OPUS

Christchurch City Council

Sockburn Service Centre PRO 1531-005

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Sockburn Service Centre

Quantitative Assessment Report

149 Main South Road

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Summary

Sockburn Service Centre
PRO 1531-005

Detailed Engineering Evaluation
Quantitative Report - Summary
Final – Revision B

Background
This is a summary of the quantitative assessment report for the Sockburn Service Centre building structure at 149 Main South Road, Sockburn, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011, visual inspections on 8 March 2011, 22 March 2011, and 25 June 2012 and available drawings.

Key Damage Observed
Key damage observed includes:

- Cracks at the base of several concrete columns.
- Cracks and spalling of the corner of the concrete strong room, at the southeast entrance.
- Cracks to wall Gib board linings and ceilings throughout the building.
- Cracks to timber wall linings throughout the building.

Critical Structural Weaknesses
The following critical structural weaknesses have been identified:

- The connections of structural elements at the first floor to the tops of the strong room walls have an apparent small amount of weld. As the strong room walls are much stiffer than the perimeter concrete frames, these connections will attract a large percentage of the seismic forces until their capacity has been exceeded. The failure mode is likely to be sudden and brittle. Once the capacity has been exceeded, further horizontal drift could result in localised collapse of the central portion of the first floor structure.
- At the first floor of the perimeter concrete frames, the precast beams are connected to the precast columns with a short length of reinforcing bar at the top and bottom of each beam. Therefore, a brittle failure could occur at these joints, leading to sudden collapse of portions of the roof and floor structure. At larger force levels, this could result in total collapse of the building.

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 less than 33% NBS and post-earthquake capacity less than 33% NBS. The building is therefore classed as an earthquake prone building.

Recommendations

a) We recommend that the building remain unoccupied given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

b) Before any further assessment or strengthening analysis is undertaken, we recommend that intrusive investigations be undertaken to determine if the assumptions made in the assessment are valid.

c) We recommend that a geotechnical study be undertaken before any strengthening design commences.

d) As the building is earthquake prone, cordoning is recommended along all sides, at a minimum distance of 12m.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council to undertake a detailed seismic assessment of the Sockburn Service Centre, located at 149 Main South Road, Sockburn, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011 and subsequent aftershocks.

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 Background Information

4.1 Building Description

4.1.1 General

The Sockburn Service Centre, located at 149 Main South Road, is a two storey building. The original building, constructed in 1958, was a single storey building and in 1971 a new structure was constructed over the top of the existing building. At that time, portions of the existing roof structure were removed and supported by steel channels and interior steel columns at the new floor level.

The main area of the building is approximately 15m x 42m, with a projection on one side of approximately 5m x 10m. The building is approximately 8.5m high. For the purposes of this report, the longitudinal direction is oriented northeast – southwest, parallel to Main South Road, and the transverse direction is oriented northwest – southeast, perpendicular to Main South Road.

Photographs of the building are shown in Appendix 1 and excerpts from the drawings are shown in Appendix 2.

4.1.2 Gravity load Resisting System

For the 1971 extension, new precast concrete columns and in-situ concrete footings were added outside the footprint of the original single storey structure. These columns are connected at the first floor level with precast concrete spandrel beams that were connected to the columns with an in-situ welded rebar detail. The tops of the columns support steel trusses that span across the building. The first floor is supported on steel channels that span between the exterior concrete columns and internal steel posts. Spanning between the channels are timber joists with tongue and groove decking. The roof consists of metal roofing over sarking over timber joists spanning between steel trusses. The floor at ground level, part of the original 1958 building, is timber framed, supported by shallow piles.

4.1.3 Lateral Load Resisting System

Lateral seismic forces in the longitudinal direction are resisted by frame action of the concrete columns and beams along grid lines A, G, and H. At the roof level, forces are transferred though the timber sarking diaphragm to the concrete columns. The columns cantilever from the first floor, with weak axis bending. At the first floor level, forces are transferred through the tongue and groove timber diaphragm into the precast concrete beams along grid lines A, G, and H. The precast concrete beams and columns, with cast in situ beam segments each side of the columns, act as moment frames to transfer forces into the foundations.

Lateral forces in the transverse direction are resisted by a combination of moment frame action at the interior grid lines and moment frame action along the perimeter concrete frames at grids 1 and 14. At the interior grid lines, the frames consist of precast concrete columns, with bending in the strong axis. Steel trusses at the roof and steel channels at the floors have spigot connections to the concrete columns, providing effective pin joints. At grids 1 and 14, the moment frames action is similar to the frames on grids A and G. Transverse seismic forces at the floor and roof will be transferred to the frames through the timber roof sarking and tongue and groove flooring.

The details of the concrete beam to column joints at the first floor perimeter frames consist of a welded rebar connection at the top and bottom of each beam. This is discussed in greater detail in Section 7 of this report.

Inside the original building are two concrete strong rooms. The tops of the strong rooms are located just below the first floor, at approximately the same elevation as the bottom of the precast concrete spandrel beams. In the 1971 extension, some of the new first floor structural elements were supported on the walls of the strong room, as described below:

- The precast portions of the spandrel beams along grid lines 4.8 and 8.2 stop short of the strong room wall corner. The cast in situ portions, which are also attached to the precast beams along grid G, bear on top of the strong rooms. The drawings do not show the length or size of the welding of the reinforcing bars which are indicated on the original drawing to connect these areas. The details for this area are shown in Figure A2-6 of Appendix 2.

- Along grids 5, 6, 7, and 8, the steel channels that support the first floor bear on the strong room walls. The drawings, as shown in Figure A2-4, do not indicate the size or length of weld connecting the steel channels to the reinforcing steel of the walls.

