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Mona Vale Fernery

PRK 0555 BLDG 007 EQ2

Detailed Engineering Evaluation

Quantitative Report

Version FINAL

63 Fendalton Road, Fendalton

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

Mona Vale Fernery

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Detailed Engineering Evaluation
Quantitative Report
Version FINAL

63 Fendalton Road, Fendalton

Christchurch City Council

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Date
19 March 2013

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Quantitative Report Summary

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Detailed Engineering Evaluation

Quantitative Report - SUMMARY

Version FINAL

65 Fendalton Road, Fendalton

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 21 September 2012 and available drawings itemised in 5.2.

Building Description

The oval building consists of an unreinforced brick masonry perimeter wall and circular hollow section columns combining to support a netted pergola roof structure. There are reinforced concrete or arched unreinforced brick masonry buttresses on the southern and northern elevations respectively. The internal floor consists of a pond, a gravel pathway and fernery beds. The walls are supported on strip footings with the columns being supported on concrete pad foundations.

Key Damage Observed

Key damage observed includes:

- Demolished or collapsed unreinforced brick masonry wall previously connected to building with a brick arch.
- Minor cracking in joints at various locations. The mortar has been repointed in numerous locations around the structure.

Building Capacity Assessment

Based on the results of the quantitative assessment the building scored 1% NBS. Therefore the building is Earthquake Prone.

Recommendations

Currently the unreinforced brick arch buttresses on the main western elevation are failing with a %NBS of 1%. Design concepts should be undertaken to strengthen the structure.

1. Background

GHD has been engaged by Christchurch City Council (CCC) to undertake a detailed engineering evaluation of Mona Vale Fernery.

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 building is located at 63 Fendalton Road, Riccarton. The building was originally constructed shortly after 1907, with glazed sections originally forming the roof being taken from a Fernery (1906-1907) in Hagley Park. The sole use of the building remains a fernery.

The unreinforced brick masonry walls remain from the original construction. These consist of the oval perimeter wall and the arched buttresses on the northern elevations. The building was restored in the 1990's, with all the additions likely added at that time. A 300mm deep concrete ring beam was added to the top of the 220mm thick unreinforced brick masonry perimeter wall. The ring beam contains ø16mm main reinforcement in each corner with ø6mm links at 500mm centres. 430mm thick concrete buttresses, added to the southern elevations, consist of a separately poured mass concrete core and a reinforced concrete border. The approximately 450mm wide border contains ø16mm main reinforcement in each corner with ø10mm links at 300mm centres. There are 250mm x 300mm reinforced concrete posts extending from the ground up to the ring beam, the majority being found on the south-eastern elevation. The timber pergola roof structure is supported by the perimeter walls and internally by 105mm CHS (Circular Hollow Section) columns. The main pergola elements are 250mm x 75mm timber sections, with 150mm x 50mm spanning between these to support the 35mm x 25mm at 140mm centre battens. These timber roof sections support a mesh covering the entire roof structure. The internal floor consists of a pond, a gravel pathway and fernery beds. The CHS are supported from concrete pad foundations and the walls are supported from strip footings.

The building is approximately 24m in length by 18m in width with a footprint of 210m^2. The radius ends of the structure are formed from successive 30 degree turns in the perimeter wall. The internal ground level varies through-out, with it being approximately 4.55m to the roof structure from its lowest point. The adjacent unreinforced brick masonry boundary wall, which was connected to the structure by a brick arch, was found in rubble. The exact cause is unclear however the recent seismic activity would be considered a factor. The flat site is approximately 10m above sea level and is approximately 200m southwest of the Avon river. The fernery is located on a recreational reserve, with other facilities such as car-park and tennis courts being located nearby.

