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Clare Park Pavilion and Toilets
PRK 0104 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report
Version FINAL

149 Burwood Road, Burwood

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Christchurch City Council

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8th March 2013

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Clare Park Pavilion and Toilets

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

149 Burwood Park, Burwood

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 and visual inspections on 31 January 2013.

Building Description

The two storey building is located at the eastern end of Clare Park, 149 Burwood Road, Burwood and is assumed to have been constructed during the 1970s. An additional storey and an adjacent garage were constructed in 1985.

The building is currently used as a pavilion with changing rooms and public toilet facilities. The building is approximately 14.5m in length by 7.2m in width with a height of 8.0m and has a footprint of approximately 105m^2. The site is approximately 1km north of the Avon River.

The upper floor structure consists of corrugated sheet metal roof cladding supported by timber purlins spanning between five steel PFC portal frames. The walls on the upper floor are timber framed and are lined internally with chipboard and clad externally with corrugated sheet metal. The floor consists of 350mm deep precast concrete double tee units with a reinforced concrete topping. The double tee units are supported by the reinforced concrete masonry walls on the ground floor.

Key Damage Observed

Residual displacements were observed in several areas of the building during inspections. The flange supported double tee precast floor units in the changing room area appear to have moved away from the supporting concrete masonry walls, reducing the seating.

Separation between the pavilion building and the garage immediately to the south appears to have occurred during the recent seismic activity. Several steel ties connecting these two buildings have pulled out. The gap between the two buildings is approximately 35mm. The front of the pavilion building also appears to have settled by approximately 20mm.

Approximately 20mm of settlement was observed at the entrance to the pavilion building. Separation of the garage addition and the pavilion building and relative movement of the external stair landing and the staircase indicates that ground movement has occurred.

Building Capacity Assessment

The pavilion building has been assessed to have a seismic capacity in the order of 7% NBS and is therefore Earthquake Prone.

The adjacent garage has been assessed to have a seismic capacity in the order of 82% NBS and is not Earthquake Prone or Earthquake Risk.

Recommendations

It is recommended that Christchurch City Council investigate potential strengthening options for the pavilion building.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the pavilion and public toilets in Clare Park.

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.

A quantitative assessment involves a full site measure of the building which is used to determine the building's bracing capacity in accordance with manufacturers' guidelines where available. When the manufacturers' guidelines are not available, values for material strengths are taken from the NZSEE guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes (2006)'. The seismic demand for the building is determined and the percentage of New Building Standard (%NBS) is assessed.

At the time of this report, no intrusive site investigation or finite element modelling of the building structure had been carried out.

2. Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

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

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

2.2 Building Act

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

Section 112 – Alterations

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

Section 115 – Change of Use

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

2.2.1 Section 121 – Dangerous Buildings

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

- In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
- In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
- There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
- There is a risk that 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 Descriptions

4.1 General

The two storey building is located at the eastern end of Clare Park, 149 Burwood Road, Burwood and is assumed to have been constructed during the 1970s. An additional storey and an adjacent garage were constructed in 1985.

The building is currently used as a pavilion with changing rooms and public toilet facilities. The building is approximately 14.5m in length by 7.2m in width with a height of 8.0m and has a footprint of approximately 105m^2. The site is approximately 1km north of the Avon River.

The roof structure consists of corrugated sheet metal roof cladding supported by timber purlins spanning between five steel PFC portal frames. The walls on the upper floor are timber framed and are lined internally with chipboard and clad externally with corrugated sheet metal.

The first floor consists of 350mm deep precast concrete double tee units with a concrete topping reinforced with 8mm diameter bars at 150mm centres. The double tee units span in alternating directions shown in Figure 2. There is a balcony on the western side of the building that extends approximately 1.5m beyond the supporting concrete masonry wall. The double tee units are supported by the reinforced concrete masonry walls on the ground floor through a combination of direct bearing of the web of the double tee units and bearing of the double tee flanges on the concrete masonry walls.

The 190mm thick concrete masonry walls on the ground floor are partially filled and are reinforced with 12mm diameter vertical bars at 600mm centres. The foundations of the building consist of strip footings beneath external concrete masonry walls and internal pad footings. The ground floor is a concrete slab-on-grade.

The building has been constructed into a slope. The ground floor wall along the eastern side of the building retains approximately 2.2m of fill. The ground slopes downwards east to west along the transverse faces of the building. There is an external timber staircase and landing supported by two 50mm square posts at the northern end of the building.

The garage addition directly adjacent to the south of the pavilion building consists of partially filled concrete masonry walls supporting a lightweight timber framed roof structure with corrugated sheet metal roof cladding. The concrete masonry walls are reinforced with 12mm diameter vertical bars at 600mm centres. The foundations consist of a concrete slab-on-grade with strip footings beneath the external walls. The garage addition is roughly 4.4m long by 4m wide and 2.4m in height.

Figure 2 and Figure 3 show the construction details. No drawings of the building were available.