- The connections described above, because the strong rooms are more rigid than the perimeter concrete frames, are likely to attract significant seismic forces until the capacity of the connections is exceeded. For the purposes of this assessment, these connections are identified as potential critical structural weaknesses. Because the connections could fail at a relatively small to moderate earthquake, the perimeter frames have been assessed with the assumption that no seismic forces will be resisted by the strong room walls.

4.2 Survey

4.2.1 Post 22 February 2011 Rapid Assessment

A structural (Level 2) assessment of the interior and exterior of the building was undertaken on 8 March 2011 by an engineer from Opus International Consultants. At that time the building was posted Green, indicating no restriction on use. The Level 2 report indicated that no damage was observed, except for what had been previously observed after the 10 September 2010 earthquake.

4.2.2 Further Inspections

On 22 March 2011, an Engineer from Opus International Consultants undertook a Level 3 Assessment to determine the level of damage in greater detail. A report was issued, dated 6 May 2011, that identified the building as potentially earthquake prone, based on a comparison of current seismic coefficients using estimates of the structural capacity made by a CCC staff engineer in 2001.

A further inspection was undertaken by an Opus engineer on 25 June 2012. At that time the building was closed. This was an inspection of the exterior of the building only.

4.3 Original Documentation

Copies of the following documentation was provided by CCC:

- Additions and Alterations to the Paparua County Council Offices Sockburn CH-CH, Architectural, Structural, and Electrical Drawings, Dated January 1971. Architectural and Electrical drawings were by Griffiths, Moffatt, & Partners. Structural drawings were by Powell Fenwick & Partners.

- Offices for the Paparua County Council Sockburn Christchurch, one Architectural and one Structural drawing, dated November 1958. Drawings were by Griffiths, Moffatt, & Partners Architects and Powell Fenwick & Partners Structural Engineers.

- Paparua County Council Extensions to Existing County Offices, one Architectural and one Structural drawing, dated May, 1967. Architect and Structural Engineer were not identified on the drawings.

- A memo from Stuart Smith, CCC Design Engineer, dated 5 September 2001, with an estimate of the building's seismic capacity.

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

Copies of the design calculations for the building (original building or additions) were not provided by CCC and may not be available.

Excerpts for the drawings of the 1971 addition are shown in Appendix 2.

5 Structural Damage

The following damage has been noted:

5.1 Foundations

No local ground settlement was observed around the base of the building. Some cracks in the southeast corner entrance to the Boiler Room were observed. There were also some cracks to the ground slab at the southeast corner of the entrance room.

5.2 Primary Gravity Structure

Cracks in the concrete columns were observed at several locations, typically near the bottom of the column; refer to Photographs 5 and 6 of Appendix 1. Cracks and spalling of the corner of the concrete strong room are seen in Photographs 7 and 8. Shoring is currently in place at this location.

5.3 Non Structural Elements

Damage to Gib board walls and timber finished panels was observed in several locations.

6 General Observations

With the exception of the southeast entrance corner, the structure appears to have generally performed well during the earthquake. It sustained moderate damage to structural elements, as well as some moderate to severe damage to non-structural elements. The observed damage is consistent with the expected building performance, following a review of the structural drawings and site investigations. Damage to internal and concealed connections is unknown because they were not observed.

7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [3] draft document prepared by the Engineering Advisory Group on 19 July 2011, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5] 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. The following potential CSW's were identified:

a) The connections of structural elements at the first floor to the tops of the strong room walls have an apparent small amount of weld. As the strong room walls are much stiffer than the perimeter concrete frames, these connections will attract a large percentage of the seismic forces until their capacity has been exceeded. The failure mode is likely to be sudden and brittle. Once the capacity has been exceeded, further horizontal drift could result in localised collapse of the central section of the first floor structure.

b) At the first floor of the perimeter concrete frames, the beams are connected to the column with a welded reinforcement detail at the top and bottom of the beams. Therefore, a brittle failure could occur at these joints, leading to sudden collapse of portions of the roof and floor structure. At larger force levels, this could result in total collapse of the building.

7.2 Quantitative Assessment Methodology

To assess the lateral load capacity of the building, the required seismic forces for new building standard were calculated by the equivalent static method, assuming the following seismic parameters:

Hazard Factor, Z:	0.30
Subsoil Class:	D
Importance Level:	2
Ductility, μ (both directions):	1.00
Structural Performance Factor, Sp:	1.0
Calculated Period, T (both directions):	0.297 seconds
Seismic Coefficient Cd(T):	0.900

Selected perimeter and interior frames were modelled using the computer programme Microstran. In this analysis, it was assumed that the connection detail between the precast spandrel beams and the precast columns was constructed in a way that provided continuity of the top and bottom beam reinforcing through the columns. The following information about these joints was obtained from the documentation provided by CCC:

- The longitudinal reinforcing bars in the spandrel beams (2-16mm top and bottom) extend 203mm from the end of the precast concrete.
- There is a space of approximately 254mm between the end of the beam and the face of the column. This space was for a cast in situ concrete closure.
- There are 4-19mm inside diameter sleeves through each column, located so that straight reinforcing bars could pass through the column and lap with the bars that extend out from the spandrel end.

- There are two shear keys cast into each end of the spandrel beams and each face of the columns.
- The bars protruding from the precast beams are welded to the round bars passing through the columns and the bars projecting from the precast concrete. Since welding of reinforcing steel is indicated elsewhere on the drawings, it is possible that the bars are welded together. A deconstruction of the joint would be required to determine the actual connection method.
- Details from the drawings are shown in Figure A2-5 of Appendix 2. Photograph 4 shows an example of a completed joint.

