Christchurch South Library

Detailed Engineering Evaluation

Quantitative Assessment Report
66 Colombo Street, Christchurch

Christchurch City Council

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Christchurch South Library
BU 1755-002 EQ2

Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

66 Colombo Street, Christchurch

Background

This is a summary of the quantitative assessment report for the building structure at 66 Colombo Street (Christchurch South), and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 14 October 2011 and 26 March 2012 and available drawings.

Key Damage Observed

Key damage observed includes:-

- Cracks to wall gib board linings and ceilings throughout the building. Several cracks through the wall linings are up to 10mm in width. Intrusive investigation of a limited number of these cracks is recommended, as in several locations steel beams are supported on precast concrete walls behind the gib wall lining and any potential damage to the precast walls cannot currently be quantified.
- Significant cracking to precast panel H.1 near the western end of the building.
- Displaced/movement of ground floor slab by the Children's Area.
- Cracks in the concrete floor, up to 40mm wide, underneath carpet and tile areas at the western end of the building. The effect of this crack on the seismic performance and water-tightness of the building needs to be quantified.
- Slumping in the ground within children's playground area.
- Cracks to external moat/pond slab in several locations.
- Opening of existing cracks in the vertical faces of ground beams along the northern sides of the building wings. No longitudinal reinforcement was visible in these cracks, which were up to 15mm wide.
- It is understood that several façade panels became loose during the 22 February 2011 earthquake.
- Outwards rotation of the precast panel at the northern end of the western most external wall. The façade panels should be removed from this panel to identify any possible signs of damage to the panel fixings.
- Significant differential settlement throughout the entire building. The overall differential settlement is in the order of 92mm, while the maximum differential settlement over a 6m length is in the order of 45mm. This settlement exceeds the maximum allowable differential settlement of 25mm over 6m as specified in Clause B1 of the New Zealand Building Code.

Critical Structural Weaknesses

The following critical structural weaknesses have been identified:

- In both directions the out-of-plane flexural capacity of the precast concrete panels is deficient. The panels were analysed in two ways. Firstly, as simply supported elements, spanning from the floor to the roof. Secondly, as cantilevered elements off the ground floor slab. In both cases, the panels lack the required capacity lateral capacity.
- In the transverse direction, the portal frame beams run over the top of the columns. The columns are welded to the underside of the beam flange. The welds are 6mm fillet welds, all around, and do not appear to be able to develop the flexural capacity of the column. No

web stiffener plates are provided on the beams. The lengths of these columns vary from full height columns through to stub columns less than 300mm high. These shorter stub columns will attract a higher level of load than the full height columns. Intrusive investigation of these stub columns is recommended.

- In both directions, the beam flexural capacity exceeds that of the column producing a strong beam-weak column condition. The strong beam-weak column condition can cause a soft storey collapse mechanism if a hinge forms at the top of the column. In the longitudinal direction, the column bends about its weak axis. This will only exacerbate the strong beam-weak column condition in this direction.
- In both directions, the lateral force resisting systems are very flexible. The flexibility of the frames means the potential drift far exceeds the code allowable drift.

Indicative Building Strength (from quantitative assessment)

Based on the information available, and from undertaking a quantitative assessment, the building's original capacity has been assessed to be in the order of 10-20% NBS and post-earthquake capacity in the order of 10-20% NBS. The building is therefore classed as an earthquake prone building.

Strengthening Options

Options for the strengthening of the building have not been considered in detail, but could consist of installing braced frames in both directions within the building. The damage to the floor slab could also be rectified during this process.

Recommendations

a) The CCC reviews the occupancy of this building.

b) All areas of the floor slab should be checked for cracking. This will require all floor coverings to be removed.

c) Intrusive investigations to several areas of the building are recommended.

d) 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.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Christchurch South Library, located at 66 Colombo 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 quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

A qualitative assessment [1] was completed in April 2012 for the building.

This report has been prepared by Opus International Consultants in conjunction with Simpson Gumpertz and Heger.

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 quantitative and detailed quantitative assessments.

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

1. The importance level and occupancy of the building.
2. The placard status and amount of damage.
3. The age and structural type of the building.
4. Consideration of any critical structural weaknesses.

Any building with a capacity of less than 33% of new building standard (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67% as required by the CCC Earthquake Prone Building Policy.

2.2 Building Act

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

Section 112 - Alterations

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

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

Section 115 – Change of Use

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

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

Section 121 – Dangerous Buildings

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

1. In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
2. In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or

3. There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

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

A moderate earthquake is defined by the building regulations as one that would generate loads 33% of those used to design an equivalent new building.

Section 124 – Powers of Territorial Authorities

This section gives the territorial authority the power to require strengthening work within specified timeframes or to close and prevent occupancy to any building defined as dangerous or earthquake prone.

Section 131 – Earthquake Prone Building Policy

This section requires the territorial authority to adopt a specific policy for earthquake prone, dangerous and insanitary buildings.

2.3 Christchurch City Council Policy

Christchurch City Council adopted their Earthquake Prone, Dangerous and Insanitary Building Policy in 2006. This policy was amended immediately following the Darfield Earthquake on 4 September 2010.

The 2010 amendment includes the following:

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

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

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

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

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

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

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

2.4 Building Code

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

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

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

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

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

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

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

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

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

3 Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The loadings are in accordance with the current earthquake loading standard NZS1170.5 [2].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [3] is presented in Figure 1 below.

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

Improvement of Structural Performance	
Legal Requirement	NZSEE Recommendation
The Building Act sets no required level of structural improvement (unless change in use) This is for each TA to decide. Improvement is not limited to 34%NBS.	100%NBS desirable. Improvement should achieve at least 67%NBS
	Not recommended. Acceptable only in exceptional circumstances
Unacceptable	Unacceptable

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

Table 1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% risk of exceedance in 50 years (i.e. 0.2% in the next year). It is noted that the current seismic risk in Christchurch results in a 6% risk of exceedance in the next year.

Table 1: %NBS compared to relative risk of failure

Percentage of New Building Standard (%NBS)	Relative Risk (Approximate)
>100	<1 time
80-100	1-2 times
67-80	2-5 times
33-67	5-10 times
20-33	10-25 times
<20	>25 times

3.1 Minimum and Recommended Standards

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

3.1.1 Occupancy

- The Canterbury Earthquake Order[i] in Council 16 September 2010, modified the meaning of "dangerous building" to include buildings that were identified as being

EPB's. As a result of this, we would expect such a building would be issued with a Section 124 notice, by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Based on information received from CERA to date, this notice is likely to prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an EPB.

3.1.2 Cordoning

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

3.1.3 Strengthening

- Industry guidelines (NZSEE 2006 [3]) 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.

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

4 Background Information

4.1 Building Description

The Christchurch South Library, located at 66 Colombo Street, is a library and community service centre serving the southern Christchurch communities. The library is relatively new, designed in 2002 and opened in August 2003. The building is a one-storey steel and precast concrete tilt panel structure.

