OPUS

Christchurch City Council

Coronation Hall
PRK 1099 BLDG 001 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Coronation Hall

Quantitative Assessment Report

71 Domain Terrace, Christchurch

Prepared By

Scott C. Darling
Structural Engineer

Reviewed By

John Meyer
Structural Engineer, SE CA 2161

Approved By

Alistair Boyce
Senior Structural Engineer, CPEng 209860

Opus International Consultants Ltd
Christchurch Office
20 Moorhouse Avenue
PO Box 1482, Christchurch Mail Centre, Christchurch 8140
New Zealand

Telephone: +64 3 363 5400
Facsimile: +64 3 365 7858

Date: February 2013
Reference: 6-QUCC1.06
Status: Final

Summary

Coronation Hall
PRK 1099 BLDG 001 EQ2

Detailed Engineering Evaluation
Quantitative Report - Summary
Final

Background
This is a summary of the quantitative report for the building structure of Coronation Hall, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections, and developed drawings.

Key Damage Observed
No significant damages resulting from the recent seismic activity were reported. However, damage from historic fires was observed to the roof trusses. The detailed engineering evaluation was performed without consideration of these damages and the effect of these damages are only included in the following report discussion.

Critical Structural Weaknesses
Critical structural weaknesses were observed at:

1. Main Structure Foundation Connection: No positive connection between the foundation under the primary structure and the wall bottom plate were observed during site investigations.

2. Addition foundation lateral stability: The addition at the west end of the structure falls outside the original foundation and no concrete foundations could be located in this area. The piles present do not appear to provide any lateral stability and leave the addition susceptible to sliding off the foundation.

Indicative Building Strength
Based on the information available, and from undertaking a quantitative assessment, Coronation Hall has been assessed to have an overall capacity of 43% NBS. The capacity is governed by the overturning capacity of the timber framed walls in the transverse direction of the structure.

The quantitative assessment assumes that foundation system is capable of carrying all lateral loads out of the structure. This assumes that the bottom plate under the main hall has been anchor bolted to the concrete strip footing around its perimeter, which was typical of construction methods of the time. Additionally, this assumes that the pile foundation under the addition at the rear (west) of the structure is properly embedded and anchored. These two assumptions need to be further investigated and confirmed.

The quantitative assessment has been completed assuming an undamaged state of the structure. The structure at Coronation Hall has significant damage from multiple historical fires which is most prevalent in the wooden roof trusses. It has been determined that the fire damage has not compromised the lateral force resisting system significantly and that the trusses remain greater than 33% NBS. However, it has been determined that the original undamaged trusses do not meet the current code design requirements for gravity loading. This is compounded by the fact that the

fire damage has further reduced the truss capacities, despite efforts to reinforce the trusses through sistered members. This does not pose an imminent threat to collapse, but a strengthening plan is recommended for the extended use of the structure.

Recommendations

1. Confirm the positive attachment of the wall bottom plate to the foundation under the original structure.

2. Inspect at least one pile location under the addition to determine the embedment depth.

3. Improve the foundation under the addition if the timber pile embedment is found to be inadequate.

4. Improve fastening of the floor framing to the foundation under the original structure and the addition (as deemed necessary by further site investigation).

5. Develop a strengthening works scheme to increase gravity capacity of the roof trusses.

6. Develop a strengthening works scheme to increase the seismic capacity of the building to at least 67% NBS.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of Coronation Hall, located at 71 Domain Terrace, Spreydon, Christchurch following the Canterbury Earthquake Sequence since September 2010.

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

The seismic assessment 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) [3] [4].

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.

Christchurch City Council requires any building with a capacity of less than 34% of New Building Standard (including consideration of critical structural weaknesses) to be strengthened to a target of 67% as required under 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).

The Earthquake Prone Building policy for the territorial authority shall apply as outlined in Section 2.3 of this report.

Section 115 – Change of Use

This section requires that the territorial authority 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 territorial authorities as being 67% of the strength of an equivalent new building or as near as practicable. 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.

Where an application for a change of use of a building is made to Council, the building will be required to be strengthened to 67% of New Building Standard or as near as is reasonably practicable.

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:

- increase in the basic seismic design load for the Canterbury earthquake region (Z factor increased to 0.3 equating to an increase of 36 – 47% depending on location within the region);

- 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)

<table>
<tr><th colspan="2">Improvement of Structural Performance</th></tr>
<tr><th>Legal Requirement</th><th>NZSEE Recommendation</th></tr>
<tr><td rowspan="2">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.</td><td>100%NBS desirable. Improvement should achieve at least 67%NBS</td></tr>
<tr><td>Not recommended. Acceptable only in exceptional circumstances</td></tr>
<tr><td>Unacceptable</td><td>Unacceptable</td></tr>
</table>

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).

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[1] 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 and from the DBH guidance document dated 12 June 2012 [6], 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/territorial authority 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.

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

4 Background Information

4.1 Building Description

Coronation Hall is located at 71 Domain Terrace, Spreydon, Christchurch, and consists of a 1-storey assembly hall built in the early 1900's. The building has a T-shaped layout that consists of a centre hall with two wings at one end. At some point following initial construction, an addition was constructed off the west end (rear) of the building. The overall plan dimensions are approximately 16m by 22m. The centre hall has a pitched roof where the ridge is approximately 6.2m above ground level.

Figure 2: Aerial View of Coronation Hall

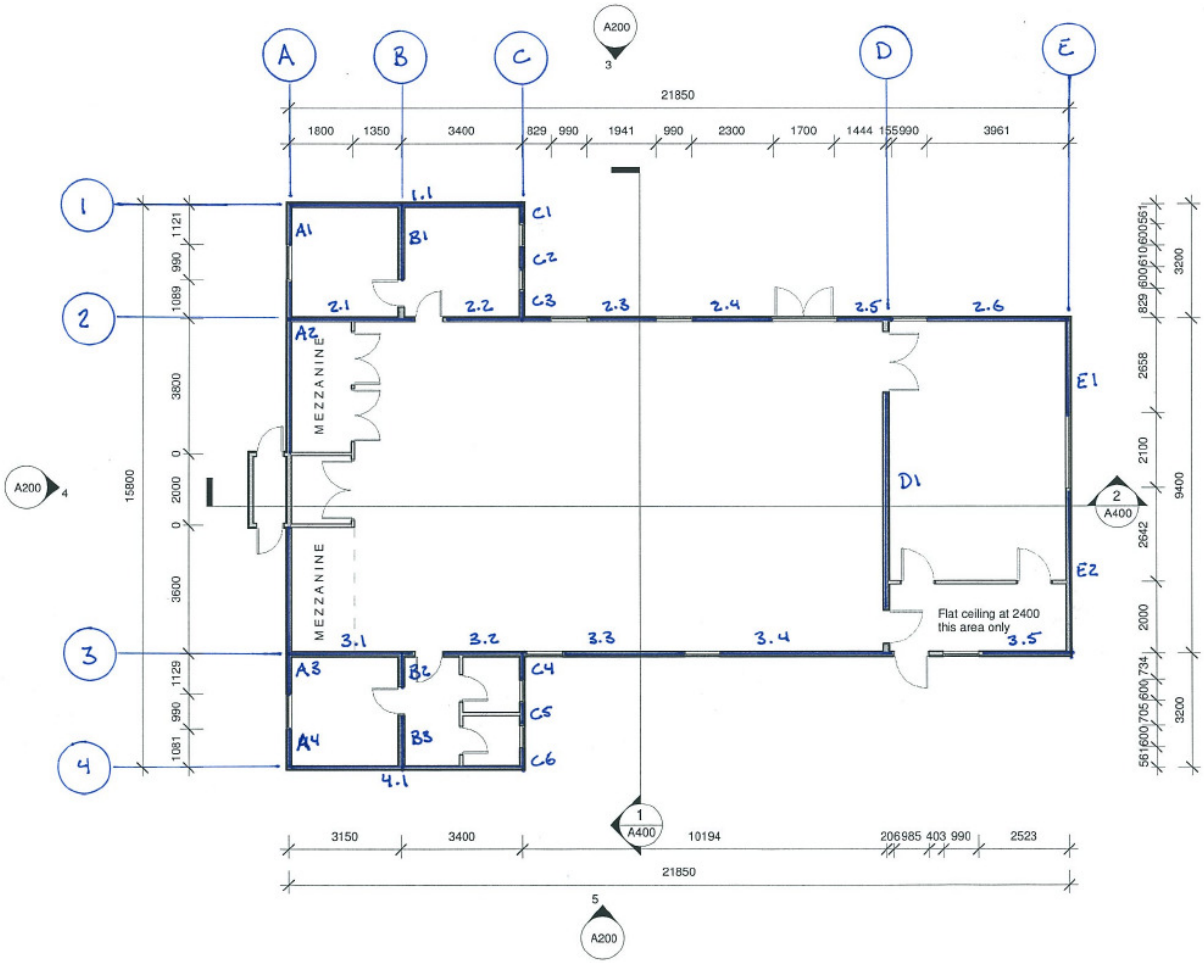


Figure 3: Coronation Hall Floor Plan and Gridlines

Coronation Hall consists of lightweight timber framed construction supporting a timber framed roof with timber trusses with steel rod tension ties. The ground level of the original building consists of a timber framed floor system supported by concrete pads and a perimeter continuous concrete foundation. The addition to the rear of the building has timber framed walls and a timber framed floor system supported by timber piles.

