Wharenui Recreation Centre Sports Hall

BU 0904-001 EQ2

Detailed Engineering Evaluation

Quantitative Assessment Report

Christchurch City Council

Wharenui Recreation Centre Sports Hall Detailed Engineering Evaluation Quantitative Report

73 Elizabeth Street
Christchurch City Council

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Contents

Executive Summary

Christchurch City Council has appointed Opus International Consultants to carry out a detailed seismic assessment of their Wharenui Recreation Centre Sports Hall. The purpose of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards.

The findings of the assessment are:

1. As a result of a number of deficiencies the building has a seismic capacity less than 34%NBS, with a minimum rating of 11%NBS. It is therefore classified as earthquake prone in terms of the Building Act.
2. It should be feasible to improve the seismic capacity to at least 67%NBS by undertaking remedial works.
3. It is recommended that the building should not be occupied given its structural weaknesses.

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the Wharenui Recreation Centre, located at 73 Elizabeth Street, Christchurch following the M6.3 Christchurch earthquake on 22 February 2011.

A stage 1 qualitative assessment[1] completed in November 2011 concluded that the building contained a number of potentially critical structural weaknesses that affects its seismic capacity and recommended that the following work should be undertaken:

1. An inspection survey to determine the damage sustained from the earthquakes and to review the damage to the external panels.
2. Further investigation to assess and identify suitable load paths, if any, to brace the mezzanine floor.
3. A survey of the eaves to determine if there are tie beams between frames in the north-south direction.
4. A quantitative assessment, following all inspection and investigation work, of the building to determine the capacities of the roof level horizontal steel bracing and of both the in-plane and out-of-plane capacities of the external wall panels including their connections.

CCC has instructed Opus to proceed and this report presents the outcomes of this work. In addition the report presents a summary of relevant compliance issues as background information for the reader.

The seismic assessment and reporting have been undertaken based on the 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

[1] *Wharenui Recreation Centre Sports Hall Detailed Engineering Evaluation, Stage One Qualitative Report*, Opus International Consultants, November 2011.

commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

This section enables the chief executive to require a building owner, insurer or mortgagee to carry out a full structural survey before the building is re-occupied.

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building definition in the Building Act). It is anticipated that CERA will adopt the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011. This document sets out a methodology for both initial qualitative and detailed quantitative assessments.

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

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

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

2.2 Building Act

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

Section 112 - Alterations

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

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

Section 115 – Change of Use

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

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

Section 121 – Dangerous Buildings

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

1. In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
2. In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
3. There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
4. There is a risk that 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 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 4th September 2010.

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,

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

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

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

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

2.4 Building Code

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

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

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

3 Earthquake Resistance Standards

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

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

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

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

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

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

Table 1: %NBS compared to relative risk of failure

4 Background Information

4.1 Building Description

The Wharenui Recreation Centre Sports Hall is located on the corner of Elizabeth Street and Matipo Street. For the purposes of this report we refer to the direction parallel to Matipo Street as the north to south direction and the direction parallel to Elizabeth Street as the east to west direction.

The original building was constructed in 1990. It is a single storey steel portal frame and precast perimeter wall building with a precast concrete floor viewing area on the east side of the building. The building has overall plan dimensions of 32 metres (north-south) and 32 metres (east-west), and a 1024 m^2 floor area.

4.1.1 Gravity Load Resisting System

The building has four 610UB steel portal frames spanning in the east to west direction. The frames are supported by 60x60L bracing running across the central bay with 89x89RHS sections providing horizontal restraint to the portal frames at third points. These frames support steel purlins and metal roofing. The columns are clad in timber framing.

The mezzanine floor to the east side of the building is a 200mm thick Dycore precast flooring system with 65mm concrete topping. These precast slabs span one way and are supported by precast panels on either side.

The main ground floor is a timber floor on 150x50mm floor joists at 600mm centres which span onto 150x75 timber bearers 1.32m centres. The timber bearers span onto concrete piles with the piles on an approximate grid of 1.6 m x 1.32m. The ground floor to the east of the building is a ground bearing concrete slab of 125mm thickness over a 75mm thick blinding slab.

The steel frame columns are supported by 1 metre wide by 4 metres long shallow concrete foundation pads. These are tied by a 300 x 300mm concrete tie beam in the east-west direction that provides moment restraint to the column bases.

There are precast panels around the full perimeter of the building. These are generally 175mm thick panels spanning between the steel frame columns. The panels on the north and south elevation span between 150UC columns. The wall panels are reinforced with D12 bars at 300mm centres. The wall panels are fixed to the columns at the centre and the top of the precast panels, and are also connected to the foundations. Each fixing consists of one 16mm diameter bolt bolted to a plate on the precast panels, with the plates being welded to the anchor bars in the panel.

