Christchurch City Council

Riccarton Community Centre PRO 0537-002

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Riccarton Community Centre

Quantitative Assessment Report

Prepared By

Benjamin Weaver
Structural Engineer

Reviewed By

Will Parker
Principal Structural Engineer, CPEng

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

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Summary

Riccarton Community Centre, 199 Clarence Street
PRO 0537-002

Detailed Engineering Evaluation
Quantitative Report - Summary
Final R2

Christchurch City Council appointed Opus International Consultants to carry out a detailed seismic assessment of the Riccarton Community Centre building located at 199 Clarence Street, Christchurch. The key outcome of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards.

Findings of the assessment are:

1. The seismic performance of the original building is governed by the original brick masonry exterior walls, which have an expected strength of 4%NBS in the transverse direction (north-south) and 2%NBS in the longitudinal direction (east-west). The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

2. The seismic performance of the 1960 addition is governed by the brick veneer walls in the longitudinal direction (east-west), which have a capacity of 5%NBS, and by the Steel portal frames in the transverse direction which have an expected strength of 22%NBS. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

3. The seismic performance of the 1968 addition is governed by the possibility of pounding, which gives an expected NBS of 60%, otherwise it is governed by the flexural capacity of the reinforced concrete frames, which have an expected strength of greater than 100%NBS in both the longitudinal and transverse directions.

4. The seismic performance of the 1986 addition is governed by the possibility of pounding, which gives an expected NBS of 60%, otherwise it is governed by the shear capacity of the concrete blockwork walls, which have an expected strength of greater than 100%NBS in both the longitudinal and transverse directions.

We recommend strengthening of the buildings be undertaken, with a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Our concept strengthening scheme to achieve this would include:

1. Introducing new seismic resisting elements with the intention of:
   - Reducing the displacement demand on the existing brick masonry piers in the transverse direction
   - Providing a complete lateral load resisting system in the transverse direction.
2. Introduce a new seismic resisting elements in the 1960 addition with the intention of:

- Providing a complete lateral load resisting system in the longitudinal direction.
- Increasing the flexural capacity of the portal frame beams in the transverse direction.

3. Providing ties between the various additions to provide strength to some of the weaker portions of the centre.
4. Repair of all current earthquake induced damage to both buildings.
5. Upgrade of building foundations as necessary to achieve the above items.

It is recommended that the original building and the 1960 addition remain unoccupied, given its structural weaknesses and the elevated level of seismic risk in Christchurch

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Riccarton Community Centre, located at 199 Clarence Street, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

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 qualitative and 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.

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	Improvement of Structural Performance	
					Legal Requirement	NZSEE Recommendation
Low Risk Building	A or B	Low	Above 67	Acceptable (improvement may be desirable)	The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
Moderate Risk Building	B or C	Moderate	34 to 66	Acceptable legally. Improvement recommended		Not recommended. Acceptable only in exceptional circumstances
High Risk Building	D or E	High	33 or lower	Unacceptable (Improvement required under Act)	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).

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 Building Description

The Riccarton Community Centre, located at 199 Clarence Street, is made up of four independent structures. These are the original building, whose original construction date is unknown, and additions in 1960, 1968, and 1986.

The original building and the additions are separated by a 10mm or less seismic separation at all levels above ground.

Figure 2: Site Plan

4.1 Original Building

The original building is a single storey hall structure, 18 metres long by 11 metres wide.

No drawings were available for the original building, but based on site observations it appears that the building dates to the 1930's or 1940's.

The gravity load system consists of triple thick brick masonry walls with concrete lintels over window and door openings. The masonry walls do not have any brick ties. The roof is pitched with a height of 7m at the peak. Timber scissor roof trusses support 100x50mm purlins under tongue and groove sarking. The roof trusses are nailed to a 200x80mm top wall plate. No bolts from the top wall plate were observed into the brick walls. The ground level is a suspended timber floor system.

The foundation system appears to be continuous concrete footings.

Lateral load resistance is provided by in-plane and out-of-plane bending in the masonry walls running in the longitudinal direction (east-west). Lateral load in the longitudinal direction is resisted by in-plane bending, while load in the transverse direction is resisted by out-of-plane bending. There are no end walls in the transverse direction. No seismic connection is present between the roof diaphragm and the wall elements.

4.2 1960 Addition

The 1960 Addition is a single storey steel portal frame structure, which fits around the west end of the original building. The addition is approximately 19 metres long by 19 metres wide, with sloping decks to match the levels of the original building.

The gravity load system consists of transverse pitched steel portal frames with a height of 7m at the peak. The portal frames support 200x50mm purlins at 700mm centres under 150x25mm diagonal sarking. The framing at the ground level is a suspended timber floor with diagonal sarking over 125x50mm joists. The suspended floor system is supported by continuous footings. The exterior walls have a 100mm thick brick veneer with 8g wire ties every fourth course.

The foundation system is continuous concrete footings.

Lateral load resistance in the transverse direction is provided by frame action in the steel portal frames. No lateral load resisting system is provided in the longitudinal direction, so lateral resistance is most likely provided by the brick veneer.

4.3 1968 Addition

The 1968 Addition is a single storey concrete frame structure, which fits around the east end of the original building. The addition is approximately 14 metres long by 21 metres wide.

The gravity load system consists of reinforced concrete frames in both directions. The roof is flat with a height of approximately 4m. Concrete frames support 300x50mm purlins under 100x25mm diagonal sarking. Frames are infilled with 250mm thick concrete blockwork, horizontally grouted at 600mm centres, and vertically grouted at wall ends. The ground level has a 100mm thick slab on grade.

The foundation system is continuous concrete footings.

Lateral load resistance is provided by frame action in both the transverse and longitudinal directions. The infill blockwork is the stiffest lateral element, but the building has been analysed assuming all strength is provided by frame action only.

4.4 1986 Addition

The 1986 Addition is a single storey concrete blockwork structure, along the south end of the original building. The addition is approximately 13 metres long by 3 metres wide.