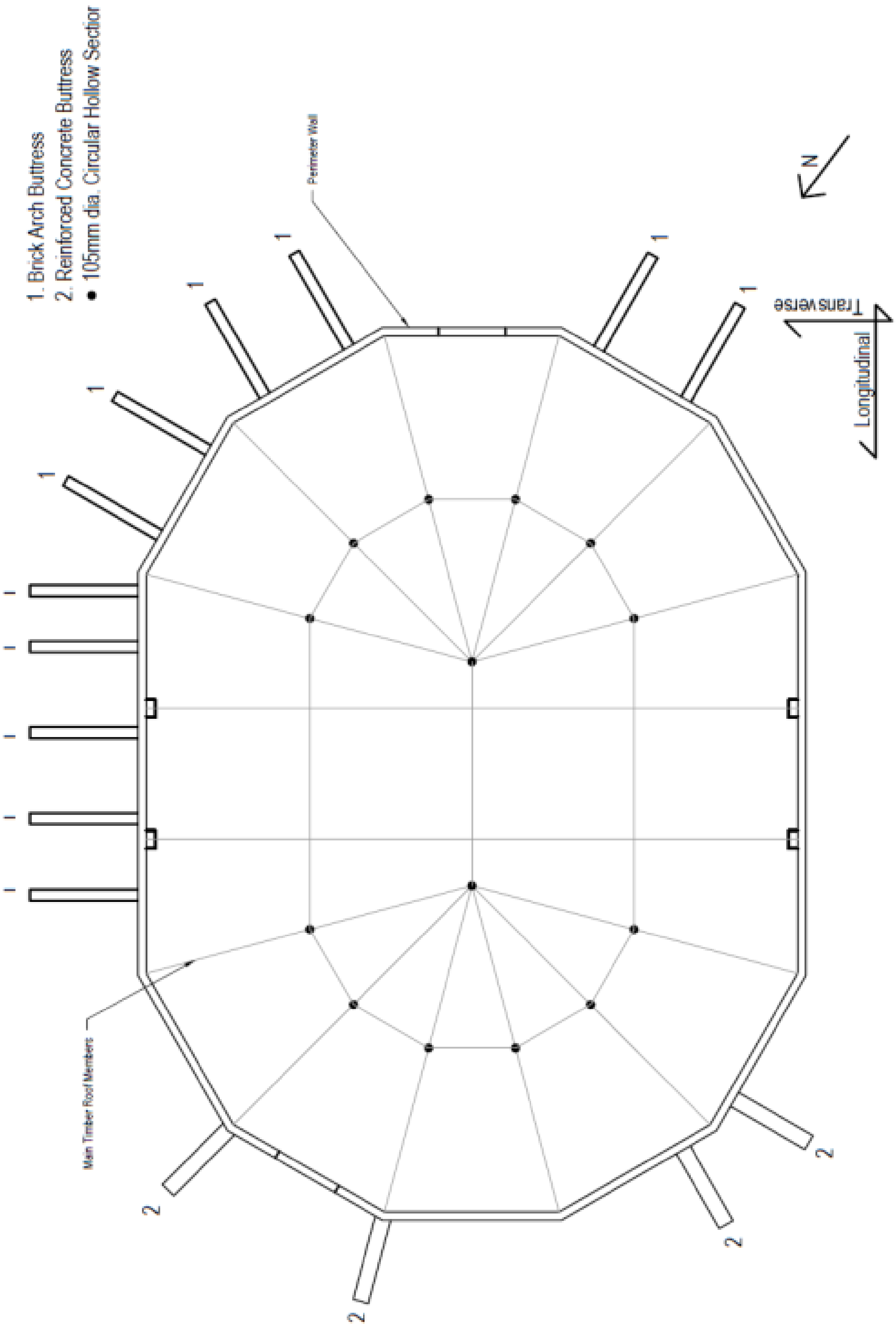


Figure 2 Plan of Structure

4.2 Gravity Load Resisting System

Gravity roof loads are transferred through the battens and timber members to the main timber roof sections. These main timber roof sections span from support provided by the perimeter unreinforced brick masonry wall or the internal Circular Hollow Section (CHS) columns. The CHS columns transfer the gravity loads down to the pad foundations where they distribute into the ground beneath. Similarly, the unreinforced brick masonry walls transfer the gravity loads from the roof downwards to the strip footings beneath.