The lateral load resisting system in Coronation Hall consists of:

- Timber framed walls along the exterior walls in the longitudinal direction. The walls resist lateral loads via the gypsum wall boards on the interior of the timber framing.

- Timber framed walls along the exterior walls and one interior wall in the transverse direction. The exterior walls resist lateral loads via the gypsum wall boards on the interior of the timber framing. The gable frame wall at the west end of the structure, which became an interior wall during construction of the addition, has gypsum board on both sides of the timber framing to resist lateral loads.

4.2 Survey

4.2.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of Coronation Hall was not available at the time of the detailed assessment.

4.2.2 Further Inspections

- Site visits were performed by Opus International Consultants on 30 August 2012 and on 20 September 2012. Field measurements were made to produce drawings for the building. Additionally, openings in the gypsum board walls were created to determine the existing conditions beneath.

- Opus International Consultants performed a fire investigation in the attic of the structure on 1 November 2012.

- A foundation specific field investigation was performed by Opus International Consultants on 5 December 2012. During this site visit, the crawl space under the addition was accessible but the area under the main hall was not accessible without intrusive work.

4.3 Original Documentation

No copies of the original construction drawings were available at the time of the detailed assessment.

Copies of the design calculations were not provided.

5 Structural Damage

5.1 Seismic Related Structural Damage

No earthquake related damage was observed during the visits by Opus International Consultants.

5.2 Fire Damage to Coronation Hall

Damage from historical fires was observed in the attic of Coronation Hall. Further investigation has revealed that two separate fires have reduced the structural integrity of the timber trusses in this area. Based on the lack of historical records or recollections by the members of the hall, it is estimated that both fires occurred prior to 1940. Evidence of the fire damage can be seen in Photographs 28-44 of Appendix 1 and the Fire Investigation field summary is in Appendix 2.

The original trusses, constructed of Rimu timber, show the most severe damage. A secondary truss has been sistered to the original truss using bolted connections following the original fire. The secondary trusses, constructed from Oregon timber, also exhibits varying levels of fire damage along the length of the building. This indicates that a second fire occurred after the repairs for the original fire had been completed.

Additionally, the gable end wall and nearby roof framing at the east end of the building exhibited significantly greater damage than the timber framing at the west end. This shows that the second fire may have been concentrated toward the east end of the structure. The roof sarking has been replaced in the three eastern-most bays following the second fire while the sarking in the final two bays remains discoloured by the second fire. This provides further evidence that the second fire was greatest in the eastern-most bays of the hall.

The roof purlins exhibit some charring and have been either sistered or replaced depending upon their level of damage.

Refer to Section 6.7 for a discussion of the effects of the fire damage on the structural capacity of the building.

6 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, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] issued on 21 December 2011.

6.1 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. The following potential CSW's were identified for the building and have been considered in the quantitative analysis.

- Main structure foundation connection: No positive connection between the foundation under the primary structure and the wall bottom plate were observed during site investigations.
- Addition foundation lateral stability: The addition at the west end of the structure falls outside the original concrete foundation and sits on timber piles. Proper embedment and fastening of these elements must be confirmed to ensure that the foundation will provide sufficient lateral stability.

6.2 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:

a. The base shear was calculated from the seismic weight of the building using the spectral values established from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1). The base shear was distributed to different storeys following NZS1170.5.

b. Coronation Hall consists of flexible diaphragms at the roof thus the horizontal forces are distributed to each individual wall lines by tributary area.

c. Average wall shear stresses in the GIB sheathing was calculated and compared to the shear capacities references in NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes"

d. The buildings were assessed as Importance Level 2.

6.3 Review of Critical Structural Weaknesses

The critical structural weaknesses identified have the potential to provide a discontinuity in the load path between the structure and the foundations. During the quantitative assessment, it has been assumed that the construction methods at Coronation Hall followed those typical of the time. This would indicate that the bottom plate of the main structure has been adequately anchor bolted to the foundation and that the timber pier foundation under the addition has been sufficiently embedded and fastened to the floor framing.

Based on these assumptions, which must be field verified, the critical structural weaknesses do not affect the capacity of the structure.

6.4 Limitations and Assumptions in Results

The observed level of damage suffered by the building was deemed low enough to not affect the capacity. Therefore the analysis and assessment of the building was based on it being in an undamaged state. There may have been damage to the building that was unable to be observed that could cause the capacity of the buildings to be reduced; therefore the current capacity of the buildings may be lower than that stated.

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:

a. Simplifications made in the analysis, including boundary conditions such as foundation fixity (as discussed in Section 6.3).
b. Assessments of material strengths based on limited drawings, specifications and site inspections.
c. The normal variation in material properties which change from batch to batch.
d. Approximations made in the assessment of the capacity of each element, especially when considering the post-yield behaviour.

6.5 Assessment

A summary of the structural performance of the building is shown in the table below. Note that the degradations due to fire damage have not been included when performing this analysis. The effects of those damages are further discussed below. 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. This can be further considered when developing the strengthening options.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode or description of limiting criteria based on displacement capacity of critical element.	% NBS based on assumed capacity
Timber framed walls in shear– Longitudinal direction (east-west)	Timber framed walls sheathed in GIB provide lateral resistance. It was assumed that the single sided GIB board was the only member providing shear resistance. These walls are controlled by shear failure of the GIB.	79%
Timber framed walls in shear – Transverse direction (north-south)	Timber framed walls sheathed in GIB provide lateral resistance. Exterior walls assumed GIB board located on a single side of the timber frame, while interior walls assumed GIB board was located on both sides of the timber framing.	58%
Timber framed walls in overturning– Longitudinal direction (east-west)	Timber framed walls sheathed in GIB provide lateral resistance. The overturning capacity of this wall line is greater than the shear capacity.	99%
Timber framed walls in overturning – Transverse direction (north-south)	Timber framed walls sheathed in GIB provide lateral resistance. These walls are generally controlled by overturning capacity of individual piers.	43%
Roof diaphragm – Longitudinal Direction (east-west)	The roof diaphragm consists of straight sheathing that distributes loads to the top of the north and south walls.	230%
Roof diaphragm – Transverse Direction (north south)	The roof diaphragm consists of straight sheathing that distributes loads to gable framed walls at the east and west ends of the structure.	104%

6.6 Discussion of Seismic Findings

Based on the quantitative assessment, the seismic load resisting system at Coronation Hall has a computed capacity of 43% NBS. The overall capacity of the structure is limited by the overturning capacity of the wall at Gridline D (see Figure 3). Due to the assumed distribution of loads, the wall at Gridline D takes a considerable amount of the horizontal shear in the transverse direction. This wall has a door at each end, which prevents it from being able to take full advantage of the hold-down forces that the longitudinal walls could provide.

Additionally, the shear strength of the walls in the transverse direction results in a capacity of less than 67% NBS along Gridlines A and D (58% and 61%, respectively. Along Gridline A, a single sheet of GIB board was assumed because it could not be confirmed if there was straight sheathing on the exterior of the wall framing. Further investigation into the construction of the exterior walls could improve the capacity ratings. Along column line D, the GIB board on both sides has already been utilized. In order to strengthen this wall above 67% NBS, additional shear capacity would need to be added.

Dissimilar to the transverse direction, the behaviour of the building when loaded in the longitudinal direction is controlled by shear failure of the piers. In this direction, the piers along Gridline 2 limit the capacity of the system. As indicated in Figure 4.2, the wall along this gridline has a number of smaller piers due to the various openings along its length. However, the top chord of this wall would help to provide continuity along the entire length such that for a single pier to fail, all piers along the same line would also have to fail. Using this logic, an average overturning capacity for the piers could be assumed to represent a realistic overturning capacity for this Gridline. Using this logic, the piers along Gridline 2 would have a capacity of 99% NBS in overturning. Therefore, the controlling mechanism in the longitudinal direction is the shear capacity of the walls (79% NBS).