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 and assumptions made about the construction details of the exterior frame beam-column joints. These assumptions include:
  - The spandrel beam reinforcing is connected through the beam-column joint by welding or other mechanical connection
  - The first floor diaphragm is well connected to the entire concrete frame.
  - The sarking provided at the roof level is sufficient to transfer lateral loads to the perimeter frames.
  - The foundation provides fixity (restraint from rotation) at the base of the columns.
  - 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.

7.4 Quantitative Assessment Results

Structural Element/System	Failure mode, or description of limiting criteria based on elastic capacity of critical element.	% NBS based on calculated capacity
Perimeter concrete frames along grids A and G (longitudinal forces)	Assessment based on assumptions of reinforcing continuity and foundation fixity. Actual capacity may be less than calculated. Mode of failure likely to be brittle, due to lack of ductility in the connections, resulting in localized or global collapse.	24% (maximum), governed by flexure
Perimeter concrete frames along grids 1 and 14 (transverse forces)	Assessment based on assumptions of reinforcing continuity and foundation fixity. Actual capacity may be less than calculated. Mode of failure likely to be brittle, due to lack of ductility in the connections, resulting in localized or global collapse.	33% (maximum), governed by flexure
Connection of first floor steel and concrete beams to top of strong walls	Size and length of weld unknown, but likely to fail in a sudden and brittle manner. Once failure occurs, localized collapse is probable.	Unknown, less than 20%
Interior frames of steel roof trusses, steel floor channels, and	Flexural failure in columns along the strong axis. Failure mode will have limited ductility, due to wide spacing of column ties.	24%
Floor and roof diaphragms	Diaphragms were not assessed, because nailing was unknown. Because the capacity of the concrete frames is so low, diaphragm strength is not likely to control capacity. If strengthening works are undertaken, the diaphragms will need to be assessed, and strengthened if necessary.	Unknown

7.5 Discussion of Results

The building has a calculated capacity of less than 20% NBS, based on the capacity of the connections at the top of the strong walls. In addition, the concrete perimeter frames do not have sufficient strength or ductility and have also been assessed as having seismic capacities of less than 33% NBS in each direction.

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 lack of redundancy and the potential brittle failure modes, we believe that the overall risk of a local or global collapse is moderate to high and that the building should be cordoned to a width of 12m around the entire perimeter.

Due to a lack of detailed structural information, calculated capacities are based on a number of assumptions. The capacities in the table above are therefore considered to be in the upper range capacities.

8 Summary of Geotechnical Appraisal

8.1 General

A geotechnical investigation was not conducted as part of this assessment. Because of the location and lack of visible ground damage, it was not considered essential to determining the seismic capacity of the building.

8.2 Liquefaction Potential

The site is indicated to have low potential for liquefaction in the ECAN study. The residential areas on the other side of Main South Road are indicated on the CERA map as being in Technical Category TC1 (grey), meaning they are unlikely to incur future land damage from liquefaction.

8.3 Further Work

If strengthening works are undertaken for the building, a geotechnical investigation will be required for the design of any foundation elements.

9 Conclusions

a) The seismic performance of the building is governed by the flexural capacity of the perimeter concrete frames, which have an expected strength of less than 33% NBS in both directions. The building is therefore considered to be earthquake prone in accordance with the Building Act 2004.

b) The assessment of less than 33% NBS is based on the following assumptions:

- The spandrel beam reinforcing is connected through the beam-column joint by welding.
- The first floor diaphragm is well connected to the entire concrete frame.
- The sarking provided at the roof level is sufficient to transfer lateral loads to the perimeter frames.
- The foundation provides fixity (restraint from rotation) at the base of the columns.

c) Due to lack of redundancy and critical structural weaknesses, we recommend the building be cordoned.

10 Recommendations

a) We recommend that the building remain unoccupied given its earthquake prone building status and the elevated level of seismic risk in Christchurch.

b) As the building is earthquake prone, cordoning is recommended along all sides, at a minimum distance of 12m.

c) Before any further assessment or strengthening analysis is undertaken, we recommend that intrusive investigations be undertaken to determine if the assumptions noted above are valid.

d) We recommend that a geotechnical study be undertaken before any strengthening design is begun.

11 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.

12 References

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

[2] NZSEE: 2012, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

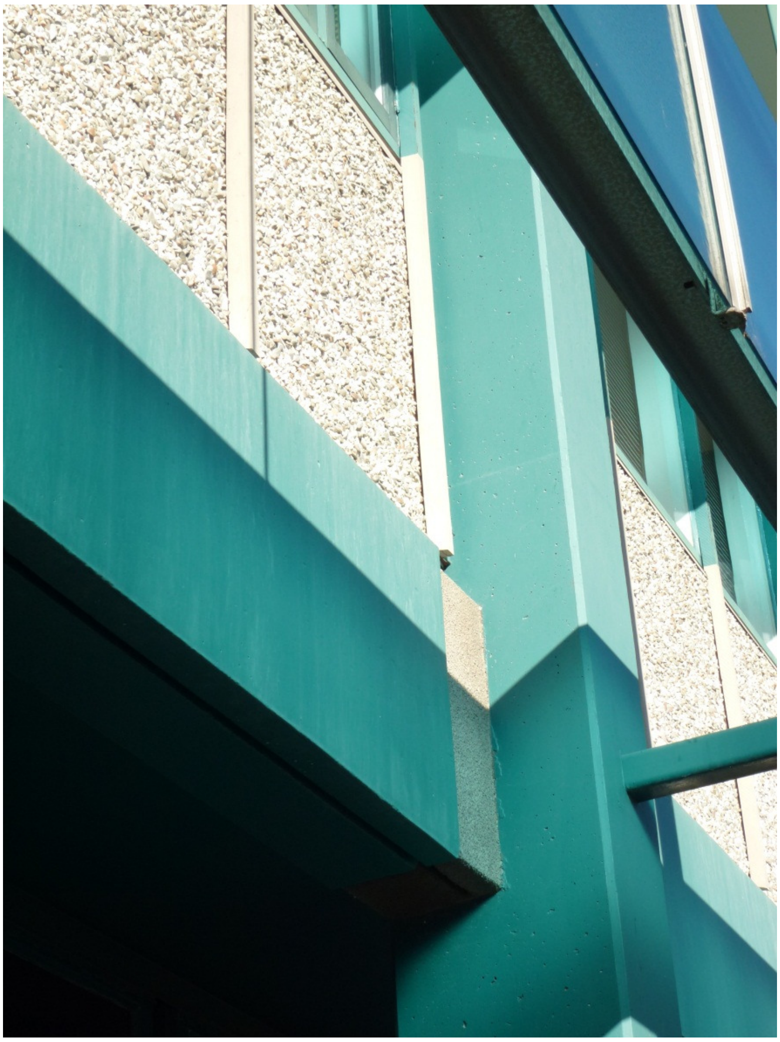
[3] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 5, 19 July 2011.

