Bishopdale Community Centre and Library

Bishopdale Mall,

129 Farrington Avenue

Bishopdale

Christchurch

Detailed Engineering Evaluation

Quantitative Assessment Report

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129 Farrington Avenue
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Quantitative Assessment Report

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Executive Summary

Bishopdale Community Centre and Library
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Christchurch City Council appointed Opus International Consultants to carry out a quantitative seismic assessment of the Bishopdale Community Centre and Library, 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.

The principal finding of the assessment is that in the worst case short columns formed by the partial height blockwork infill panels have a seismic capacity equivalent to only 4% of the current New Building Standard. This equates to over 25 times the risk of collapse than that of a new building. The columns are situated in the north-west wall and south-west wall. Similar columns elsewhere in the building are also short columns and have seismic capacities only marginally higher. These short columns have a brittle shear failure mechanism which occurs before the columns can develop their bending capacity and could result in a partial collapse of the building.

The assessment has also identified three other critical structural weaknesses as follows:

a) The position of the lift shaft results in significant plan irregularity.

b) Overloading of the reinforced concrete stair flights, due to both ends being built in.

c) Lack of restraint of the roof due to the lack of bracing in the roof in the northwest to south east direction.

It is recommended that the building not be occupied given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Bishopdale Community Centre and Library, located at Bishopdale Mall, 129 Farrington Avenue, 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 will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building Act). It is anticipated that CERA will adopt 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.

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

2.2 Building Act

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

Section 112 - Alterations

This section requires that an existing building complies with the relevant sections of the Building Code to at least the extent that it did prior to the alteration.

This effectively means that a building cannot be weakened as a result of an alteration (including partial demolition).

Section 115 – Change of Use

This section requires that the territorial authority (in this case Christchurch City Council (CCC)) is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

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

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone if its ultimate capacity would be exceeded in a 'moderate earthquake' and it would be likely to collapse causing injury or death, or damage to other property.

A moderate earthquake is defined by the building regulations as one that would generate 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 or to close and prevent occupancy to any building defined as 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 4th September 2010.

The 2010 amendment includes the following:

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

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

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

4. Repair works for buildings damaged by earthquakes will be required to comply with the above.

The council has stated their willingness to consider retrofit proposals on a case by case basis, considering the economic impact of such a retrofit.

If strengthening works are undertaken, a building consent will be required. A requirement of the consent will require upgrade of the building to comply 'as near as is reasonably practicable' with:

- The accessibility requirements of the Building Code;

- The fire requirements of the Building Code. This is likely to require a fire report to be submitted with the building consent application.

2.4 Building Code

The Building Code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

After the February 2011 earthquake, Compliance Document B1: Structure was amended on 19 May 2011 to include increased seismic design requirements for Canterbury as follows:

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

- Increased serviceability requirements.

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). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

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

Table 1: %NBS compared to relative risk of failure

4 Background Information

4.1 Building Description

The Bishopdale Community Centre and Library is situated in the Bishopdale Mall at 129 Farrington Avenue, Bishopdale, Christchurch. It was constructed in 1974 and is approximately 30m long by 25m wide. The main axis of the building runs from north-west to south-east.

The building is a two storey reinforced concrete frame structure with a precast concrete first floor formed with 1.20m wide double T floor beams. The external walls comprise reinforced concrete columns with partial height infill blockwork panels on two sides and a cavity block and stone veneer on the other sides. Internally the first floor is supported on two reinforced concrete beams spanning across the building onto four internal columns.

At first floor level on the north-east and south-west elevations there are precast concrete panels fixed to the reinforced concrete columns under the windows.

In the south-east corner there is a reinforced concrete lift shaft which partly supports and is attached to the first floor and also supports the reinforced concrete staircase which runs up from ground to first floor.

In the north-west corner of the building is an emergency staircase. This is a straight flight of insitu reinforced concrete and is built into the first floor.

The Bishopdale Community Centre and Library is situated in a yellow (TC2) residential zone. It is not therefore considered that liquefaction is likely to be a major problem in this area. As such a geotechnical survey has not been carried out at this stage.

4.2 Building Damage Assessments

4.2.1 Post 22 February 2011 Rapid Assessment

Structural (Level 2) assessments of the structure were undertaken on 29 September 2010 and 28 February 2011 by Tony Raper of Opus International Consultants Limited. These inspections included external and internal visual inspections of all structural elements, without the benefit of opening up works.