The results of the quantitative assessment assume that sufficient positive connection exists between the foundation and floor framing of the structure. Under the original structure, further investigation is required to determine if there is positive connection between the sill plate and the foundation. Under the addition, proper embedment of the timber piers and proper fastening to the floor framing needs to be confirmed.

6.7 Discussion of Fire Damage Findings

A description of the fire damage has been included in Section 5.2. As described, following the first fire, secondary trusses were sistered to the original trusses. These trusses were then damaged by a second fire. Additionally some of the purlins have been replaced or sistered and the straight sheathing has been replaced in the bays where fire caused significant damage. Based on the observed levels of damage we had concerns that the structural capacity for lateral and gravity loadings was insufficient.

The roof trusses do not provide significant resistance for lateral loading of the structure. Therefore, the damage to these elements has not compromised the lateral force resisting system significantly. However, the gable framed walls at Gridlines A and D play a significant role in the lateral force resisting system because they must resist the loads from the roof diaphragm when the structure is loaded in the transverse direction. The primary concern is that the fire has reduced the effective cross section of the gable end wall vertical members to an extent that they could not reliably hold the wall board nails.

None of the members in the gable end walls appeared to be sistered or replaced following the second fire. Thus, the available cross-sectional area had to be field measured to determine their remaining capacity. The outer char on these members was scraped to reveal the hardwood beneath and outer dimensions recorded. These measurements were further reduced by 9.5mm on each side with char to account for the thermal degradation with an additional factor of safety. The degradation is generally considered to be contained within the first 1/4inch (6.4mm) beneath the char. Based on this effective area analysis, it was determined that sufficient cross sectional area remained beneath the heat affected zone to develop the shear strength of the nailing connections. Therefore, we do not believe the fire damage has reduced the lateral capacity of the building.

Next, the trusses were analysed for gravity loads. It was determined that the original trusses, prior to any degradation due to fire loading, do not provide sufficient capacity to meet the current code criteria for gravity loading. Our calculations indicate the trusses can marginally support dead loads depending on the assumptions made for allowable stresses of the timber. The trusses are highly overstressed for dead plus code imposed loads. The fire damage to the members in these trusses has further reduced the section properties of these members, and therefore reduced the capacity of the original trusses. The secondary trusses with additional fire damage do not provide sufficient additional cross sections properties to make up for the losses to the original truss. Based on these facts, the combined properties do not provide sufficient capacity to meet the code requirements for gravity loading. Further compounding this issue is the fact that the second fire has compromised the bolting connections used to sister the two trusses, which calls into question their ability to act compositely. This can be seen in Photographs 39-41 of Appendix 1.

Based on the gravity loading, it is pertinent that a strengthening plan is developed to improve the behaviour of the roof trusses if the structure is to see extended continued use. However, these trusses have behaved well for an estimated 70+ years since the second fire. Thus, it is believed that these elements do not pose an imminent collapse hazard and the structure does not need to be vacated.

7 Summary of Geotechnical Appraisal

This section is a brief summary of the geotechnical desktop study report contained in Appendix 4 of this report.

7.1 General

Christchurch City Council commissioned Opus International Consultants to undertake a desktop study of the ground conditions at the Coronation Hall building. Geotechnical information herein is based on the findings of that study.

The northern half of the Coronation Hall building is founded on shallow concrete strip footings and the southern addition of the building is founded on 200mm square timber piles. No evidence of liquefaction was observed on the site after the 4 September earthquake and the aftershocks of 22 February and 13 June 2011, or the 24 December 2011 earthquake.

7.2 Liquefaction Potential

The 2004 ECan Solid Facts Liquefaction Study indicates the building is in an area designated as 'no liquefaction ground damage potential'. Areas 200m to the north of Coronation Hall are reported as areas designated as 'low liquefaction ground damage potential'.

7.3 Summary

Based on current evidence, the existing foundation of the Coronation Hall has performed well. The risk of liquefaction damage is considered low for this site and the foundations are considered suitable for future earthquake events.

8 Conclusions

Coronation Hall has been assessed to have an overall capacity of 43% NBS. This capacity is limited by the overturning capacity of the transverse wall along Gridline D. This capacity level implies the building is considered a moderate risk for seismic performance but is legally accepted under the 2004 Building Act.

Structural damage due to seismic loadings has not been reported for Coronation Hall. The damage to the structure is limited to the fire damage to the roof trusses that was estimated to have occurred prior to 1940. These trusses do not meet the code requirements for gravity loading and need to be strengthened for extended use of the structure. However, this does not pose an imminent danger of collapse and does not require the building to be vacated.

The connection between the bottom plate of the main structure and the foundations and the embedment and connection of the timber piles under the addition could not be verified during our site inspection. An intrusive investigation is required in order to verify that the assumptions made regarding the strengths of these elements are appropriate.

9 Recommendations

1. Confirm the positive attachment of the wall bottom plate to the foundation under the original structure.
2. Inspect at least one pile location under the addition to determine the embedment depth.
3. Improve the foundation under the addition if the timber pile embedment is found to be inadequate.
4. Improve fastening of the floor framing to the foundation under the original structure and the addition (as deemed necessary by further site investigation).
5. Develop a strengthening works scheme to increase gravity capacity of the roof trusses.
6. Develop a strengthening works scheme to increase the seismic capacity of the building to at least 67% NBS.

10 Limitations

1. This report is based on site investigations of the structure of the building and focuses on the structural damage resulting from the 4 September 2010 Darfield Earthquake and the 22 February 2011 Canterbury Earthquake and aftershocks. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.
2. 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.
3. 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.

11 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] SESOC (2011), Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes, Structural Engineering Society of New Zealand, 21 December 2011.

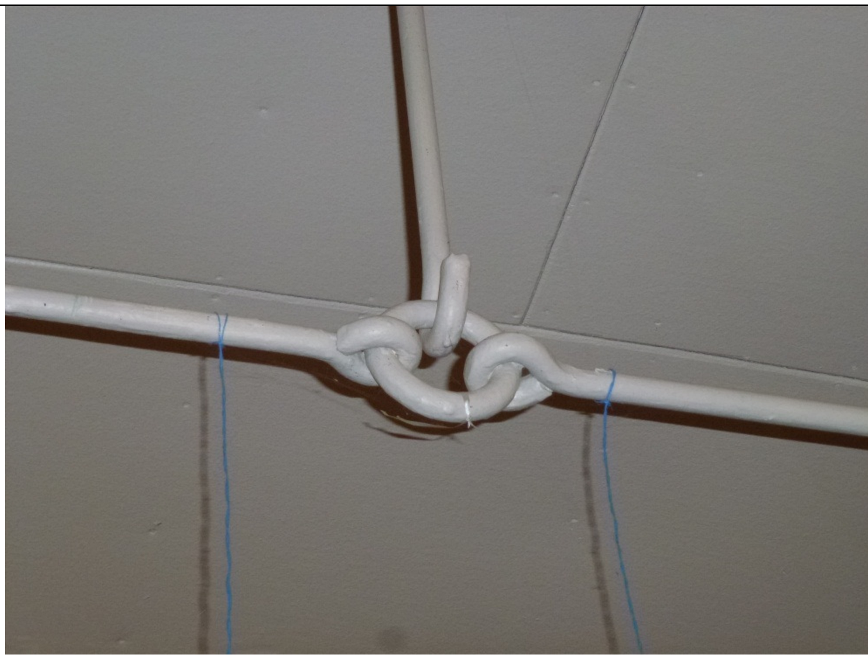
[6] DBH (2012), Guidance for engineers assessing the seismic performance of non-residential and multi-unit residential buildings in greater Christchurch, Department of Building and Housing, June 2012.