The library is located in Beckenham and is bordered on the north and east by Hunter Terrace and by Colombo Street to the west. Hunter Terrace follows the path of the Heathcote River on its southern edge. At its closest point, the river is approximately 30m from the building. For the purposes of this report, the north-south directions and east-west directions are referred to as transverse and longitudinal, respectively. Refer to the site plan in Figure 2 below.

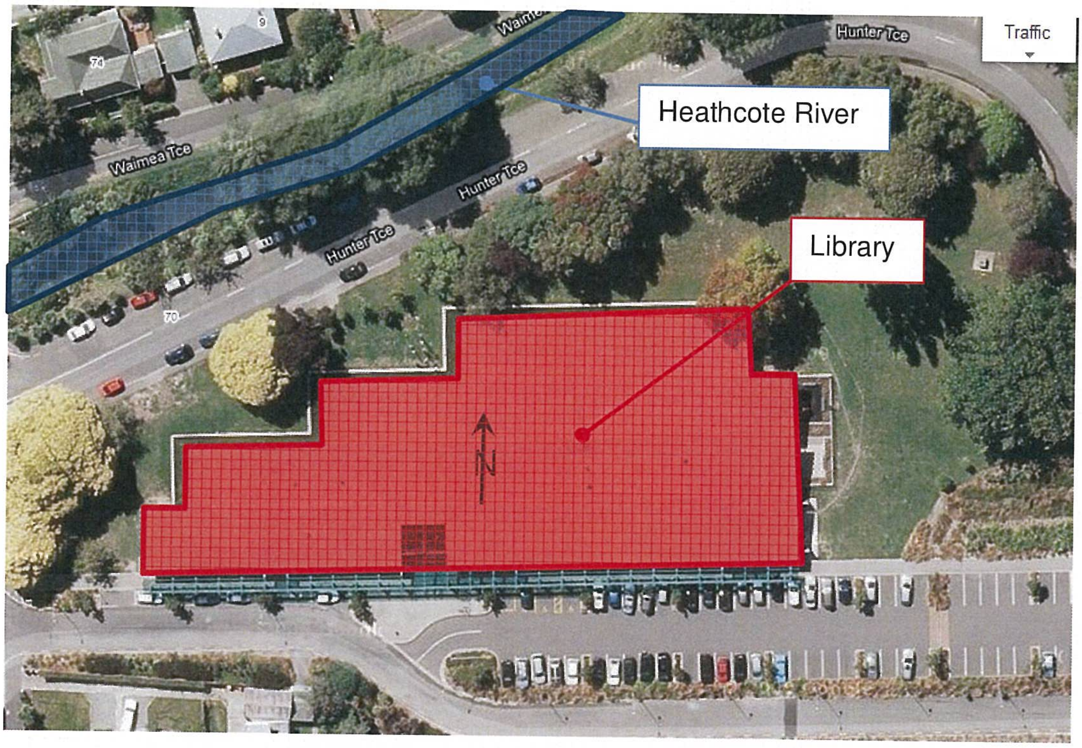


Figure 2: Christchurch South Library site plan (source: Google Maps)

The library has overall dimensions of approximately 94m long (longitudinal direction) and 36m wide (transverse direction), with an overall floor area of approximately 2,470 square metres. The roof has a saw-toothed shape with a pitch from north to south in the transverse direction. The saw-toothed shape enables light to enter in to the building through the gap between the low and high roof portions along the whole length of the building. The high point of the roof is typically 4.4m above the ground, with the low point approximately 3.2m above the ground. Steel portal frames run in the transverse direction. They are typically composed of 250UB31 beams and 150UC23 columns. Exterior columns and posts down to precast wall panels are framed with square hollow sections (SHS) and rectangular hollow sections (RHS) respectively. Roof purlins span longitudinally to the transverse portal frame beams. Additionally, some universal beams (UB's) run longitudinally between the portal frames. Partial height interior precast wall panels are present in several locations south of gridline F. The roof is sheathed with 12mm plywood, with some discrete locations on the exterior supplemented with diagonal steel flat bracing.

The library has a reinforced slab on grade with variable thickness over a moisture barrier and a layer of compacted fill. The minimum and maximum thicknesses are 125mm and 225mm, respectively. The interior portal frame columns are supported on reinforced concrete foundation pads with no tie beams. The top of the foundation pad is flush with the top of the slab on grade, and has no ties into the slab on grade. A continuous strip footing runs around the perimeter of the library, with a moat along the north face.

4.2 Seismic Load Resisting System

In both directions, lateral loads are resisted by frame action in the saw-tooth portal frames. The strong axes of the columns are oriented in line with the transverse direction. The beams run over the top of the column, with the column welded to the bottom flange. No continuity plates are provided in the beam web. The longitudinal beams are then welded into the transverse beam members. All beam to column welds are 6mm fillet welds, all around. As noted above, the columns are supported on isolated concrete foundation pads, with no tie beams connecting them. This means the portal frames are effectively an idealized "pin" connection at the base.

Interior precast reinforced concrete tilt panels are also provided, primarily in the transverse direction. The panels are partial height and do not attach directly to the roof framing members. Typically, posts extend from the top of the panels to support the transverse frame beams. The posts are welded to an embedded plate at the top of the panel, which is fixed into the concrete via two flat straps with butterflied ends. This connection provides the out of plane support for the panel. The panels are typically singly reinforced, 120mm thick precast concrete.

4.3 Original Documentation

Copies of the following construction drawings were provided by CCC on 27 February 2012:

- Christchurch South Library and Service Centre Structural Drawings, dated 5 July 2002.

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

No architectural drawings were available for review.

A set of the structural calculations was viewed at the CCC Civic Office on 6 July 2012. It is not known whether these were the full set of calculations. The calculations viewed showed that the building was designed for a seismic coefficient of 0.48, based on a ductility factor of 1.25.

5 Survey

5.1 Post 22 February 2011 Rapid Assessment

A structural (Level 3) assessment of the above buildings/property was undertaken on 22 March 2011 by Opus International Consultants.

The site was posted with a Green (G2) placard on 22 March 2011, indicating that the building access is not restricted.

5.2 Further Inspections

Further inspections were undertaken by Opus International Consultants on 14 June 2011, 10 October 2011 and 26 March 2012.

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.

Intrusive investigations to identify any signs of damage and to confirm the thickness of several precast concrete panels were completed on 5 July 2012.

A level survey was undertaken by Opus on 4 July 2012. Refer to Appendix 4 for a plan of the level survey results.