The gravity load system consists of concrete blockwork walls in both directions. The roof has a height of approximately 3m. The roof is framed with 150x50mm sloping purlins at 600mm centres under 18mm ply sarking. The purlins are nailed to a 200x50mm top plate on the blockwork wall which is in turn bolted with an M12 bolt at 1200mm centres. The ground level has a suspended timber floor.

The foundation system is continuous concrete footings.

Lateral load resistance is provided by in-plane bending in the blockwork walls. The walls are grouted and reinforced at 800mm centres.

4.5 Original Documentation

Copies of the following construction drawings were provided by CCC on 24 May 2011:

- Additions and Alterations To Riccarton Town Hall, Architectural Drawings, Griffiths and Moffat Registered Architects, September 1960
- Additions and Alterations To Riccarton Town Hall, Structural Drawings, Powell Fenwick and Partners Consulting Engineers, September 1960
- Alterations and Additions to the Riccarton Town Hall, Architectural Drawings, Griffiths Moffat and Partners, February 1960
- Alterations and Additions to the Riccarton Town Hall, Structural Drawings, Griffiths Moffat and Partners, February 1960
- Alterations and Addition 1987 Riccarton Town Hall, Architectural Drawings, Austin and Warren Registered Architects, February 1987

The drawings have been used to confirm the structural systems, investigate potential critical structural weaknesses (CSW) and identify details which required particular attention.

Structural drawings have not been located for the original building.

5 Survey

5.1 Post 22 February 2011 Rapid Assessment (Level 2)

A structural (Level 2) assessment of the above buildings/property was undertaken on 16 March 2011 by Opus International Consultants.

5.2 Post 22 February 2011 Rapid Assessment (Level3)

A structural (Level 3) assessment of the above buildings/property was undertaken on 21 March 2011 by Opus International Consultants.

5.3 Further Inspections

A further inspection was undertaken by Opus International Consultants on 30 June 2011.

These inspections included external and internal visual inspections of all structural elements above foundation level, and areas of damage to structural and non-structural elements.

6 Damage Assessment

The structural damage observed prior to 13 June 2011 has been captured in the Riccarton Community Centre Structural Damage Report (Level 3 Assessment), issued to Christchurch City Council on 4 April 2011. Post 13 June 2011 damage has been captured in the Riccarton Community Centre Qualitative Assessment Summary issued to the Christchurch City Council on 8 August 2011. Both reports should be referred to in addition to this report.

The following damage has been noted:

6.1 Original Building

The brick (3 layers) cavity walls on the northern and southern sides of the hall (above the internal windows) sustained moderate cracking of the plaster and possibly brick work. The wall and pilaster columns (external) appear to be out of plumb by approximately 20mm. The cracking appears to be caused by rotation of the concrete lintel at the lintel seat in the masonry wall.

6.2 1960 Addition

Vertical and horizontal cracks to the brick veneer gable end wall on the western side of the building above the window line.

6.3 1968 Addition

No damage was observed.

6.4 1986 Addition

No damage was observed.

6.5 Ground Damage

Site photos taken on 16th March 2011 show no liquefaction at the site. A walkover inspection of the exterior of the building and surrounding sites was completed on 14 July 2011. Interior access was not obtained during the survey. No cracking was noted in the car park area, or between the building and adjacent surfacing. No evidence of liquefaction, such as sand boils, was noted around the buildings or in the car park area. One sand boil was mapped beside the eastern-most of the two concrete water tanks to the south of the Community Centre. The sand boil was approximately 1 m x 0.5 m size. No differential settlement between the ground and the building appears to have occurred.

7 General Observations

The original building and its additions appear to have generally performed well during the earthquake.

The original building sustained moderate damage to structural elements, as well as some moderate damage to non-structural elements. The 1960 addition sustained some minor damage to non-structural elements. The observed damage is light compared to some of the possible deficiencies noted in the buildings, following a review of the structural drawings and site investigations.

8 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.

8.1 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. During the initial qualitative stage of the assessment the following potential CSW's were identified for each of the buildings and have been considered in the quantitative analysis.

8.1.1 Original Building

a) Pounding potential on the on the east, west, and south sides due to the proximity with the 1960, 1968, and 1986 additions, respectively.

b) The lateral system in the transverse direction is unconfined masonry shear walls. These walls have limited or no ductility due to a lack of reinforcement and no confinement. Any failure in these walls, especially under high cyclic loading, can lead to a failure of lateral and gravity load carrying capacity.

c) There is no apparent lateral load resisting system in the north south direction. The walls on the north and the south end of the building must resist the lateral load via out of plane bending. These walls have limited or no ductility due to a lack of reinforcement and no confinement. Any failure in these walls, especially under high cyclic loading, can lead to a failure of lateral and gravity load carrying capacity.

d) The top plate which the scissor trusses affix to is not bolted to the masonry walls. Shear transfer is currently provided by friction between the top plate and the wall. High roof accelerations could cause the roof to lose bearing and collapse.

8.1.2 1960 Addition

a) The bottom flange of the transverse portal frames is unbraced for the full length of the portal frames. This can lead to a premature lateral torsional buckling failure before the full flexural capacity of the beam section can be reached.

b) Pounding potential on the east side from the original building.

c) There is no explicit lateral system in the east-west direction. The architectural brick veneer is the only element to resist lateral load. This is a non-ductile lateral load resisting element.

8.1.3 1968 Addition

a) Pounding potential on the east side from the original building.

8.1.4 1986 Addition

a) Pounding potential on the north side from the original building.

8.2 Quantitative Assessment Methodology

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

The original building and additions were analysed using a variety of calculation methods. A mixture of hand calculation and two and three dimensional computer modelling were employed during the quantitative assessment phase as noted below:

- Original Building: Hand calculations.
- 1960 Addition: Hand calculations, 2D model of steel portal frames in the computer analysis program RISA 3D.
- 1968 Addition: Hand calculations, 3D model of the building in the computer analysis program ETABS, section analysis in the computer analysis program XTRACT.
- 1986 Addition: Hand calculations.