Appendix 1 - Photographs

Coronation Hall		
No.	Item description	Photo
General		
1.	East elevation of Coronation Hall	
2.	South-east corner of Coronation Hall	

3.	South elevation of Coronation Hall	
4.	South-west corner of Coronation Hall addition	

5.	North-west corner of Coronation Hall. Addition shown in front of west gable end wall (Gridline D)	
6.	North elevation of Coronation Hall	

7.	Interior of main hall in the original building. Tension rods of roof truss shown extending below ceiling	
8.	Connections of tension rods to vertical hanger at the centre of the truss	

9.	Wall opening	
10.	Straight board sheathing beneath wall opening	

11.	Discontinuous straight board sheathing beneath a second wall opening	
Foundation Investigation		
12.	Crawl space beneath addition	

13.	Floor framing of addition sitting on top of timber piles	
14.	Floor framing of addition sitting on top of timber piles	

15.	Base of timber pile buried in soil (depth unknown)	
16.	View of concrete foundation under the original structure from the crawl space under the addition	

17.	View of concrete foundation under the original structure from the crawl space under the addition. Timber posts are shown outboard of original concrete foundation	
18.	View of concrete foundation under the original structure from the crawl space under the addition. Timber posts are shown outboard of original concrete foundation	

19.	View of crawl space under the original structure through an opening in concrete strip foundation under Gridline D. Timber framing is supported on concrete pads	
20.	Concrete pads under floor framing of original building	

21.	Concrete pads under floor framing of original building	
22.	Pad appears to be a piece of stone with floor framing off-centre	

23.	Deteriorated concrete pier under floor framing of original building	
24.	View looking north along concrete strip foundation under Gridline D	

25.	View looking south along concrete strip foundation under Gridline D	
26.	Beam connection into concrete strip foundation adjacent to opening in same foundation	

27.	Beam connection into concrete strip foundation adjacent to opening in same foundation	
	Fire Investigation	
28.	Fire damage to east gable end wall	

29.	East gable end wall looking toward the south-east corner. Note new straight sheathing and sistered roof purlins in adjacent bay	
30.	Fire damage to timber stud and top chord of east gable end wall	

31.	East gable end wall looking toward north-east corner. Note wall stops short to allow for soffit (exterior view of soffit in pictures 1-2)	
32.	Fire damage to stud of east gable end wall	

33.	Layout of typical timber trusses	
34.	Connection of centre tension rod at ridgeline of roof truss	

35.	Connection of bottom chord, centre web members and tension rod. Note the sistered secondary truss on the far side of the original truss for all members. The bottom chord has a second sistered member to attach ceiling framing	
36.	Connection of outer web member and tension rod to bottom chord	

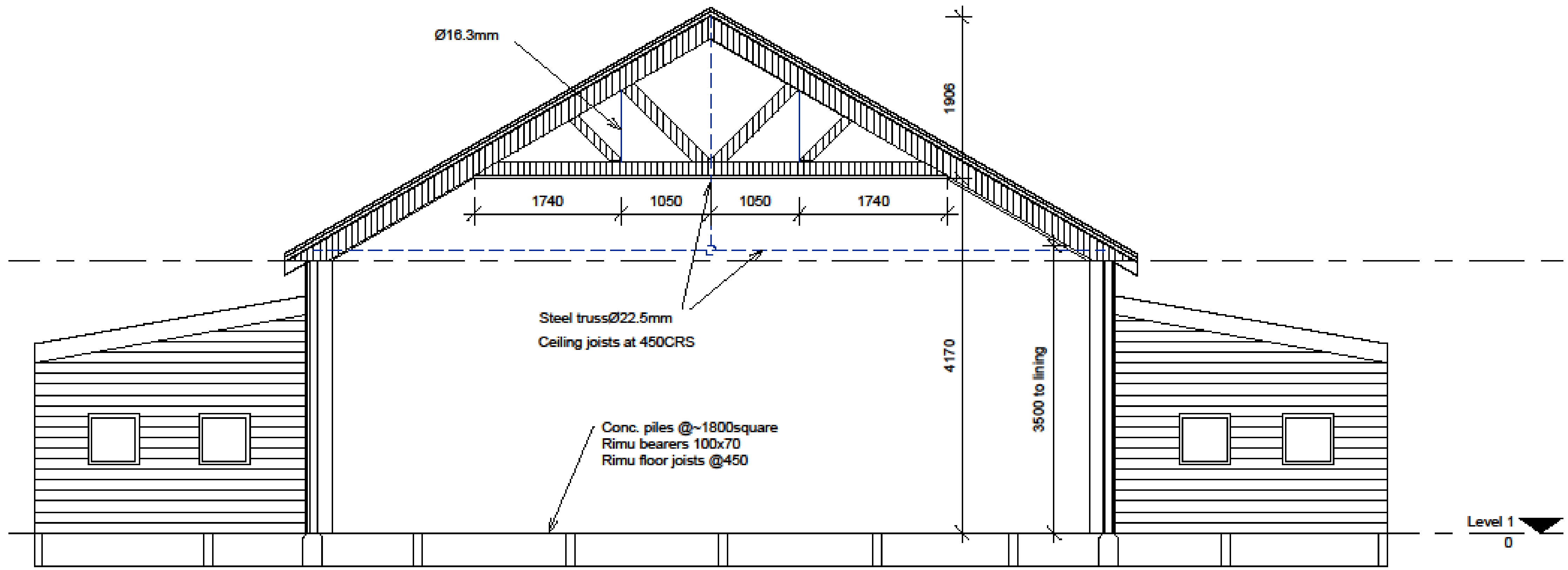
37.	Connection of centre web member and outer tension rod to top chord	
38.	Connection of original centre web member and secondary truss web member to original top chord member	

39.	Connection of original outer web member, secondary truss web member, and secondary truss top chord to the original top chord member. Note that char has been removed from the end of the web members and the bolted connection between sistered truss has deteriorated	
40.	Deterioration of tension rod connection at the top chord	

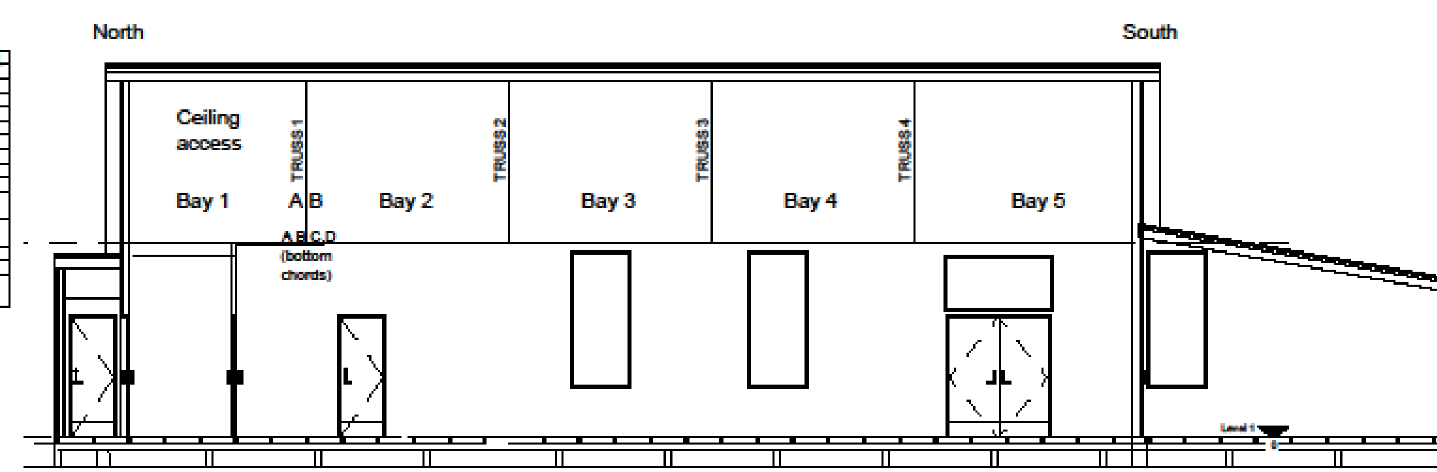
41.	Deterioration of bolted connection of secondary truss to the original truss	
42.	West gable end wall with visible discoloration but no charring	

43.	View looking northwest of west gable end wall. Note the straight board sheathing and roof purlins have not been replaced in this bay	
44.	West gable end wall	