6 Structural Damage

The following damage has been noted:

a) Cracks to wall gib board linings and ceilings throughout the building. Several cracks through the wall linings are up to 10mm in width.

b) A significant 4mm wide crack was identified at the eastern end of the southern face of precast panel H.1 following intrusive investigations. A number of other cracks greater than 0.5mm width were also identified in this location. Intrusive investigation of all precast panels along this line is recommended so that all damage to the precast walls can be identified.

c) Displaced/movement of ground floor slab by the Children's Area.

d) Three significant cracks in the concrete slab, ranging from 10 to 40mm wide, underneath carpet and tile areas in the western and central areas of the building. The effect of this crack on the water-tightness of the building needs to be quantified. Only isolated areas of the slab have been checked for cracking and it is recommended that the entire slab be investigated.

e) Slumping in the ground within children's playground area.

f) Lateral spreading induced cracks to external moat/pond slab in several locations.

g) Opening of existing cracks in the vertical faces of ground beams along the northern sides of the building wings. No longitudinal reinforcement was visible in these cracks, which were up to 15mm wide.

h) Outwards rotation of the precast panel at the northern end of the western most external wall. The façade panels should be removed from this panel to identify any possible signs of damage to the panel fixings.

i) It is understood that several façade panels became loose during the 22 February 2011 earthquake.

j) Significant differential settlement throughout the entire building. The overall differential settlement is in the order of 90mm, while the maximum differential settlement over a 6m length is in the order of 45mm. This settlement exceeds the maximum allowable differential settlement of 25mm over 6m as specified in Clause B1 of the New Zealand Building Code at Serviceability Limit State.

7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [3] 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" [4] 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" [6] issued on 21 December 2011.

7.1 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 quantitative stage of the assessment the following potential CSW's were identified:

a) In both directions the out-of-plane flexural capacity of the precast concrete panels is deficient. The panels were analysed in two ways. Firstly, as simply supported elements, spanning from the floor to the roof. Secondly, as cantilevered elements off the ground floor slab. In both cases, the panels lack the required lateral capacity.

b) In the transverse direction, the portal frame beams run over the top of the columns. The columns are welded to the underside of the beam flange. The welds are 6mm fillet welds, all around, and do not appear to be able to develop the flexural capacity of the column. No web stiffener plates are provided on the beams. The lengths of these columns vary from full height columns through to stub columns less than 300mm high. These shorter stub columns will attract a higher level of load than the full height columns. Intrusive investigation of these stub columns is recommended.

c) In both directions, the beam flexural capacity exceeds that of the column, producing a strong beam-weak column condition. The strong beam-weak column condition can cause a soft storey collapse mechanism if a hinge forms at the top of the column. In the longitudinal direction, the column bends about its weak axis. This will only exacerbate the strong beam-weak column condition in this direction.

d) In both directions, the lateral force resisting systems are very flexible. The flexibility of the frames means the potential drift far exceeds the code allowable drift.

7.2 Seismic Coefficients

The seismic design parameters based on current design requirements from NZS1170:2002 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170:2002
- Site hazard factor, $Z = 0.3$, B1/VM1 clause 2.2.14B
- Return period factor, $R_u = 1.3$ from table 3.5, NZS 1170:2002, for an Importance Level 3 structure with a 50 year design life. This allows for the building to have an occupancy level of greater than 300 people. The building has also been assessed as an Importance Level 2 structure (occupancy less than 300 people) for comparison purposes.
- Ductility factors:
    - $\mu = 1.25$ for the steel frames. The ductility factor chosen is based on the detailing at the beam-column joint and the strong beam-weak column condition.
    - $\mu = 1.0$ for the connection between the tops of the precast wall panels and the steel posts.
    - $\mu = 2.0$ for the dowel connection at the base of the precast wall panels.

7.3 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 will 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.

7.4 Quantitative Assessment Results

A summary of the structural performance of the building is shown in the following tables. Note that the values given represent the worst performing elements in the building, when redistributed can be relied on as these effectively define the building's capacity.

Table 2: Summary of Seismic Performance (Importance Level 3)

Structural Element/System	Failure mode, or description of limiting criteria based on displacement capacity of critical element.	% NBS based on calculated capacity
Out-of-plane flexural capacity embedded at top of precast panels	Brittle concrete blowout failure in the embedded plate at the panel edge which connects to the steel posts supporting the transverse portal frame beam. The embedded plate has very little edge distance, so while there is additional capacity in the steel elements, the concrete limits the capacity of the connection. Once the connection fails, the panel can no longer resist shear and flexural demand, and the wall behaves as a cantilever off the ground floor slab	6% (Gridline 2, Panel H.1) 190% (Gridline 2, Panel J.1) 19% (Gridline 3, Panel J.1) 18% (Gridline 6, Panel J.2) 10% (Gridline 9, Panel J.4)
Out-of-plane flexural capacity embedded at bottom of precast panels	Yielding of the H16 dowel from the thickened ground floor slab to the precast concrete panels. The failure mechanism itself is not brittle, but once the mechanism occurs, there is no redundancy to resist further lateral load demand on the panel. When the strength of the connection degrades, collapse will initiate. This connection becomes critical once the connection at the top of the panel fails.	19% (Gridline 2, Panel H.1) 18% (Gridline 2, Panel J.1) 25% (Gridline 3, Panel J.1) 23% (Gridline 6, Panel J.2) 21% (Gridline 9, Panel J.4)
Weld capacity from the column to the underside of the transverse portal frame columns	Brittle failure of the weld group from the 150UC23 to the underside of the 250UB31. The weld cannot develop the flexural capacity of the column, so no column yielding can occur without brittle failure of the weld group.	38%
Column flexural capacity – transverse direction	Yielding of the column about the strong axis. The columns are idealized as portal frames with a "pin" base. When the columns hinge at the top, a collapse mechanism can form.	63% (Gridline 2) 70% (Gridline 3) 68% (Gridline 6) 68% (Gridline 9)
Column flexural capacity – longitudinal direction	Yielding of the column about the weak axis. The columns are idealized as portal frames with a "pin" base. When the columns hinge at the top, a collapse mechanism can form.	27% (Gridline C) 39% (Gridline D) 36% (Gridline F)
Beam flexural capacity – transverse direction	Yielding in flexure of the portal frame beams in the transverse direction.	93%

Structural Element/System	Failure mode, or description of limiting criteria based on displacement capacity of critical element.	% NBS based on calculated capacity
Beam flexural capacity – longitudinal direction	Yielding in flexure of the portal frame beams in the longitudinal direction.	59%
Drift – transverse direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements, and premature collapse due to P-Delta effects.	37%
Drift – longitudinal direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements, and premature collapse due to P-Delta effects.	22%

Table 3: Summary of Seismic Performance (Importance Level 2)

