Southland Museum & Art Gallery

Detailed Engineering Evaluation
Quantitative Assessment Report

Invercargill City Council

Southland Museum & Art Gallery

Detailed Engineering Evaluation
Quantitative Assessment Report

Prepared By

Keith Palmer
Structural Engineer

Reviewed By

John Meyer
Principal Structural Engineer

Approved for Release By

Eddie McKenzie
Engineering Project Team Leader, Civil

Opus International Consultants Ltd

Christchurch Office
20 Moorhouse Avenue
PO Box 1482, Christchurch Mail
Centre, Christchurch 8140
New Zealand

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

Date: 29 November 2013
Reference: 6-VI030.00 / 005gv
Status: Final

Contents

Summary

The Invercargill City Council (ICC) appointed Opus International Consultants Ltd (Opus) to carry out a detailed seismic assessment of the Southland Museum and Art Gallery on Gala Street in Invercargill. The key outcome required of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards.

Findings of the assessment are:

Background

This is a summary of the quantitative assessment report for the building structure at 108 Gala Street in Invercargill, and is based on the Detailed Engineering Evaluation Procedure document [3], visual inspections, selective intrusive investigations and available drawings.

Critical Structural Weaknesses

The following critical structural weaknesses have been identified:

<u>Original 1940 building</u>

a) Flexural failure of exterior concrete moment frame beams at the first story;

b) Lack of roof diaphragm;

c) Pounding with adjacent buildings;

d) Restraint of north stairwell within 1988 structure and lack of visible gap between the two structures in this location;

e) Out-of-plane wall anchorage of concrete wall at roof level;

f) Out-of-plane support of brick in-fill wall;

g) Out-of-plane support of exterior brick veneer;

<u>1959 Addition</u>

a) Inadequate out-of-plane support of ground floor concrete wall along grid line B, 5, and 7;

<u>1960 Addition</u>

a) Pounding with adjacent buildings;

b) Torsional response at ground floor: Concrete frame along line B is infilled with masonry panels whereas frame along line E is not. This results in a torsional response under east-west earthquake loading.

<u>1988 Addition</u>

a) Pounding with adjacent buildings;

b) Shear failure of short-span concrete beam at second floor;

c) Stiffness incompatibility between the steel structure of the pyramid and the concrete frame. Steel diagonals of the pyramid between GL8 and 9 connect to the 1st and 2nd floor of the concrete frame and act as braces between floors. The connections of these members are not adequate to resist the imposed forces.

d) Failure of bolts connecting the pyramid roof diagonals to the perimeter concrete columns;

e) Lack of bracing at the south side of the mezzanine to the south of the 1940 museum.

Indicative Building Strength

Based on the information available, and from the results of the quantitative assessment, the buildings have the following expected capacity, based on the weakest element:

1940 Original Museum: <34%NBS. Classified as Earthquake Prone
1959 Addition: <34%NBS. Classified as Earthquake Prone
1960 Addition: 30%NBS. Classified as Earthquake Prone
1988 Addition: <34%NBS. Classified as Earthquake Prone

Recommendations

A staged approach is recommended as follows in order to understand and manage the economic impact of any proposed remedial actions:-

a) An outline scheme for structural strengthening – with a view to achieving a minimum level of 34%NBS, or to a recommended level of 67%NBS, should be further developed with sufficient information so that costing can be put on the proposed works. This will expand upon the remedial options discussed in Section 9.

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

c) Carry out a geotechnical investigation.

d) Carry out detailed design of a scheme for the strengthening of the structure.

1 Introduction

Opus International Consultants Limited has been engaged by Invercargill City Council to undertake a detailed seismic assessment of the Southland Museum and Art Gallery, located at 108 Gala St, Invercargill, to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation document [3].

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 Invercargill at present.

2.1 Building Act – Legislative Basis for the Earthquake Prone Building Policy

The sections of the Building Act that refer to earthquake prone buildings (EPBs) are in subpart 6 of Part 2 of the Act.

- Section 122 and its associated regulations define an earthquake-prone building.
- Sections 124 to 130 provide power for territorial authorities to act on EPBs and set out how this action is to be taken.
- Sections 131 and 132 require territorial authorities to establish EPB policies and specify how the policies are to be established, what they are to include and when they are to be reviewed.

2.1.1 Section 122 – Meaning of Earthquake Prone Building

Section 122 of the Building Act 2004 deems a building to be earthquake prone if having regard to its condition and to the ground on which it is built, and because of its construction, the buildings ultimate capacity is exceeded in a “moderate earthquake” and it would be likely to collapse causing injury or death to persons in the building or to persons on any other property, or damage other property.

The Building Regulations (2005) define a moderate earthquake as an earthquake that would generate shaking at the site of the building that is one-third as strong as the earthquake shaking that would be used to design an equivalent new building, but of the same duration.

2.1.2 Section 124 – Power of Territorial Authorities

If the building is found to be earthquake prone, the territorial authority has the power under section 124 of the Building Act to require strengthening work to be carried out, or to close the building to prevent occupancy.

2.1.3 Section 131 – Earthquake Prone Building Policy

Section 131 of the Building act requires all territorial authorities to adopt a policy on dangerous, earthquake prone, and insanitary buildings within its district.

2.2 Invercargill City Council Policy

The Invercargill City Council (ICC) adopted their policy on EPB on 8th November 2005. Under this policy ICC undertook the following actions:

2.2.1 Identifying Earthquake Prone Buildings

Within 24 months of the date of this Policy being adopted (8th November 2005), the Council will identify and classify as “Potentially Earthquake Prone” from its records as far as practicable which pre-1930s non-residential buildings could be earthquake prone. In so doing it will take into account work which has been carried out over the life of that building.

2.2.2 Taking Action on Earthquake Prone Buildings

Where a building is classified as potentially earthquake prone, Council will:

i. Advise the owner in writing and make an appropriate note on the Council’s property file
ii. Encourage the owner to address the hazard, acting on the advice of an appropriately qualified structural engineer
iii. Note on any application for a PIM or a LIM that the building has been identified as potentially earthquake prone
iv. Consider any appeal from the owner as to the classification

Council will follow the procedure set out in Sections 124-130 of the Building Act 2004 where an earthquake prone building is also dangerous or insanitary.

The Invercargill City Council concludes that the appropriate time to require structural upgrade of a building for enhanced earthquake performance will normally be at the time of reclassification of the building, which is defined under the change of use criteria of the Building Act 2004 (Section 115). The owner is recommended to upgrade the building as far as reasonably practicable to meet 100% of current performance standards.

2.3 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.

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

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

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

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

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

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

3 Earthquake Resistance Standards

For this assessment, the building's earthquake resistance is compared with the current New Zealand Building Code requirements for a new building constructed on the site. This is expressed as a percentage of new building standard (%NBS). The loadings are in accordance with the current earthquake loading standard NZS1170.5 [1].