4.3 Lateral Load Resisting System

There is no diaphragm or braced action in the roof due to the absence of sheathing and diagonal members respectively. The roof structure is formed from timber members which aren't rigidly connected. The timber roof members are all connected to the perimeter wall in a perpendicular orientation, resulting in all lateral loads from the roof being applied to walls in the out of plane direction. Where out-of-plane walls are buttressed, all the lateral roof loads, along with those from the adjacent wall panels, are expected to be restrained by the buttresses. These adjacent unreinforced brick masonry wall panels span vertically from the ground to the reinforced concrete ring beam which in turn, span horizontally between buttresses. Where unreinforced brick masonry panels have no buttressing, the roof is not expected to transfer restraint from other buttress locations, leaving the panels effectively free-standing. Wall panels and buttresses resist lateral in-plane loads by panel action. CHS columns are not expected to provide any lateral restraint given the lack of base fixity.

5. Assessment

5.1 Site Inspection

An inspection of the building was undertaken on the 21st of September 2012. Both the interior and exterior of the building were inspected. The main structural components of the building 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 elements in the building. In the case of conflicting results, the most conservative bar diameter was chosen for the capacity calculations.

5.2 Available Drawings

No construction drawings were made available.

6. Damage Assessment

6.1 Surrounding Buildings

The adjacent unreinforced brick masonry boundary wall, which was connected to the structure by a brick arch, was found in rubble. The exact cause is unclear, however the recent seismic activity would be considered a factor.

Significant damage was sustained to other unreinforced brick structures on the site. These buildings remain closed with temporary propping installed to various brick elements.

6.2 Residual Displacements and General Observations

There were no settlement issues identified during the inspection of the structure. There was minor cracking noted in the unreinforced brick masonry joints. The mortar has been repointed in numerous locations. While the cracking may be a result of recent seismic activity, it may also be attributed to the age and type of structure.

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)	1.0
Performance Factor (Sp), based on NZS 3.1.0.1	1.0
Gravitational Constant (g)	9.81 m/s^2

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

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:

$$C_d(T_1)=\frac{C(T_1)\cdot S_p}{k_\mu}=\frac{0.90\cdot 1.0}{1.0}=0.90$$

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

7.3 Dependable Capacity

7.3.1 Concrete Beam Shear Capacity

The shear capacity of the concrete beam was calculated using Section 9.3 of NZS 3101:2006.

Shear links are present in the beam, however the spacing is too large to be effective, hence only the shear capacity of the concrete was used.

This shear capacity of the concrete, V_c, was based on the following equations:

$$V_c = v_c A_{cv}$$

$$v_c = k_d k_a v_b$$

$$v_b = (0.07 + p_w)\sqrt{f'_c}$$

$$p_w = A_s/bd$$

Where

A_{cv} = shear area of beam;

k_d = 1.0 for current beam dimensions;

k_a = 1.0 for 20mm aggregate size;

f'_c = concrete strength;

A_s = area of tension reinforcement;

b = width of beam;

d = depth of beam.

7.3.2 Reinforced Concrete Beam Moment Capacity

The following method was used to calculate the moment capacity of the reinforce concrete beam.

$$\emptyset M_n = \emptyset(d - c - a) f_{yt} A_s$$

$$a = {}^{A_s f_{yt}}\!\big/_{\emptyset b f'_c}$$

Where

d = depth of beam

c = cover to reinforcement

A_s = area steel

f_{yt} = steel strength

b = width of beam

f'_c = concrete strength

7.3.3 Unreinforced Brick Masonry Capacity

The capacities of the unreinforced brick masonry were calculated in accordance with NZSEE guideline.