4.2.2 Further Inspections

A further inspection was undertaken by Andrew Blacker of Opus International Consultants Limited on 28 October 2011.

4.3 Original Documentation

Copies of the following construction drawings were provided by CCC on 14 October 2011:

- Construction drawings number 2565/Sheets 1 to 22.

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

Some specification information is available but no structural calculations for the building have been located.

4.4 Qualitative Assessment

A qualitative assessment [1] for the building was completed in November 2011 following the 22 February 2011 earthquake. The findings of this report were that the building had some critical weaknesses which affected the likely seismic capacity of the building. The evaluated capacity of the building was determined to be 17%NBS by qualitative assessment. The damage sustained to the building was minor, but there was some evidence of the perceived structural weaknesses resisting seismic loads. A quantitative assessment was recommended following the completion of the qualitative assessment report.

5 Structural Damage

A damage assessment survey was carried out by Andrew Blacker of Opus International Consultants Limited on 28 October 2011.

5.1 Surrounding Buildings

This building abuts the Westpac building on the north-east elevation. This is a single storey structure with its roof at approximately the same level as the first floor of the Community Centre and Library. There is no evidence to suggest that any damage has been caused to either building by pounding effects during the recent seismic events.

5.2 Residual Displacements and Damage

The damage noted has been reported in the qualitative report for this property issued by Opus International Consultants Limited on 2 November 2011.

The report identified that there are cracks on a number of the reinforced columns, most notably on the north-west side but also seen on the south-west and south-east sides. These walls all have partial height infill panels of blockwork or cavity work as described above.

On the north-east wall, which has all the panels between the columns fully filled with blockwork, there is some opening up of the joints between the blockwork panels and the reinforced concrete column in the south corner of the building.

Some stepped cracks were noted in the facing panels.

Internally there are numerous cracks to partition walls and some fine cracks particularly at junctions between the columns and the main ring beams at the first floor and roof levels.

Note: Photographs showing the structural damage noted above are included with the Opus Qualitative Assessment report dated November 2011.

5.3 Foundations

No evidence of ground damage or foundation settlement has been noted at the site.

5.4 Primary Gravity Structure

As noted above and in the qualitative report some cracking damage has been noted to the reinforced concrete columns most notably to the north west elevation where the partial height infill panels of reinforced blockwork and small height windows create “short columns”.

5.6 Non Structural Elements

Damage has been noted to internal partitions. Adjacent to the main entrance staircase movement of the stair itself has damaged the plaster finish on the walls.

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 Detailed Engineering Evaluation Procedure [3] (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 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 critical structural weaknesses have been identified for this building:

a) The presence of "short columns" formed by the partial height blockwork infill panels in between the concrete columns.

b) The roof level has no structural bracing in the north-west to south-east direction.

c) The position of the lift shaft in the south corner of the building results in a significant plan irregularity and induces large torsional forces into the building.

d) The stair flights are fully built into the building and may be expected to attract loads in a seismic event for which they have not been designed.

6.2 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 building was reviewed and the most critical elements identified. This shows that the reinforced columns and particularly those partially confined by the blockwork are likely to be the most critical elements. The probable flexural and shear capacities of the columns were calculated.

A displacement based analysis was carried out to identify the expected failure mode of the structure and the likely displacement demand which would be imposed on the structure in a design earthquake.

6.3 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. 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:

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

6.4 Quantitative Assessment Results

A summary of the structural performance of the building is shown in the table 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 will have significantly greater capacity when compared with the governing elements.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode or description of limiting criteria based on elastic capacity of critical element	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Partially infilled concrete columns	Brittle shear failure in the columns due to the partial height infill blockwork panels (short column effect).	Yes	4% NBS
Roof level lateral load resisting system	No lateral load resisting system in the north-west to south-east direction at roof level. This will result in out of plane bending in the roof trusses and limited load transfer to the first floor lateral load resisting elements.	No	<10% NBS
Lift shaft walls	Shear failure of the insitu concrete walls. Potential uplift of the foundations. These walls are particularly affected by the significant plan irregularity of the building.	Yes	<34% NBS
Fully infilled concrete frame (north-east elevation)	Shear failure of the masonry infill panels.	No	40-50% NBS

6.5 Discussion of Results

We have considered the displacement effect upon the short columns in the north east wall of the building using the methods described by Professor Nigel Priestley in Displacement-Based Seismic Design of Structures and as referred to in the NZSEE assessment guide. This shows that in the worst case the column is likely to fail in shear and only has a capacity equal to approximately 4%

NBS (New Building Standard). Based on this method, the shear capacity is reached before the yield moment can be developed in the column. Therefore the failure mode for the short column, and therefore the building, is a brittle shear failure at 4% NBS.