A generally accepted classification of earthquake risk for existing buildings in terms of %NBS that has been proposed by the NZSEE 2006 [2] is presented in Figure 1 below.

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

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).

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 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 Territorial Authority guidelines.

3.1.2 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.

3.1.3 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 Territorial Authorities of potentially dangerous buildings; this would include earthquake prone buildings.

4 Background Information

4.1 Building Description

The Southland Museum and Art Gallery, located on Gala Street at Invercargill, consists of three independent structures and the remains of a fourth. These are the original building, which was constructed in 1940; the addition constructed in 1960 to the northwest of the original building, and another addition built in 1988 to the east of the 1960 building. This final addition included the construction of a pyramid that enclosed all of the buildings. In this report, the buildings are referred to as the original museum (or the 1940 museum), the 1960 addition, and the 1988 addition, respectively. Additionally, a one-storey building, located to the east of the 1960 building was demolished prior to construction of the 1988 addition. Some of the columns and walls of this building remain in place on the ground level inside the 1988 addition and these remains are referred to as the 1959 addition. Appendix 1 shows photographs of the buildings and surrounding area.

The original museum is a two-storey brick-clad concrete structure. The lateral force resisting system consists of concrete shear walls at each storey in each direction and is broken up by windows along each side. There are interior concrete columns in the first storey supporting the first floor cast-in-place beam and slab system. Piers within the perimeter concrete walls support these beams on the perimeter. This frame system will also contribute to the lateral resistance of the first story. There are no interior columns in the second storey, and the roof structure consists of timber trusses that typically span between the north and south walls except at the centre portion of the building where the trusses spans east west. The north-south spanning trusses are deepest at the north end and taper to a very small depth at the south end. This results in tall cantilevered parapet on the south side. The south, east and west sides of the museum are clad in 115 or 230mm brick. This summary is based on a limited set of structural and architectural drawings and visual observations at the site.

No drawings were provided for the 1960 addition and the structure of this building was determined through field observations. This building is two stories. The roof consists of plywood over timber joists supported by steel portal frames. The portal frames have wide flange beams and tube steel columns. The 1st floor consists of precast double-tees spanning in the east-west direction supported by reinforced concrete beams and columns.

The 1988 addition consists of a two-storey concrete frame building and a steel framed pyramid that forms the exterior of the museum. The 1st floor of the concrete frame building, located to the east of the 1960 addition and north of the original museum, consists of a Stalhton system with concrete topping supported by concrete beams and concrete columns. Additionally, concrete beams and columns were constructed on the outside of the 1960 addition and the original building. The 2nd floor of the 1988 addition spans over the roofs of the original building and the 1960 addition and is a combination of the Stalhton system and precast double-tees supported by concrete beams. Some of the columns that support this floor penetrate the roof and 1st floor of the 1960s building. The pyramid roof structure, which consists of structural steel tube and I-section framing clad with insulated panels enclose all three buildings and forms the exterior skin of the museum. The lateral force resisting system for the 1988 addition is a concrete moment frame system in each direction.

The buildings are typically separated by approximately 50mm in most locations except at some locations around the original museum stairs at the interface of the 1988 building, where there is little to no separation.

Figure 4.1 below shows the location of the site within the City of Invercargill and Figure 4.2 shows the location of the Southland Museum. Figures 4.3 to 4.6 consist of the floor plans and sections of the building showing various structures within the museum.

