building scored 22% NBS. Therefore the building is Earthquake Prone.

Recommendations

Currently the external side walls are failing out-of-plane with a %NBS of 22%. Design concepts should be undertaken to strengthen the structure to a minimum of 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.

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 Hockey Pavilion was built on the southeast corner of Wycola Park in 1976. The park lies within a suburban area at 34 Manurere Street. No additions have been made to the structure since its original construction.

The building is approximately 9.2m long, 4.8m wide and 3.5m in height. The overall footprint of the building is approximately 44m^2.

The roof structure consists of timber trusses at 900mm centres supporting 75x50mm timber purlins at 800mm centres. This roof structure supports the lightweight metal cladding externally and a plasterboard lined ceiling internally.

The 190mm thick partial filled concrete masonry walls are reinforced with ø16mm bars in each of the four corners and along the side walls of the building irregular locations. There is a bond beam reinforced with a ø12mm bar at the wall head. Internal storage areas are formed by partial height lightweight timber frame partitions. The construction plans show the foundations to be a strip footing 250mm wide with a depth of 300mm and reinforced with 4No. ø12mm bars. The floor consists of a 100mm concrete slab on 150mm of hardfill.

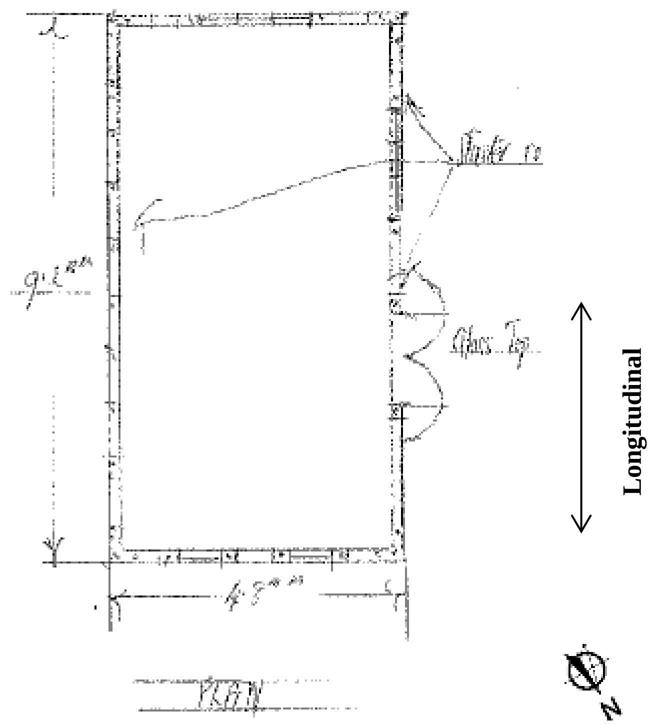

Figure 2 Plan of Structure

The Hockey Pavilion is located approximately 10m from two residential houses. The predominantly flat site is 5km Northwest of Heathcote River and is found at an elevation of 25m above sea level.

Construction plans are provided in Appendix B.