Structural Element/System	Failure mode, or description of limiting criteria based on displacement capacity of critical element.	% NBS based on calculated capacity
Out-of-plane flexural capacity embedded at top of precast panels	Brittle concrete blowout failure in the embedded plate at the panel edge which connects to the steel posts supporting the transverse portal frame beam. The embedded plate has very little edge distance, so while there is additional capacity in the steel elements, the concrete limits the capacity of the connection. Once the connection fails, the panel can no longer resist shear and flexural demand, and the wall behaves as a cantilever off the ground floor slab	7% (Gridline 2, Panel H.1) 190% (Gridline 2, Panel J.1) 26% (Gridline 3, Panel J.1) 24% (Gridline 6, Panel J.2) 12% (Gridline 9, Panel J.4)
Out-of-plane flexural capacity embedded at bottom of precast panels	Yielding of the H16 dowel from the thickened ground floor slab to the precast concrete panels. The failure mechanism itself is not brittle, but once the mechanism occurs, there is no redundancy to resist further lateral load demand on the panel. When the strength of the connection degrades, collapse will initiate. This connection becomes critical once the connection at the top of the panel fails.	25% (Gridline 2, Panel H.1) 25% (Gridline 2, Panel J.1) 33% (Gridline 3, Panel J.1) 32% (Gridline 6, Panel J.2) 28% (Gridline 9, Panel J.4)
Weld capacity from the column to the underside of the transverse portal frame columns	Brittle failure of the weld group from the 150UC23 to the underside of the 250UB31. The weld cannot develop the flexural capacity of the column, so no column yielding can occur without brittle failure of the weld group.	49%
Column flexural capacity – transverse direction	Yielding of the column about the strong axis. The columns are idealized as portal frames with a "pin" base. When the columns hinge at the top, a collapse mechanism can form.	78% (Gridline 2) 92% (Gridline 3) 91% (Gridline 6) 87% (Gridline 9)
Column flexural capacity – longitudinal direction	Yielding of the column about the weak axis. The columns are idealized as portal frames with a "pin" base. When the columns hinge at the top, a collapse mechanism can form.	36% (Gridline C) 51% (Gridline D) 49% (Gridline F)

Structural Element/System	Failure mode, or description of limiting criteria based on displacement capacity of critical element.	% NBS based on calculated capacity
Beam flexural capacity – transverse direction	Yielding in flexure of the portal frame beams in the transverse direction.	120%
Beam flexural capacity – longitudinal direction	Yielding in flexure of the portal frame beams in the longitudinal direction.	77%
Drift – transverse direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements, and premature collapse due to P-Delta effects.	48%
Drift – longitudinal direction	Excessive drift in portal frames can lead to high damage levels for non-structural elements, and premature collapse due to P-Delta effects.	29%

7.5 Discussion of Results

Based on the building being assessed as an Importance Level 3 structure, the building has a calculated capacity of 10-20% NBS based on the capacity of the precast concrete panels and the portal frame stub columns on gridline J on the southern elevation of the building.

If the number of people occupying the building is restricted to less than 300 people so that the building could be deemed to be an Importance Level 2 structure, the building has a calculated capacity of 10-25% NBS.

While the lack of damage observed throughout the building does not correlate to the results from the quantitative assessment, the detailing of the seismic load resisting systems lacks robustness and strength. In our view the actual seismic performance of the building was enhanced by secondary structural system (architectural features) along the northern elevation(s) of the building. These features comprise external Hardiboard linings and internal gib board linings, but stop short of the floor and roof. Based on discussions with library staff it is understood that these elements were damaged following the 22 February 2011 earthquake but were repaired prior to Opus inspecting the building.

Of particular concern are the precast concrete panels, many of which are only 120mm thick. The 120mm thick panels along the southern elevation form part of the seismic load resisting system by bending out of plane, however the fixing between the top of the panel and the steel RHS post lacks robustness and strength. The precast concrete panel on the eastern elevation adjacent to the plant room is also only 120mm thick, and is not restrained at roof level. This panel therefore cantilevers 3.1m, and has insufficient capacity at the base.

The panels themselves represent a fall/collapse hazard as does a section of roof for which gravity support is provided by the panels.

As the building has a seismic capacity less than 34% NBS it is classed as an earthquake prone building in accordance with the Building Act. Due to the form of the structural systems, the secondary structural system and the lightweight nature of the building we believe that the overall risk of a local or global collapse is low.

We do believe that temporary securing works could be installed to support the southern elevation precast panels and to provide a secondary load path for gravity loads along this line should the stud connection to the panels fail. The CCC building consent department would need to be consulted on the regulatory requirements for any strengthening works.

The site visits have revealed that the building has been significantly affected by lateral spreading. While this is unlikely to have adversely affected the superstructure the effects of the damage induced to the building by the lateral spreading needs to be considered when designing any permanent strengthening works.

8 Summary of Geotechnical Appraisal

The geotechnical desktop study completed as part of the qualitative assessment recommended that a ground investigation programme comprising four CPT's be completed for the quantitative assessment.

The CPT testing was completed on 6 July 2012 however the geotechnical interpretation of the results had not been completed at the time this report was completed. This report will be updated to include the geotechnical investigation findings once they are available.

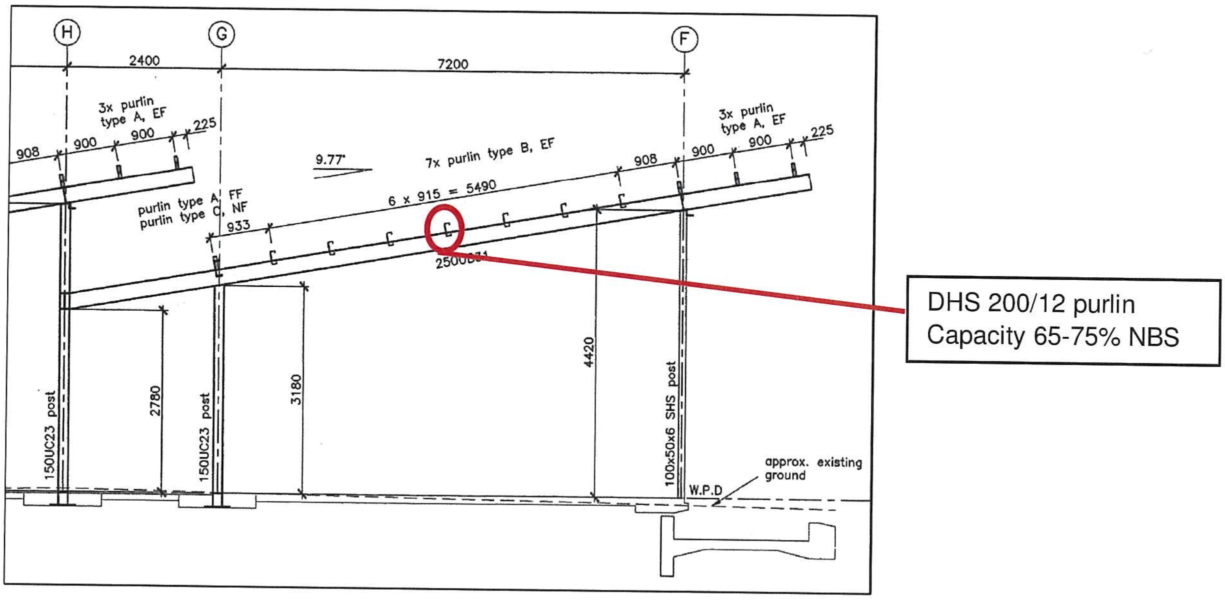
9 Snow Loading Assessment

As part of this quantitative assessment a separate check has been undertaken on the capacity of the roof superstructure to support snow loads. The snow load assessment is based on the 2008 increase in design level snow load for the Canterbury region, as specified in the New Zealand Building Code Clause B1/VM1 clause 2.2.13. The assessment makes allowance for snow drift on the saw tooth profiled roof.