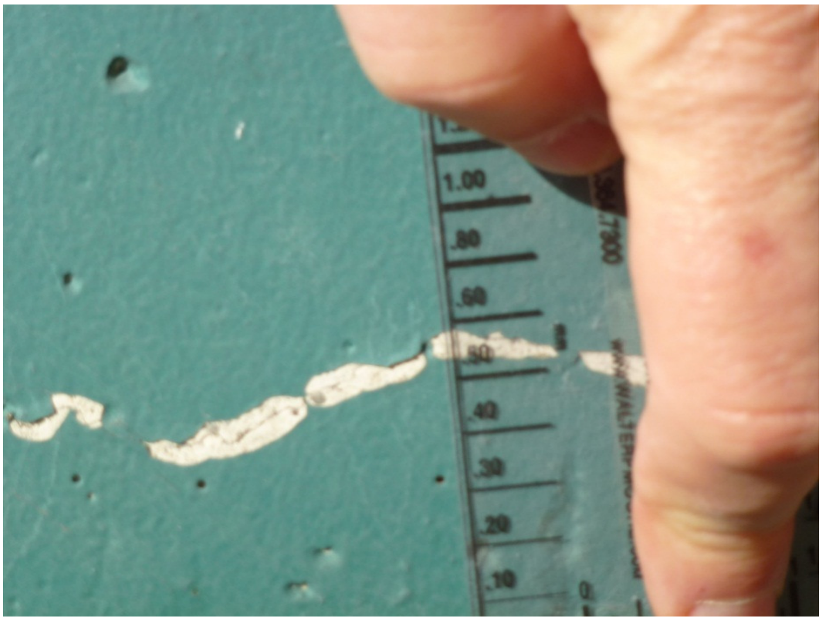
[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, 13 December 2011.

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

Appendix 1 - Photographs

CCC Sockburn Service Centre - Main South Road		
No.	**Item description**	**Photo**
1.	Front view of building, looking south	
2.	Front view of building, looking east	

3.	Rear view of building, looking north	
4.	Typical spandrel beam at column	

5.	Example of crack in concrete column. This column is on the north east side of the building	
6.	Close up view of column crack from photograph 5.	

7.	Cracks in corner of Strong Room at southeast entrance	
8.	Close up view of cracks from photograph 7	

Appendix 2 - Drawings

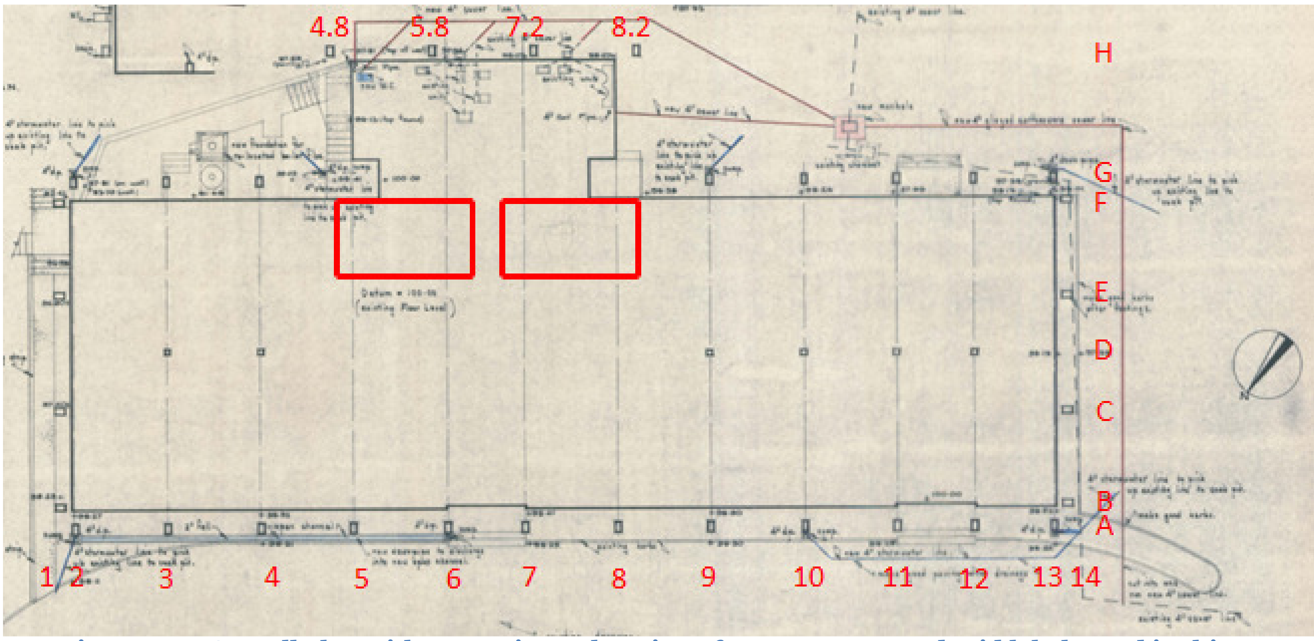


Figure A2-1 Overall plan with approximate location of strong rooms and grid labels used in this assessment. Grid line A is parallel to Main South Road.