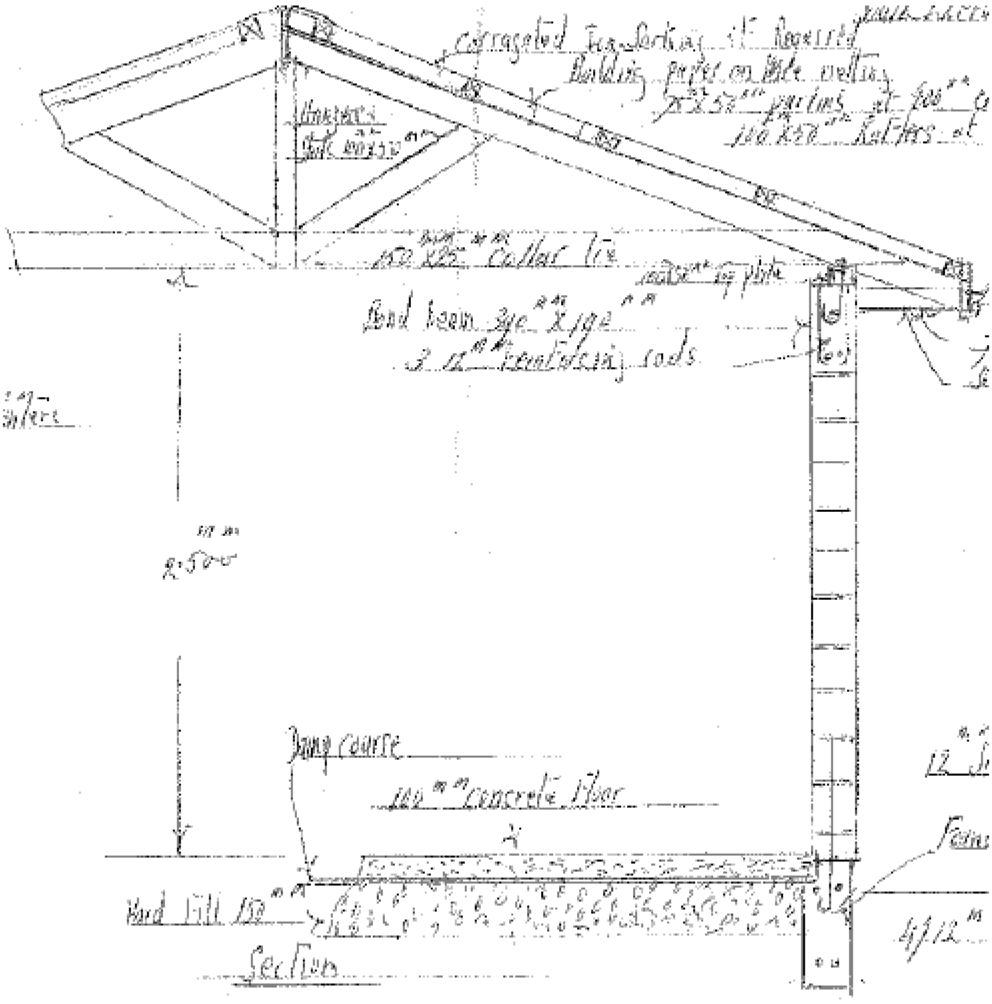

Figure 3 Section of Structure

4.2 Gravity Load Resisting System

Gravity roof loads are transferred via the lightweight metal cladding and timber purlins to the timber roof trusses. These timber roof trusses span between the concrete masonry walls which support the roof loads. The load bearing concrete masonry walls transfer the gravity roof loads downwards to the foundations where the they are distributed into the ground beneath. Gravity floor loads are transferred directly through the concrete floor slab to the ground beneath.

4.3 Lateral Load Resisting System

Given the lightweight construction of the roof, the ceiling will have sufficient capacity to span horizontally between the walls in the plane of loading. These in-plane walls will resist the minimal lateral roof loads and those from wall self-weight by the panel action of the concrete masonry. These lateral loads will be transferred via the foundation to the ground beneath.

The gable walls will span vertically from the foundation level to the reinforced concrete bond beam at the wall head, which in turn spans horizontally between the orthogonal in-plane walls. These in-plane walls again resist the lateral loads by the panel action of the concrete masonry.

The absence of a reliable ceiling diaphragm to transfer the high lateral load demands from the side walls to the in-plane gable walls will require these side walls to cantilever from the foundations.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 13 of December 2012. Both the interior and exterior of the building were inspected. The main structural components of the building, except those located in the enclosed roof space, were all able to be viewed. 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 partial fill concrete masonry walls. 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

The construction drawings of the structure were made available.

All drawings are attached as Appendix B.

6. Damage Assessment

6.1 Surrounding Buildings

There was no notable damage to any surrounding buildings however visual inspection of the structures immediately adjacent was prevented by a perimeter fence

6.2 Residual Displacements and General Observations

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

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 requirements from the Department of Building and Housing B1 amendment 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 the absence of a reliable roof diaphragm. Elements were considered individually and subject to loads from seismic self-weight and those loads from tributary areas directly applied.

7.3 Dependable Capacity

7.3.1 Reinforced Masonry-Shear Capacity

The shear capacity of the reinforced concrete masonry shear 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. In this instance there was no shear steel reinforcement found in the structure.

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 = t wall thickness when fully filled;

d = length of wall,

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 Bond Beam 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 = bond beam thickness

A_s = area steel

A_m = area of concrete masonry

f'_m = masonry strength

A similar method was used to calculate the in-plane moment capacities of the wall.