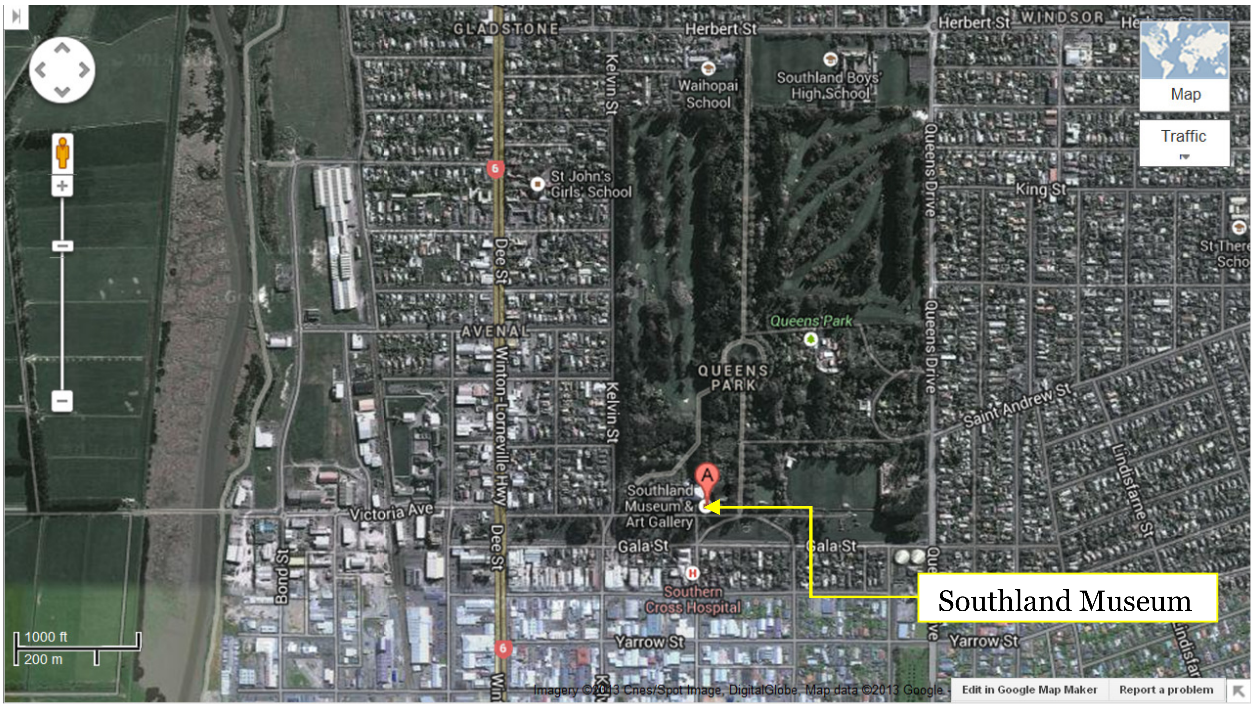

Figure 4.1 – Site Map

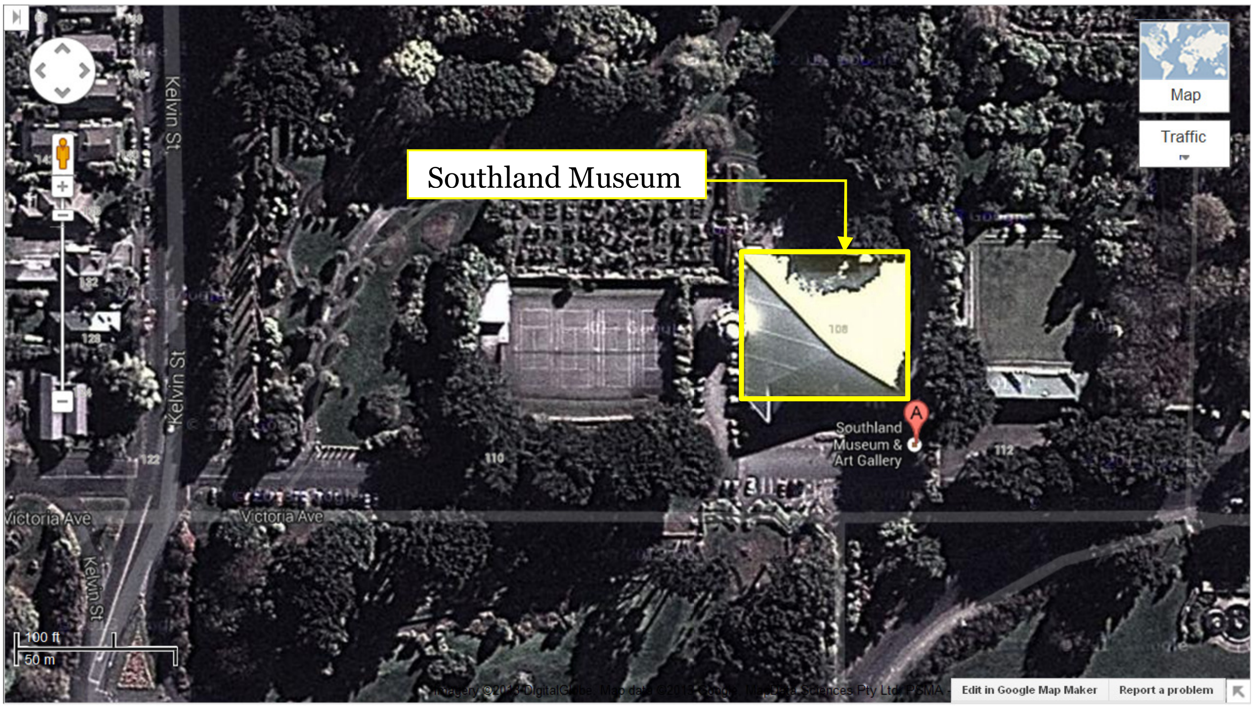

Figure 4.2 –Building Footprint

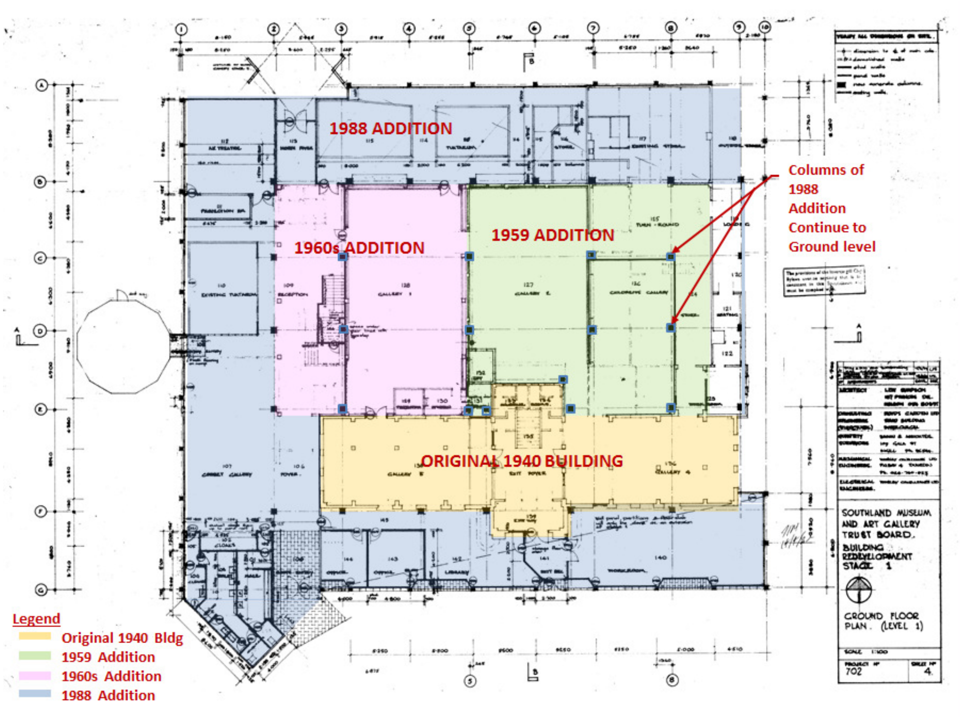

Figure 2.3 – Ground Floor Plan

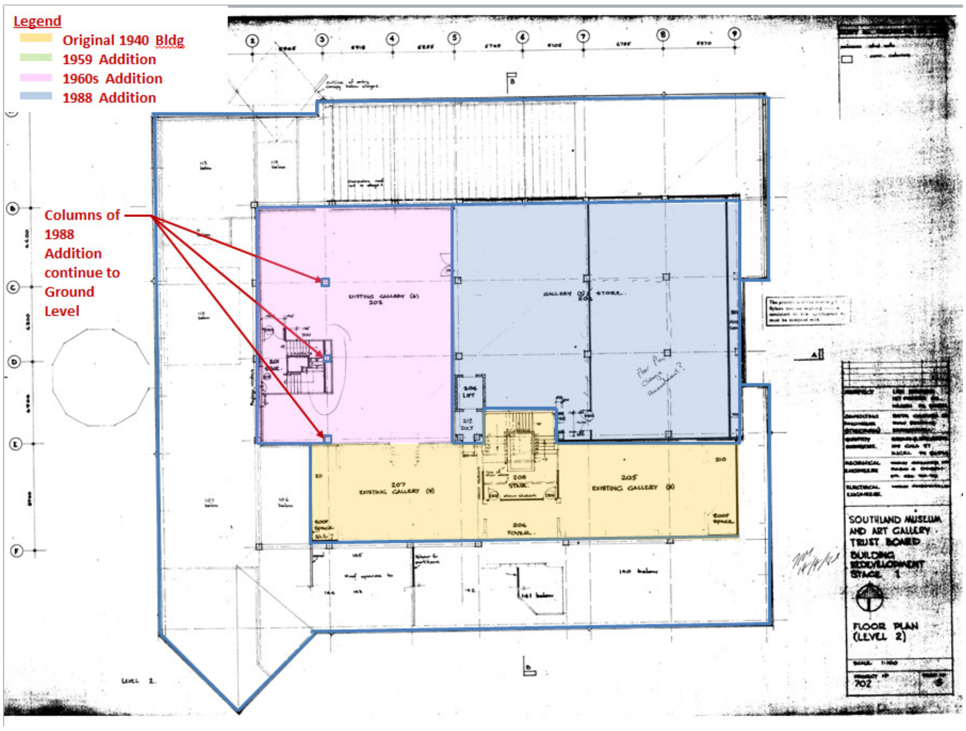

Figure 4.4 – 1st Floor Plan

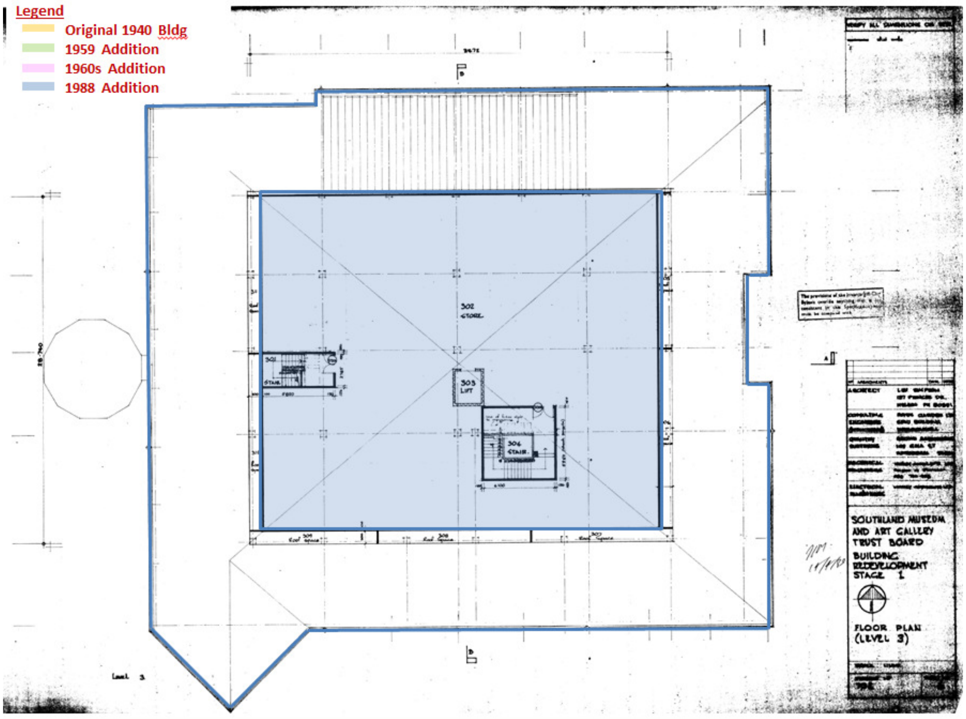

Figure 4.5 – 2nd Floor Plan

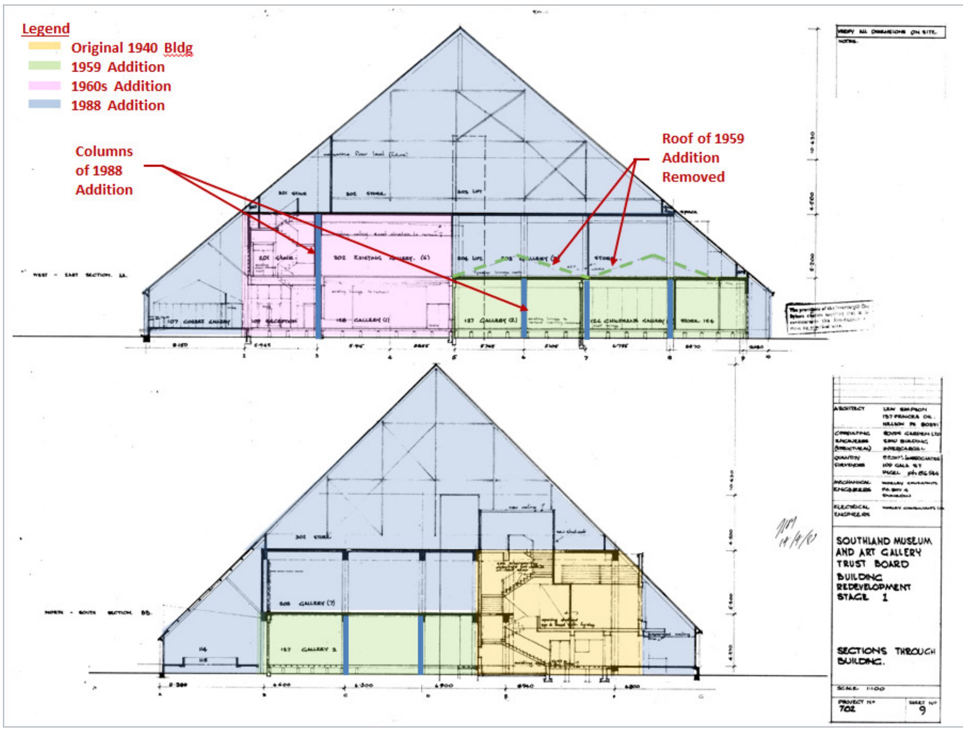

Figure 4.6 – Building Sections

4.2 Original Documentation

Copies of the following construction drawings were provided to Opus and included:

- Southland Museum & Art Gallery Trust Board Building Redevelopment Stage 1 drawings, dated December 1988 (set of 14 structural drawings consisting of structural plans and moment frame elevations, most of the details are missing. Also includes 10 architectural drawings).
- Plan of Proposed Southland Centennial Memorial Museum, dated 21/6/40 (this is one drawing showing the structure of the ground floor, south elevation and two building sections).
- Proposed New Art Gallery Addition to Southland Centennial Memorial Museum, dated 15/5/59 (one drawing consisting of a ground floor plan and six elevations and sections).
- Plans of Southland Centennial Memorial Museum dated sometime in the 1940s but date is too worn to read clearly (two drawings consisting of basic architectural plans, sections and elevations with very little information noted).

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

Structural drawings have not been located for the 1960 addition and no design calculations of any of the buildings were obtained.

5 Survey

5.1 Inspections

On June 27 and 28, 2013, Paul Cordova and Brenton Easson of Opus International Consultants visited the museum site to investigate the existing structure and document potential deficiencies.

Roslyn Clarke of Opus International Consultants visited the site on multiple occasions to perform selected intrusive investigation to determine the makeup of existing structural elements.

6 General Observations

The building appears to be in reasonably good condition. However, many cracks were observed at the topping slab at the 2nd floor of the 1988 addition (see photograph in Appendix A). In addition, various elements were partially removed during the various additions and alterations.

Due to architectural finishes, some of the primary lateral force elements were not visible, and so an assessment of these areas could not be made. Selective destructive openings were made to verify components of the lateral force resisting system. It should be noted that the site inspections revealed some as-built conditions did not always match the details shown on the available structural drawings.

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, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [5]. ASCE/SEI 41-06 [6] methodology was also implemented as a guide in determining element ductility's.