The significant plan irregularity results in the lift shaft walls resisting the resultant large torsional forces.

The lack of roof level bracing in the north-west to south-east direction results in the roof trusses bending out of plane and transferring little load to the lateral load resisting elements.

6.6 Risk

Although the building has four critical structural weaknesses, we do not consider that it poses a risk to the adjacent thoroughfares or the adjacent premises. This evaluation is based on the observation that the building's capacity has not been reduced by the earthquake damage, and also as the anticipated failure mode of the short columns is considered not likely to result in a global collapse of the building. It is however noted that predicting building behaviour after the onset of failure is difficult and has a level of uncertainty surrounding it.

While we do not consider that it is necessary to put up any barricades around the building we do recommend that the building is inspected following significant aftershocks to monitor any change in the building's condition.

7 Conclusions

a) The overall seismic performance of this building is governed by the short columns created by the partial height blockwork infill panels on the three exposed walls. In the worst case on the wall adjacent to the service corridor these columns have a capacity of around 4% NBS, and will fail in shear immediately below first floor level before they can develop their full moment capacity. The relative risk associated with this value is approximately 25 times that of a new building.

b) Three other critical structural weaknesses have been identified within the building structure as follows:

   i) The position of the lift shaft resulting in significant plan irregularity.

   ii) The stair flights are locked into each level.

   iii) The lack of horizontal bracing within the roof structure.

   All of these items would need to be addressed to arrive at a scheme for repair and strengthening.

8 Recommendations

a) Strengthening options be developed for increasing the seismic capacity of the building to at least 67% NBS.

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

c) If a scheme for strengthening proves to be economically viable a full design should be carried out to produce this scheme. This will need to take into account all the structural weaknesses identified within the property.

d) It is recommended that the CCC review the on-going occupancy of this building.

9 Limitations

a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some 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 visual and non-intrusive, no linings or finishes were removed to expose structural elements. Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time.

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

10 References

[1] Detailed Engineering Evaluation, Stage 1 Qualitative Report; November 2011, Opus International consultants

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

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

Appendix A – Assessment Assumptions and Methodology

1. Assumptions

No structural information or original calculations for the building are available. The following assumptions for materials strengths have been made in accordance with the recommendations of the NZSEE [3] and professional judgement

Concrete base strength 20MPa - for use in design this is increased by a factor of 1.5 to allow for normal overstrength in manufacture and age hardening.

Mild steel reinforcement – 275MPa (approx. 40,000psi)

Deformed or high tensile reinforcement 414MPa (approx. 60,000psi).

At this stage of the assessment any secondary effects and existing damage to the structure (which may reduce its capacity) have been ignored.

2. Methodology

This quantitative analysis has been carried out using the methods described in NZSEE: 2006, "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes", New Zealand Society for Earthquake Engineering and in particular section 7 "Detailed Assessment of Reinforced Concrete Structures" – section 7.2.3 "Displacement-Based Procedure for Framed Structures". This also refers to other sections and where applicable these have been used.

Earthquake loading and displacement demand has been calculated using NZS 1170 "Structural Design Actions" Section 5 Earthquake Actions – New Zealand". This also refers to other sections of the document and where applicable these have been used.

The hazard factor used for the analysis is 0.3 in accordance with the amendment made to the Building Code B1 Structure on 19 May 2011 subsequent to the February Earthquake.

The shear and flexural capacities of the most vulnerable columns have been calculated together with the displacement required to mobilise these conditions.

The displacement demand for the building has been assessed using a substitute single degree of freedom analogy in the method as described by Professor Nigel Priestly in his book "Displacement-Based Seismic Design of Structures".

Appendix B – CERA DEE Spreadsheet

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