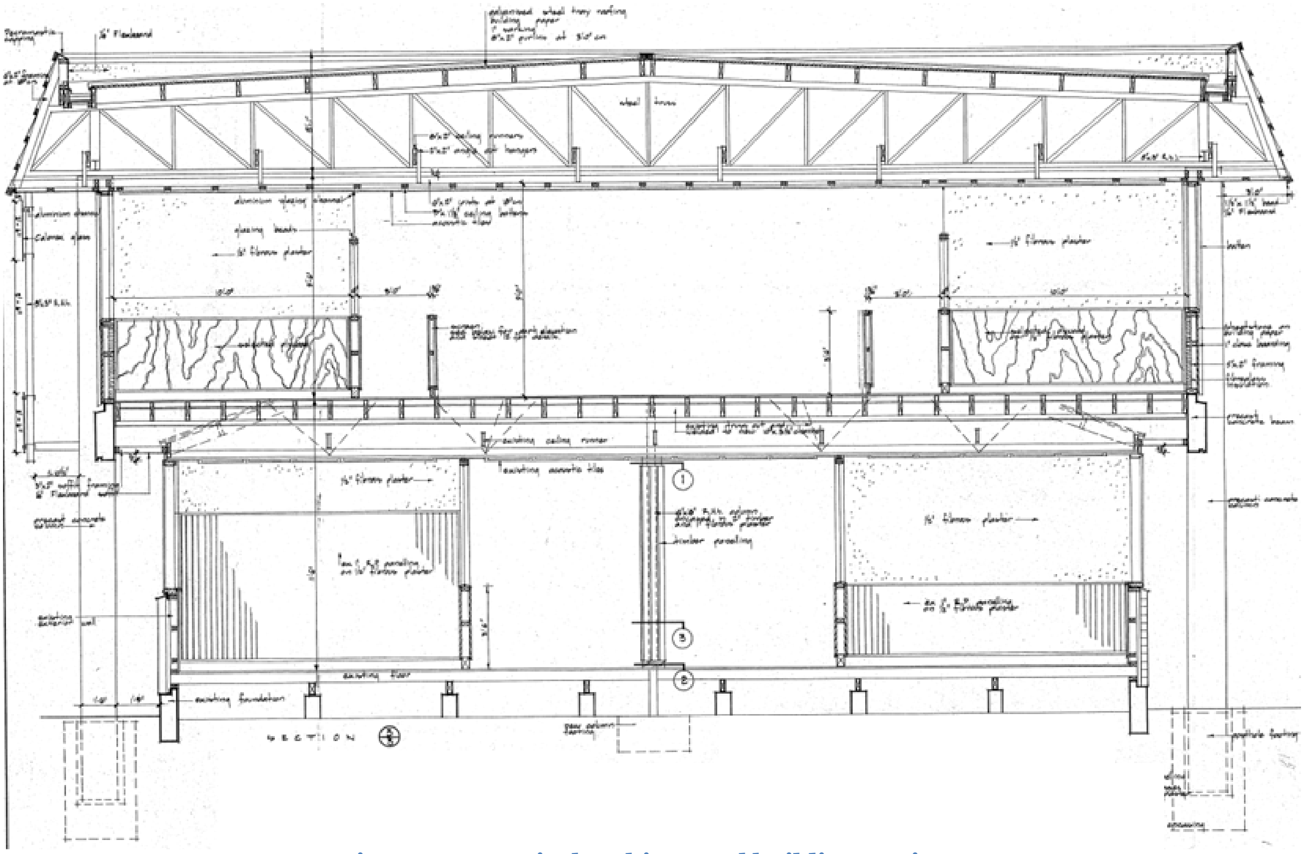


Figure A2-2 Typical architectural building section

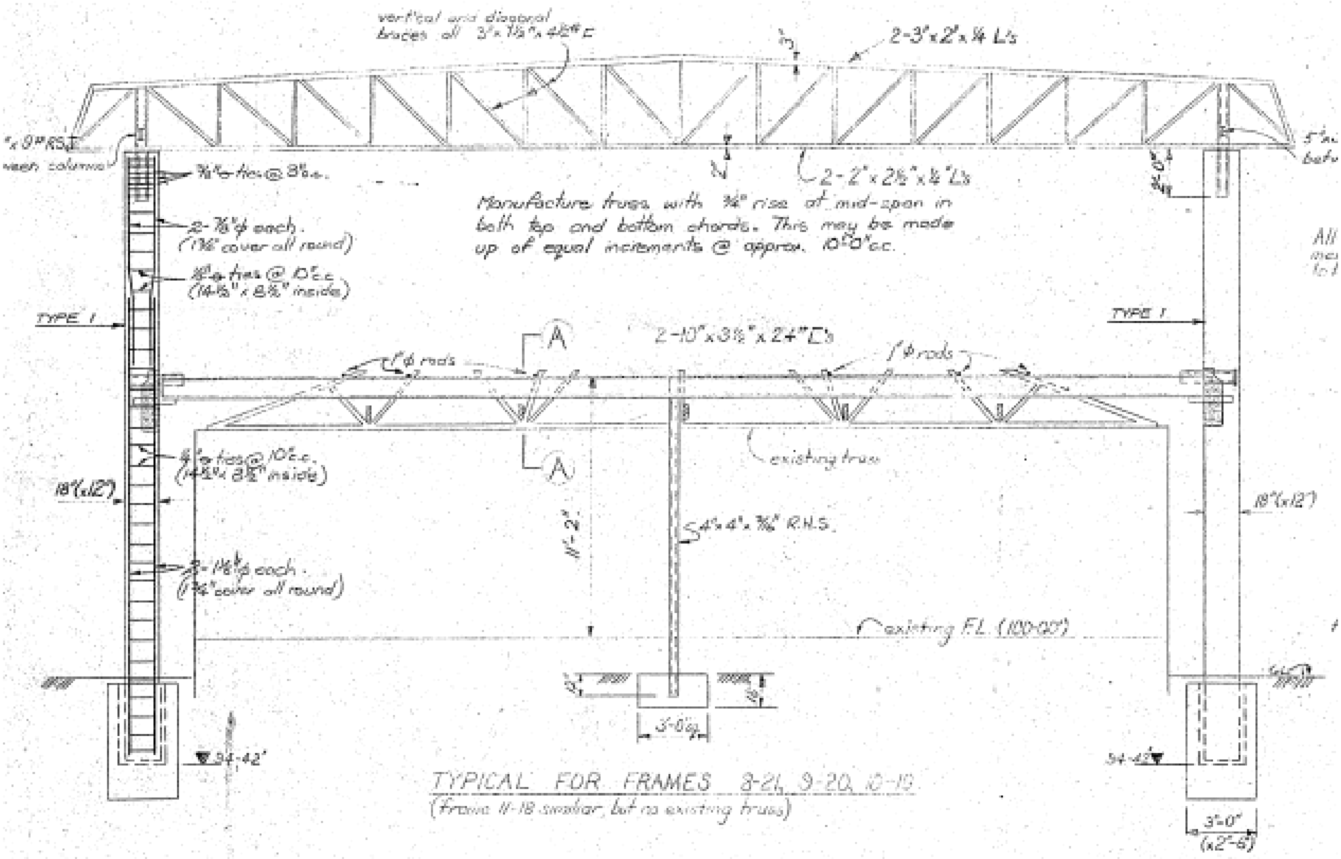


Figure A2-3 Typical structural building section

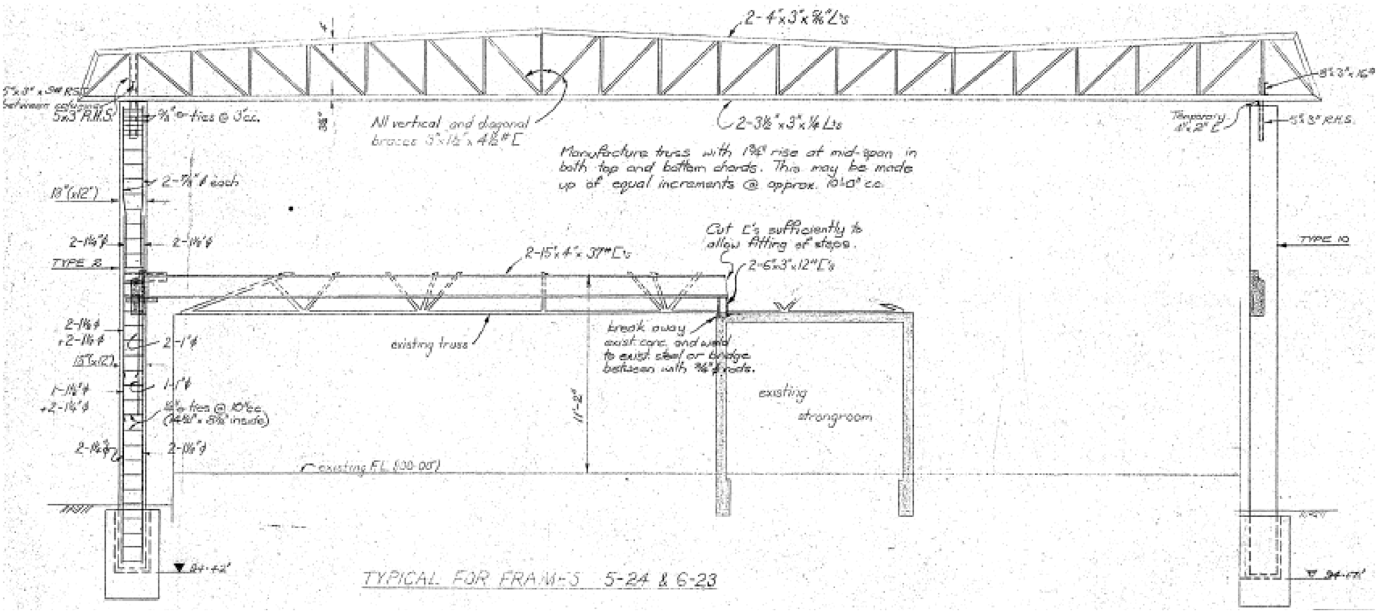


Figure A2- 4 Structural building section at strong room

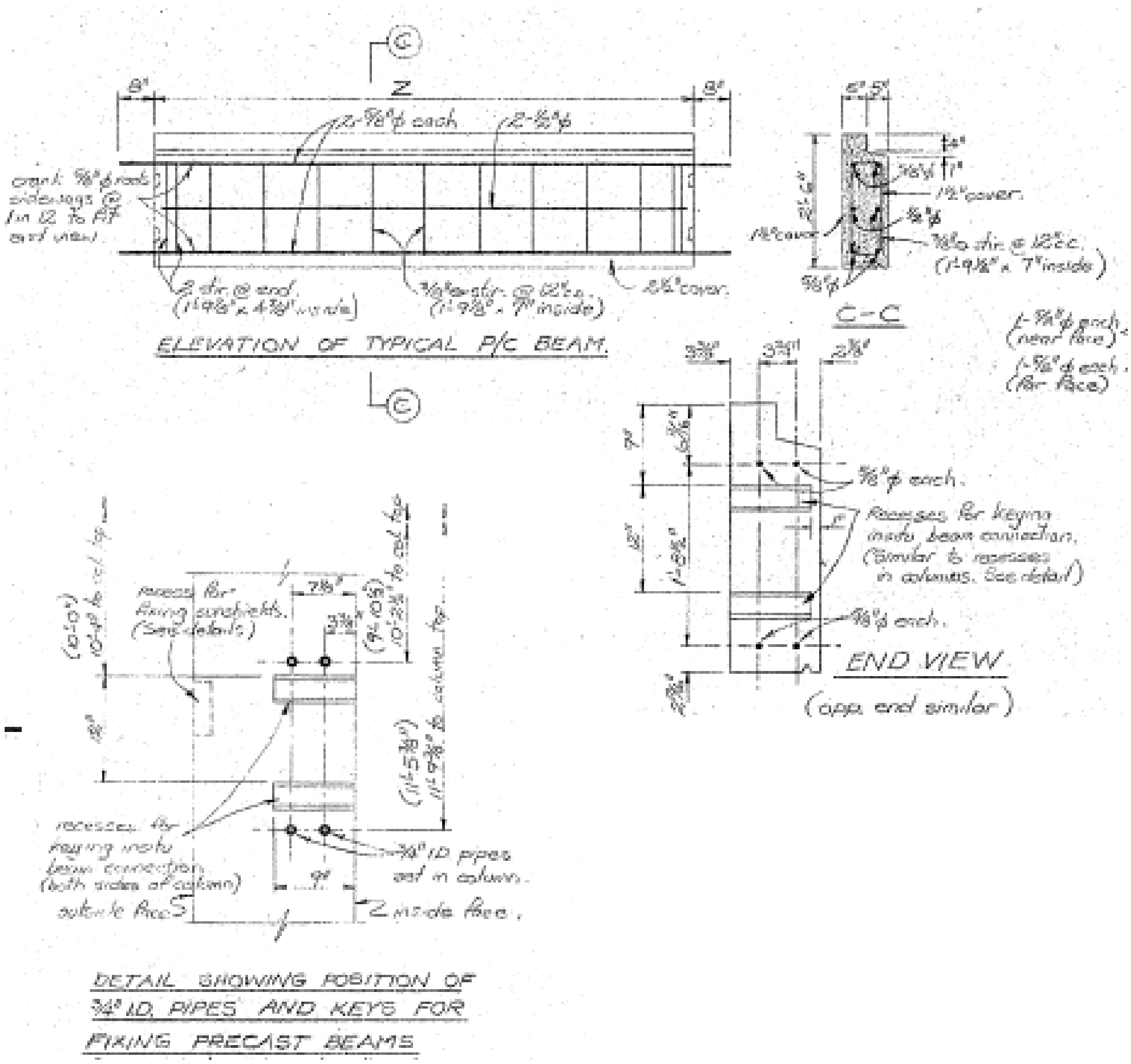


Figure A2- 5 Precast spandrel beam details

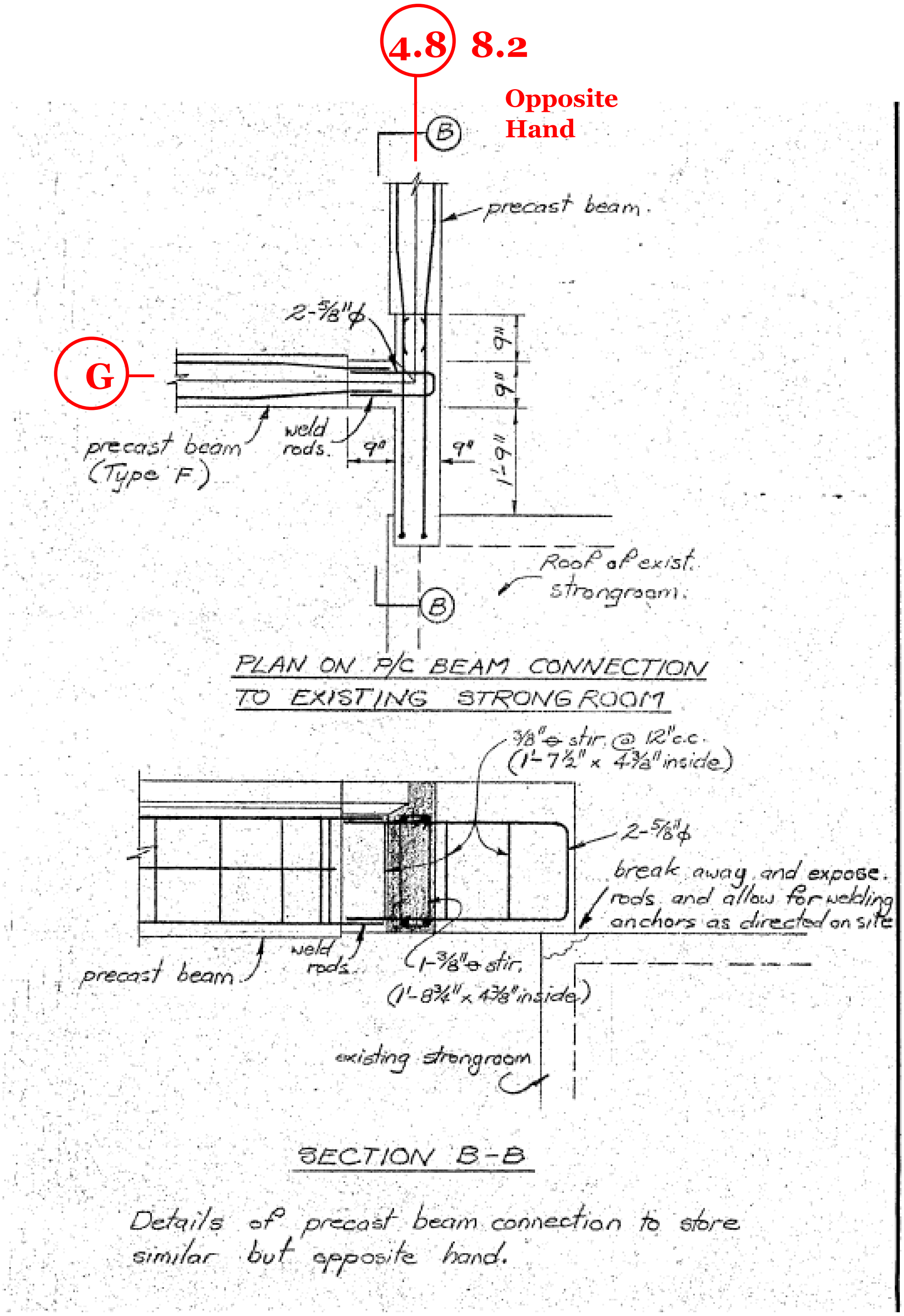


Figure A2- 6 Precast spandrel beam details at strong room corner

Appendix 3 - CERA DEE Data Sheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	CCC Sockburn Service Centre		Reviewer:	Jan Stanway
	Unit / No: / Street		CPEng No:	222291
Building Address:	149	Main South Road, Sockburn	Company:	Opus International Consultants, Ltd.
Legal Description:			Company project number:	6-QUCCC.68
			Company phone number:	(3) 365 7858
	Degrees / Min / Sec			
GPS south:	43	32 / 19.00	Date of submission:	31-Jan-14
GPS east:	172	33 / 20.00	Inspection Date:	
			Revision:	Revision B
Building Unique Identifier (CCC):	PRO 1531 005		Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	mixed	Soil Profile (if available):	