7.1 Critical Structural Weaknesses

The term Critical Structural Weakness (CSW) refers to a component or structural feature of a building that could contribute to increased levels of damage or cause premature collapse of a building. An initial desktop review of the available documentation was performed to pre-screen for potential CSW's for closer inspection during the quantitative phase of the assessment. The following potential CSW's were identified during this review and following the quantitative assessment:

<u>Original 1940 building</u>

a) Flexural failure of exterior concrete moment frame beams at the first story;

b) Lack of roof diaphragm and connection to concrete walls;

c) Pounding with adjacent buildings;

d) Restraint of north stairwell within 1988 structure and lack of visible gap between the two structures in this location;

e) Out-of-plane wall anchorage of concrete wall at roof level;

f) Out-of-plane support of brick in-fill wall;

g) Out-of-plane support of exterior brick veneer;

<u>1959 Addition</u>

a) Inadequate out-of-plane support of ground floor concrete walls along grid lines B, 5, and 7;

<u>1960 Addition</u>

a) Pounding with adjacent buildings: the 1960s addition is surrounded by adjacent buildings, 1988 addition to the north, east, and west and the original 1940 building towards the south. While there are gaps (approximately 50mm) at some locations between the buildings, it is unclear if this gap is carried throughout all interfaces between the buildings. Additionally, the columns of the 1988 building run through the roof and 1st floor slab of the building. Lateral displacement of the building will load these columns at mid height.

b) Torsional response at ground floor: Along line B, the concrete frame is infilled with a masonry wall. Along line E, although there are some masonry infills, the lengths are short and the infills have large openings, thus are ineffective in providing resistance. The difference in stiffness along these two sides results in a torsional response in the east-west direction loading.

<u>1988 Addition</u>

a) Pounding with adjacent buildings

b) Shear failure of short-span concrete beam at second floor

c) Stiffness incompatibility between the steel structure of the pyramid and the concrete frame. Steel diagonals of the pyramid between GL8 and 9 connect to the 1st and 2nd floor of the concrete frame and act as braces between floors. The connections of these members are not adequate to resist the imposed forces

d) Failure of bolts connecting the pyramid roof diagonals to the perimeter concrete columns

e) Lack of bracing at the south side of the mezzanine to the south of the 1940 museum.

7.2 Quantitative Assessment Methodology

The probable seismic performance of the building has been assessed in accordance with the recommendations of the NZSEE publication "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" [2]. The following sections in particular have been used in this assessment:

– Section 4 "*Detailed Assessment - General Issues*";

– Section 7 "*Detailed Assessment of Reinforced Concrete Structures*";

– Section 10 "*Detailed Assessment of Unreinforced Masonry Buildings*".

The probable Earthquake loading for this building has been calculated from NZS 1170 - Structural design actions, Part 5 Earthquake Action [1]. The building has been classed as Importance Level 3 (IL3) in accordance with AS/NZS1170.0.

The building has been assessed using a force based approach by applying the forces that may be expected to be applied to the building by the design earthquake. Calculations were performed on beam, column, wall and connection elements of the building in order to assess their likely performance in an earthquake. This performance has been measured as a %NBS (New Building Standard), that is, as a percentage of the capacity that would be required for the design of an equivalent new building on this site.

7.3 Limitations and Assumptions in Results

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

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 levels of 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.

7.4 Quantitative Assessment

A summary of the structural performance of the building is shown in the following table. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements. This will be considered further when developing the strengthening options.

Table 7 .1: Summary of Seismic Performance

Structural Element/System	Failure mode, or description of limiting criteria based on capacity of critical element.	Critical Structural Weakness	Assessed Ductility Factor, μ	% NBS based on calculated capacity
Original 1940 Building:				
In-plane action of the concrete walls in the North-South direction at the upper storey	Flexural failure of concrete walls and piers due to inadequate vertical reinforcement.	No	2.0	89%
In-plane action of the concrete walls in the North-South direction at the ground storey	Flexural failure of walls at the NE, SW and SE corners	No	2.0	67%
In-plane action of the concrete walls in the East-West direction at the ground storey	Flexural and shear capacity in this direction not an issue.	No	2.0	>100%
Concrete moment frame columns in the North-South direction at the ground storey	Flexural failure of interior columns	No	2.0	52%
Concrete moment frame beams in the North-South direction at the first floor	Flexural failure of beams at exterior columns	Yes	2.0	23%
1st floor concrete collector beam in the North-South direction	Tensile failure of concrete and yielding of steel reinforcement in the beams connecting the concrete shear walls at the north and south extensions in the middle of the building. This may trigger a diaphragm	Yes	2.0	50%

Structural Element/System	Failure mode, or description of limiting criteria based on capacity of critical element.	Critical Structural Weakness	Assessed Ductility Factor, μ	% NBS based on calculated capacity
	shear failure at these two north walls and cause a redistribution of forces to other concrete walls and moment frames which will reduce the %NBS of the lateral system in the North-South direction to 29%.			
Roof diaphragm and connections	Diaphragm has been removed in a number of locations and no connection exists at the roof to connect diaphragm and roof structure to concrete walls	Yes	-	<33%
Out of plane support of exterior concrete walls	No connection between the roof trusses and the concrete wall requiring the concrete to cantilever the entire upper storey from the first floor diaphragm. Failure of this wall will result in loss of gravity load carrying capacity and given the lack of connection to roof trusses, collapse of the roof can be expected.	Yes	2.0	24%
Out of plane support of exterior brick veneer	No ties connecting the brick veneer back to the concrete walls	Yes	1.0	<33%
Out of plane support of infill brick wall bearing on top of the first floor at the north side	Out-of-plane failure causing potential collapse of wall and falling hazard	Yes	1.0	20%
Restraint of the north stairwell by adjacent building	The restraint of the stair well is caused by it being bounded by the 1988 building and the lack of gap between the two structures. The stairs are not adequately braced and may pose a local collapse hazard.	Yes	1.0	<33%
1959 Addition:				
Out of plane support of concrete walls	During the 1988 addition, concrete walls of the 1959 addition were left in place even though the roof was removed. Nominal braces were added to brace the top of the concrete walls for out-of-plane loading. The connections of these braces are unknown. We anticipate	Yes	NA	<34%

Structural Element/System	Failure mode, or description of limiting criteria based on capacity of critical element.	Critical Structural Weakness	Assessed Ductility Factor, μ	% NBS based on calculated capacity
	that the strength is inadequate.			
1960s Addition:				
North-south lateral load resistance at 1st storey – combination of timber framed walls sheathed with GIB and steel portal frame along lines 2 and 5	The timber walls sheathed with GIB will initially attract load given that they are stiffer than the steel frame. The strength is governed by the shear capacity of these walls. Once the GIB wall softens, the steel frame acts as the lateral load resisting element. The strength of the frame is governed by the columns in flexure.	No	2.0	50%
East-west lateral load resistance at 1st storey – steel portal frames.	Lateral load resisting system at consists of steel portal frames with wide flange beams and tube steel columns. The capacity is governed by the beam to column connections. Some connections consist of fillet welds between flanges of the beam to face of tube steel column, which are non-ductile. However, there is an alternative load path at this level that consists of GIB-lined timber walls along grids B and E. Therefore, it is unlikely this will pose a collapse risk to the structure.	No	1.0	34%
North-south lateral load resistance at ground floor – concrete frame along grid lines 2, 3 and 5.	Only limited information was available on beam and column reinforcement and details, therefore we considered the lower-bound estimated strengths of column and beam. The frames are anticipated to be governed by the flexural strengths of the beams.	Potentially	2.0	25 - 40%
East-west lateral load resistance at ground floor – concrete frame infilled with partially grouted concrete masonry along line B.	Failure mode is expected to be in sliding shear between infill masonry panel and the beam above and/or shear in columns due to effect of the infill panel.	Yes	1.0	<30%