For hand and 2D calculations, a static analysis has been carried out using ordinate of the hazard spectrum at the period of vibration, T1, from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1). The period of vibration was calculated using Empirical Method A.

For 3D analyses, a modal response spectrum analysis has been carried out using the spectral values established from NZS1170.5, with an updated z factor of 0.3 (B1/VM1). This analysis was used to establish the actions on the structural elements.

Based on the actions determined from the analyses, demand to capacity ratios (DCR's) were determined for each component in question. The highest DCR was then converted to a %NBS for the structure.

8.3 Quantitative Assessment

A summary of the structural performance of the building is shown in the following tables. Note that the values given are generally the worst performing as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity (for example the walls added to the original building in 1986). This will be considered further when developing the strengthening options.

Table 2: Summary of Seismic Performance – Original Building

Structural Element/System	Failure mode and description of limiting criteria based on capacity of critical element	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Walls – North South	Shear failure due to in-plane loading, resulting in compression failure of unconfined masonry under repeated cycles of load and lack of confining reinforcement. Once the wall becomes unstable the wall loses its ability to take gravity load i.e. support of the roof above.	Yes	4%
Walls – North South	Flexural failure in the out-of-plane direction, resulting in compression failure of cantilevered unconfined unreinforced brick masonry under repeated cycles of load and lack of confining reinforcement. Once the wall becomes unstable in the potential plastic hinge zone, the wall loses its ability to take gravity load i.e. support of the roof above. There is no seismic resisting system in the transverse direction, so the walls on the north and south must resist the lateral load in the out-of-plane direction.	Yes	2%
Truss connection to masonry walls	Shear transfer failure from the truss and top plate connection to the masonry wall. No positive connection between the top plate and the masonry wall was observed. Shear transfer is currently provided by friction between the top plate and the wall. High roof accelerations could cause the roof to lose bearing and collapse.	Yes	0%
Pounding	There is a pounding hazard with the 1960, 1968 and 1986 Additions. The additions were built without sufficient seismic gaps. The additions were built with a variety of lateral load resisting systems, so the various buildings will have different fundamental periods of vibration. Although this is unlikely be the initiator of collapse, damage will be increased because of this effect.	May be critical	60%

Table 3: Summary of Seismic Performance – 1960 Addition

Structural Element/System	Failure mode and description of limiting criteria based on capacity of critical element	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Frames – North South Direction	Lateral torsional buckling failure in the transverse portal frame beams. High cyclic loading in the frames can lead to a loss of lateral and then gravity load carrying capacity for the building.	Yes	22%
Brick Veneer – Both Directions	Compression failure under cyclic loading, due to limited wire ties in the brick veneer resulting in a loss of lateral load carrying capacity.	Yes	5%
No Lateral System – East West Direction	No lateral load resistance, due to the lack of any discernible lateral load resisting system in the east west direction. A review of the drawings and various site visits show there is no lateral load resisting in this direction. This leaves the brick veneer, as the stiffest element in the load path, to resist lateral load in this direction. The brick veneer lacks the required lateral load carrying capacity.	Yes	5%
No diaphragm to distribute lateral load in the East-West Direction.	No diaphragm is present to distribute lateral load to any present or future lateral load resisting elements in the east west direction. This is an incomplete load path.	Yes	0%

Structural Element/System	Failure mode and description of limiting criteria based on capacity of critical element	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Pounding	There is a pounding hazard with the original building and the 1968 and 1986 Additions. The additions were built without sufficient seismic gaps. The additions were built with a variety of lateral load resisting systems, so the various buildings will have different fundamental periods of vibration. Although this is unlikely be the initiator of collapse, damage will be increased because of this effect.	May be critical	60%

Table 4: Summary of Seismic Performance – 1968 Addition

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
Pounding	There is a pounding hazard with the original building and the 1960 and 1986 Additions. The additions were built without sufficient seismic gaps. The additions were built with a variety of lateral load resisting systems, so the various buildings will have different fundamental periods of vibration. Although this is unlikely be the initiator of collapse, damage will be increased because of this effect.	May be critical	60%

Table 5: Summary of Seismic Performance – 1986 Addition

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
Pounding	There is a pounding hazard with the original building and the 1960, 1968 Additions. The additions were built without sufficient seismic gaps. The additions were built with a variety of lateral load resisting systems, so the various buildings will have different fundamental periods of vibration. Although this is unlikely be the initiator of collapse, damage will be increased because of this effect.	May be critical	60%

8.4 Discussion of Results

The seismic performance of the original building is governed by the strength of the unreinforced brick masonry walls, which have an expected strength of 4%NBS in the longitudinal direction (north-south) and 2%NBS in the transverse direction (east-west). The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

The seismic performance of the 1960 addition is governed by the shear strength of the architectural brick veneer, which have a capacity of 5%NBS in the longitudinal direction (east-west). The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

The 1968 and 1986 additions are considered to have a capacity of greater than 100%NBS. These additions are therefore not considered to be earthquake prone in accordance with the Building Act 2004.

All of the buildings are potentially affected by pounding against adjacent structures, however this is not expected to initiate collapse but would cause an increase in damage.

8.5 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 building to be reduced; therefore the current capacity of the building 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.
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.

9 Summary of Geotechnical Appraisal

9.1 Ground Conditions

A desk study of well logs in the area obtained from Environment Canterbury records identified four drill logs from boreholes located within the pump station area immediately to the south of the Community Centre building. These boreholes were drilled for the public water supply in 1945 and 1950. The borehole logs indicate the area is underlain by a 15.5 m – 17.5 m thick layer of sand and clay, which is underlain by gravel.