The results of the snow loading assessment are as follows:

Element	% NBS based on calculated capacity
Cantilevered east-west beams at the eastern and western ends of the building	>100%
Main roof – 250 PFC purlins	>100%
Main roof – 2 No. 100x50x6 RHS purlins	>100%
Main roof – DHS 200/12 purlins	65%
Main north-south portal frames	>100%

The results of the snow loading assessment show that the majority of the superstructure elements have sufficient capacity to resist the increased snow loads. The only structural element with a capacity less than 100% NBS is the DHS 200/12 purlin near the low point in each section of the saw tooth roof. The snow loading applied to this purlin in the assessment is considered to be conservative due to the amount of snow of snow drift that was considered to occur, and it is expected that the actual capacity of this element will be in the range 65-75% NBS.



10 Remedial Options

Options for strengthening the building have not been considered in detail, but could consist of installing braced frames in both directions within the building. The damage to the floor slab could also be rectified during this process.

11 Conclusions

a) The seismic capacity of the building is governed by the out-of-plane strength of the precast concrete panel connections which is calculated in this quantitative assessment to be around 10-20% NBS. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

b) Due to the form of the structural systems, the presence of secondary structure, and the lightweight nature of the building we believe that the overall risk of a local or global collapse is low. However given the observed damage to a precast concrete panel and the low seismic capacity it is recommended that the CCC review the occupancy of this building.

c) The building has undergone significant differential settlement, with the settlement in several areas exceeding the maximum allowable differential settlement specified in the Building Code.

d) Three significant cracks, ranging in width from 10mm to 40mm were observed in the floor slab. Not all areas of the floor slab have been checked for cracking.

e) Significant cracks, one of which is 4mm wide, have been identified in precast panel H.1.

f) Intrusive investigations to the short stub columns and the internal precast concrete walls should be undertaken.

g) The geotechnical investigation has not yet been completed. This report will be updated once the interpretation of the CPT ground investigation results is completed.

12 Recommendations

a) The CCC reviews the occupancy of the building.

b) All areas of the floor slab should be checked for cracking. This may require all floor coverings to be removed.

c) Intrusive investigations to several areas of the building are recommended.

d) 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.

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 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 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.

14 References

[1] Opus International Consultants, *Christchurch South Library Qualitative Assessment Report*, April 2012.

[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.

[4] 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.

[5] 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.

[6] SESOC, Practice *Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

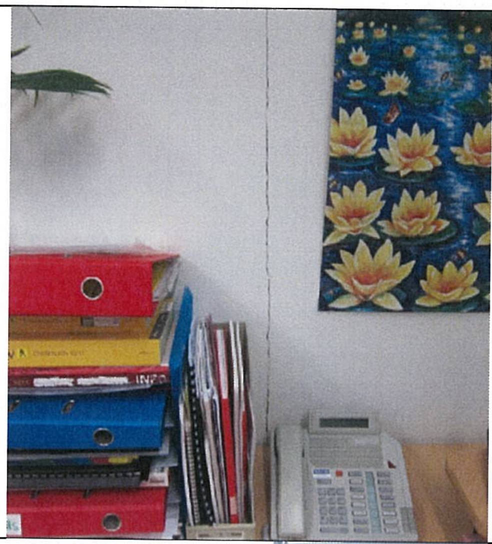
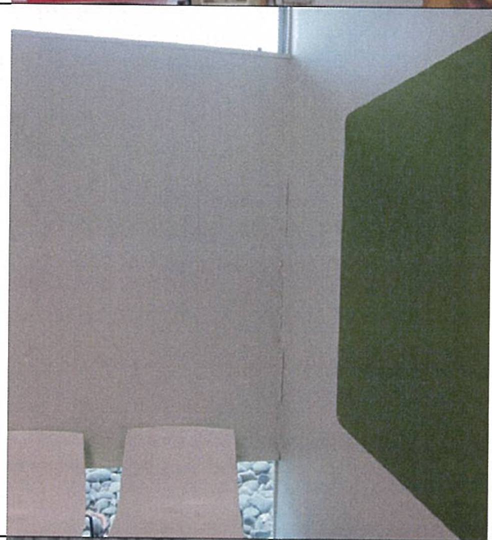
Appendix 1 – Photographs

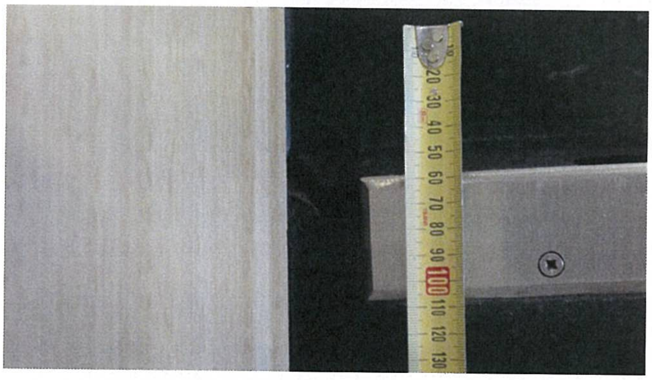
Christchurch South Library		
No.	**Item description**	**Photo**
General		
1.	Looking south at the northwest corner	
2.	Looking east at the west face of the building	

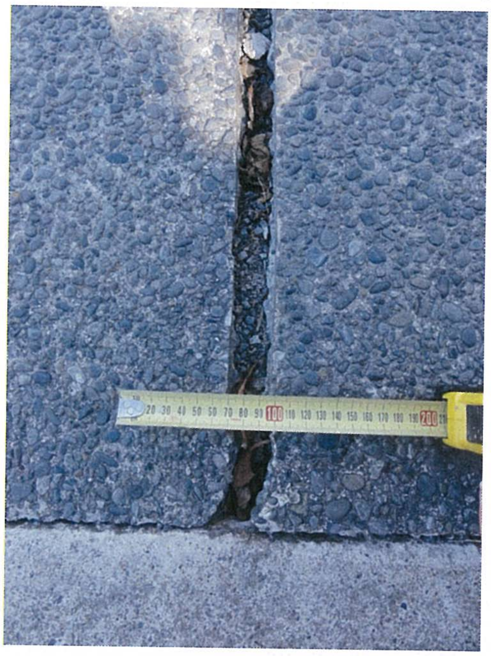
3.	Looking southwest at the north face	
4.	Looking west at the east face	
5.	Looking west at the south face	