Structural Element/System	Failure mode, or description of limiting criteria based on capacity of critical element.	Critical Structural Weakness	Assessed Ductility Factor, μ	% NBS based on calculated capacity
East-west lateral load resistance at ground floor – concrete frame along line E	Only a short length of masonry infill exists along this line. And the infill has various large openings and therefore, the infill is ineffective in resisting lateral load. Limited information was available on beam and column reinforcement and details, therefore we considered the lower-bound estimated strengths of column and beam. The frames are anticipated to be governed by the flexural strengths of the beams.	Yes	2.0	20 - 30%
Horizontal roof diaphragm – plywood sheathing	Roof diaphragm consists of unblocked plywood sheathing. The failure mode is in shear of the diaphragm.	No	2.0	60%
Horizontal diaphragm at 1st storey –double-T with 50mm topping slab	Based on our field investigation, the double-T has approximately 50mm topping slab. The connection between the topping and the concrete frame below is unknown. For our assessment, we assumed that some positive connection exists to connect the topping to the beams below.	No	1.0	50%
Pounding with adjacent structure.	Calculated lateral deflection of the building at the 1st floor is 100mm whereas the gap between buildings is only 50mm; therefore pounding is expected to occur. Where the floor levels align (between 1960 and 1988), localized damage to the slab/ beam is expected to occur. Where the floor levels are not aligned (between 1960s addition and original building), pounding between buildings will damage the ground floor columns.	Yes	NA	NA
1988 Addition:				
North-South lateral load resistance at ground storey	Flexural failure of beam due to inadequate amount of bottom steel reinforcement at column B-9.	No	2.0	36%

Structural Element/System	Failure mode, or description of limiting criteria based on capacity of critical element.	Critical Structural Weakness	Assessed Ductility Factor, μ	% NBS based on calculated capacity
East-West lateral load resistance at ground storey	Flexural failure of beam due to inadequate amount of bottom steel reinforcement at column E-5.	No	2.0	26%
North-South and East-West lateral load resistance at ground storey	Column flexural failure due to inadequate amount of longitudinal reinforcement (notwithstanding the columns discussed below as a result of impact from adjacent building).	No	2.0	63%
Impact with adjacent 1960s addition.	Columns along GL 3 are susceptible to impact from the 1960s building and will most likely be subjected to a large part of this structure's seismic load.	No	2.0	46%
North-South lateral load resistance at upper storey	Flexural failure of beam due to inadequate amount of bottom steel reinforcement at column B-2.	No	2.0	46%
East-West lateral resistance at upper storey	Shear failure of beam due to large amount of flexural reinforcement and very little shear reinforcement in beam along GL E spanning between GL 5 and GL 5.3.	Yes	2.0	27%
East-West lateral load resistance at upper storey	Flexural failure of column at GL E-5.3 due to inadequate amount of longitudinal reinforcement.	No	2.0	37%
North-South and East-West lateral load resistance at upper storey	Column flexural failure due to inadequate amount of longitudinal reinforcement.	No	2.0	57%
Beam-column joint shear strength at second floor	Failure of joint in shear.	No	1.0	77%
Beam-column joint shear strength at first floor	Failure of joint in shear	No	1.0	79%
Column shear strength	Columns along GL B between GLs 3 and 7 have inadequate stirrups to resist flexural hinges forming at each end. This is a brittle type failure.	No	1.0	73%

Structural Element/System	Failure mode, or description of limiting criteria based on capacity of critical element.	Critical Structural Weakness	Assessed Ductility Factor, μ	% NBS based on calculated capacity
First floor diaphragm shear	Concrete shear failure at GL 2 frame	No	1.0	79%
First and second floor diaphragm shear around masonry elevator core	Concrete shear failure around the elevator core due to the relatively high stiffness of the walls. This will result in load redistribution to the concrete moment frames which have been assumed to resist the entirety of the building inertia in this assessment. This will cause localized damage of the slab around the core.	No	NA	NA
Connection of the pyramid roof structural steel to the concrete structure between GLs 8 and 9	Five steel members are connected to the second and first floor between GLs 8 and 9 and provide bracing between the two levels that will attract a large amount of load. The connections are inadequate to resist this and will fail during an event. This may cause partial collapse of the pyramid roof structure.	Yes	1.0	<10%
Connection of the four corner pyramid main diagonals to the concrete structure at 2nd floor	Shear failure of four bolts connecting these struts to the concrete structure can result in roof instability and collapse.	No	1.0	46%
Connection of pyramid main diagonals to the top of the corner columns (A-1, A-9, G-1, and G-9).	Combined shear and tensile failure of bolted connection to column. This is a critical structural weakness and may cause the partial collapse of pyramid roof structure at the perimeter.	Yes	2.0	<33%
Splice connections of four corner pyramid main diagonals at the second floor level.	Combined shear and tensile failure of bolted connection between the UB diagonals above the second floor and the UB diagonals below the second floor.	No	2.0	50%
Pounding with adjacent structures	As discussed above for the original 1940 museum.	Yes	NA	NA
Inter-storey drift of the second storey frames in the transverse direction	The displacement between the second floor and first floor is 225 mm which is 3% of the storey height.	No	2	85%

8 Summary of Geotechnical Appraisal

8.1 General

A geotechnical appraisal was not part of this assessment. Soil Class D was used based on the report "Amplified ground shaking and liquefaction susceptibility, Invercargill City" by Glassey and Heron.

9 Discussion

Original 1940 Museum

The Original museum built in 1940 is considered Earthquake Prone due to a number of reasons. There is a lack of connection between the roof diaphragm and truss structure to the concrete walls. Therefore, the concrete walls must cantilever above the second storey floor and lack the out-of-plane flexural capacity to resist the inertial loads in this direction. The brick veneer and infill also does not have the strength to resist out-of-plane inertial loading. No ties between the veneer and the structural concrete were found during the investigation.

The main lateral force resisting system at the first story is a combination of concrete shear walls and moment frames and these members will be loaded in proportion to their stiffness. The two north walls that flank the stairs in the middle of the building are relatively stiff and will attract a large percentage of the North-South inertial force. The beams spanning between these walls and the south walls that are parallel to it will act as collectors and transfer the diaphragm shear to these walls. The steel reinforcement shown on the drawings for these collectors is not adequate to transfer the tensile load to the walls. Failure of these collectors will result in a diaphragm shear failure at the north walls and will result in a redistribution of load to the other walls and frames. This will increases the demands on these elements resulting in a %NBS of less than 34%.