8. Geotechnical Consideration

8.1 Site Description

The site is situated within a recreational reserve, within the inner suburb of Fendalton, Christchurch. It is relatively flat at approximately 10m above mean sea level. It is approximately 60m west and 100m south of the Avon River, and 12km west of the coast (Pegasus Bay).

8.2 Public Information on Ground Conditions

8.2.1 Published Geology

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

8.2.2 Environmental Canterbury Logs

Information from Environment Canterbury (ECan) indicates numerous boreholes are located within a 200m radius of the site. Six boreholes are located within 100m, of these boreholes, four were considered to have lithographic logs (see Table 2). However, only one of these logs recorded the lithographic layers between 1.5 and 21m bgl. The remaining log indicate the site geology to be a surface layer of clays, sands and grit underlain by stratified gravel and peat/clay layers.

Table 2 ECan Borehole Summary

Bore Name	Log Depth	Groundwater	Distance & Direction from Site
M35/2430	~55.4m	~1.6m bgl	~97m SW
M35/2096	~55.1m	~1.4m bgl	~55m NW
M35/2056	~55.7m	~1.9m bgl	~95m SSE
M35/12953	~1.52m	N/A	~100m S

It should be noted that the purpose of the boreholes the well logs are associated with, 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.

8.2.3 EQC Geotechnical Investigation

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 Fendalton[2]. One investigation point was undertaken approximately 250m east of the site, as summarised below in Table 3..

Table 3 EQC Geotechnical Investigation Summary Table

Bore Name	Grid Reference	Log Summary	
CPT-FND-01	2478670 mE 5742558 mN	0 – 1	Surface Soil
		1 – 2.5	SAND, with minor silt; medium dense to dense
		2.5 – 8	Silty CLAY and clayey SILT
		8 – 9	SAND, with minor silt and clay; medium dense
		9 – 10	CLAY
		10 – 17.7	Silty SAND and SAND; medium dense to dense
			(WT at 1.5m bgl)

Initial observations of the CPT results indicate the soils are layers of sand, silt and clay, of varying density and strength.

8.2.4 Additional Geotechnical Investigations

In addition to the above intrusive geotechnical information, one CPT was undertaken on 02 April 2012 in the In order to get better understanding of soil conditions. The testing location was approximately 100m northeast of the fernery. The testing results are summarised in Table 4

Table 4 Summary of CPT undertaken on 02 April 2012-Inferred Lithology

Bore Name	Grid Reference	Depth (m)	Lithology
CPT-29	2478451 mE 5742533 mN	0 – 1.5	Peat/Clay/Silty Clay
		1.5 – 5.0	Sandy silt / Silty sand
		5.0 – 12.5	Clay/Sensitive fine grained soil
		12.5 –18.0	Sand/Silty Sand

8.2.5 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.

The site itself is a non-residential properties in urban area of Christchurch (but within the Green Zone), therefore it has not been given a CERA land zoning technical category. However, a property to the west

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

of the subject building has been given a technical category of TC2 (yellow) – CERA indicates that this means that future land damage from liquefaction is unlikely and the standard foundations for concrete slabs or timber floors can be used

Canterbury Earthquake Recovery Authority (CERA) has indicated that these means that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

8.2.6 Post February Aerial Photography

Aerial photography taken following the 22 February 2011 earthquake shows signs of moderate liquefaction outside the building footprint, and significant liquefaction in the Christchurch Girls' High School grounds adjacent to the site, as shown in Figure 3**.**

Figure 3 Post February 2011 Earthquake Aerial Photography3

8.2.7 Summary of Ground Conditions

From the information presented above, the ground conditions have been found to comprise strata of sand, silt and clay to ~18m bgl, underlain by layers of sand and gravel, and peat and clay.