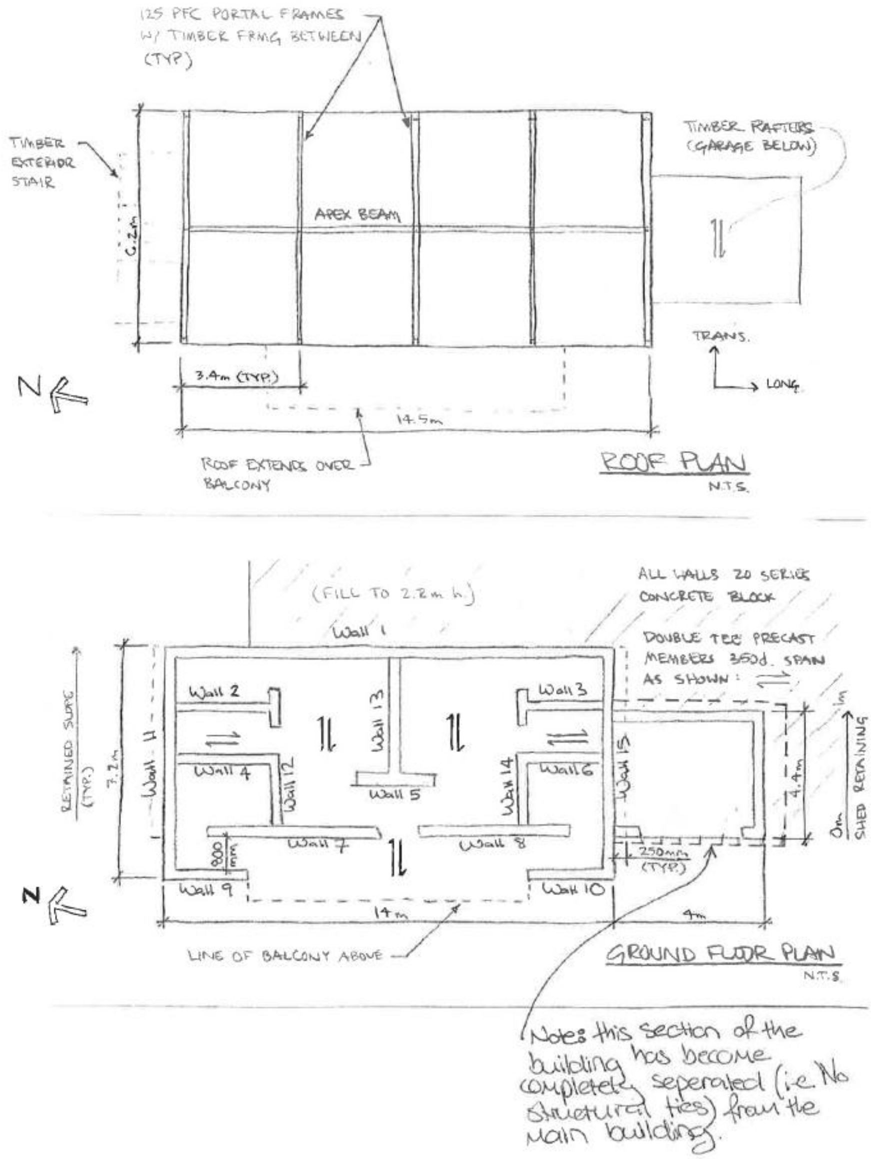


Figure 2 Ground floor and first floor plans

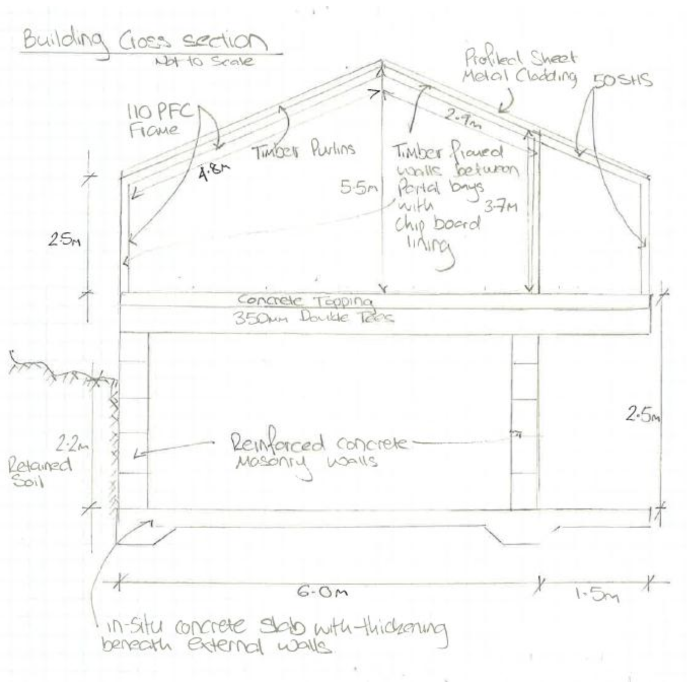


Figure 3 Typical section through building

4.2 Gravity Load Resisting Systems

Gravity loads acting on the building are resisted by steel PFC portal fames at first floor level and load bearing concrete masonry walls on the ground floor. Gravity loads from the corrugated steel roof are transferred via the timber purlins to the steel PFC portal frames. The gravity loads from the timber framed walls, PFC portal frames and the double tee precast concrete flooring units at first floor level are transferred through the external and internal concrete masonry walls to the concrete strip footings where they are distributed into the ground. Ground floor gravity loads are transferred through the concrete slab to the underlying ground.

