Fern House, Botanic Gardens
PRK 1566 BLDG 018 EQ2

Detailed Engineering Evaluation (Rev R2)

Quantitative Assessment Report

Christchurch City Council

Fern House
Detailed Engineering Evaluation

Quantitative Assessment Report

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Date: February 2014

Reference: 6-QUCCC.40
Status: R2

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Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

Hagley Park, Botanic Gardens, Christchurch

Background

This is a summary of the quantitative report for the building structure at Hagley Park, Botanic Gardens (Fern House), and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 09 March 2011 and November 2012, available drawings and calculations.

Key Damage Observed

Key damage observed includes:-

- No damage to the building structure was identified during the damage survey on November 2011.
- There are no surrounding buildings to consider within immediate proximity of the structure.
- No evidence of ground damage or surface expression of liquefaction was visible in the immediate vicinity of the building, and no surface expression was observed elsewhere on the site.
- No signs of settlement have been observed in the floor or walls of the building. This is consistent with the observations of adjacent buildings.
- The form and depth of the foundations is unknown, however it is expected that the building is supported on shallow concrete strip footings which are assumed to be undamaged.

Critical Structural Weaknesses

a) No critical structural weaknesses have been identified in either the qualitative or quantitative assessments.

Indicative Building Strength (from quantitative assessment)

Based on the current strengthening of the existing wall plate, and from undertaking a quantitative assessment, the building's post-earthquake capacity is calculated as >67% NBS.

Recommendations

It is recommended that:

a) A strengthening works scheme be developed to increase the seismic capacity of the building to at least 67% NBS, this will need to consider compliance with accessibility and fire requirements.

b) An inspection of the timber wall plate to head of concrete wall connection is conducted.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic evaluation of Fern House, located in the Christchurch Botanic Gardens, following the M6.3 Christchurch earthquake on 22 February 2011.

This report follows on from the qualitative assessment report produced in February 2012 which was undertaken to ascertain an initial capacity assessment using a desktop study. The results concluded that the building is potentially earthquake prone.

The purpose of which is to determine if the building is classed as being earthquake prone in accordance with the Building Act 2004.

The seismic evaluation and reporting have been undertaken based on the quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) 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 to carry out a full structural survey before the building is re-occupied.

We understand that CERA 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). CERA have adopted the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

It is anticipated that a number of factors, including the following, will determine the extent of evaluation and strengthening level required:

1. The importance level and occupancy of the building.

2. The placard status and amount of damage.

3. The age and structural type of the building.

4. Consideration of any critical structural weaknesses.

Any building with a capacity of less than 34% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

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 the 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)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by CCC as being 67% of the strength of an equivalent new building. This is also the minimum level recommended by the New Zealand Society for Earthquake Engineering (NZSEE).

Section 121 – Dangerous Buildings

This section was extended by the Canterbury Earthquake (Building Act) Order 2010, and defines a building as dangerous if:

1. 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
2. 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
3. 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

4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. 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 (EPB) 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 loads 33% of those 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 on 4 September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;

2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;

3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,

4. 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.

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.

On 19 May 2011, Compliance Document B1: Structure was amended to include increased seismic design requirements for Canterbury as follows:

- 36% increase in the basic seismic design load for Christchurch (Z factor increased from 0.22 to 0.3);
- Increased serviceability requirements.

2.5 Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics

One of the core ethical values of professional engineers in New Zealand is the protection of life and safeguarding of people. The IPENZ Code of Ethics requires that:

Members shall recognise the need to protect life and to safeguard people, and in their engineering activities shall act to address this need.

1.1 Giving Priority to the safety and well-being of the community and having regard to this principle in assessing obligations to clients, employers and colleagues.

1.2 Ensuring that responsible steps are taken to minimise the risk of loss of life, injury or suffering which may result from your engineering activities, either directly or indirectly.

All recommendations on building occupancy and access must be made with these fundamental obligations in mind.

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 loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 below.

Description	Grade	Risk	%NBS	Existing Building Structural Performance
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)

Improvement of Structural Performance	
Legal Requirement	NZSEE Recommendation
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
	Not recommended. Acceptable only in exceptional circumstances
Unacceptable	Unacceptable

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

Table 1 below 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

3.1 Minimum and Recommended Standards

Based on governing policy and recent observations, Opus makes the following general recommendations:

3.1.1 Occupancy

- The Canterbury Earthquake Order[i] in Council 16 September 2010, modified the meaning of “dangerous building” to include buildings that were identified as being EPB’s. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

- Where there is an overhead falling hazard, or potential collapse hazard of the building, the areas of concern should be cordoned off in accordance with current CERA/Christchurch City Council guidelines.