8.3 Seismicity

8.3.1 Nearby Faults

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

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

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

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	20 km	W	7.1	~15,000 years
Hope Fault	100 km	N	7.2~7.5	120~200 years
Kelly Fault	100 km	NW	7.2	150 years
Porters Pass Fault	60 km	NW	7.0	1100 years

Recent earthquakes since 22 February 2011 have identified the presence of a previously unmapped active fault system underneath Christchurch City and the Port Hills. Research and published information on this system is in development and not generally available. Average recurrence intervals are yet to be estimated.

This 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.

8.3.2 Ground Shaking Hazard

This 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.

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.

In addition, the geology is anticipated to be stratified alluvial deposits of varying density, and the site has a 475-year PGA (peak ground acceleration) of ~0.4 (Stirling et al, 20024). Bedrock is anticipated to be in excess of 500m deep, and hence ground shaking is expected to be moderate to high.

8.4 Slope Failure and/or Rockfall Potential

The site is located in the typically flat suburb of Fendalton. Global slope instability is considered negligible. However, any localised retaining structures and/or embankments should be further investigated to determine the site-specific slope instability potential.

In addition, the site is located adjacent to the Avon River. Should significant liquefaction occur in the area, it is considered possible and likely that lateral spreading will occur along the river. This is reinforced by evidence of lateral spreading in nearby areas following the February earthquake.

[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

8.5 Liquefaction Potential

Due to the presence of sands and silts, and evidence from the post-earthquake aerial photography, it is considered highly possible that liquefaction will occur in layers and locations where sands and silts are present.

8.6 Conclusions & Recommendations

This assessment is based on a review of the geology and existing ground investigation information, and observations from the Christchurch earthquakes since 4 September 2010, as well as on the results based on the piezo CPT site test undertaken on 02 April 2012.

The site appears to be situated on stratified alluvial deposits, comprising sand and silt. Associated with this the site also has a moderate to severe liquefaction potential, in particular where sands and/or silts are present.

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

9. Results of Analysis

The elements in the structure were considered individually and separately. The structure can be broken down into wall panels which are most definable by the support conditions and loads applied. As the structure is not completely orthogonal, the longitudinal and transverse directions are mainly used to deal with the lateral restraint to the roof.