Finally, the north stairs are located in an appendage that is restrained by the 1988 building with little to no gap between the two structures. During shaking in the east-west direction, these stairs will initially attract a relatively large amount of load which they do not have the capacity to resist and will most likely result in failure of the connections and potential collapse.

1960 Addition

The 1960 addition of the museum is considered Earthquake Prone. The capacity of the building is limited by the ground floor concrete frames in the east-west direction. Along line B, the frame is partially in-filled with concrete masonry blocks. The infill block wall does not have adequate shear strength to resist the lateral load. Additionally, the infill wall will interact with the surrounding concrete frame columns and will result in potential shear failure of the columns. Along line E, some infill masonry walls exists between the concrete frame members. These walls are relatively short and punched by various openings thus the remaining portions are not effective in resisting lateral load. The concrete frame along this line is expected to have limited strength to resist lateral load.

The capacity of the concrete frames along the north-south direction at the ground floor are expected to be 20 - 40% NBS. The uncertainty in the reported strength is due to the lack of information and drawings that show details and reinforcement of the members. The foundations of this building are also not known. The available drawings imply the foundations to be spread footings interconnected by some nominal ground beams. The ability of the foundation system to prevent rotation at the column base is not known. Therefore, we conservatively bounded our solutions using both "fix-base" as well as "pin-base" assumptions.

Additionally, the concrete frame at the ground floor level is flexible, and pounding with the adjacent structures is anticipated. Since the 1st floor levels are not aligned between the 1960 addition and the original 1940 building, pounding will result in damage at ground floor columns.

<u>1988 Addition</u>

The 1988 addition of the museum is considered Earthquake Prone. This is mainly due to the connections between the pyramid roof structural steel and the concrete structure of the 1988 addition and the perimeter concrete support columns. Five steel roof members are connected to the concrete structure at the first and second floors and between GLs 8 and 9 and will attract a large amount of load. These members do not have the capacity to resist this load and will buckle in compression. When loaded in the opposite direction, the connections to the concrete structure are not strong enough to resist the applied tension and shear on these members. The remaining members of the perimeter steel roof beyond the 1988 addition are connected to perimeter concrete columns through a baseplate and two bolts. The four main diagonals will act as struts and transfer a relatively large amount of seismic load to the corner columns. These connections do not have the capacity to resist this load. Likewise, the rest of the connections to the perimeter concrete columns may have similar issues although it is very difficult to predict precisely given the unknown details of the base connections of these columns, the steel reinforcement, and their relative flexibility.

Additionally, the concrete perimeter columns, particularly at the corners, do not have the flexural capacity to resist the thrust from the roof members, although their strength is predicted to be larger than the connections so it is expected that the connections will fail first, in which case the columns will be not be subjected to sufficient loading to cause them to fail. But the above conditions may cause collapse of the pyramid roof and therefore deemed a Critical Structural Weakness.

There are three concrete frame beams with %NBS less than 34% due to inadequate flexural and/or shear reinforcement at the connection regions. One column has a capacity less than 34% but the majority of the columns have capacities above 67%. The elements with low %NBS will cause localized damage but these do not present a total collapse hazard. In order to present a collapse hazard, the majority of the beams and/or columns must fail. One way of demonstrating this potential is through a mechanism analysis which has been performed on the first story frames in each direction. This analysis results in 68%NBS and 90%NBS in the North-South and East-West directions, respectively; demonstrating that the concrete frame structure is not likely to collapse but may exhibit localized areas of heavy damage.

Pounding between this structure and the adjacent structures is also a concern and will potentially cause large amounts of distress in the elements adjacent to these structures.

10 Remedial Options

Original 1940s Museum

The Original 1940 museum is considered Earthquake Prone and strengthening to a minimum of 34%NBS is recommended. Strengthening to 67%NBS or greater would require a similar amount of labour and detailing but more material. If strengthening is chosen as the option, the following items must be addressed regardless of the level:

a) Provide connection between the brick veneer to the concrete wall at all locations.

b) Provide connections between the roof framing and the concrete walls to provide out-of-plane support to the walls and the ability to transfer diaphragm shear to the concrete walls.

c) Provide out-of-plane support of brick in-fills.

d) Strengthen the collector beams and their connections to the interior north and south walls at the first floor. This can be achieved by an overlay of Fiber Reinforced Polymer (FRP).

e) Strengthen the concrete walls via shotcrete or provide a new bracing system (if item ‘d’ above is not addressed).

f) The north stair tower should be isolated from the 1988 structure to provide adequate separation that would allow the movements expected during a seismic event. This would most likely involve demolishing this part of the building and rebuilding with an appropriate gap between the two structures.

1960 Building

The 1960 addition is considered Earthquake Prone and strengthening to a minimum of 34%NBS is recommended. To achieve an expected strength of 34%NBS, or greater, the following items must be addressed:

a) Increase the lateral load resisting capacity at the ground floor along grid B. This can be achieved by removing existing masonry infill and replacing with reinforced concrete walls or reinforced fully grouted masonry walls that are properly doweled into the concrete frame.

b) Install additional lateral load resisting element along grid E. This can be achieved by infilling portions of the concrete frame with reinforced concrete walls or fully grouted masonry walls that are properly doweled into the concrete frame.

c) Install additional concrete walls or bracing in the north-south direction at the ground floor to reduce drift and potential for pounding.

Should strengthening be required to achieve an expected strength of 67%NBS, or greater, in addition to the items identified for the 34%NBS scheme, the following items must also be addressed:

a) Strengthen the 1st floor diaphragm. This can be achieved by overlay of FRP on existing double-Ts or reducing the diaphragm loads by constructing additional interior shear walls.

b) Strengthen existing roof diaphragm by addition of rod bracing.

c) Reinforce steel portal frame connections. Alternatively, provide new lateral system, such as steel braces or plywood sheathing, within plane of the existing gib walls.

<u>1988 Building</u>

The 1988 addition is considered Earthquake Prone and strengthening to a minimum of 34%NBS is recommended. To achieve an expected strength of 34%NBS, or greater, the following items must be addressed:

a) Provide a sliding connection for the pyramid steel members at the first floor between GLs 8 and 9 to remove their ability to attract and resist lateral loading.

b) Provide sliding connections at the top of the perimeter concrete columns supporting the pyramid roof structure.

c) Strengthen the connections of the pyramid roof steel to the second floor concrete structure. This can be accomplished by attaching new steel components to the roof steel and installing adhesive anchors in the concrete structure.

d) Strengthen concrete beams to increase their flexural and shear capacity at the joints.

e) Provide bracing at the southern perimeter of the mezzanine. This will also require new foundations beneath this bracing.