9.2 Liquefaction Hazard

The 2004 ECan Christchurch Liquefaction Study shows the Riccarton area was predicted to have various liquefaction potential, from no liquefaction potential to high liquefaction potential, under low groundwater conditions. The Riccarton Community Centre is within, or very close to, an area of high liquefaction potential. Ground damage is expected to be moderate, indicating subsidence is likely to be 100 mm – 300 mm (Environment Canterbury, 2004).

9.3 Summary

Only very minor liquefaction has occurred at the site following the earthquakes. No evidence of building settlement was observed during the site walkover, nor was any evidence of differential settlement observed during the interior inspection of the building. Therefore it is unlikely the damage to the Community Centre was caused by ground damage. No further geotechnical investigations will be required at the site prior to building repair and remediation. The existing foundations appear to have performed well under seismic loading.

10 Remedial Options

Strengthening of the buildings, to a target of increasing the seismic performance to as near as practicable to 100%NBS, and at least 67%NBS. Our concept strengthening scheme to achieve this would include:

1. Introducing new seismic resisting elements to the original building with the intention of reducing the seismic demand on the original shear walls in the in-plane direction.
2. Introduce new lateral load resisting elements at the east and west ends of the original building with the intention of providing an adequate lateral load resisting system in the transverse direction.
3. Introduce new wall ties from the roof to the existing north and south masonry walls in the original building, with the intention of reducing the out-of-plane bending demands on the original shear walls.
4. Strengthen the roof diaphragm of the original building so that lateral load can be delivered to the new lateral load resisting elements at the east and west ends of the building.
5. Introduce new seismic resisting elements in the east west direction of the 1960 addition with the intention of reducing the drift in this direction and thus the lateral load demand on the brittle architectural brick veneer.
6. Introduce new bracing elements within selected portal frames in the north south direction of the 1960 addition, with the intention of reducing the drift in this direction and thus the lateral load demand on the brittle architectural brick veneer.
7. Distribute lateral roof loads to new and existing lateral load resisting elements in the 1960 addition.
8. Add new lateral bracing for the portal frame beams in the transverse direction of the 1960 addition increasing the flexural capacity of the portal frame beams.
9. Repair of all current earthquake induced damage to both buildings.

11 Conclusions

a) The seismic performance of the original building is governed by the strength of the unreinforced brick masonry walls, which have an expected strength of 4%NBS in the longitudinal direction (north-south) and 2%NBS in the transverse direction (east-west). The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

b) The seismic performance of the 1960 addition is governed by the shear strength of the architectural brick veneer, which have a capacity of 5%NBS in the longitudinal direction (east-west). The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

c) The 1968 and 1986 additions have a potential risk of pounding (with an expected strength of 60%NBS), but are otherwise considered to have a capacity of greater than 100%NBS. These additions are therefore not considered to be earthquake prone in accordance with the Building Act 2004.

d) All of the buildings are potentially affected by pounding against adjacent structures, however this is not expected to initiate collapse but would cause an increase in damage.

12 Recommendations

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) A quantity surveyor be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.

c) It is recommended that the original building and the 1960 addition remain unoccupied, given their earthquake prone building status and the elevated level of seismic risk in Christchurch.

13 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 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

b) Our inspections have been both visual and intrusive, and some linings or finishes were removed to expose structural elements. Calculations and analyses have been performed to reach the conclusions discussed herein. 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) Due to the limited earthquake related ground effects, the geotechnical appraisal was limited to a desk top study.

d) 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.

14 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 - Assessment Assumptions and Methodology

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.

- Faculty of Engineering, The University of Auckland, *Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance*, Draft Prepared by the Faculty of Engineering, The University of Auckland, February 2011.

Analysis Parameters

The following parameters were used for the seismic analysis:

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

- Seismic hazard factor — Cl. 2.2.14$_B$, 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 = 1.0 (Original Building – Both Directions)
  μ = 1.25 (1960 Addition – Transverse Direction)
  μ = 1.0 (1960 Addition – Longitudinal Direction)
  μ = 3.0 (1968 Addition – Both Directions)
  μ = 2.0 (1968 Addition – Both Directions)

- Structural performance factor Cl. 2.6.2.2, NZS3101:2006
  S_p = 1.0 (Original Building – Both Directions)
  S_p = 0.7 (1960 Addition – Transverse Direction)
  S_p = 0.7 (1960 Addition – Longitudinal Direction)
  S_p = 0.7 (1968 Addition – Both Directions)
  S_p = 0.7 (1960 Addition – Both Directions)

- Material properties

Table A2: Analysis Material Properties

	All Buildings
Average brick compressive strength, f'_b (MPa) (1)	26.9
Concrete compressive strength, f'_c (MPa) (2)	41
Mild reinforcing yield strength, f_y (MPa)	300
Portal frame steel yield strength, f_y (MPa)	250

Notes:
1. Base on guidance from *Assessment and Improvement of Unreinforced Masonry Buildings for Earthquake Resistance*, Table 2.2 for Medium Hardness Scratch Index
2. Based on guidance from *NZSEE 2006*, probable concrete compressive strength is based on a value of 1.5 times the nominal compressive strength (Cl 7.1.1)

- Effective section properties

Table A3: Effective section properties from NZS3101:2006

Table C6.6 – Effective section properties, I_e

Type of member	Ultimate limit state		Serviceability limit state		
	f_y = 300 MPa	f_y = 500 MPa	μ = 1.25	μ = 3	μ = 6
1 Beams					
(a) Rectangular ¶	0.40 I_g (use with E_{40}) §	0.32 I_g (use with E_{40}) §	I_g	0.7 I_g	0.40 I_g (use with E_{40})§
(b) T and L beams ¶	0.35 I_g (use with E_{40}) §	0.27 I_g (use with E_{40}) §	I_g	0.6 I_g	0.35 I_g (use with E_{40})§
2 Columns					
(a) $N^*/A_g f'_c > 0.5$	0.80 I_g (1.0 I_g) ‡	0.80 I_g (1.0 I_g) ‡	I_g	1.0 I_g	As for the ultimate limit state values in brackets
(b) $N^*/A_g f'_c = 0.2$	0.55 I_g (0.66 I_g) ‡	0.50 I_g (0.66 I_g) ‡	I_g	0.8 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.40 I_g (0.45 I_g) ‡	0.30 I_g (0.35 I_g) ‡	I_g	0.7 I_g	
3 Walls ¶					
(a) $N^*/A_g f'_c = 0.2$	0.48 I_g	0.42 I_g	I_g	0.7 I_g	As for the ultimate limit state values
(b) $N^*/A_g f'_c = 0.1$	0.40 I_g	0.33 I_g	I_g	0.6 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.32 I_g	0.25 I_g	I_g	0.5 I_g	
4 Diagonally reinforced coupling beams	0.6I_g for flexure Shear area, A_{shear}, as in text		I_g 1.5 A_{shear} for ULS	0.75 I_g 1.25 A_{shear} for ULS	As for ultimate limit state