7.3.3 Unreinforced Masonry Out-of-Plane Moment Capacity

The out-of-plane flexural capacity of the unreinforced concrete masonry walls was determined using Section 10.3.4 of the NZSEE guidelines “Assessment & Improvement of the Structural Performance of Buildings in Earthquakes (2006)”. The overall out-of-plane capacity of each wall was evaluated by comparing the likely displacement of the wall during an earthquake and the displacement that would cause instability of the wall. The out-of-plane capacity of each wall is,

$$\%NBS = 0.72 \frac{\Delta_i}{D_{ph}}$$

Where

Δi = out-of-plane deflection that would cause instability

Dph = out-of-plane displacement response demand for a wall panel

8. Geotechnical Consideration

8.1 Site Description

The site is situated in the suburb of Hei Hei, west of Christchurch. The site is relatively flat at approximately 25m above mean sea level. It is approximately 5km northwest of Heathcote River, 500m north of the Main South Line Railway, and 18km west of the coast (Pegasus Bay).

8.2 Public Information on Ground Conditions

8.2.1 Published Geology

The geological map of the area1 indicates that the site is underlain by:

- Yaldhurst Member of the Springston Formation, dominantly alluvial gravel, sand, and silt of historic river flood channels, Holocene in age.

Due to the low-lying location of the site, shallow ground water table is anticipated.

8.2.2 EQC Geotechnical Investigations

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

8.2.3 Environmental Canterbury Logs

Information from Environment Canterbury (ECan) indicates that there are six boreholes located within 200m of the site. Two boreholes with significant information regarding the site are shown in the table (see **Table 2**).

These indicate that the area is underlain by sand and gravel with varying amount of silt.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35-1841	42 m	13m bgl	190m NW
M35-1868	72.5 m	14.8m bgl	190m W

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

8.2.4 Additional Geotechnical Investigation

Records from one piezocone CPT investigation that was previously conducted near to the site at 12 Wycola Avenue, indicate that dense gravels are present at approximately 1m bgl.

[1] Forsyth, P. J., Barrell, D. J. A., & Jongens, R. (2008): *Geology of the Christchurch Urban Area*. Institute of Geological and Nuclear Sciences 1:250,000 Geological Map 16. IGNS Limited: Lower Hutt.

8.2.5 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has classified the site as "Green Zone, N/A – Urban Non-residential" category. Land in the green zone is generally considered suitable for residential construction. An "N/A" technical category indicates the site is a non-residential property in urban area beyond the extent of land damage mapping. However, the neighbouring properties are classified as "Green Zone, TC1 (grey)". This indicates that future land damage from liquefaction is unlikely.

8.2.6 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows no signs of liquefaction outside the building footprint or adjacent to the site, as shown in Figure 4**.**

Figure 4 Post February 2011 Earthquake Aerial Photography2

8.2.7 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise predominantly gravel, with gravel and sand strata along with varying amount of silt.

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.

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	120 km	NW	~8.3	~300 years
Greendale (2010) Fault	14 km	SW	7.1	~15,000 years
Hope Fault	102 km	NW	7.2~7.5	120~200 years
Kelly Fault	102 km	NW	7.2	150 years
Porter Pass Fault	55 km	NW	7.0	1100 years

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

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.

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

8.3.3 Slope Failure and/or Rockfall Potential

Given the site's location in Hei Hei, 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.

8.3.4 Liquefaction Potential

The site is considered to have a negligible susceptibility to liquefaction, due to the following reasons:

- No previous liquefaction or settlement at the site following the February (Mw 6.3, 2.0g) and June (Mw 6.0-6.3, 1.5g) events; and,
- Anticipated presence of predominantly gravel beneath the site;

However the ground information available indicates there are sand layers and silt in the area that may be present. Such layers may be highly liquefiable.

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

[4] GNS Active Faults Database

8.3.5 Conclusions & Recommendations

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

The site appears to be situated on alluvial deposits. However, nearby investigations indicate dense gravel/sand. Associated with this the site also has a negligible liquefaction potential.

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 was found to be relatively symmetrical with the opposing wall panels achieving similar performances. The structure is divided into two groups of wall panels as identified in Figure 9-1.