3.1.3 Strengthening

- Industry guidelines (NZSEE 2006 [2]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A strengthening solution to anything less than 67%NBS would not provide an adequate reduction to the level of risk.

- It should be noted that full compliance with the current building code requires building strength of 100%NBS.

3.1.4 Our Ethical Obligation

- In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

[i] This Order only applies to buildings within the Christchurch City, Selwyn District and Waimakariri District Councils authority

4 Background Information

4.1 General

Fern House was constructed in 1955 to serve as a conservatory to house New Zealand ferns for the Hagley Park Botanic Gardens. The building is located in Hagley Park within the Botanic Gardens. Refer to site location plan in Figure 2 below.

Access to the house is from the path to the South, which runs along the North side of Townend House, through double doors in the South elevation.

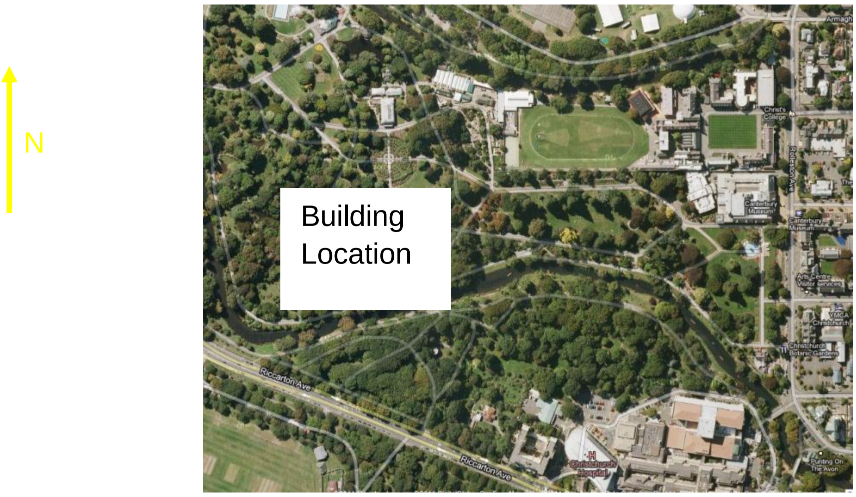


Figure 2 - Site Location Plan

The building is in the form of a cross on plan and the super-structure consists of two parts.

A steel arch roof springs off the top of the reinforced concrete perimeter walls. The complex roof is then formed with angle struts off the arches supporting flat roof and sloping timber joists and lantern light at the centre. The lantern light is fully glazed and the sloping sections are covered with corrugated sheets with areas of translucent corrugated sheets.

No inspection of the foundations has been carried out however; it has been assumed that for this structure simple spread foundations have been provided to the external wall perimeter.

4.2 Gravity Load Resisting System

An overview of the existing gravity load system has been described below:

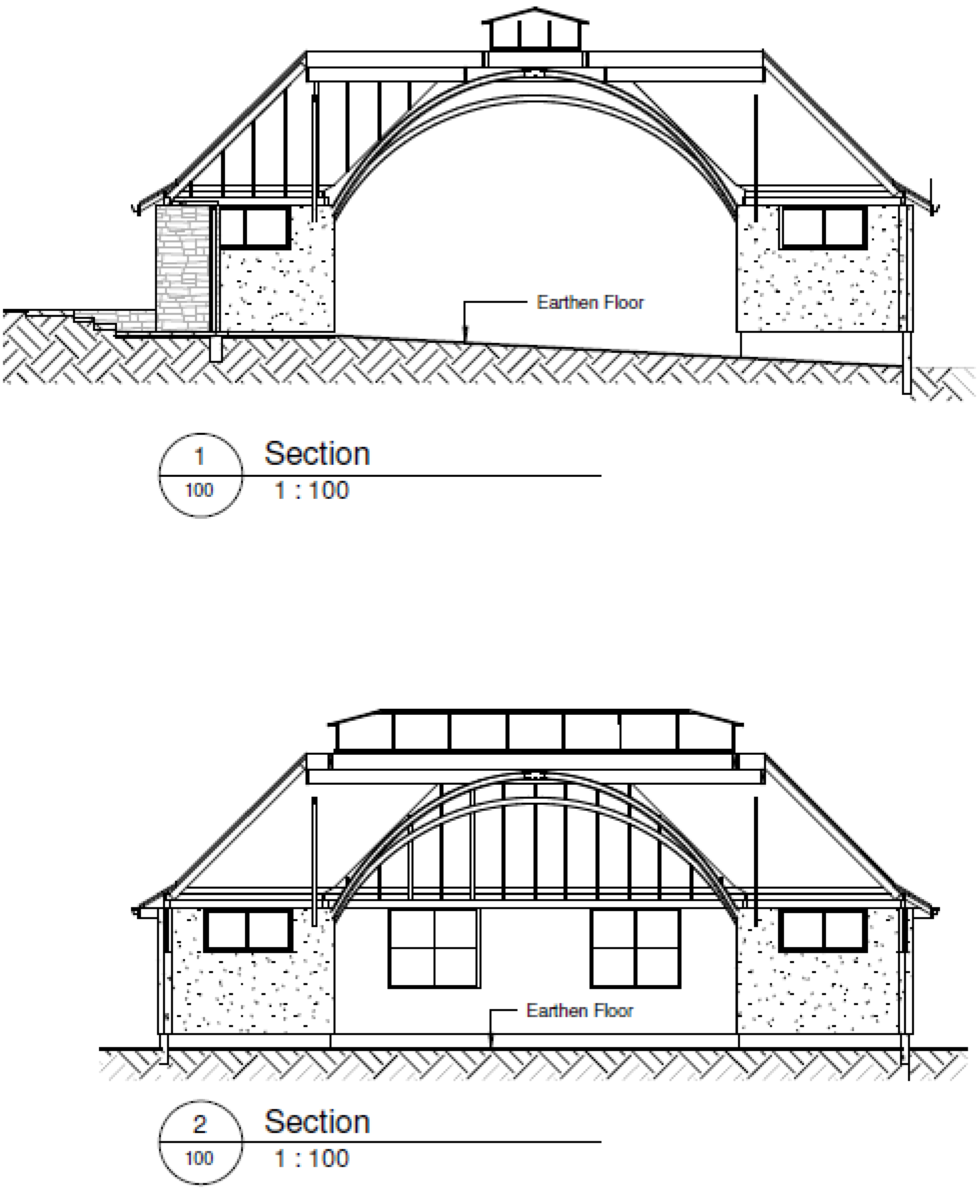


Figure 3 – Fern House typical sections through glasshouse

Glass and light steel structure high level lantern light roof running east west is supported by timber joist ring beams which in turn are supported by angle struts off semi-circular steel arches.

The flat roof areas to the north and south of the lantern light are formed with timber joists spanning onto timber ring beams which in turn are supported by angle struts off the steel arches.

The roof slopes are formed with timber rafters which span between eaves wallplate and the high level timber ring beam. The roof is hipped at the outer corners and forms valleys at the internal corners. The vertical loads are transferred to the perimeter walls at eaves level from the steel arches through the upper timber ring beam.

There are six steel arches, four of which spring from the internal corners of the perimeter walls parallel to each elevation while the remaining two spring from the internal wall corners but cross diagonally over the centre of the building to the opposite corner. These diagonal arches meet at the centre of the building and utilise a fabricated box section as the apex connection. Buttress action through the perimeter walls supports the loads from the arches and transfers them to the foundations and ground.

The details of the foundations are not known as no investigation has been carried out.