4.3 Lateral Load Resisting Systems

At first floor level, the chipboard panel ceiling lining provides a diaphragm to transfer seismic forces through the roof structure to the chipboard lined timber framed walls and steel portal frames in the plane of loading. Lateral seismic loads in the longitudinal direction are resisted by the chip board lined timber framed walls in the plane of loading. Lateral seismic loads in the transverse direction are resisted by frame action of the steel PFC portal frames and panel action of the chip board lined timber framed walls. The timber framed walls and portal frames transfer the seismic loads to the supporting ground floor concrete masonry walls through diaphragm action of the precast concrete double tee units and reinforced concrete topping.

At ground floor level, the precast double tee flooring units and reinforced topping provide a diaphragm to transfer seismic forces from the upper storey to the reinforced concrete masonry walls in the plane of loading. Panel action of the longitudinal and transverse concrete masonry walls resists the lateral seismic loads. The concrete masonry walls transfer the seismic loads to the foundations through shear and bending where they are then distributed into the ground.

The ground floor concrete masonry walls are restrained out-of-plane by the diaphragm provided by the double tee units and reinforced topping. The walls span vertically between the ground floor concrete slab and the first floor diaphragm to resist out-of-plane seismic loading.

The central internal walls (Walls 5 and 13, Figure 2) and entrance wing walls (Walls 9 and 10, Figure 2) at ground floor level have no connection to the diaphragm at first floor level. As a result, these concrete masonry walls are unrestrained along their top edges and rely on cantilever action to resist out-of-plane seismic loading.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 31st of January 2013. Both the interior and exterior of the building was inspected. 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 presence of reinforcement in the concrete masonry walls. Where reinforcement was detected, the position, depth and diameter of the reinforcement were recorded. The results of the reinforcement scanning were used as part of the element capacity calculations

5.2 Available Drawings

Drawings of the structure were not available.

Sketches of the key structural features of the building are attached as Appendix B.

5.3 Damage Assessment

5.3.1 Surrounding Buildings

The building is located in a suburban area with open park areas adjacent to the site to the west. There are a number of residential houses adjacent to the site to the east which showed no signs of seismic damage.

5.3.2 General Observations

Residual displacements were observed in several areas of the building during inspections. The flange supported double tee precast floor units in the changing room area appear to have moved away from the supporting concrete masonry walls, reducing the seating.

Separation between the pavilion building and the garage immediately to the south appears to have occurred during the recent seismic activity. The separation can be seen in Photograph 9. Several steel ties connecting these two buildings have pulled out. The gap between the two buildings is approximately 35mm. The front of the pavilion building also appears to have settled by approximately 20mm.

No other signs of seismic damage to the building were observed while on-site.

5.3.3 Ground Damage

Approximately 20mm of settlement was observed at the entrance to the pavilion building. Separation of the garage addition and the pavilion building and relative movement of the external stair landing and the staircase indicates that ground movement has occurred.

6. Geotechnical Consideration

6.1 Site Description

The pavilion, toilet is situated in Clare Park in Burwood, in north Christchurch. It is relatively flat at approximately 4.5-5.0m above mean sea level. The site is approximately 1km to the north of the Avon River, and 2.5m west of the coast.

6.2 Published Information on Ground Conditions

6.2.1 Published Geology

The geological map of the area[1] indicates that the site is underlain by Holocene marine soils of the Christchurch Formation, dominantly sand of fixed and semi-fixed dunes and beaches. The site is close to the boundary of alluvial sand and silt overbank deposits.

6.2.2 Environment Canterbury Logs

Information from Environment Canterbury (ECan) indicates that a number of boreholes are located within a 200m radius of the site.

Of these boreholes, four of them had lithographic logs (see Table 2 ECan Borehole Summary), which indicate the area is typically underlain by 30m of sand, overlying layers of gravel and sand at depth.

The log M35/13264 indicates strata containing peat to be present between 2.6m and 2.9m bgl, and log M35/4133 between 32.9 and 33.5m.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/3660	153m	7.3m bgl	~80m NE of buildings
M35/4133	152.4m	Not noted	~100m NE of buildings
M35/13264	3.2m	Not noted	~20m E of building
M35/14919	1.8m	Not noted	~170m S of buildings

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.

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

6.2.3 EQC Geotechnical Investigations

The Earthquake Commission has undertaken geotechnical testing in the area of the site. Information pertaining to this investigation is included in the Tonkin & Taylor Report for Burwood[2]. Three CPT investigation points were located approximately 200m around the property, as summarised below in Table 3.

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Grid Reference	Depth (m bgl)	Log Summary
CPT-BUR-100	2484596 mE 5746345 mN	0 – 1m	Surface Soil, sensitive fine grained
		1m ~ 5m	SAND to silty Sand, moderate dense
		5m ~ 6m	Silty CLAY/Peat
		>6m	SAND, dense
CPT-BUR-102	2484302 mE 5746420 mN	0 – 1.2m	Surface Soil, sensitive fine grained
		1.2m ~ 3m	SAND to silty Sand, moderate dense
		3m ~ 5.5m	Silty CLAY/Peat
		>5.5m	SAND, dense
CPT-BUR-105	2484387 mE 5746131 mN	0 – 1m	Surface Soil, sensitive fine grained
		1m ~ 3.2m	SAND to silty Sand, moderate dense
		3.2m ~ 4.3m	Silty CLAY/Peat
		>4.3m	SAND, dense

Initial observations of the CPT results indicate that the soil deeper than 1.0m is sand with one thick layer of clay/peat at the 3m depth.