Should strengthening be required to achieve an expected strength of 67%NBS, or greater, in addition to the items identified for the 34%NBS scheme the following items must also be addressed:

a) If possible, provide a larger gap between the columns along line 3 and the 1960s building to prevent the 1960s building from impacting these columns. This most likely will not be required if walls are added to the 1960s building.

b) Increased strengthening of the pyramid connections beyond that required for 34%NBS.

c) Strengthen concrete beams and columns to increase their flexural and shear capacity. This can be accomplished through FRP or jacketing.

We understand that an expansion of the museum is currently being planned. Based on preliminary drawings of the addition, a 3rd level is proposed above the existing 2nd floor of the 1988 addition. This will likely increase the mass of the building and thus increase the seismic demands on the existing structural elements. The expansion will likely trigger a requirement for a seismic upgrade of the building to a minimum of 67%NBS. We anticipate the strengthening scheme to accommodate the addition will be similar to the 67% NBS scheme but the amount of strengthening will be more.

11 Conclusions

a) The original 1940 museum is considered to be Earthquake Prone in accordance with the Building Act 2004. This is due to a number of items which include inadequate shear and flexural strength of the concrete diaphragm and walls for both in and out-of-plane loading. No connections were found between the brick veneer and the concrete walls. Additionally, the north stair tower is restrained and does not have adequate connections or resistance to resist the lateral load that will be imparted to it.

b) The 1960 addition is considered to be Earthquake Prone in accordance with the Building Act 2004. This is primarily due to limited lateral load resistance in the east-west direction at the ground floor. Based on currently available information, the concrete frames in the north-south direction at the ground floor are of marginal strength. Additional investigations may show that the strengths of these frames are higher than that assumed in our analysis. However these frames are relatively flexible, thus in a seismic event, the building will likely pound against the adjacent 1940s building to the south and result in damage to the columns.

c) The 1988 building is considered Earthquake Prone due to the inadequate connections of the pyramid roof structure and lack of bracing at the southern mezzanine.

d) It is recommended that targeted strengthening be performed to increase the seismic performance to a minimum of 34%NBS.

12 Comments on Planned Expansion

ICC requested that Opus review the preliminary expansions plans and make high-level comments regarding its effect on the existing structure. Architectural drawings describing "Sketch Scheme option 6 – June 2013" were provided and were reviewed. The following items are noted and discussed:

a.) A new building is shown to the west of the existing structures and the first floors of each are connected by a bridge.

   1. The new building should be seismically isolated from the existing structures to allow relative movement between the two and to prevent one from loading the other. This will require a joint at the roof between them and also a joint at the support for the bridge.

b.) The existing pyramid roof structure will be cut back at the interface over the new bridge and the ground level which appears to be open to above up to the new roof.

   1. This does not pose any major issues.

c.) A new floor level will be added above the existing storage (to be gallery).

   1. This will increase the mass of the building and the seismic demands on the existing structure, including the foundations. The existing structure requires strengthening in various locations and the additional floor will increase the required strengthening. The foundations will also need to be checked for the additional load and will possibly require strengthening.

d.) Usage of the first and second floors of the existing museum has changed. This includes rearranging the storage area on the first floor and adding a classroom, and changing the usage on the second floor from storage to gallery.

1. It appears that the area of storage on the first floor does not change considerably from the area as shown on existing drawings and therefore should not be a major issue; however, the existing floor should be checked to ensure it can resist the new configuration. The code-mandated loading for a gallery is slightly less than that for storage and therefore, changing from storage to gallery theoretically decreases the loading on this floor.

e.) Adding, removing, and shifting stair and lift locations in the existing museum.

1. This will require the removal of existing floor structure and the addition of new floor structure but should not pose any major issues. The floor system surrounding the new lift should be allowed to slide relative to the lift walls to prevent transfer of any lateral load to the lift walls.

f.) Possible demolition of the original museum.

1. If the original museum is demolished and a new structure is built in its place, this will require either: 1.) its own lateral system and seismic isolation between the existing structure or 2.) attachment to the existing structure and the consideration of this additional mass in the strengthening scheme for the existing structure.

13 Recommendations

A staged approach is recommended as follows in order to understand and manage the economic impact of any proposed remedial actions:

a) An outline scheme for structural strengthening – with a view to achieving a minimum level of 34%NBS, or to a recommended level of 67%NBS, should be further developed with sufficient information so that costing can be put to the proposed works. This will expand upon the remedial options discussed in Section 9.

b) Review planned expansion in more depth (including any updated plans) to study how it may impact structural strengthening scheme.

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

d) Carry out detailed design of a selected scheme for the strengthening of the structure.

14 Limitations

a) This report is based on an inspection of the structure of the buildings and available structural documentation.

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 ICC to assist with assessing the necessity of remedial works required for the Southland Museum and Art Gallery. It is not intended for any other party or purpose.

15 References

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

[2] NZSEE: 2006, Assessment and improvement of the structural performance of buildings in earthquakes, New Zealand Society for Earthquake Engineering, with Corrigendum No.2, 15 July 2012.

[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 7, May 2012.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Non-residential buildings, Part 3 Technical Guidance*, Draft Prepared by the Engineering Advisory Group, Rev 3, May 2012.

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

[6] American Society of Civil Engineers (ASCE), *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-06), 2007, Reston, Virginia.

Appendix A

Photographs

Southland Museum		
No. / Location	**Item description**	**Photo**
General Photos		
View from South	South elevation	
View from North	North elevation	

View from Southwest	Southwest corner	
View from Southeast	East elevation	

Interior View from South	Original museum entrance	
Southwest corner of interior of 1988 addition	West brick veneer of original museum	

Interior View from Southwest	1988 concrete frame on the exterior of the 1960 concrete frame and original museum. Diagonal bracing for pyramid roof structure also shown.	
West of grid 2.	Gap between 1988 structure and 1960 structure along grid 2.	

Original Building	Interior view showing first storey concrete beams and columns.	
View from South at ground level	Mezzanine structure at south side of original building. Original brick veneer seen in the background.	

View from southeast corner	Column at intersection of grids F and 2 of 1988 addition. Precast double-T structure of 1988 addition can be seen along with pyramid insulated panel roof.	

Pyramid roof	Structural steel and roofing panels of pyramid roof.	

1988 Addition	Cracking in Level 3 topping slab	
Original Museum	Upper level ceiling joists bearing on concrete wall beam	

Original Museum	Roof truss bottom chord supported in pocket in concrete wall	
Original Museum	Roof truss of original museum. Stahlton slab system of 1988 addition shown spanning above original roof	

Original Museum	Roof connection to concrete wall.	
Pyramid Roof Structure	Main corner diagonal (member "B") connection to third floor concrete structure at B-8.	

Pyramid Roof Structure	Member "A" pyramid connection at GL 9 to third floor concrete structure	
Pyramid Roof Structure	Connection of member "G" to concrete column at A-5.	

1988 addition and pyramid	Connection of SHS pyramid strut to concrete column	
1988 addition and pyramid	Connection of SHS pyramid strut to concrete beam	

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

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