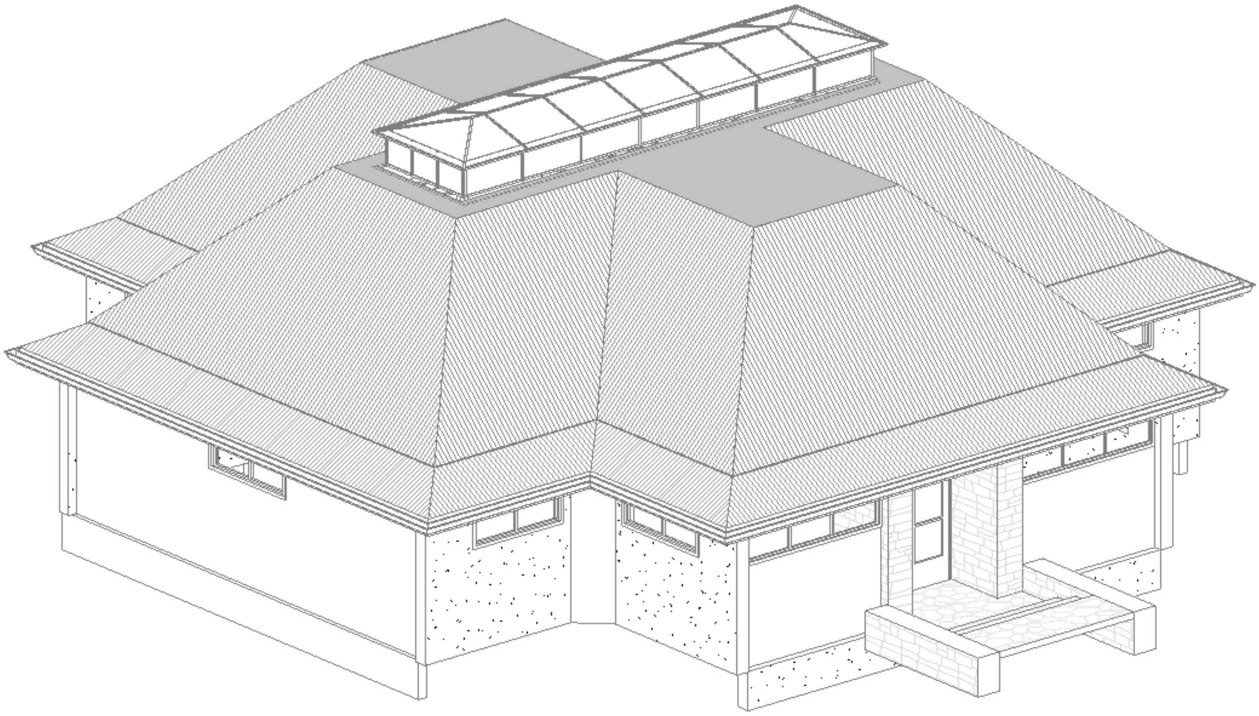
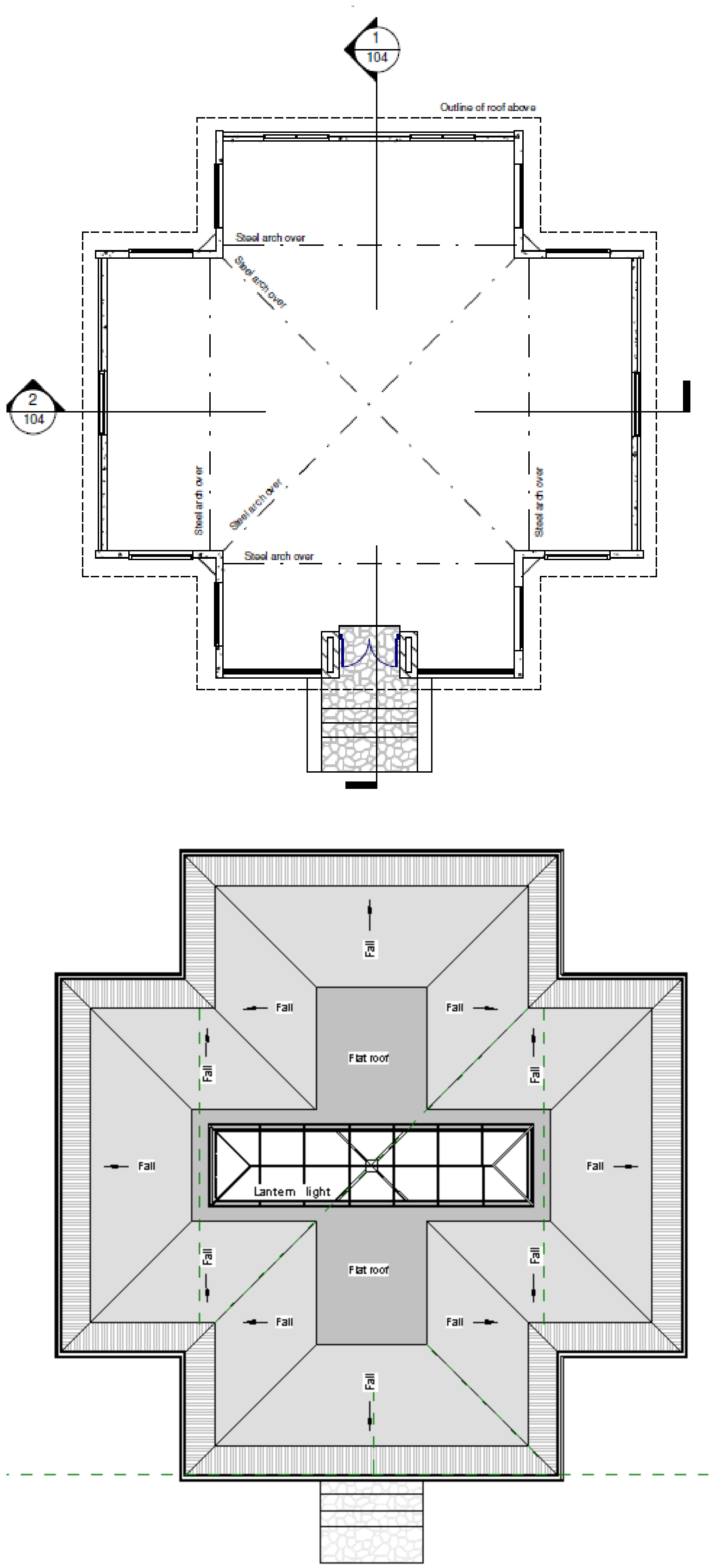