6.2.4 Land Zoning

Canterbury Earthquake Recovery Authority (CERA) has published areas showing the Green Zone Technical Category in relation to the risk of future liquefaction and how these areas are expected to perform in future earthquakes.

This site is classified as Green Zone, indicating the land is generally suitable for repair and rebuilding to take place. It is also categorised Technical Category Not Applicable, as the property is considered non-residential.

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

6.2.5 Post February Aerial Photography

Aerial photography taken within days following the 22 February 2011 earthquake doesn't show any sign of liquefaction around the pavilion, as shown in Figure 4.

Figure 4 Post February 2011 Earthquake Aerial Photography[3]

6.2.6 Summary of Ground Conditions

From the information presented above, the ground conditions underlying the site are anticipated to comprise sands with one thick layer of peat at the level of approximately 3m.

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

6.3 Seismicity

6.3.1 Nearby Faults

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

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

Known Active Fault	Distance from Site	Direction from Site	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	130 km	NW	~8.3	~300 years
Greendale (2010) Fault	30 km	SW	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	110 km	NW	7.2	150 years
Porters Pass Fault	60 km	NW	7.0	1100 years

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

6.3.2 Ground Shaking Hazard

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

Post February 2011 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.

The presence of marine and/or estuarine sands of varying density and bedrock anticipated to be in excess of 500m deep with, a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 2002[4]), , ground shaking is likely to be relatively high.

6.4 Slope Failure and/or Rockfall Potential

Given the site's location in Burwood, a flat suburb in north Christchurch, global slope instability is considered negligible. However, any localised retaining structures or embankments should be further investigated to determine the site-specific slope instability potential.

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

[5] GNS Active Faults Database

6.5 Liquefaction Potential

Due to the anticipated presence of dense sands, in addition to liquefaction evidence in post-earthquake aerial photography it is considered unlikely that liquefaction will occur at the site.

6.6 Recommendations

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

According to all above, there is no need for any further site investigation.

6.7 Conclusions & Summary

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

The site appears to be situated predominantly on the dense sands. Associated with this the site also has a minor 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 comprising at least one CPT should be conducted to target depth of 20m.

7. Structural Analysis

7.1 Seismic Parameters

Seismic loading on the structure has been determined using New Zealand Standard 1170.5:2004.

◗ 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 (R_u)	
(Table 3.5, NZS 1170.5:2004)	1.0 (ULS)

First Floor

Longitudinal Direction

◗ Ductility Factor (μ)	3.0
◗ Ductility Scaling Factor (k_μ)	2.14
◗ Performance Factor (S_p)	0.7

Transverse Direction

◗ Ductility Factor (μ)	3.0
◗ Ductility Scaling Factor (k_μ)	2.14
◗ Performance Factor (S_p)	0.7

Ground Floor

Longitudinal Direction

◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (S_p)	0.925

Transverse Direction

◗ Ductility Factor (μ)	1.25
◗ Ductility Scaling Factor (k_μ)	1.14
◗ Performance Factor (S_p)	0.925

An increased Z factor of 0.3 for Christchurch has been used in line with recommendations from the Department of Building and Housing.

The structural performance factor, S_P, was calculated in accordance with Clause 4.4.2 NZS 1170.5.

$$S_P = 1.3 - 0.3\mu \geq 0.7$$

The seismic weight coefficient 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 0.4 was assumed for both directions of the building. The coefficient was then calculated using Equation 5.2(1);

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

Where

$$k_\mu = \frac{(\mu - 1)T_1}{0.7} + 1$$

7.2 Lateral Force Distribution

The distribution of lateral forces from the roof in both the longitudinal and transverse directions of the building follows the bracing design procedure discussed in Section 5 of NZS 3604:2011. The seismic bracing demand in each direction was resolved into bracing units (BUs) and compared to the bracing capacity of the timber walls. At first floor level in both the longitudinal and transverse directions, a ductility factor of 3.0 has been assumed based on the relatively flexible, lightweight timber framed walls resisting lateral seismic loads.

At first floor level, lateral seismic forces were calculated in accordance with NZS 1170.5:2004. The lateral seismic forces have been distributed to the concrete masonry walls assuming that the double tee precast units with a reinforced topping at first floor level behave as a rigid diaphragm. An accidental eccentricity of 10% has been assumed in each direction. The structure is considered to be nominally ductile. As a result, 30% loading from the other orthogonal direction has been included when determining the loading on the masonry walls for an earthquake in a particular direction as per NZS 1170.5:2004 requirements.

A ductility factor of 1.25 has been assumed in both the longitudinal and transverse directions at ground floor level based on the reinforced concrete masonry wall system that resists lateral seismic loading. The structure is expected to have nominally ductile behavior given the relatively lightly reinforced concrete masonry construction.

7.3 Capacity of Structural Elements

7.3.1 Timber Framed Wall Bracing Capacity

The bracing capacity of the first floor timber framed walls in both the longitudinal and transverse directions was calculated in accordance with NZS 3604:2011 and the NZSEE guidelines. The demand for each building was calculated in accordance with NZS 1170.5:2004 and resolved into Bracing Units (BUs) for comparison.

There is no reliable information available regarding the bracing capacities of the chipboard lining to the timber framed walls. Assumptions regarding the likely bracing capacity of the chipboard lined timber walls have been made in accordance with Table 11.1 of the in NZSEE guidelines. A bracing capacity value of 3 kN/m (60 BU/m) and a strength reduction factor of 0.7 have been used in calculations.