Appendix 2 – Fire Investigation



MEMBER	NOMINAL SIZE HEIGHT (mm)	SIZE BURNED HEIGHT (mm)	SIZE SCRAPED HEIGHT (mm)	NOMINAL SIZE WIDTH (mm)	SIZE BURNED WIDTH (mm)	SIZE SCRAPED WIDTH (mm)	MATERIAL
Top Chrod A	10"?	206.83	181.93		50.93	45.08	Rimu
Top Chrod B	secondary truss	144.41	136.78		50	44	Oregon
Bottom Chrod A	3 SIDES MEASURED			30	27.62	23.93	Oregon
Bottom Chrod B	3 SIDES MEASURED			70	53.75	50.23	Rimu
Bottom Chord C	3 SIDES MEASURED			50	47.08	45.8	Oregon
Bottom chord D	Secondary truss					has not been	scraped
Truss Web A	100	92.09	80.25	70	56.45	54.64	Rimu
Truss Web B	100	94.7	81.12	45	41.5	40.92	Oregon
Ceiling Jiost Centre	100	100	82	3 SIDES MEASURED	75.33	50.73	Rimu
Ceiling Joist Typ.	100	100	84	3 SIDES MEASURED	49.44	41.16	Rimu
Roof Purlin	100	97.27	93.64	50	47.13	45.09	Rimu
Gable Studs (North)	3 SIDES MEASURED	~100	86.68	50	~50	33.14	Rimu
Gable Top Chord (North)	3 SIDES MEASURED	~100	88.9		~50	43.11	Rimu



OPUS
Christchurch Office

Coronation Hall
Christchurch
CCC

Fire Investigation
Notes

6 QUCC1.06
6/401/13/8602
201
R0

Original Sheet Size A3 [420x297] Plot Date 20.11.2012 @ 10:15 a.m. File Path C:\Revit\Coronation Hall\fire investigation.dwg

Appendix 3 - Methodology and Assumptions

A3.1. Referenced Documents

- AS/NZS 1170.0:2002, *Structural design actions, Part 0: General principles,* Standards New Zealand.

- AS/NZS 1170.1:2002, *Structural design actions, Part 1: Permanent, imposed and other actions,* Standards New Zealand.

- NZS 1170.5:2004, *Structural design actions, Part 5: Earthquake actions – New Zealand,* Standards New Zealand.

- NZS 3101: Part 1: 2006, *Concrete Structures Standard, The Design of Concrete Structures,* Standards New Zealand.

- NZS 3101: Part 2: 2006, *Concrete Structures Standard, Commentary on the Design of Concrete Structures,* Standards New Zealand.

- NZBC, *Clause B1 Structure, Verification Method B1/VM1,* Department of Building and Housing.

- NZSEE: 2006, *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes,* New Zealand Society for Earthquake Engineering.

- 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.

- ASCE/SEI 41-06, *Seismic Rehabilitation of Existing Buildings,* Structural Engineering Institute of the American Society of Civil Engineers, 2007.

A3.2. Analysis Parameters

The following parameters are used for the seismic analysis:

- Site soil category — Cl. 3.1.3, NZS1170.5
  D (deep or soft soil)

- Seismic hazard factor — Cl. 2.2.14B, B1/VM1
  Z = 0.30

- Return period factor — Table 3.5, NZS1170.5
  R_u = 1.0 (*Importance* Level 2 structure, 50 year design life)

- Ductility factor — Cl. 2.6.1.2, NZS3101:2006
  μ = 2.0 (nominally ductile)

- Structural performance factor — Cl. 2.6.2.2, NZS3101:2006
  S_p = 0.925

- Material properties

Table A1: Analysis Material Properties for all buildings

Shear capacity of Gypsum Board (kN/m)	3

Notes:
1. Based on guidance from *NZSEE 2006*, probable strength values for existing materials in wood frame construction (Table 11.1)

- Earthquake load combination — Cl. 4.2.2, AS/NZS1170.0
  $G + E_u + \Psi_E Q$

- Floor live loading — Table 3.1 Part G, AS/NZS1170.1
  Q = 1.5 kPa – General Areas
  Q = 0.5 kPa – Non-habitable roof spaces

- Earthquake combination factor — Table 4.1, AS/NZS1170.0
  $\Psi_E = 0.3$

- Building seismic weight — Cl. 4.2, NZS1170.5
  $W_t = G + \Psi_E Q$

  Building seismic weight was calculated as 240 kN

A3.3. Assessment Methodology

Static Analysis

The seismic assessment was undertaken by completing static analysis for the building in accordance with NZS 1170.5:2004. Diaphragms of the buildings consist of timber sheathing or GIB ceiling and are considered to be flexible diaphragms. Thus lateral load are distributed based on tributary area and are inputted to each individual wall lines.

A simple 2D model was set up using the structural analysis program ETABS to assess the capability of the exterior wall at Line E to transfer shears around the opening in the wall. It was determined that the strap forces developed exceeded the capacity of the walls and that individual piers needed to be considered when assessing the overturning capacity along Line E.

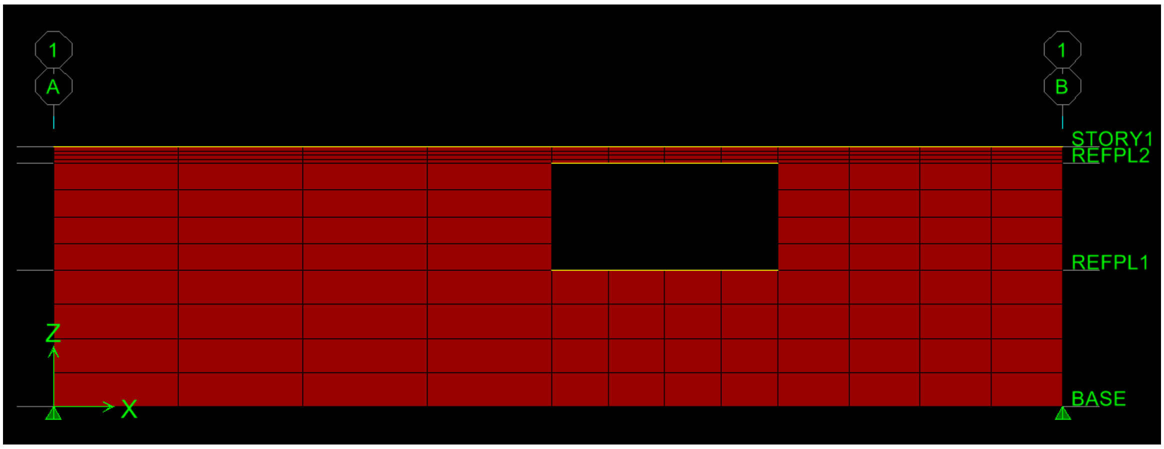


Figure A1: 2D ETABS model of 2-Storey Apartment single front wall section

The fundamental building periods were assumed to be less than the lower bound limit of 0.4seconds which is a conservative assumption.

An equivalent static analysis was carried out to perform the seismic assessment of the building. The base shears resulting from the equivalent static method is 96.5 kN.

The building was analysed as having limited ductility (μ = 2.0) and the design actions were applied separately in each perpendicular direction.

Element Demand to Capacity

Element force demands were extracted from the equivalent static analysis and compared to calculated capacities based on the material properties. The results of these demand to capacity checks are summarized in further detail in the report and reported as %NBS.

Appendix 4 – Geotechnical Desktop Study

Opus International Consultants Ltd
Christchurch Office
20 Moorhouse Avenue
PO Box 1482, Christchurch Mail Centre, Christchurch 8140
New Zealand

t: +64 3 363 5400
f: +64 3 365 7858
w: www.opus.co.nz

14 January 2013

Michael Sheffield
Property Asset Manager
Christchurch City Council
PO Box 237
Christchurch 8140

6-QUCC1.06/025HC

Dear Michael,

Geotechnical Desktop Study - Coronation Hall

1 Introduction

This report summarises the findings of a Geotechnical Desktop Study and the site walkover completed by Opus International Consultants (Opus) for Christchurch City Council at the above property on 5 July 2012. The Geotechnical desk study follows the Canterbury Earthquake Sequence initiated by the 4 September 2010 earthquake.

The purpose of the Geotechnical Desktop Study is to record observed ground damage and to assess the current ground conditions and the potential geotechnical hazards that may be present at the site, and determine whether further subsurface geotechnical investigations are necessary.

This Geotechnical Desktop Study has been prepared in accordance with the Engineering Advisory Group's Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Revision 5, 19 July 2011.

The Geotechnical Desktop Study forms part of a Detailed Engineering Evaluation prepared by Opus. Based on a site inspection by Opus Engineers, no ground damage or visual evidence of differential settlement has been observed at the site. A level survey has not been undertaken. The Geotechnical Desktop Study has been undertaken without the benefit of any site specific investigations and is therefore preliminary in its nature.

It is our understanding this is the first inspection by a Geotechnical Engineer of this property following the Canterbury Earthquake Sequence.