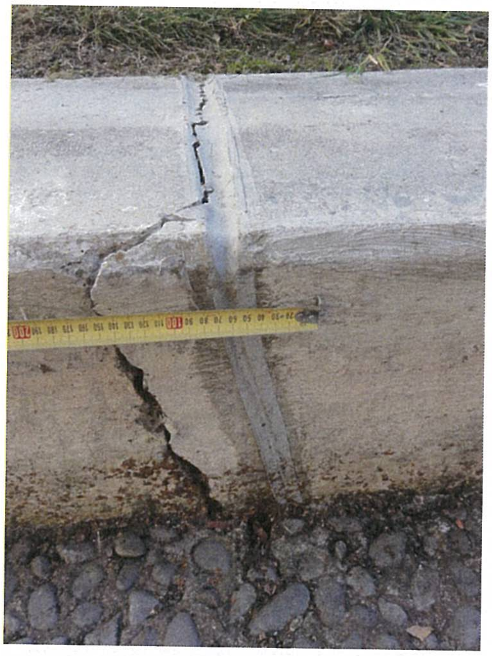
6.	Looking east through the Library Workroom	
7.	Looking east at the Library Services station	
8.	Cracking in the gib board	

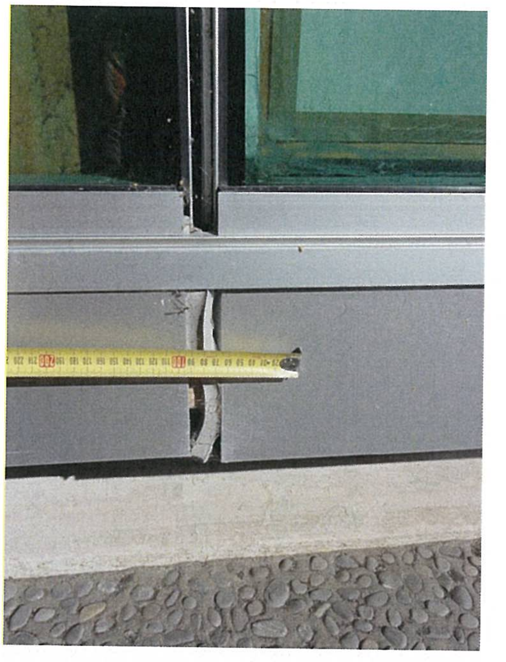
9.	Cracking in the gib board	
10.	Cracking in the gib board	
11.	Cracking in the gib board	

12.	Cracking in the gib board		
13.	Cracking in the gib board		
14.	Dislodged stone façade piece on the west face		

15.	Cracking in the moat and perimeter ground beam		
16.	Cracking in the ground floor slab due to ground movement		
17.	Differential settlement of the exterior paving at the library entrance		

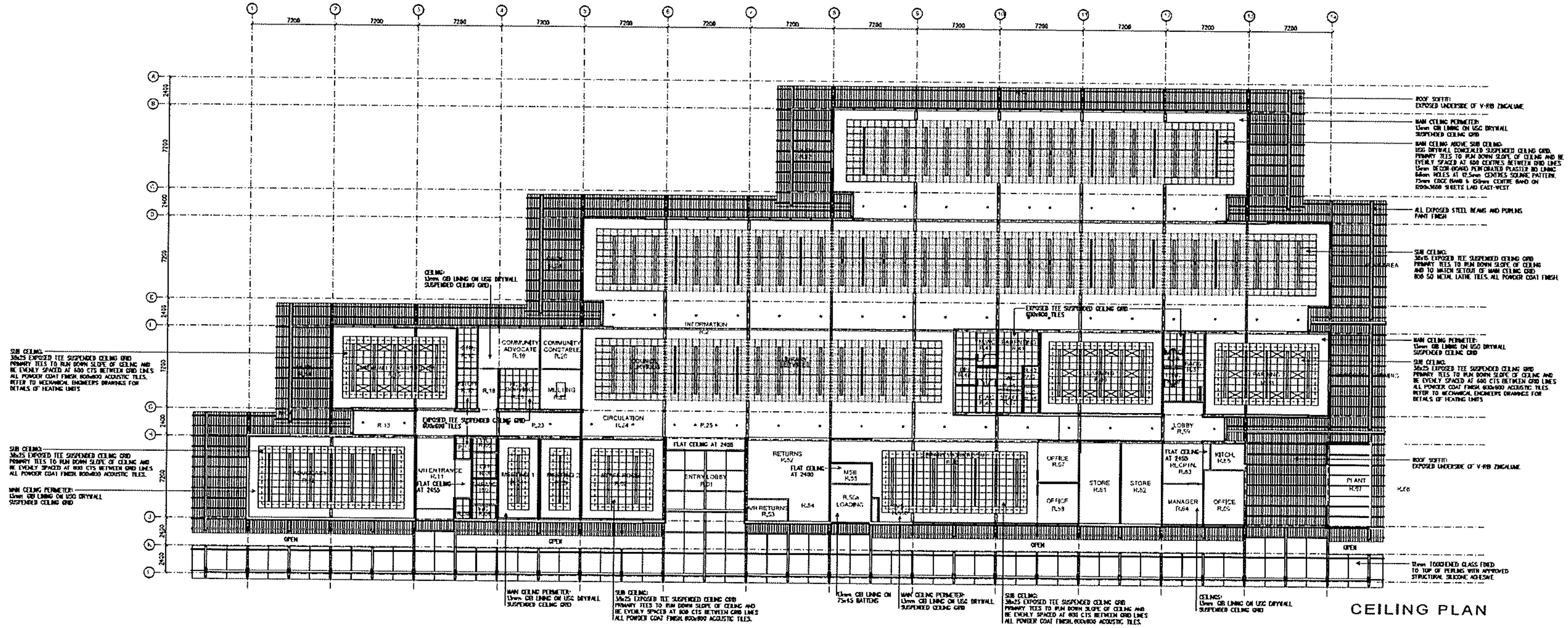
18.	Separation of the paving along the exterior walkway due to ground movement	
19.	Differential settlement in the paving along the exterior walkway due to ground movement	