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):		If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	20.00

Building

No. of storeys above ground:	2	single storey = 1	Ground floor elevation (Absolute) (m):	
Ground floor split?	yes		Ground floor elevation above ground (m):	0.50
Storeys below ground	0			
Foundation type:	isolated pads, no tie beams		if Foundation type is other, describe:	
Building height (m):	8.50		height from ground to level of uppermost seismic mass (for IEP only) (m):	9
Floor footprint area (approx):	650			
Age of Building (years):	41		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):	public			
Use notes (if required):	Council Service Centre			
Importance level (to NZS1170.5):	IL2			

Gravity Structure

Gravity System:	frame system		
Roof:	steel truss	truss depth, purlin type and cladding	1m deep, timber purlins, sarking & metal roof
Floors:	timber	joist depth and spacing (mm)	250mm deep joists @ .5m
Beams:	steel non-composite	beam and connector type	double channel, nailed and bolted to timber
Columns:	precast concrete	typical dimensions (mm x mm)	457 x 305
Walls:	non-load bearing	0	

Lateral load resisting structure

Lateral system along:	non-ductile concrete moment frame	**Note: Define along and across in detailed report!**		3.9
Ductility assumed, μ:	1.00		note typical bay length (m)	
Period along:	0.30	0.47 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):	55		estimate or calculation?	calculated
maximum interstorey deflection (ULS) (mm):	23		estimate or calculation?	calculated
Lateral system across:	non-ductile concrete moment frame			7.2
Ductility assumed, μ:	1.00		note typical bay length (m)	
Period across:	0.30	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	117		estimate or calculation?	calculated
maximum interstorey deflection (ULS) (mm):	65		estimate or calculation?	calculated

Separations:

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

Non-structural elements

Stairs:	timber	describe supports	spans floor to floor
Wall cladding:	other light	describe	timber with boards
Roof Cladding:	Metal	describe	metal roof on timber sarking
Glazing:	timber frames		
Ceilings:	light tiles		
Services(list):			

Available documentation

Architectural	full	original designer name/date	Griffiths, Moffat & Partners 1971
Structural	full	original designer name/date	powell Fenwick & Partners 1971
Mechanical	partial	original designer name/date	Griffiths, Moffat & Partners 1971
Electrical	full	original designer name/date	Griffiths, Moffat & Partners 1971
Geotech report	none	original designer name/date	

Damage

Site: (refer DEE Table 4-2)

Site performance:	Good	Describe damage:	Cracks on long side of column
Settlement:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

	Current Placard Status:	red		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	Not safe to enter - observations on exterior on.
	Describe (summary):	Limited damage observed		
Across	Damage ratio:	0%		
	Describe (summary):	Limited damage observed		
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:	significant structural and strengthening	Describe:	New frames required at all sides
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	do not occupy	Describe:	Building currently closed
Along	Assessed %NBS before e'quakes:	20%	0% %NBS from IEP below	If IEP not used, please detail assessment methodology: Calculations for individual frames
	Assessed %NBS after e'quakes:	20%		
Across	Assessed %NBS before e'quakes:	20%	0% %NBS from IEP below	
	Assessed %NBS after e'quakes:	20%		

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