Figure 4 – Fern House 3D view



Figure

5 – General arrangement floor and roof plan Fern House

4.3 Seismic Load Resisting System

4.3.1 East to West and North to South Directions

Horizontal loads imposed on the upper roof structure are transferred through diaphragm action of the roof coverings and the timber rafters on the slopes to the steel arches and the top of the perimeter concrete walls. The hips and valleys will also stiffen the roof structure in the absence of any direct diagonal bracing.

The thrusts from the steel arches will be transferred to ground by the shear action of the perimeter walls buttressing the arches. The lateral load on the perimeter walls between the buttresses will be supported by cantilever action of the wall down to the foundations.

4.4 Survey

4.4.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of the building was carried out on 9th March 2011 by Opus International Consultants Limited. These inspections included external and internal visual inspections of all the structural elements only, without the benefit of any opening up works.

The site was graded as a G1 placard on 9th March 2011.

4.4.2 Further Inspections

A damage survey was conducted in November 2011 by Opus International Consultants Limited, refer to section 6 and Appendix A (photographs) of the Qualitative Report.

4.5 Original Documentation

Drawings of the structure were not made available.

5 Structural Damage

The following damage has been noted:

5.1 Surrounding Buildings

No buildings are within immediate proximity of Fern House

5.2 Residual Displacements

No evidence of ground damage or surface expression of liquefaction was visible in the immediate vicinity of the building, and no surface expression was observed elsewhere on the site. No signs of settlement have been observed in the floor or walls of the building. This is consistent with the observations of adjacent buildings.

5.3 Foundations

The form and depth of the foundations is unknown, however it is expected that the building is supported on shallow concrete strip footings – assumed to be undamaged.

6 General Observations

The general condition of the building appears to be reasonable considering the age. Detailed Seismic Assessment

7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011.

7.1 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix 3 of the report due to the technical nature of the content. A brief summary follows:

An assessment of the building structural element capacities was made based on the actions determined by equivalent static forces established from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1).

7.2 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component of a building that could contribute to increased levels of damage or cause premature collapse of a building.

No singular component or structural system forming the building structure has been considered a Critical Structural Weakness throughout the qualitative and quantitative assessments.

7.3 Limitations and Assumptions in Results

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity.
- Assessments of material strengths based on limited drawings, specifications and site inspections
- The normal variation in material properties which change from batch to batch.
- Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

7.4 Quantitative Assessment

A summary of the structural performance of the building is shown in the following tables. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements.

Table 2: Summary of Seismic Performance – Fern House

Structural Element/System	Failure Mode, or description of limiting criteria based on displacement capacity of critical element.	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity (ULS)
Timber hip/ valley members	Member buckling due to axial and bending force. Ductility factor, μ = 1.25.	No	>100%
Reinforced concrete perimeter walls	In-plane capacity governed by shear strength of walls, μ = 1.25.	No	>100%
Timber wall plate to head of reinforced concrete wall connection	Loss of vertical and lateral support of roof structure through differential movement between wall and wall plate.	No	50%

7.5 Discussion

The timber hip and valley members that provide a gravity system and brace the roof structure against lateral forces are robustly framed into the ring beams which form the high level roof light and flat roof areas. The members themselves have adequate axial and bending capacities and are fully restrained against buckling along their lengths by the timber rafters. This combined results in a %NBS rating in excess of 100% (ULS).