20.	Cracking in the ground beam along the perimeter moat	
21.	Cracking in the perimeter ground beam	

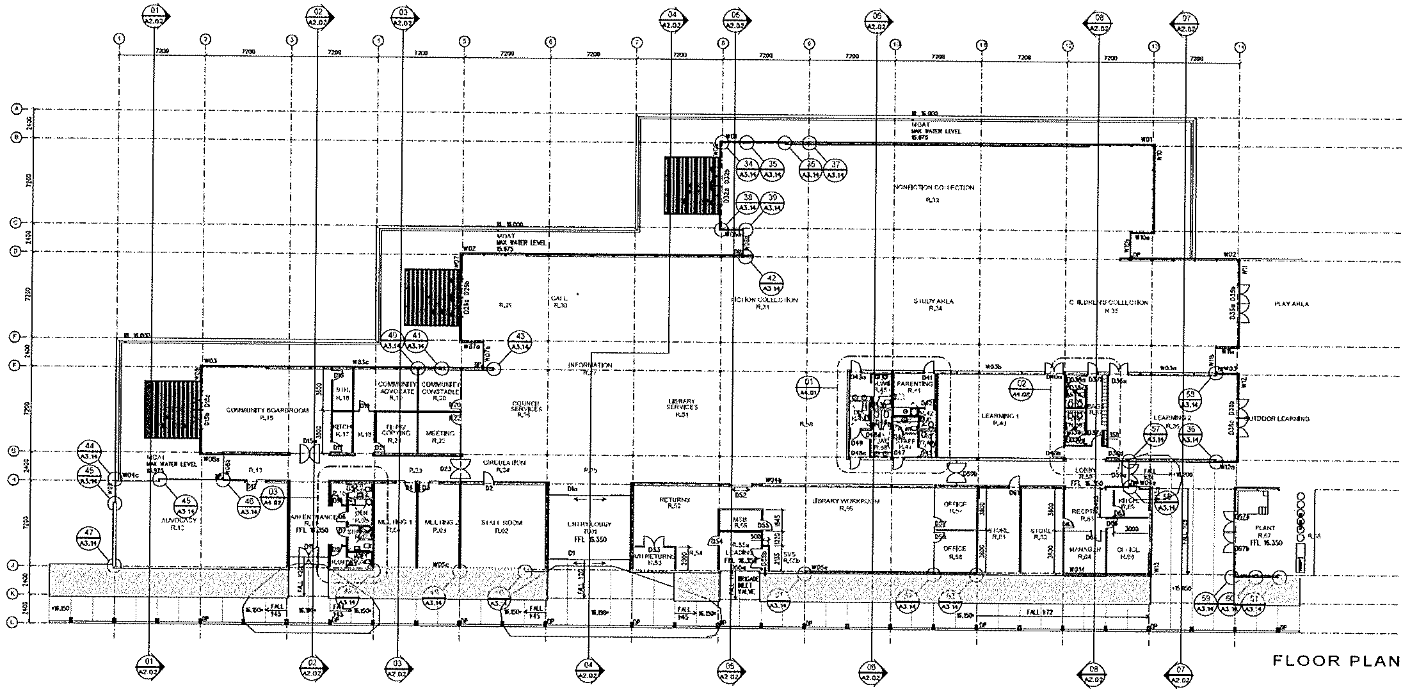
22.	Separation of the façade pieces due to ground movement	
23.	Cracking to precast panel H.1	

Appendix 2 – Floor Plan

REVISIONS
A 16-09-02 RAMPS TO ENTRANCES ADJUSTED



CEILING PLAN



FLOOR PLAN

CONSULTANTS

BECA CARTER HOLLINGS & FERNER LTD
Mechanical Consultant

CITY SOLUTIONS

HARSTON DAVIES LTD

SOUTH CHRISTCHURCH LIBRARY & SERVICE CENTRE

CEILING PLAN
FLOOR PLAN

SCALE	
DATE	$$$$DATE$$$$
JOB No	
DRAWN	
CHECKED	

A 1.02 (A)

Auckland | Wellington | Christchurch

Victoria Street
P O Box 25086
Christchurch 8030
New Zealand
tel +64 (3) 961 5926
fax +64 (3) 961 5935
email wam@wam.co.nz
www.wam.co.nz

architecture Warren and Mahoney

Appendix 3 – DEEP Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	South Christchurch Library
Building Address:	Unit: No: Street: 66 Colombo Street
Legal Description:	
GPS south:	Degrees 43, Min 33, Sec 41.07
GPS east:	Degrees 172, Min 38, Sec 16.75
Building Unique Identifier (CCC):	
Reviewer:	Alistair Boyce
CPEng No:	209860
Company:	Opus International Consultants
Company project number:	6-QUCCC.07 / 005SC
Company phone number:	+64 3 363 5400
Date of submission:	20/07/2012
Inspection Date:	various incl. 19/07/2012
Revision:	Final
Is there a full report with this summary?	yes

Site

Site slope:	flat
Soil type:	silt
Site Class (to NZS1170.5):	D
Proximity to waterway (m, if <100m):	31
Proximity to clifftop (m, if < 100m):	
Proximity to cliff base (m,if <100m):	
Max retaining height (m):	0
Soil Profile (if available):	D, Soft Soil
If Ground improvement on site, describe:	
Approx site elevation (m):	16.00

Building

No. of storeys above ground:	1 (single storey = 1)
Ground floor split?	no
Storeys below ground	0
Foundation type:	isolated pads, no tie beams
Building height (m):	4.50
Floor footprint area (approx):	
Age of Building (years):	
Ground floor elevation (Absolute) (m):	16.35
Ground floor elevation above ground (m):	0.35
if Foundation type is other, describe:	
height from ground to level of uppermost seismic mass (for IEP only) (m):	4.4
Date of design:	1992-2004
Strengthening present?	no
If so, when (year)?	
And what load level (%g)?	
Brief strengthening description:	
Use (ground floor):	public
Use (upper floors):	
Use notes (if required):	
Importance level (to NZS1170.5):	IL3

Gravity Structure

Gravity System:	frame system
Roof:	steel framed
Floors:	
Beams:	steel non-composite
Columns:	structural steel
Walls:	load bearing concrete
rafter type, purlin type and cladding	DHS purlins, with some PFC purlins
beam and connector type	UB, with welded conn to column
typical dimensions (mm x mm)	150x150
#N/A	

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	welded and bolted steel moment frame
Ductility assumed, μ:	2.00
Period along:	0.43 (0.43 from parameters in sheet)
Total deflection (ULS) (mm):	
maximum interstorey deflection (ULS) (mm):	
note typical bay length (m)	7.2
estimate or calculation?	calculated
estimate or calculation?	
estimate or calculation?	
Lateral system across:	welded and bolted steel moment frame
Ductility assumed, μ:	2.00
Period across:	0.43 (0.00)
Total deflection (ULS) (mm):	
maximum interstorey deflection (ULS) (mm):	
note typical bay length (m)	7.2
estimate or calculation?	calculated
estimate or calculation?	
estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	
Wall cladding:	other light (describe: mixture of glass, stone, metal exteriors)
Roof Cladding:	Metal (describe: Zincalume coated steel)
Glazing:	aluminium frames
Ceilings:	fibrous plaster, fixed
Services(list):	

Available documentation

Architectural	full	original designer name/date	WAM, 2002
Structural	full	original designer name/date	C. Bedford, 06/2002
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:	
Describe damage:	
Settlement:	25-100m
Differential settlement:	1:150 or more
Liquefaction:	none apparent
Lateral Spread:	0-50mm
Differential lateral spread:	none apparent
Ground cracks:	0-20mm/20m
Damage to area:	slight
notes (if applicable):	

Building:

Current Placard Status:	green
Along — Damage ratio:	
Along — Describe (summary):	
Describe how damage ratio arrived at:	
Across — Damage ratio:	#DIV/0!
Across — Describe (summary):	
Diaphragms — Damage?:	no
Describe:	
CSWs — Damage?:	no
Describe:	Significant cracking of precast panel(s)
Pounding — Damage?:	no
Describe:	
Non-structural — Damage?:	yes
Describe:	

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening
Building Consent required:	yes
Interim occupancy recommendations:	do not occupy
Describe:	
Along — Assessed %NBS before:	##### %NBS from IEP below
Along — Assessed %NBS after:	10%
If IEP not used, please detail assessment methodology:	
Across — Assessed %NBS before:	##### %NBS from IEP below
Across — Assessed %NBS after:	10%

IEP

Use of this method is not mandatory - more detailed analysis may give a different answer, which would take precedence. Do not fill in fields if not using IEP.