Section 11.4 of the NZSEE guidelines suggests that shear panels may utilise their full bracing capacity for aspect ratios (height-to-width) up to 2:1. For aspect ratios greater than 2:1 and up to 3.5:1 a limiting factor may be applied in accordance with the NEHRP Recommended Provisions (BSSC, 2000) as follows;

$$\text{Aspect Ratio Factor} = \frac{2 \times \text{Width}}{\text{Height}}$$

Any sections of wall with an aspect ratio greater than 3.5:1 were not included in the bracing calculations.

The buildings were also checked against the current requirements in NZS 3604:2011 for spacing of bracing lines, minimum bracing line values, diaphragm spans and the bracing capacities of walls supporting diaphragms.

7.3.2 Reinforced Masonry Shear Capacity

The shear capacity of the reinforced concrete masonry walls was determined using NZS 4230:2004. As there are no details as to the level of supervision during the construction stage, an Observation Type of B was used in accordance with Table 3.1. The strength reduction factor, ϕ, for shear and shear friction was taken as 0.85 in accordance with NZSEE guidelines. The overall shear capacity of the wall was calculated from Clause 10.3.2.1, Equation 10-4.

For reinforced concrete 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

v_m = shear strength of masonry

b_w = t wall thickness when fully filled

d = 0.8 x 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

7.3.3 Reinforced Masonry In-Plane Moment Capacity

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

$$\emptyset M_n = \emptyset \left[\sum F_{si}(x_i - c) + C_m \left(c - \frac{a}{2} \right) + N \left(\frac{L_w}{2} - c \right) \right]$$

Where

$$\sum F_{si} - C_m + N = 0$$

F_{si} = tension or compression force in the vertical wall reinforcement

x_i = vertical reinforcing bar position

c = neutral axis depth

C_m = masonry compressive force

a = βc = masonry compression block parameter

N = axial load

7.3.4 Reinforced Masonry Out-of-Plane Moment Capacity

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

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

$$a = {}^{A_s f_{yt}} \big/ {}_{0.85 f'_m b}$$

Where

t = thickness of the masonry wall

b = unit width of wall

A_s = area of steel reinforcement

A_m = area of masonry

f'_m = specified compressive strength of masonry

f_y = the strength of steel as specified by the NZSEE guidelines

7.3.5 %NBS

The shear and moment capacities of the structural elements were compared to their respective demands to determine the overall %NBS for each element.

$$\%NBS = \frac{\emptyset V_n}{V^*} \text{ x } 100$$

$$\%NBS = \frac{\emptyset M_n}{M^*} \text{ x } 100$$

8. Results

The New Zealand Society for Earthquake Engineering (NZSEE) publication ‘Assessment & Improvement of Structural Performance of Buildings (2006)’ and the relevant New Zealand material standards were used to provide a framework and method for the analysis. Our analysis applied live loads, imposed dead loads and seismic loads to the structure. The elements were then assessed against their respective load capacities.

Our calculations show that the structure achieves **7% NBS** and is therefore **Earthquake Prone**.

The structural analysis results are discussed in the following sections.

8.1 Pavilion Building

8.1.1 Timber Framed Walls

The chipboard lined timber framed walls achieve **61% NBS** under in-plane shear seismic loading.

The bracing demand was determined by evaluating the seismic weight of the upper storey of the building and multiplying this value by the horizontal design action coefficient corresponding to the timber framed walls. The demand was then resolved into bracing units (BUs) for comparison with bracing capacities of timber framed walls.

The total bracing capacity of the building in the longitudinal direction was evaluated by determining the lengths of plasterboard lined timber framed walls available that satisfy the aspect ratio limit of 3.5:1 suggested in the NZSEE guidelines.

The timber framed wall bracing system for the upper floor of the building satisfies current NZS 3604:2011 requirements for minimum bracing line capacities, maximum spacing of bracing lines and ceiling diaphragm requirements.

8.1.2 Reinforced Concrete Masonry Walls

<u>In-Plane Shear</u>

The reinforced concrete masonry walls achieve **7% NBS** under in-plane shear seismic loading.

<u>In-Plane Moment</u>

The reinforced concrete masonry walls achieve **7% NBS** when considering in-plane bending of the walls.

The assessed in-plane shear and in-plane moment score is limited by the connection between the diaphragm at first floor level and the concrete masonry walls. The concrete masonry walls rely on friction between the web of the double tee units and the top of the concrete masonry wall to transfer seismic forces from the diaphragm to the concrete masonry walls.

<u>Out-of-Plane Moment</u>

The reinforced concrete masonry walls achieve **54% NBS** when considering out-of-plane bending of the walls.

The ground floor concrete masonry wall on the eastern side of the building (Wall 1, Figure 2) retains approximately 2.2m of fill. As a result, the out-of-plane demand on the wall during an earthquake is increased due to the combined lateral soil loading and out-of-plane inertia effects.

8.2 Garage

The garage directly adjacent to the south of the pavilion building has been assessed separately.

8.2.1 Reinforced Concrete Masonry Walls

In-Plane Shear

The reinforced concrete masonry walls achieve **100% NBS** under in-plane shear seismic loading.