NOTES –
(§) With these values the E value should be the elastic modulus for concrete with a strength of 40 MPa regardless of the actual concrete strength.
(‡) The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.
(¶) For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

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

- Building seismic weight — Cl. 4.2, NZS1170.5
  W_t = G
  W_t =245 kN (Original Building)
  W_t =260 kN (1960 Addition)
  W_t =307 kN (1968 Addition)
  W_t =96 kN (1986 Addition)

Assessment Methodology

Static & Modal Spectrum Analysis

Original Building:

The seismic assessment was performed via hand calculation using the method set forth in the Draft Guideline for the Seismic Assessment of URM Buildings prepared by the University of Auckland. Because no drawings were available, calculations were based on default material listed in Section 2 of the Draft document. In-plane wall actions and response were checked in accordance with Section 4 and 8 of the Guideline.

Out-of-plane wall actions and response were checked in accordance with Sections 6 and 11.

Diaphragm actions and response were checked in accordance with Sections 5, 7, and 10.

1960 Addition:

The seismic assessment was undertaken by a combination of calculation methods.

In the longitudinal direction, hand calculations were performed per the method described above as for the original building.

In the transverse direction a static analysis of the building was performed in accordance with NZS 1170.5:2004. A 2D model of the portal frames was set up using the structural analysis program SAP2000.

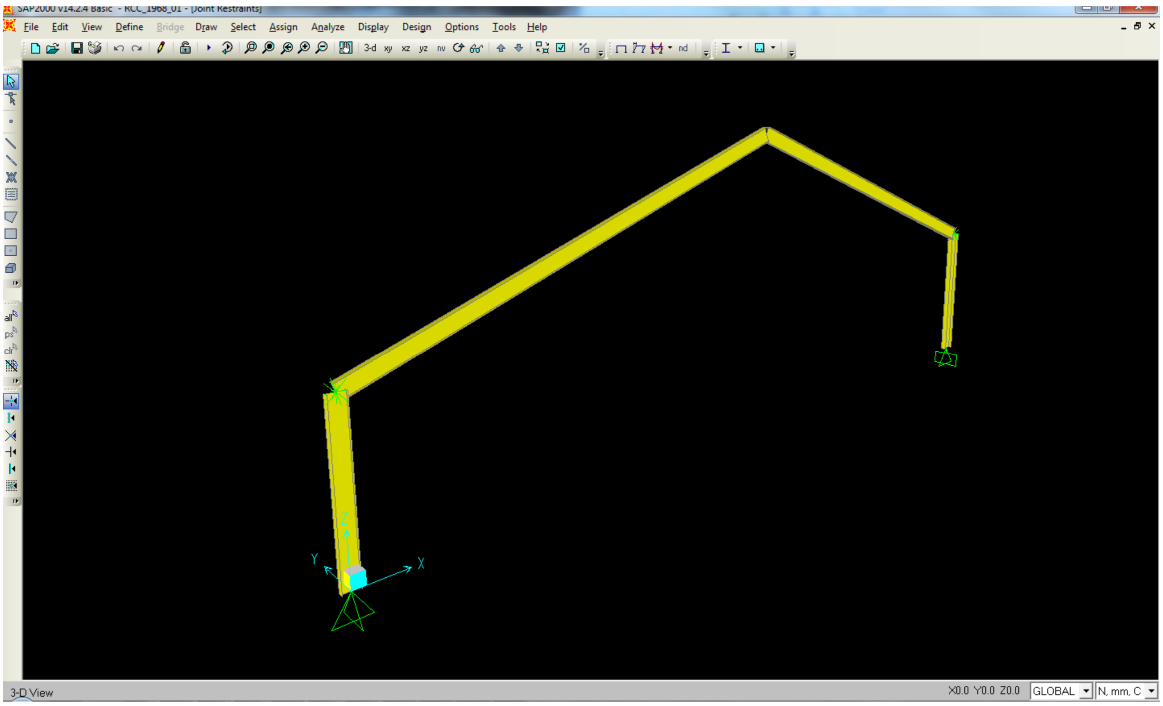


Figure A1: SAP2000 model of 1960 Addition

The fundamental building periods output from ETABS were:

$T_1 = 0.24$ sec

Based on the fundamental building period and assumed ductility capacity, the following equivalent static seismic coefficient was calculated from NZS1170.5, Clause 5.2:

$C_d = 0.95$ N/S direction

1968 Addition:

The seismic assessment was undertaken by completing a static and modal response spectrum (MRS) analysis for the building in accordance with NZS 1170.5:2004.

A 3D model was set up using the structural analysis program ETABS, and effective section properties for structural members were taken from Table A2 above.