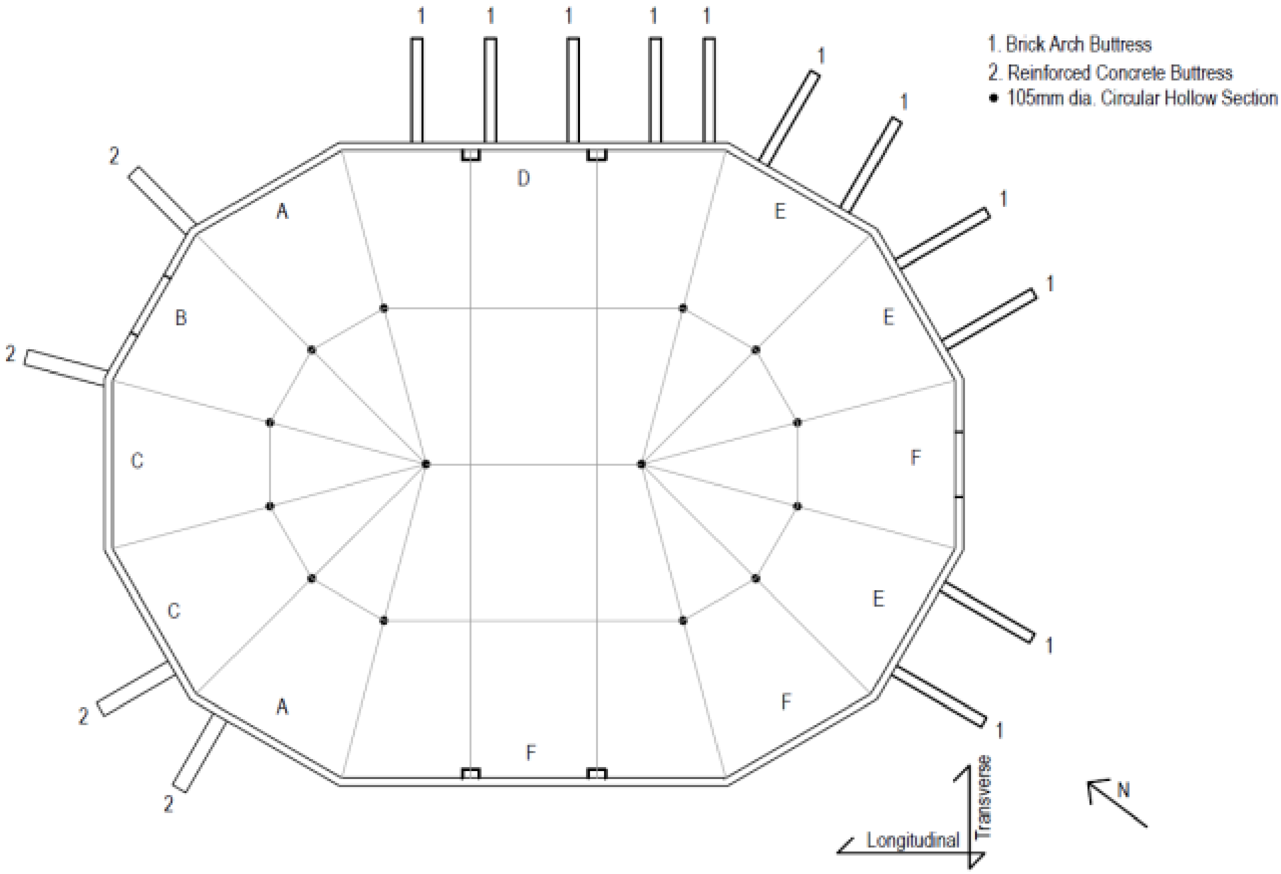


Figure 4 Plan showing wall panels as considered.

All panels (A/B/C) on the north-western end of the structure is expected to span vertically from the ground to the reinforced concrete rind beam. The reinforced concrete ring beam is expected to span horizontally for the width of the single panel (A/B) except where it spans for the width of two panels (C) given the larger spacing of the reinforced concrete buttresses. The capacity of the reinforced concrete buttresses' were checked for the combined demand from the unreinforced brick masonry panels and the timber pergola roof structure.

Table 6 %NBS of Structural Elements (North-western)

Element	% NBS
Panel A/B/C spanning vertically	36%
Ring Beam A/B	100%
Ring Beam C	37%
Buttresses for Longitudinal Load	43%

In the transverse direction, the UBAB (unreinforced brick arch buttresses) provide restraint to the timber pergola roof structure and the adjacent unreinforced brick wall panel (D). The locations of the UBAB do not require the reinforced concrete ring beam to distribute loads from the wall, given their central locations. In other locations, the UBABs will provide restraint to the adjacent panel (E) only. Where there is no buttressing to wall panels (F), they are expected to behave as free standing walls. Similarly, in the out-of-plane direction the UBAB, as found at panels D/E, are to behave as free standing walls.

Table 7 %NBS of Structural Elements

Element	% NBS
Panel D	1%
Panel E	2%
Panel F	15%
UBAB free standing	18%

It could be expected that the reinforced concrete buttresses on the north-western end of the structure could provide some lateral restraint to the roof structure in the transverse direction, however the absence of similar lateral restraint at the opposite (south-eastern) end of the structure precludes this.

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 Mona Vale Fernery was designed in 1907 and was likely designed without any design codes or seismic considerations in mind. The additions provided in the 1990's restoration provided for a more robust structure, however it did not deal with the demand of a design seismic event adequately.

Elements of the structure were found to have varying performance in regard of %NBS, with the controlling value being found to be 1%NBS for Wall Panel D. The low %NBS score is due to the high demand on this element from the roof structure and panel, and the poor performance of unreinforced masonry in a seismic event. Buttressing at Wall Panel D was found to be the only lateral restraint for the roof structure in the transverse direction.