In-Plane Moment

The reinforced concrete masonry walls achieve **100% NBS** when considering in-plane bending of the walls.

Out-of-Plane Moment

The reinforced concrete masonry walls achieve **82% NBS** when considering out-of-plane bending of the walls.

The lightweight timber framed garage roof is unlikely to restrain the concrete masonry walls out-of-plane. As a result, the concrete masonry walls in the garage have been assumed to resist out-of-plane loads through cantilever action.

8.3 Summary

<table>
<tr><th>Element</th><th>Seismic Action</th><th>%NBS</th></tr>
<tr><td colspan="3">First Floor</td></tr>
<tr><td colspan="3">Longitudinal Direction</td></tr>
<tr><td>Timber Framed Walls</td><td>In-Plane Shear</td><td>100</td></tr>
<tr><td colspan="3">Transverse Direction</td></tr>
<tr><td>Timber Framed Walls</td><td>In-Plane Shear</td><td>61</td></tr>
<tr><td colspan="3">Ground Floor</td></tr>
<tr><td colspan="3">Longitudinal Direction</td></tr>
<tr><td rowspan="3">Reinforced Concrete Masonry Walls</td><td>In-Plane Shear</td><td>81</td></tr>
<tr><td>In-Plane Bending</td><td>81</td></tr>
<tr><td>Out-of-Plane Bending</td><td>54</td></tr>
<tr><td colspan="3">Transverse Direction</td></tr>
<tr><td rowspan="3">Reinforced Concrete Masonry Walls</td><td>In-Plane Shear</td><td>7</td></tr>
<tr><td>In-Plane Bending</td><td>7</td></tr>
<tr><td>Out-of-Plane Bending</td><td>58</td></tr>
<tr><td colspan="3">Garage</td></tr>
<tr><td rowspan="3">Reinforced Concrete Masonry Walls</td><td>In-Plane Shear</td><td>100</td></tr>
<tr><td>In-Plane Bending</td><td>100</td></tr>
<tr><td>Out-of-Plane Bending</td><td>82</td></tr>
</table>

Table 5 Summary of %NBS scores

8.4 Discussion of Results

The results obtained from the analysis are reasonably consistent with those expected for a building of this age and construction type.

The building is assumed to have been designed in the early 1970s and was likely designed in accordance with the earlier loading standard, NZS 1900:1965. The design loads used are likely to have been less than those required by the current loading standard.

The critical aspect of the seismic assessment of the overall building is the performance of the connection between the diaphragm at first floor level and the ground floor masonry walls. Several of the concrete masonry walls rely on friction between the web of the double tee units and the top of the concrete masonry wall to transfer seismic forces from the diaphragm to the concrete masonry walls. As a result, the lateral load resisting capacity of these walls is severely limited by the strength of these connections.

9. Conclusions and Recommendations

The pavilion building has been assessed to have a seismic capacity in the order of 7% NBS and is therefore Earthquake Prone. The adjacent garage has been assessed to have a seismic capacity in the order of 82% NBS and is not Earthquake Prone or Earthquake Risk . It is recommended that Christchurch City Council investigate potential strengthening options for the pavilion building.

10. Limitations

10.1 General

This report has been prepared subject to the following limitations:

- The foundations of the building were unable to be inspected beyond those exposed above ground level externally.
- 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.

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

The advice tendered in this report is based on a visual geotechnical appraisal. No subsurface investigations have been conducted. An assessment of the topographical land features have been made based on this information. It is emphasised that geotechnical conditions may vary substantially across the site from where observations have been made. Subsurface conditions, including groundwater levels can change in a limited distance or time. In evaluation of this report cognisance should be taken of the limitations of this type of investigation.

An understanding of the geotechnical site conditions depends on the integration of many pieces of information, some regional, some site specific, some structure specific and some experienced based. Hence this report should not be altered, amended or abbreviated, issued in part and issued incomplete in any way without prior checking and approval by GHD. GHD accepts no responsibility for any circumstances, which arise from the issue of the report, which have been modified in any way as outlined above.