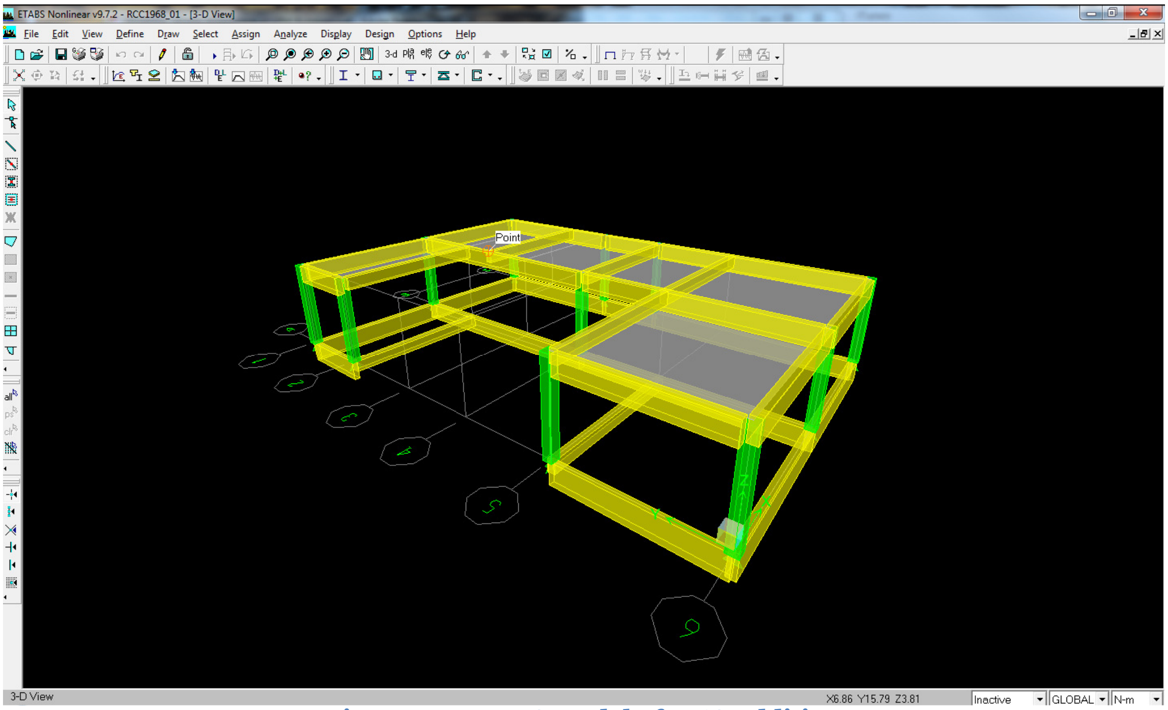


Figure A2: ETABS model of 1968 Addition

The fundamental building periods output from ETABS were:

T_1 = 0.29 sec (N/S direction)
T_1 = 0.30 sec (E/W direction)

The structural irregularity features of Clause 4.5 were checked, and the 1968 Addition was found to be irregular in plan due to torsional sensitivity. Due to this plan irregularity, the model response spectrum analysis was scaled to 100% of the equivalent static base shear (Cl. 5.2.2.2).

This addition was analysed as being ductile (μ = 3.0), so the design actions were assumed to act separately in each of the two horizontal directions. Allowance was made for accidental eccentricity in the application of actions, as required by Clause 5.3.2.

An equivalent static analysis was carried out as a consistency check of the MRS analysis outputs. Based on the fundamental building periods and assumed ductility capacities, the following equivalent static seismic coefficients were calculated from NZS1170.5, Clause 5.2:

C_d = 0.38 N/S direction
C_d = 0.38 E/W direction

1986 Addition:

The seismic assessment was undertaken by completing a static analysis for the building in accordance with NZS 1170.5:2004.

The fundamental building period was estimated per NZS1170.5, Clause 4.1:

T_1 = 0.29 sec (Both Directions)

The structural irregularity features of Clause 4.5 were checked, and no irregularities were found.

This addition was analysed as being ductile ($\mu = 2.0$), so the design actions were assumed to act separately in each of the two horizontal directions.

Based on the fundamental building periods and assumed ductility capacity, the following equivalent static seismic coefficient was calculated from NZS1170.5, Clause 5.2:

$C_d = 0.52$ (Both Directions)

Appendix 2 - Photographs

Riccarton Community Centre – 199 Clarence Street		
No.	**Item description**	**Photo**
1.	View of the east side of the 1968 addition.	
2.	The interior of the original building.	
3.	Scissor trusses in the original building.	

4.	Connection of the scissor truss to the top plate on the masonry wall.	
5.	North wall of the original building.	
6.	Cracking in masonry wall and plaster at the concrete lintel seat on the north wall of the original building.	

7.	Cracking in masonry wall and plaster at the concrete lintel seat on the north wall of the original building.	
8.	South wall of the original building.	
9.	Interface of the 1960 addition with the original building.	
10.	Easternmost portal frame of the 1960 addition.	

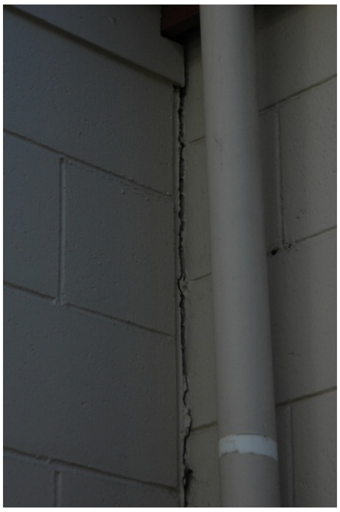
11.	Portal frame knee in the 1960 addition.	
12.	Portal frame knee connection in the 1960 addition.	
13.	Portal frame in the 1960 addition.	

14.	Portal frame in the 1960 addition.	
15.	Portal frame in the 1960 addition.	
16.	Portal frame knee connection in the 1960 addition.	

17.	North wing of the 1968 addition.	
18.	Concrete column with block infill in the 1968 addition.	
19.	Northeast corner beam-column connection of the 1968 addition.	

20.	Northeast corner beam-column connection of the 1968 addition.	
21.	Interface of the 1968 addition with the original building.	
22.	Cracking in masonry wall at the concrete lintel seat on the north wall of the original building.	