10. Conclusions and Recommendations

The building overall has been assessed as having a seismic capacity of 1% NBS and is therefore classified as being ‘Earthquake Prone’.

Currently the building is found to be Earthquake Prone, as such it would be advised to proceed with strengthening concepts.

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

Photo 1. North-western Elevation.

Photo 2. Interior of structure.

Photo 3. Posts supporting pergola roof structure.

Photo 4. Interior of perimeter wall.

Photo 5. Reinforced concrete buttress..

Photo 6. Unreinforced Brick Arch Buttress.

Photo 7. Reinforced Concrete Ring Beam.

Photo 8. Pad Foundation for Post.

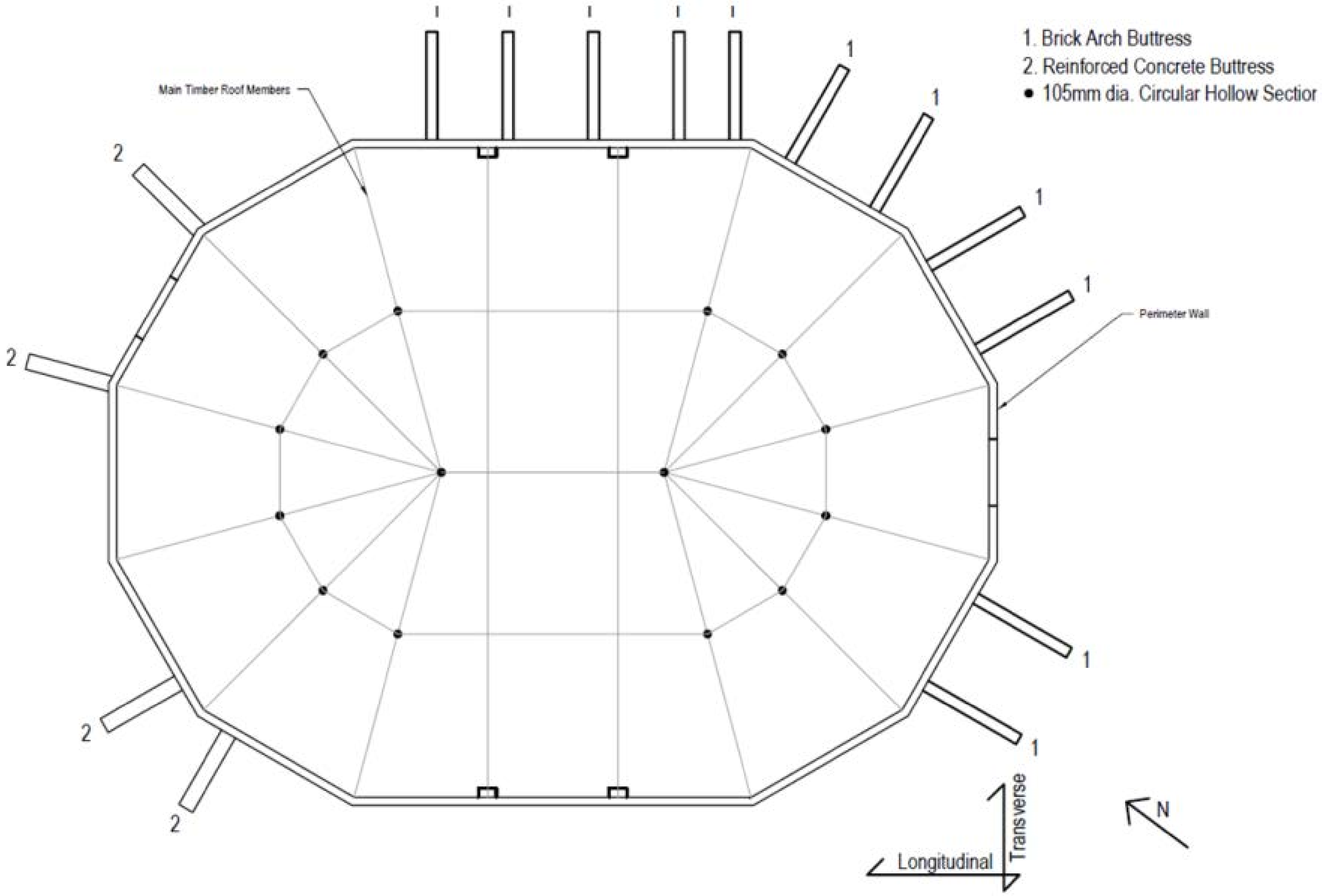
Photo 9.   Repointed Unreinforced Brick Masonry Joints.