Appendix A
Photographs

Photograph 1 View of the building from the north-west

Photograph 2 Western elevation of the building

Photograph 3 Southern elevation of the building

Photograph 4 Interior of upper storey

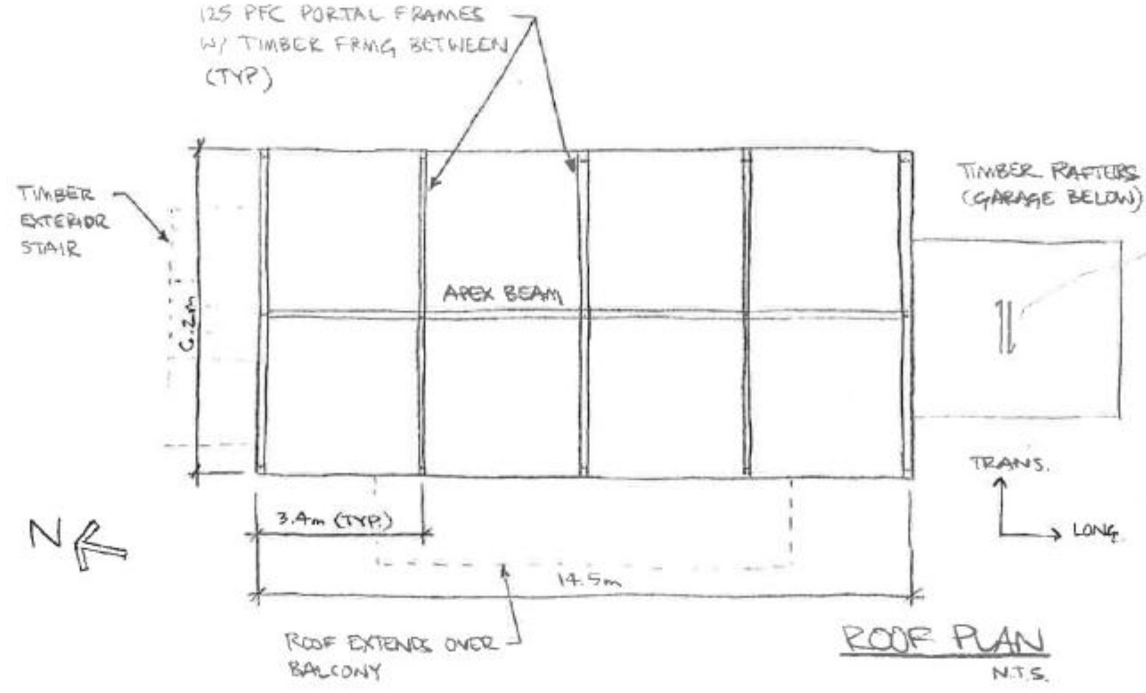
Photograph 5 Overhanging double tee units

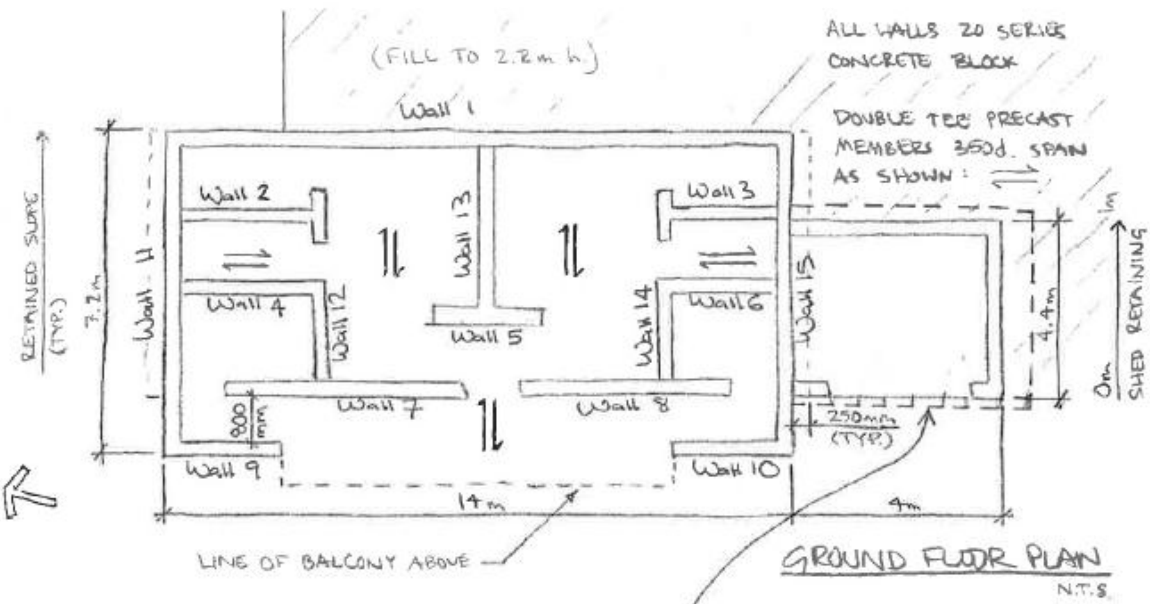
Photograph 6 Double tee webs bearing on the concrete masonry walls