23.	Cracking at the interface of the masonry walls of the original building with the veneer of the 1960 addition.	
24.	South face of the 1986 addition.	
25.	Interface of the veneer of the 1960 addition with the walls of the 1986 addition.	

26.	Interface of the wall of the 1986 addition with the walls of the 1968 addition.	
27.	North wall of the original building and part of the 1960 addition.	

Appendix 3 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Riccarton Community Centre - Original Building	Reviewer:	Alistair Boyce
	Unit / No: 199 / Street: Clarence Street	CPEng No:	209860
Building Address:	199 Clarence Street	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.11
		Company phone number:	6433657858
GPS south:	Degrees 43, Min 31, Sec 51.53	Date of submission:	4-Sep-13
GPS east:	Degrees 172, Min 36, Sec 5.84	Inspection Date:	
		Revision:	Final R2
Building Unique Identifier (CCC):	BU 0537-002 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
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:	N/A
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	5.00

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	5.50
Ground floor split?	yes	Ground floor elevation above ground (m):	0.50
Storeys below ground	0		
Foundation type:	pads with tie beams	if Foundation type is other, describe:	
Building height (m):	5.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	4.2
Floor footprint area (approx):			
Age of Building (years):		Date of design:	1935-1965
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	load bearing walls		
Roof:	timber truss	truss depth, purlin type and cladding	
Floors:	timber	joist depth and spacing (mm)	
Beams:			
Columns:	brick masonry	typical dimensions (mm x mm)	
Walls:	load bearing brick	#N/A	

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - brick	**Note: Define along and across in detailed report!**	note wall thickness and cavity	
Ductility assumed, μ:	1.00			
Period along:	0.40	0.40 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	unreinforced masonry bearing wall - brick		note wall thickness and cavity	
Ductility assumed, μ:	1.00			
Period across:	0.40	0.00	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):		leave blank if not relevant
east (mm):	0	
south (mm):	0	
west (mm):	0	

Non-structural elements

Stairs:	cast insitu	notes	
Wall cladding:			
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	none	original designer name/date	
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	Minor evidence of liquefaction and ground cracking	Describe damage:	
Settlement:		notes (if applicable):	
Differential settlement:		notes (if applicable):	
Liquefaction:	0-2 m²/100m³	notes (if applicable):	
Lateral Spread:		notes (if applicable):	
Differential lateral spread:		notes (if applicable):	
Ground cracks:	0-20mm/20m	notes (if applicable):	
Damage to area:		notes (if applicable):	

Building:

	Current Placard Status:	red		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	yes	Describe:	Heavy cracking to window lintels
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Cracking to gib board, plaster ceiling

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	
Along	Assessed %NBS before:	4% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative assessment
	Assessed %NBS after:	4%		
Across	Assessed %NBS before:	2% ##### %NBS from IEP below		
	Assessed %NBS after:	2%		

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Riccarton Community Centre - 1960 Addition		Reviewer:	Alistair Boyce
	Unit No:	Street	CPEng No:	209860
Building Address:	199	Clarence Street	Company:	Opus International Consultants
Legal Description:			Company project number:	6-QUCCC.11
			Company phone number:	6433657858
	Degrees / Min	Sec		
GPS south:	43 / 31	51.53	Date of submission:	4-Sep-13
GPS east:	172 / 36	5.84	Inspection Date:	
			Revision:	Final R2
Building Unique Identifier (CCC):	BU 0537-002 EQ2		Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
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:	N/A
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	5.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	5.20
Ground floor split?	no		Ground floor elevation above ground (m):	0.20
Storeys below ground	0			
Foundation type:	pads with tie beams		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	5
Floor footprint area (approx):				
Age of Building (years):			Date of design:	1935-1965
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	other (note)	describe sytem	Suspended timber floor
Beams:	steel non-composite	beam and connector type	
Columns:	structural steel	typical dimensions (mm x mm)	
Walls:	non-load bearing	0	

Lateral load resisting structure

Lateral system along:	unreinforced masonry bearing wall - brick	**Note: Define along and across in detailed report!**	note wall thickness and cavity	Brick veneer
Ductility assumed, μ:	1.25			
Period along:	0.47	0.40 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	welded and bolted steel moment frame		note typical bay length (m)	19
Ductility assumed, μ:	1.25			
Period across:	0.47	0.00	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):	0	leave blank if not relevant
east (mm):	0	
south (mm):	0	
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:	brick or tile	describe (note cavity if exists)	
Roof Cladding:	Membrane	substrate	
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	none	original designer name/date	
Structural	full	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:	Minor evidence of liquefaction and ground cracking	Describe damage:	
Settlement:		notes (if applicable):	
Differential settlement:		notes (if applicable):	
Liquefaction:	0-2 m²/100m³	notes (if applicable):	
Lateral Spread:		notes (if applicable):	
Differential lateral spread:		notes (if applicable):	
Ground cracks:	0-20mm/20m	notes (if applicable):	
Damage to area:		notes (if applicable):	

Building:

	Current Placard Status:	yellow		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	Cracks to brick veneer

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	
Along	Assessed %NBS before:	5%	##### %NBS from IEP below	If IEP not used, please detail assessment methodology: Quantitative assessment
	Assessed %NBS after:	5%		
Across	Assessed %NBS before:	5%	##### %NBS from IEP below	
	Assessed %NBS after:	5%		

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Riccarton Community Centre - 1968 Addition	Reviewer:	Alistair Boyce
	Unit No: Street	CPEng No:	209860
Building Address:	199 Clarence Street	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.11
		Company phone number:	6433657858
	Degrees Min Sec		
GPS south:	43 31 51.53	Date of submission:	4-Sep-13
GPS east:	172 36 5.84	Inspection Date:	
		Revision:	Final R2
Building Unique Identifier (CCC):	BU 0537-002 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
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:	N/A
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	5.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m):	5.20
Ground floor split?	no		Ground floor elevation above ground (m):	0.20
Storeys below ground	0			
Foundation type:	pads with tie beams		if Foundation type is other, describe:	
Building height (m):	3.80		height from ground to level of uppermost seismic mass (for IEP only) (m):	3.8
Floor footprint area (approx):				
Age of Building (years):			Date of design:	1965-1976
Strengthening present?	no		If so, when (year)?	
			And what load level (%g)?	
Use (ground floor):	public		Brief strengthening description:	
Use (upper floors):				
Use notes (if required):				
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	timber framed	rafter type, purlin type and cladding	
Floors:	other (note)	describe sytem	Suspended timber floor
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	
Walls:	partially filled concrete masonry	thickness (mm)	