2 Desktop Study

2.1 Site Description

Coronation Hall is located on the northeast corner of Spreydon Domain, approximately 330m southeast of the intersection of the Domain Terrace and Lincoln Road, Spreydon, approximately 4km southwest of the centre of Christchurch. The site is relatively flat, but the hall is situated at the crest of a gentle slope leading south away from the hall. There are residential areas to the north and east of the site.

The building is a single storey timber frame structure, with a footprint measuring approximately 20m by 16m.

2.2 Structural Drawings

No structural drawings showing the foundations have been received at this time. Site observations indicate that the northern half of the building is founded on a concrete strip footing of unknown width. The southern half of the building where it stands on the gentle slope is founded on 200mm square timber piles. The floor and internal columns/walls are assumed to be supported on timber piles.

2.3 Regional Geology

The 1:25,000 Geological Map of Christchurch Urban Area (GNS 2008) indicates the site is underlain by grey river alluvium compromising gravel, sand and silt.

According to Environment Canterbury Regional Council records, groundwater is anticipated to be greater than 1.5m below ground level.

2.4 Expected Ground Conditions

A review of the Environmental Canterbury (ECan) wells database showed three wells located within approximately 450m of the site. The locations of Boreholes and CPT's undertaken by the Earthquake Commission have also been reviewed. The nearest CPT is located 290m east of the site; the CPT refused on a shallow dense layer of sand or gravel at 1.4m below ground level.

The approximate locations of the boreholes relative to the hall are shown on the attached Site Location Plan. The logs of the ECan boreholes are presented in Appendix A.

The investigation logs available from ECan records have been used to infer the ground conditions beneath the site, and are summarised in Table 1 below.

Table 1 Interpreted Ground Conditions

Stratigraphy	Thickness (m)	Depth Encountered From (m)
Interbedded layers of SILT and PEAT	7.0m	Surface
SAND and GRAVEL	5.0m to 14.0m	Surface
Interbedded layers of CLAY and PEAT	6.0m to 11.4m	7.0m to 14m
GRAVELS (Riccarton Formation)	-	14.0m to 20.0m

2.5 Ground Damage

No evidence of liquefaction was observed in aerial photographs taken after the 4 September earthquake, and the aftershocks of 22 February and 13 June 2011, or the 23 December 2011 earthquake.

2.6 Liquefaction Hazard

The Earthquake Commission (EQC) has prepared maps (Project Orbit, 2012) showing areas of liquefaction interpreted from high resolution aerial photos for the 4 September earthquake, and the aftershocks of 22 February and 13 June 2011. An interpretation of these maps indicates the site itself did not suffer from liquefaction in any of the Canterbury earthquakes initiated by the 4 September 2010 earthquake.

Liquefaction was reported 480m west of the site following the 4 September 2011 earthquake, and liquefaction was reported in areas 200m to 300m to the south, southwest and southeast following the 22 February 2011 aftershock. Further liquefaction was also reported greater than 260m to the north and west of the site following the 13 June 2011 aftershock.

The 2004 Environment Canterbury Solid Facts Liquefaction Study indicates the site is in an area designated as 'no liquefaction ground damage potential'. Areas 200m to the north and northwest of the site are reported as areas designated as 'low liquefaction ground damage potential'. According to this study, based on a low groundwater table, these areas may be affected by up to 100mm of ground subsidence.

The Christchurch Earthquake Recovery Authority (CERA) last uploaded 11 December 2011 has classified Spreydon Domain and surrounding residential properties as Green Zone, indicating the repair and rebuilding process can begin.

The maps that were released by the Department of Building and Housing (DBH) on 9 February 2012 indicate that the site is classified as urban non-residential. Residential properties north, west and south of the site are classified as Technical Category 2 (yellow), which indicates that minor to moderate land damage from liquefaction is possible in future significant earthquakes.

3 Site Walkover Inspection

A walkover inspection of the interior of the building and surrounding land was carried out by a Senior Opus Engineering Geologist on 5 July 2012. Access to the inside of the building was not possible at the time of the site visit.

The following observations were made (refer to the Walkover Inspection Plan and Site Photographs attached to this report):

External:

- The northern half of the structure is founded on a concrete strip footing of unknown width;

- The southern half of the structure where it stands on the gentle slope is founded on 200mm square timber piles (Photograph 4);

- 3mm wide crack in the concrete footings in the eastern corner of the hall (Photograph 5);

- Hairline cracks in the concrete footings on the northeast facing side of the hall (Photograph 6);

- Hairline cracks in the footings on the northeast corner of the hall (Photograph 7);

- Two 5mm wide cracks in the asphalt pavement located by the northwest site of the hall (Photograph 8);

- Dense growth of shrubbery along west facing side of the hall obscures the concrete strip footings (Photograph 9);

4 Discussion

Minor damage has occurred to the foundations of the hall, potentially due to the Canterbury Earthquake Sequence following the 4 September 2010 earthquake. There has been some cracking of the asphalt pavement on the northwest site of the hall.

No visual evidence of lateral displacement, settlement or heave of the ground around the hall was observed.

The foundations of the hall appear to have suffered only minor damage. It is unknown if the pile supported floor has experienced any kind of differential settlement. A floor level survey would be required to determine if any differential settlement has occurred.

The northern half of the building is founded on a concrete strip footing of unknown width. The southern half of the building where it stands on the gentle slope is founded on 200mm square timber piles. The floor and internal columns/walls are assumed to be supported on timber piles. This means that the building has mixed "Type A" and "Type B" foundations in accordance with DBH guidelines.

No damage to the concrete footings and timber piles was recorded and indicates that no foundation repairs are likely to be required. An internal inspection by an Opus Structural Engineer has been completed. No evidence of differential settlement has been observed.

GNS Science indicates an elevated risk of seismic activity is expected in the Canterbury region as a result of the earthquake sequence following the 4 September 2010 earthquake. Recent advice[1] indicates there is a 12% probability of another Magnitude 6 or greater earthquake occurring in the next 12 months in the Canterbury region. This event

[1] GNS Science reporting on Geonet Website: http://www.geonet.org.nz/canterbury-quakes/aftershocks/ updated on 14 November 2012.

may cause liquefaction induced land damage at the site; however it is dependent on the location of the earthquakes epicentre. There is currently a minor risk of liquefaction and differential settlements occurring at this site. It is expected that the probability of occurrence is likely to decrease with time following periods of reduced seismic activity.

Based on current evidence, the existing foundations of the Coronation Hall have performed well and are considered suitable for future earthquake events.

5 Recommendations

- Existing foundations are deemed suitable at this site.
- Assessment by a Structural Engineer to determine the level of repair/rebuild required.

6 Limitation

This report has been prepared solely for the benefit of Christchurch City Council as our client with respect to the particular brief given to us. Data or opinions in this desk study may not be used in other contexts, by any other party or for any other purpose.

It is recognised that the passage of time affects the information and assessment provided in this Document. Opus's opinions are based upon information that existed at the time of this production of this Geotechnical Desktop Study. It is understood that the Services provided allowed Opus to form no more than an opinion on the actual conditions of this site at the time the site was visited and cannot be used to assess the effect of any subsequent changes in the quality of the site, or its surroundings or any laws or regulations.

Figures:

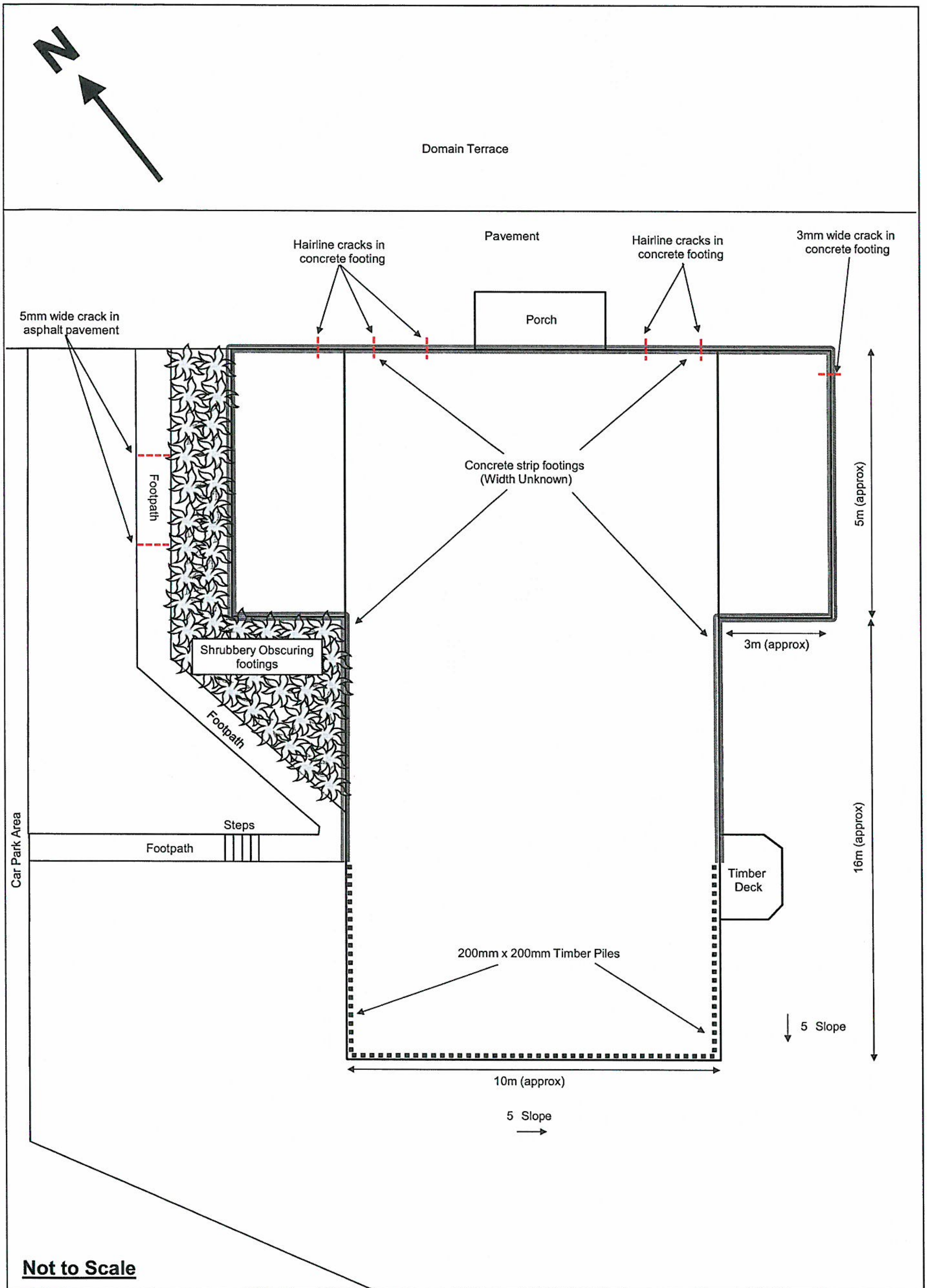
Site Location Plan

Walkover Inspection Plan

Site Photographs

SOURCE: Stronger Christchurch SCIRT Viewer (Accessed on 04/07/12)

Opus International Consultants Ltd Christchurch Office 20 Moorhouse Ave PO Box 1482 Christchurch, New Zealand Tel: +64 3 363 5400 Fax: +64 3 365 7857	**Project:** Coronation Hall Geotechnical Desktop Study **Project No.:** 6_QUCC1.06/025HC **Client:** Christchurch City Council	**Site Location Plan** **Drawn:** 5/07/2012 Senior Engineering Geologist **Date:** 5/07/2012



Not to Scale

Opus International Consultants Ltd
Christchurch Office
20 Moorhouse Ave
PO Box 1482
Christchurch, New Zealand
Tel: +64 3 363 5400 Fax: +64 3 365 7857

Project:	Coronation Hall Geotechnical Desktop Study
Project No.:	6-QUCC1.06/025HC
Client:	Christchurch City Council

Walkover Inspection Plan

Completed by:	5/07/2012 Senior Engineering Geologist
Date Drawn:	5/07/2012

Photograph 1: View of north facing side of Coronation Hall.

Photograph 2: View of southwest facing side of Coronation Hall.

Photograph 3: View of south facing side of Coronation Hall.

Photograph 4: View of the 200mm square timber piles where the southern half of the structure stands on the gentle slope.

Photograph 5: View of the 3mm wide crack in the concrete footings on the eastern corner of the hall.

Photograph 6: View of the hairline cracks on the concrete footings on the northeast facing side of the hall.

Photograph 7: View of the hairline cracks in the concrete footings on the northeast corner of the hall.

Photograph 8: View of the two 5mm wide cracks in the asphalt pavement located by the northwest side of the hall.

Photograph 9: View of the dense growth of shrubbery along west facing side of the hall obscures the concrete strip footings.

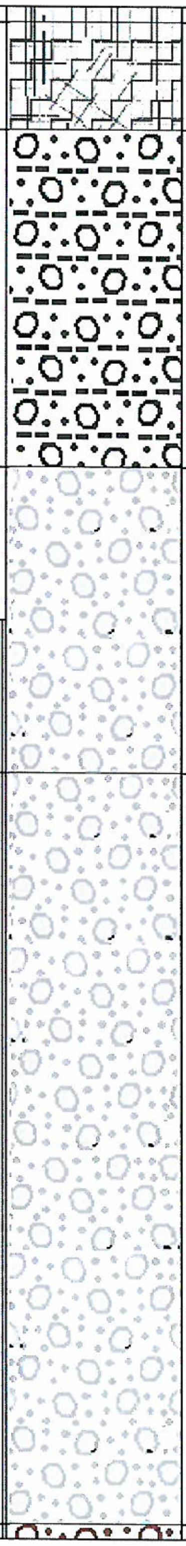
Appendix A:

ECan Well Logs

Borelog for well M36/7907

Gridref: M36:77251-39169 Accuracy : 3 (1=high, 5=low)
Ground Level Altitude : 14 +MSD
Driller : C W Drilling and Investigations Ltd
Drill Method : Hollow Stem Auger
Drill Depth : -5m Drill Date : 3/05/2005

Environment Canterbury



Scale(m)	Water Level	Depth(m)	Full Drillers Description	Formation Code
		-0.05m	asphalt	fi
		-0.40m	silt, light brown, with sand/gravel and concrete/bricks - fill	fi
		-1.50m	sandy silt with gravels/cobbles, light and dark grey areas	sp
		-2.50m	sandy gravel, grey, with some silt	sp
		-4.95m	gravel, grey, with some sand, becoming very sandy	sp
		-5.08m	sand, with some gravels, sed/brown stained, fine	sp

Scale: -0.2, -0.4, -0.6, -0.8, -1, -1.2, -1.4, -1.6, -1.8, -2, -2.2, -2.4, -2.6, -2.8, -3, -3.2, -3.4, -3.6, -3.8, -4, -4.2, -4.4, -4.6, -4.8

Borelog for well M36/4073 page 1 of 5

Gridref: M36:7722-3925 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 12 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Cable Tool
Drill Depth : -125.1m Drill Date : 20/11/1989

Environment Canterbury
Your regional council

Water Level: Artesian

Scale(m): -5, -10, -15, -20, -25

Depth(m)	Full Drillers Description	Formation Code
-0.75m	Topsoil & fill	fi
-2.00m	Grey silt	sp
-3.00m	Peat & timber	sp
-5.69m	Blue silts	sp
-7.00m	Peat & timber	sp
- 13.9m	Blue clay	sp
- 16.3m	Peat	sp
- 17.9m	Hard Green clay	sp
- 18.4m	Peat & clay lenses	sp
- 20.5m	Good clean gravel	ri
- 24.9m	Clean loose gravel	ri
- 27.9m	Tighter sandier gravel	ri

Borelog for well M36/4073

Gridref: M36:7722-3925 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 12 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Cable Tool
Drill Depth : -125.1m Drill Date : 20/11/1989

Environment Canterbury
Your regional council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
	Tighter sandier gravel	ri
- 27.9m	Very loose stained gravel	fi
- 31.0m	Yellow clay	
- 31.2m	Sandy stained gravel	ri
- 34.6m	Brown gravel	fi
- 36.0m	Yellow clay	
- 36.2m	Loose Brown gravel	ri
- 38.4m	Clay washed gravel	ri
- 39.4m	Claybound gravel	ri
- 40.4m	Very loose small gravel	ri
- 42.3m	Brown sand	br
- 49.3m	Blue gravel	br
- 49.6m	Blue silt	br
- 50.2m		

Scale(m): -30, -35, -40, -45, -50

Borelog for well M36/4073 page 3 of 5

Gridref: M36:7722-3925 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 12 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Cable Tool
Drill Depth : -125.1m Drill Date : 20/11/1989