Period of design of building (from above): 1992-2004

h_n from above: 4.4m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building

Design Soil type from NZS4203:1992, cl 4.6.2.2:

along across

	along	across
Period (from above):	0.43	0.43
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final $(\%NBS)_{nom}$:	0%	0%

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, cl 3.1.6:	1.00	
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3:	0.00
Z_{1992}, from NZS4203:1992	0.8
Hazard scaling factor, **Factor B**:	#DIV/0!

2.4 Return Period Scaling Factor

Building Importance level (from above):	3
Return Period Scaling factor from Table 3.1, **Factor C**:	0.90

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	2.00	2.00
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.00	1.00
Ductility Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.700	0.700
Structural Performance Scaling Factor **Factor E**:	1.428571429	1.428571429

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

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

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

3.1. Plan Irregularity, factor A:		1
3.2. Vertical irregularity, Factor B:		1
3.3. Short columns, Factor C:		1
3.4. Pounding potential	Pounding effect D1, from Table to right	1.0
	Height Difference effect D2, from Table to right	1.0
	Therefore, Factor D:	1
3.5. Site Characteristics		1

Table for selection of D1	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Alignment of floors within 20% of H	**0.7**	**0.8**	**1**
Alignment of floors not within 20% of H	**0.4**	**0.7**	**0.8**

Table for Selection of D2	Severe	Significant	Insignificant/none
Separation	0<sep<.005H	.005<sep<.01H	Sep>.01H
Height difference > 4 storeys	0.4	0.7	1
Height difference 2 to 4 storeys	0.7	0.9	1
Height difference < 2 storeys	1	1	1

3.6. Other factors, Factor F

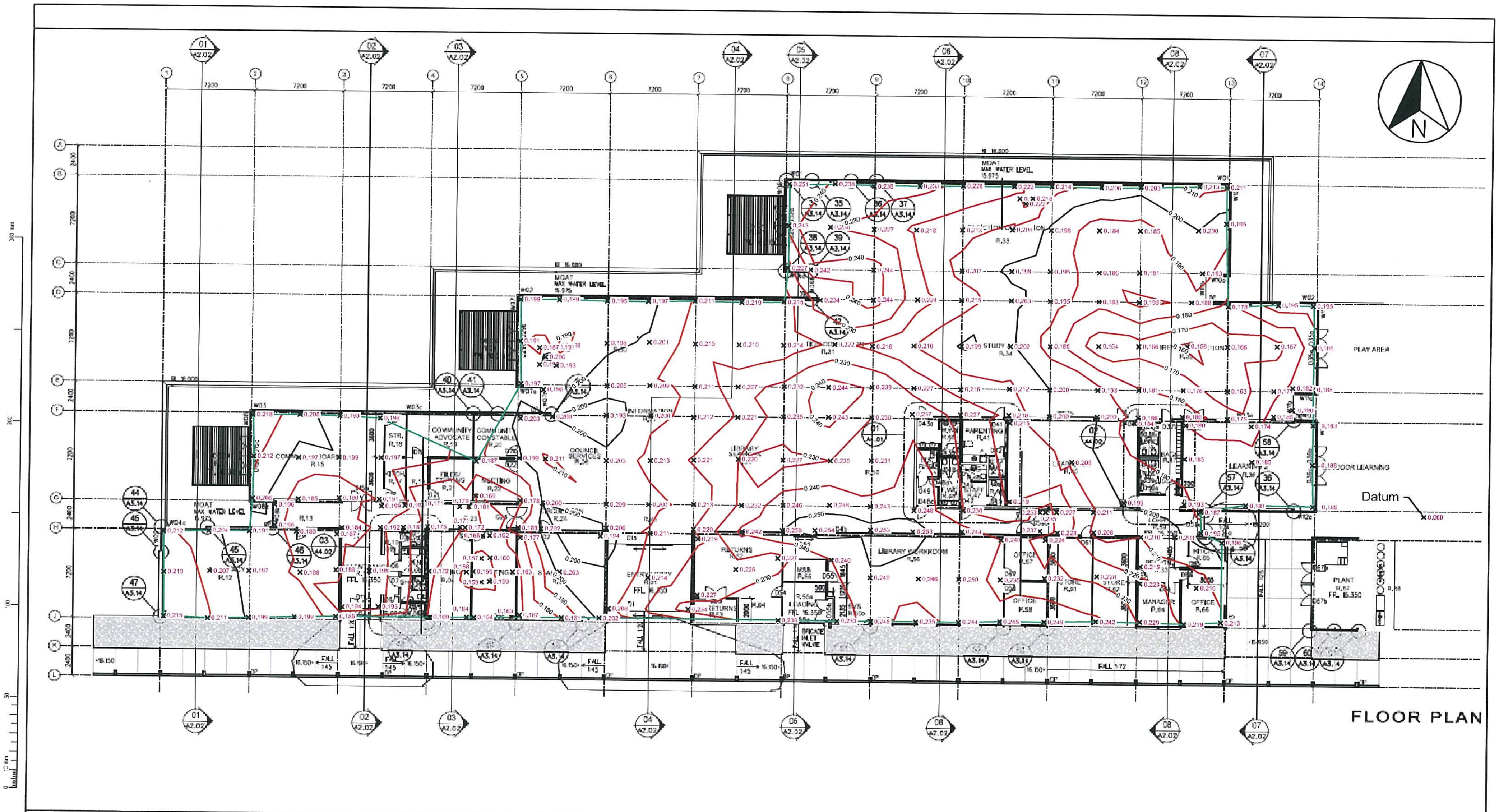
	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum		
Rationale for choice of F factor, if not 1		

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)

List any: Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

	along	across
3.7. Overall Performance Achievement ratio (PAR)	0.00	0.00
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	#DIV/0!	#DIV/0!
4.4 Percentage New Building Standard (%NBS), (before)		#DIV/0!

Appendix 4 – Level Survey Drawing



Note. All levels are in terms of an assumed datum.
Datum Point: Is a MH to the immediate east of the building. This point has been given the RL 0.000m

OPUS	Christchurch Office P O Box 1482 Christchurch 8140, New Zealand +64 3 363 5400
Project	Christchurch South Public Library Floor Level Survey
Drawn	J. Morrow
Approved	S. Becker
Project No.	6QUCCC.91 / 45SC
Scale	1:150 @ A1, 1:300 @ A3
Drawing No.	6/1366/271/2604
Sheet No.	1/1
Revision	R0

Original Sheet Size A1 [841x594] Plot Date 05/07/12 @ 12:18

OPUS