Photo 10. Collapsed Boundary Unreinforced Brick Masonry Wall adjacent to structure.

Photo 11. Brick arch previously connecting the boundary wall to structure.

Appendix B

Sketches


1. Brick Arch Buttress
2. Reinforced Concrete Buttress
● 105mm dia. Circular Hollow Section
Main Timber Roof Members
Perimeter Wall
1
2
Longitudinal
Transverse
N

Appendix C

CERA Form

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Mona Vale Fernery	Reviewer:	Stephen Lee
	Unit / No: / Street	CPEng No:	1006840
Building Address:	63 Fendalton Road	Company:	GHD
Legal Description:		Company project number:	
		Company phone number:	04 472 0799
	Degrees / Min / Sec		
GPS south:		Date of submission:	11/14/2012
GPS east:		Inspection Date:	9/21/2012
		Revision:	
Building Unique Identifier (CCC):	PRK 0555 BLDG 007 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	silt	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):
Ground floor split?			Ground floor elevation above ground (m):
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	and pads
Building height (m):	4.55	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	210		
Age of Building (years):	105	Date of design:	Pre 1935
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):		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 framed	rafter type, purlin type and cladding	
Floors:			
Beams:			
Columns:	structural steel	typical dimensions (mm x mm)	
Walls:	load bearing brick	#N/A	

Lateral load resisting structure

Lateral system along:	other (note)	**Note: Define along and across in detailed report!**		Buttresses - RC or URBM
Ductility assumed, μ:	1.00		describe system	
Period along:	0.40	0.00	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)			Buttresses - RC or URBM
Ductility assumed, μ:	1.00		describe system	
Period across:	0.40	0.00	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

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

Available documentation

Architectural	full	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:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:			
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:		Describe:	
CSWs:	Damage?:		Describe:	
Pounding:	Damage?:		Describe:	
Non-structural:	Damage?:		Describe:	

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

Recommendations

	Level of repair/strengthening required:		Describe:	
	Building Consent required:		Describe:	
	Interim occupancy recommendations:		Describe:	
Along	Assessed %NBS before e'quakes:	1%	#### %NBS from IEP below	If IEP not used, please detail assessment methodology: Detailed Calculations
	Assessed %NBS after e'quakes:	1%		
Across	Assessed %NBS before e'quakes:	1%	#### %NBS from IEP below	
	Assessed %NBS after e'quakes:	1%		

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): Pre 1935

h_n from above: m

Seismic Zone, if designed between 1965 and 1992:

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:
Z_{1992}, from NZS4203:1992
Hazard scaling factor, **Factor B:** #DIV/0!

2.4 Return Period Scaling Factor

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

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.00	1.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity Scaling Factor, **Factor D**:	0.00	0.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	1.000	1.000
Structural Performance Scaling Factor **Factor E**:	1	1

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

	along	across
$\%NBS_b$:	**#DIV/0!**	**#DIV/0!**

Global Critical Structural Weaknesses: (refer to NZSEE IEP Table 3.4)

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 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 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

3.6. Other factors, Factor F

	Along	Across
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!**

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Document Status

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