Environment Canterbury
Your regional council

Water Level	Depth(m)	Full Drillers Description	Formation Code
Artesian	50.2m	Blue silt	br
	- 50.6m	Peat	br
	- 51.3m	Teal Green sticky clay	br
	- 52.2m	Yellow clay	br
	- 52.5m	Tight Brown sand	br
	- 52.7m	Yellow clay & gravel	li-1
	- 55.0m	Brown gravel	li-1
	- 55.3m	Sandy seam	li-1
	- 56.9m	Good big gravel	li-1
	- 57.5m	Yellow clay	li-2
	- 58.3m	Blue silty clay	li-2
	- 59.2m	Grey silt	li-2
	- 59.5m	Peat	li-2
	- 59.8m	Lenses peat & Grey clay	li-2
	- 60.3m	Very tight claybound gravel	li-2
	- 61.7m	Clean Water-bearing gravel	li-2
	- 62.0m	Yellow clay	li-2
	- 63.9m	Brown gravel	li-2
	- 65.0m	Good Water-bearing gravel	li-2
	- 65.3m	Rust stained gravel	li-2
	- 69.4m	Blue clay & silt	li-2
	- 69.7m	Peat	li-2
	- 70.5m	Grey clay	li-2
	- 71.5m	Yellow clay	li-2
	- 73.0m	Brown gravel Yellow clay lenses	li-3
	- 73.5m	Very sandy Brown gravel	li-3
	- 75.6m	Brown sand gravel lenses	li-3

Scale(m): -55, -60, -65, -70, -75

Borelog for well M36/4073 page 4 of 5

Gridref: M36:7722-3925 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 12 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Cable Tool
Drill Depth : -125.1m Drill Date : 20/11/1989

Environment Canterbury
Your regional council

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
75.6m	Brown sand gravel lenses	li-3
	Brown gravel Yellow clay	li-3
- 77.2m	Yellow clay	he
- 78.0m	Blue clay & vegetation	he
- 78.5m	Yellow clay	he
- 79.5m	Silty Yellow clay	he
- 80.0m	Coarse sand	he
- 80.7m	Fine free flowing Brown sand	he
- 91.0m	Tight claybound sand heavy clay content	he
- 93.0m	Blue clay	he
- 93.9m	Peat	he
- 94.6m	Lenses gravel in very soft Yellow silt	bu
- 95.5m	Very sandy Brown stained gravel	bu
- 96.8m	Sand	bu
- 97.5m	Peat	bu
- 99.1m	Brown gravel Yellow clay	bu
- 99.3m	Sandy gravel	bu
- 99.8m	Yellow clay	bu
- 100.0m		
- 101.3m		

Scale(m): -80, -85, -90, -95, -100

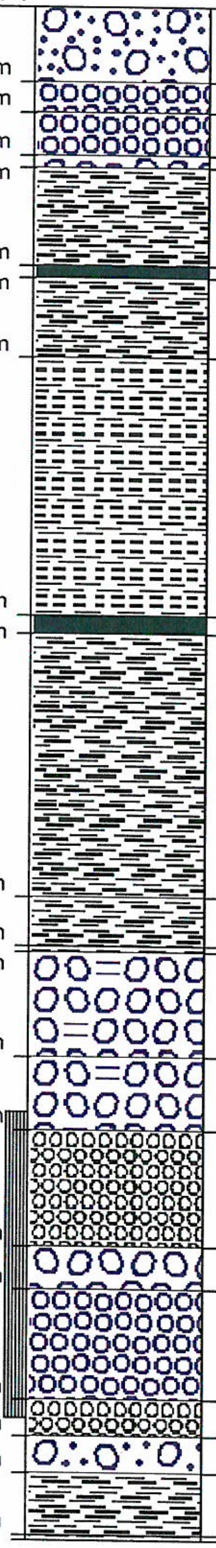
Borelog for well M36/4073 page 5 of 5

Gridref: M36:7722-3925 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 12 +MSD
Driller : Clemence Drilling Contractors
Drill Method : Cable Tool
Drill Depth : -125.1m Drill Date : 20/11/1989

Environment Canterbury
Your regional council

Scale(m): -105, -110, -115, -120, -125

Water Level: Artesian



Depth(m)	Full Drillers Description	Formation Code
- 101.3m	Sandy well sorted gravel	bu
- 101.8m	Good well sorted gravel	bu
- 102.5m	Brown stained gravel	bu
- 102.7m	Tight claybound gravel	bu
- 104.3m	Blue clay	sh
- 104.5m	Peat	sh
- 105.8m	Blue clay	sh
- 110.0m	Blue silty pug	sh
- 110.3m	Peat	sh
- 114.6m	Blue clay	sh
- 115.4m	Hard Yellow clay	sh
- 115.5m	Hard Yellow clay & Brown gravel	wa
- 117.2m	Claywashed loose gravel	wa
- 118.4m	Tight claywashed gravel lens Yellow clay	wa
- 120.3m	Small well sorted loose Blue/Brown gravel	wa
- 121.0m	Tighter big gravel	wa
- 122.8m	Good clean well sorted gravel	wa
- 123.4m	Pea gravel	wa
- 124.0m	Yellow clay/coarse sandy gravel	wa
- 125.0m	Yellow clay	wa
- 125.1m	Peat & wood	wa

Borelog for well M36/1021

Gridref: M36:772-391 Accuracy : 4 (1=best, 4=worst)
Ground Level Altitude : 13.5 +MSD
Driller : not known
Drill Method : Unknown
Drill Depth : -91.4m Drill Date : 1/05/1938

Water Level: Artesian

Depth(m)	Full Drillers Description	Formation Code
	Surface shingle	
- 14.0m	Blue clay	sp
- 20.0m	Brown shingle, water 0.9m from surface best at 23.1m	sp
- 39.6m	Blue clay & peat	ri
- 52.4m	Brown shingle, water 0.6m from surface	br
- 56.6m	Blue clay & peat	li-1
- 61.5m	Brown shingle, water rise 0.3m above	li-2
- 64.6m	Blue clay	li-2
- 70.4m	Brown shingle, water rises 0.6m above	li-2
- 76.5m	Brown sand	li-3
- 88.9m	Blue clay	he
- 89.0m	Brown shingle, flows at 98.2m3/d at surface & rises 2.1m	
- 91.4m	*>> description did not fit on this page.*	bu

Appendix 5 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Coronation Hall	Reviewer:	Alistair Boyce
		CPEng No:	209860
Building Address:	Unit: — No: 71 Street: Domain Terrace	Company:	Opus International
Legal Description:		Company project number:	6-QUCC1.06
		Company phone number:	+64 3 363 5400
GPS south:	Degrees 43, Min 33, Sec 17.90	Date of submission:	4-Feb-13
GPS east:	Degrees 172, Min 35, Sec 58.80	Inspection Date:	30-Aug-12
		Revision:	Final
Building Unique Identifier (CCC):	PRK_1099_BLDG_001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	silty sand	Soil Profile (if available):	
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):	14.00

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	14.50
Ground floor split?	yes	Ground floor elevation above ground (m):	0.50
Storeys below ground	0		
Foundation type:	strip footings	if Foundation type is other, describe:	
Building height (m):	4.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	6
Floor footprint area (approx):	352		
Age of Building (years):	100	Date of design:	Pre 1935
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):	public		
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	timber purlins, metal roof over sarking
Floors:	timber	joist depth and spacing (mm)	150x50 @ 450
Beams:	none	overall depth x width (mm x mm)	
Columns:	timber	typical dimensions (mm x mm)	
Walls:	non-load bearing		0

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	lightweight timber framed walls		note typical wall length (m)	3
Ductility assumed, μ:	2.00			
Period along:	0.20	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated
Lateral system across:	lightweight timber framed walls		note typical wall length (m)	4
Ductility assumed, μ:	2.00			
Period across:	0.20	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:	other (specify)	describe	None
Wall cladding:	other light	describe	Timber
Roof Cladding:	Metal	describe	
Glazing:	timber frames		
Ceilings:	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:	Okay	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:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Not assessed
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
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:	none	Describe:	
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before:	43%	##### %NBS from IEP below
	Assessed %NBS after:	43%	
Across	Assessed %NBS before:	79%	##### %NBS from IEP below
	Assessed %NBS after:	79%	

Opus International Consultants Ltd
20 Moorhouse Avenue
PO Box 1482, Christchurch Mail Centre,
Christchurch 8140
New Zealand

t: +64 3 363 5400
f: +64 3 365 7858
w: www.opus.co.nz