The reinforced concrete perimeter walls possess sufficient shear capacity to resist in-plane forces generated by the seismic weight of the roof structure above. The arrangement of the walls is favourable in respect of load distribution to ground level given the symmetry of the layout in each direction. The seismic weight of the roof is comparably low with that of the shear capacity of the walls in each direction which provides a %NBS rating in excess of 100% (ULS).

The connection between the timber wall plate and the head of the reinforced concrete walls is currently unknown and for the purposes of this report has been given a notional value. This element is essential to the gravity and lateral load resisting systems and has thus been notionally rated at 50% NBS (ULS) until an inspection of the connections as they exist is conducted. Initial calculations indicate that a minimum of 5 M8 bolts or equivalent are required to achieve 100%NBS

8 Summary of Geotechnical Appraisal

The building is located in an area that is assessed to have shallow gravels and a low risk of liquefaction. Further investigations are recommended to be undertaken at design stage to assess the risk of liquefaction and mitigation measures if the building is to be strengthened.

9 Remedial Options

The assessment has identified a potentially critical element notionally rated at “moderate risk” of failure. It is therefore recommended that the building should be improved by increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Our conceptual strengthening scheme to achieve this includes:

a) An inspection of the connection between the timber wall plate and reinforced concrete walls forming the perimeter of the building should be carried out by a structural engineer. The adequacy of the connection will depend upon the type, quantity and condition of the fixings between the wall plate and the concrete wall, if any. A suitable connection will be one that achieves a minimum of 67% NBS, an existing connection has the potential to be rated greater than 100%.

b) In the case an adequate connection is not identified following an inspection, options for a connection should be developed to increase seismic capacity to at least 67% NBS.

10 Conclusions

a) The seismic performance of the building is rated at 50% NBS, as governed by the potential nonexistence of a connection between the timber wall plate and the head of the perimeter reinforced concrete walls.

b) An inspection of the timber wall plate to head of concrete wall connection is required.

c) The building should be strengthened to achieve a rating of at least 67% NBS.

11 Recommendations

a) A strengthening works scheme should be developed to increase the seismic capacity of the building to at least 67% NBS, this will need to consider compliance with accessibility and fire requirements.

b) An inspection of the timber wall plate to head of concrete wall connection be undertaken.

12 Limitations

a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some structural damage is described but this is not intended to be a complete list of damage to structural items.

b) Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time.

c) This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not intended for any other party or purpose.

13 References

[1] NZS 1170.5: 2004, *Structural design actions, Part 5 Earthquake actions,* Standards New Zealand.

[2] NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

[5] NZSEE, *Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance*, Draft prepared by The University of Auckland, February 2011.

Appendix 1 - Photographs

Fern House, Botanic Gardens, Christchurch		
No.	**Item description**	**Photo**
	General	
1	View onto building entrance	
2	Steel arch springing from perimeter walls	

3	Steel arches crossing at apex and supporting roof lantern over	
4	Typical reinforced concrete wall	

Appendix 2 – Quantitative assessment methodology and assumptions

Quantitative Assessment

Methodology and Assumptions

1.1. Material Strength

Structural drawings were not available, the following material strengths were assumed:

Structural steel – f_y = 270 MPa

Concrete – f'_c = 30 MPa

1.2. Building Weights

Roof/glazing – 0.52 kPa

Density of concrete – 24 kN/m^3

1.3. Seismic Parameters

T (estimated) = 0.40 sec (for walls)

Z = 0.30

Importance Level 2

R = 1.0

N(T,D) = 1.0

Site subsoil class = D

μ = 1.25 for timber hip/ valley members

μ = 1.25 for in-plane wall assessment

1.4. Analysis Procedure

Hand calculation was used to estimate the seismic weights and corresponding equivalent static forces of the roof and walls. Gravity and seismic forces were distributed between the hip and valley members (longitudinal and transverse direction are symmetrical) by area given the equal stiffness of the beams.

The in-plane capacity of the perimeter walls were checked against forces generated from approximately 40% of the roof acting on a 3.4m length of wall and ignoring any tensile contribution that the roof may configuration may possess.

Appendix 3 – CERA DEEP data sheet

OPUS