Photograph 7 View of internal masonry walls and double tee units

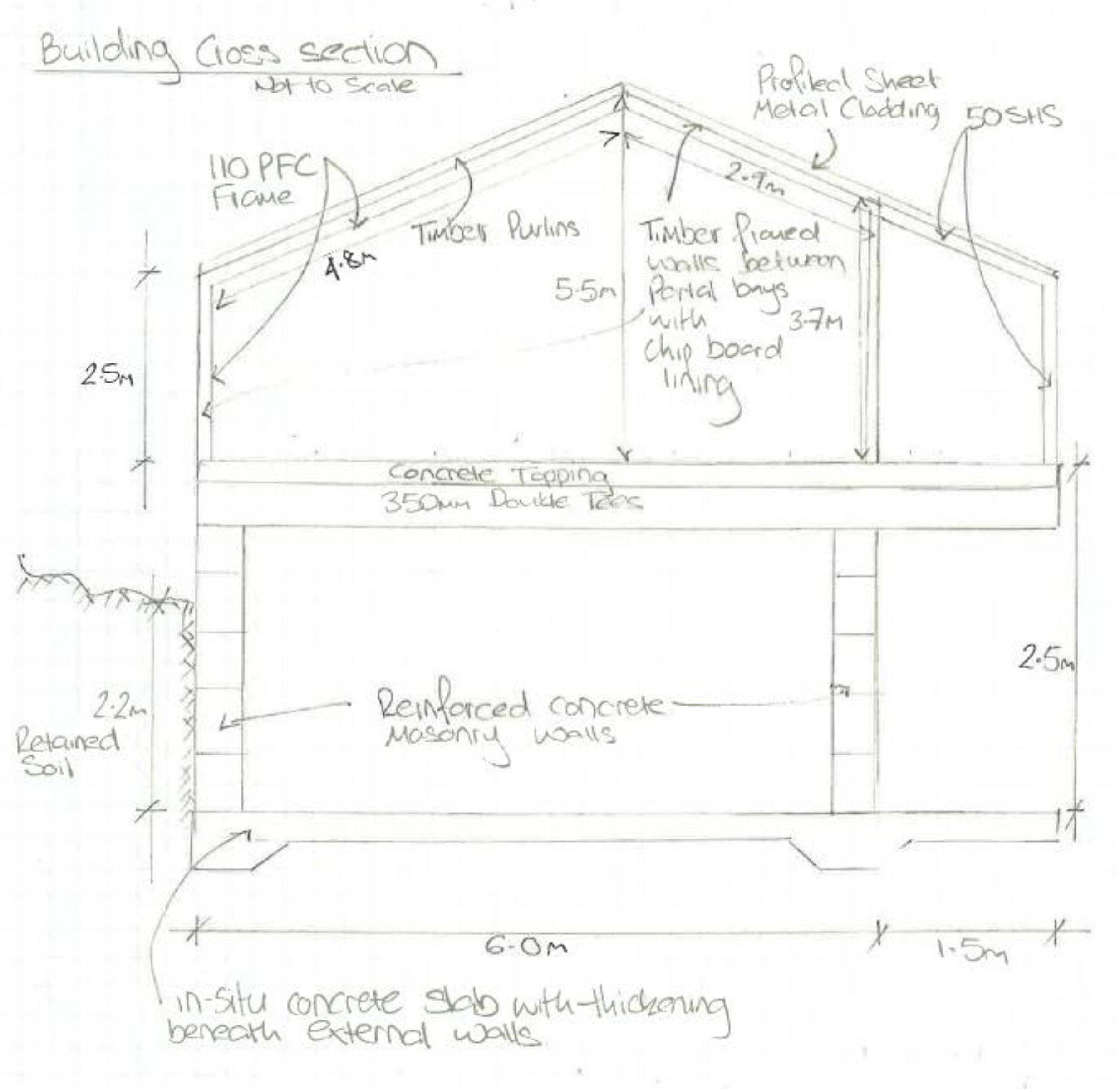
Photograph 8 Seperation between the pavilion building and adjacent garage

Appendix B
Sketches





Notes this section of the building has become completely seperated (i.e. No structural ties) from the main building.


Building Cross section
Not to Scale
Profiled Sheet Metal Cladding
50SHS
110 PFC Frame
Timber Purlins
Timber framed walls between portal bays with chip board lining
2.9m
4.8m
5.5m
3.7m
2.5m
Concrete Topping
350mm Double Tees
2.5m
2.2m
Retained Soil
Reinforced concrete Masonry walls
6.0m
1.5m
in-situ concrete slab with thickening beneath external walls

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Clare Park Toilet & Pavilion	Reviewer:	Stephen Lee
	Unit / No: 149 / Street	CPEng No:	1006840
Building Address:	149 Burwood Road	Company:	GHD
Legal Description:		Company project number:	51/30902/04
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:	43 / 29 / 25.48	Date of submission:	8/03/2013
GPS east:	172 / 40 / 59.11	Inspection Date:	31/01/2012
		Revision:	FINAL
Building Unique Identifier (CCC):	PRK 0104 BLDG 001-EQ2	Is there a full report with this summary?	yes

Site

Site slope:	slope < 1in 10	Max retaining height (m):	2.2
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	5.00

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	5.00
Ground floor split?	no		Ground floor elevation above ground (m):	0.00
Storeys below ground	0			
Foundation type:	strip footings		if Foundation type is other, describe:	
Building height (m):	8.00		height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	100			
Age of Building (years):	37		Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	other (specify)		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):	Public Toilet & Pavilion			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	precast concrete with topping	unit type and depth (mm), topping	Double Tee
Beams:			200 x 90
Columns:			
Walls:	partially filled concrete masonry	thickness (mm)	190

Lateral load resisting structure

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

Separations:

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

Non-structural elements

Stairs:	timber	describe supports	External concrete pads.
Wall cladding:	profiled metal	describe	
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
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:		Describe damage:	
Settlement:	0-25mm	notes (if applicable):	Front of building
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:		notes (if applicable):	
Lateral Spread:	0-50mm	notes (if applicable):	Movement of garage shed
Differential lateral spread:	0-1:400	notes (if applicable):	Movement of landing relative to stairs
Ground cracks:	0-20mm/20m	notes (if applicable):	Near stair
Damage to area:	slight	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Minor damage to the structure observed.
	Describe (summary):	Minor damage to the structure observed.		
Across	Damage ratio:	0%		
	Describe (summary):	Minor damage to the structure observed.		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	Strengthening recommended
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	54%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Detailed Assessment
	Assessed %NBS after e'quakes:	54%		
Across	Assessed %NBS before e'quakes:	7%	##### %NBS from IEP below	
	Assessed %NBS after e'quakes:	7%		

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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
FINAL	Alex Baylis	Stephen Lee		Nick Waddington		8/03/13