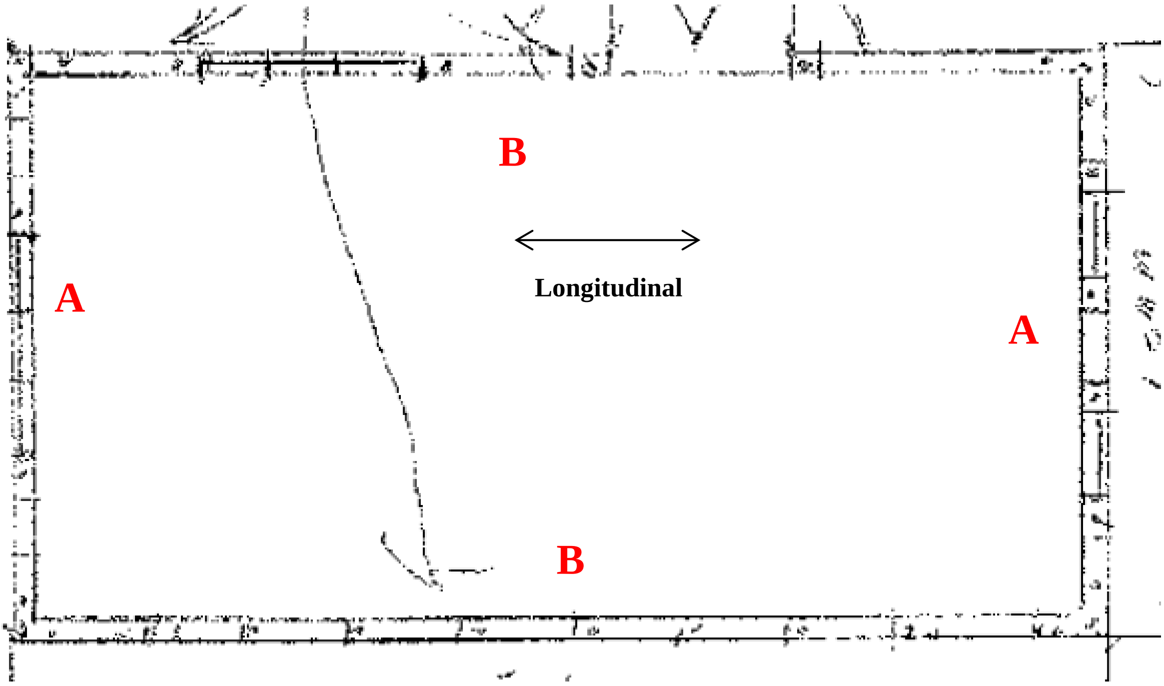

Figure 5 Plan identifying wall panels.

The critical loading condition for the concrete masonry panels in this structure is lateral loading perpendicular to the wall panels. Hence the critical load condition for Wall A is lateral loads in the longitudinal direction. Similarly, the critical load condition for Wall B and therefore the overall structure, is lateral loads in the transverse direction. The performance of each wall group for each load direction is quantified in Table 4 below.

Table 4 %NBS of Structural Elements

Element	Load Direction	% NBS
Wall A	Longitudinal	38%
	Transverse	>100%
Wall B	Longitudinal	>100%
	Transverse	22%

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 construction type, founded on Class D soils.

The Wycola Park Hockey Pavilion was designed in 1976 and was likely designed in accordance with the previous loading standard, NZS 1900:1965, superseded that year. The design loads used are likely to have been less than those required by the current loading standard. This is compounded by the absence of a reliable roof diaphragm, which require the structure to resist seismic demand inefficiently. In addition, inconsistent reinforcement detailing renders some wall sections unreinforced, with unreinforced masonry having been identified as performing poorly in seismic events.

The critical structural element identified by detailed analysis was Wall B with a New Building Standard of 22%. The absence of sufficient roof diaphragm capacity to provide lateral restrain to the top of the walls render them unstable against out-of-plane lateral loads.

10. Conclusions and Recommendations

The building overall has been assessed as having a seismic capacity of 22% NBS and is therefore classified as being ‘Earthquake Prone’.

It is recommended that strengthening concepts be developed to increase the structure’s performance to a minimum of 67% of the New Building Standard.

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 South elevation.

Photograph 2 View of the structure from the Southwest.

Photograph 3 Northern Elevation.

Photograph 4 Pavilion Interior.

Photograph 5. Partial Height Partition.

Photograph 6 Pavilion Interior.

Appendix B

Existing Drawings

Appendix C

CERA Form

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Document Status

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