Lateral load resisting structure

Lateral system along:	concrete frame with infill	**Note: Define along and across in detailed report!**	note total length of wall at ground (m):	26.5
Ductility assumed, μ:	1.50		wall thickness (m):	0.19
Period along:	0.02	0.02 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	concrete frame with infill		note total length of wall at ground (m):	26.5
Ductility assumed, μ:	1.50		wall thickness (m):	0.19
Period across:	0.02	0.02 from parameters in sheet	estimate or calculation?	
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

north (mm):	0	leave blank if not relevant
east (mm):	0	
south (mm):	0	
west (mm):		

Non-structural elements

Stairs:			
Wall cladding:			
Roof Cladding:	Membrane	substrate	
Glazing:	aluminium frames		
Ceilings:	fibrous plaster, fixed		
Services(list):			

Available documentation

Architectural	full	original designer name/date	
Structural	full	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:	Minor evidence of liquefaction and ground cracking	Describe damage:	
Settlement:		notes (if applicable):	
Differential settlement:		notes (if applicable):	
Liquefaction:	0-2 m²/100m³	notes (if applicable):	
Lateral Spread:		notes (if applicable):	
Differential lateral spread:		notes (if applicable):	
Ground cracks:	0-20mm/20m	notes (if applicable):	
Damage to area:		notes (if applicable):	

Building:

	Current Placard Status:	yellow		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	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:	60% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	Quantitative assessment
	Assessed %NBS after:	60%		
Across	Assessed %NBS before:	60% ##### %NBS from IEP below		
	Assessed %NBS after:	60%		

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name: Riccarton Community Centre - 1986 Addition
Building Address: Unit: No: 199 Street: Clarence Street
Legal Description:
GPS south: Degrees 43, Min 31, Sec 51.53
GPS east: Degrees 172, Min 36, Sec 5.84
Building Unique Identifier (CCC): BU 0537-002 EQ2

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International Consultants
Company project number: 6-QUCCC.11
Company phone number: 6433657858
Date of submission: 4-Sep-13
Inspection Date:
Revision: Final R2
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: silty sand
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m):
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):

Max retaining height (m): 0
Soil Profile (if available):
If Ground improvement on site, describe: N/A
Approx site elevation (m): 5.00

Building

No. of storeys above ground: 1 (single storey = 1)
Ground floor split? no
Storeys below ground: 0
Foundation type: strip footings
Building height (m): 3.00
Floor footprint area (approx):
Age of Building (years):

Ground floor elevation (Absolute) (m): 5.20
Ground floor elevation above ground (m): 0.20
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m):
Date of design: 1976-1992

Strengthening present? no
Use (ground floor): public
Use (upper floors):
Use notes (if required):
Importance level (to NZS1170.5): IL2

If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Gravity Structure

Gravity System: load bearing walls
Roof: timber framed — rafter type, purlin type and cladding:
Floors: other (note) — describe sytem: Concrete S.O.G.
Beams:
Columns:
Walls: load bearing brick — #N/A

Lateral load resisting structure

Lateral system along: partially filled CMU
Ductility assumed, μ: 1.25
Period along: 0.40 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

Note: Define along and across in detailed report!

note total length of wall at ground (m): 12.8
wall thickness (m): 0.19
estimate or calculation?
estimate or calculation?
estimate or calculation?

Lateral system across: partially filled CMU
Ductility assumed, μ: 1.25
Period across: 0.40 ##### enter height above at H31
Total deflection (ULS) (mm):
maximum interstorey deflection (ULS) (mm):

note total length of wall at ground (m): 5.6
wall thickness (m): 0.19
estimate or calculation?
estimate or calculation?
estimate or calculation?

Separations:

north (mm): 0 (leave blank if not relevant)
east (mm):
south (mm):
west (mm):

Non-structural elements

Stairs:
Wall cladding:
Roof Cladding: Membrane — substrate
Glazing: aluminium frames
Ceilings: fibrous plaster, fixed
Services(list):

Available documentation

Architectural: none — original designer name/date:
Structural: none — original designer name/date:
Mechanical: none — original designer name/date:
Electrical: partial — original designer name/date:
Geotech report: none — original designer name/date:

Damage

Site:
(refer DEE Table 4-2)

Site performance: Minor evidence of liquefaction and ground cracking
Describe damage:

Settlement: — notes (if applicable):
Differential settlement: — notes (if applicable):
Liquefaction: 0-2 m²/100m³ — notes (if applicable):
Lateral Spread: — notes (if applicable):
Differential lateral spread: — notes (if applicable):
Ground cracks: 0-20mm/20m — notes (if applicable):
Damage to area: — notes (if applicable):

Building:

Current Placard Status: green

Along — Damage ratio: 0%
Describe (summary):
Describe how damage ratio arrived at:

Across — Damage ratio: 0%
Describe (summary):

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

Diaphragms — Damage?: no — Describe:
CSWs: — Damage?: no — Describe: Heavy cracking to window lintels
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: no — Describe:

Recommendations

Level of repair/strengthening required: none — Describe:
Building Consent required: no — Describe:
Interim occupancy recommendations: full occupancy — Describe:

Along — Assessed %NBS before: 60% ##### %NBS from IEP below
Assessed %NBS after: 60%

If IEP not used, please detail assessment methodology:

Across — Assessed %NBS before: 60% ##### %NBS from IEP below
Assessed %NBS after: 60%

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