4.1.2 Seismic Load Resisting System

Seismic forces in the east-west direction are resisted by four portal frames. Forces in the north-south direction are resisted by the in-plane response of the precast walls on the east and west elevations. The horizontal roof level bracing is provided to transfer the lateral forces in the north-south direction to the walls.

The walls parallel to the north-south direction resist the forces from their own tributary weight, while the central and east elevation walls also resist the mass of the mezzanine floor. A suitable mechanism for resisting seismic forces in the east-west direction on the mezzanine floor does not appear to exist.

4.2 Survey

An initial inspection of the building was undertaken on 29 September 2011 by Alistair Boyce and Alex Laird of Opus International Consultants.

A further inspection was undertaken on 6 December 2011 by Josiah Thompson and Joel Stratford of Opus International Consultants.

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

4.3 Original Documentation

Copies of the following construction drawings were provided by CCC:

- Wharenui Recreation Centre structural drawings, sheets S1-S11 (Evans Douglas consulting engineers), stamped 4 December 1990.

- Wharenui Recreation Centre architectural drawings, sheets A1-11 (John Warren and Associates Ltd), stamped 4 December 1990.

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

No design calculations were available.

5 Structural Damage

A report on the building inspection that was undertaken as part of this stage of the engineering evaluation is appended to this report (Appendix 1). In summary the findings were as follows:

1. A number of the panels have sustained cracking and spalling apparently as a result of being overstressed both in plane and out of plane. Crack widths are up to 1mm.
2. The south end of the mezzanine floor is braced by a 150mm concrete block wall and precast panels. There is no equivalent bracing at the north end. The floor is attached via the wall panels to the steel portal frames columns with M16 bolted connections; however it is apparent that these connections have limited capacity.
3. There is no eaves member to distribute loads from the roof bracing uniformly to the precast panels that brace the building in the north-south direction.

Other than the damage listed above, the building has apparently withstood the Darfield and Christchurch earthquakes in good condition.

6 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [2] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the Detailed Engineering Evaluation Procedure [3] (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

6.1 Critical Structural Weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document issued by the Structural Engineering Society (SESOC) on 9 July 2011, 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 the building.

The following critical structural weaknesses have been identified for the building:

a) The precast panels span horizontally between columns and are connected to the columns at two locations, at the top of the wall and at the centre height of the wall. The connection is by one 16mm diameter bolt at each location. These connections are potentially a critical structural weakness as they do not appear to be sufficient to transfer the loads to the columns. The panels have sustained cracking and in particular two panels, one on the south elevation and one on the west elevation have failed. These panels do not appear to be sufficient to resist the out of plane loading.

b) In the north-south direction there does not appear to be a suitable load path for transferring the roof load to the shear walls. No tie beams have been indicated on the drawings at the top of the portal frame columns.

c) There does not appear to be a reliable load path in the east-west direction for the mezzanine floor.

6.2 Detailed Seismic Assessment Results

A summary of the structural performance of the building is shown in Table 2 below. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing elements. This will be considered further when developing the strengthening options.

The north-south and east-west directions are parallel to Matipo St and Elizabeth St respectively. The steel portal frames span in the east-west direction.

Structural Element/System	**Failure Mode or description of limiting criteria based on capacity of critical element.**	**% NBS based on calculated capacity**
Walls – north-south direction	Out of plane flexural failure (μ=2.0)	41%
Walls – east-west direction	Out of plane flexural failure (μ=2.0)	57%
Panel Connections to Portal Frame Columns	Shear failure of connections in concrete panel (μ=1.0)	25%
Roof Bracing Diagonal Struts	Elongation (μ=1.25)	38%
Roof Bracing Diagonal Struts	Buckling (μ=1.25)	11%
Mezzanine floor	Tension failure of floor/wall connections to portal frame columns, loading in east-west direction (μ=1.0)	22%

Table 2: Summary of Seismic Performance

6.3 Discussion of Results

The precast concrete wall cladding panels are generally 8.5m wide and up to 7.2m high. They are restrained into the foundations at their base, and bolted to portal columns or steel frames at two locations on the edges. They are reinforced with D12 bars at 300mm centres. The results reported in Table 2 are for the worst cases where large openings in the panels reduce the effectiveness of the base restraint so that the panels are effectively spanning 8.5m horizontally between steel supports. It was the panels containing these openings that received the most damage in the February earthquake (refer to Appendix 1).

Seismic forces in the north-south direction are transferred via diagonal steel roof plane bracing into the concrete panels on the east and west elevation walls. Because there are no distributor ties at the junctions of the bracing and the panels to distribute the bracing loads amongst the panels, the entire load from the bracing is transferred to the panels via two M16 bolts at the connection with one portal column on each side. Consequently the connections are over-loaded.

The roof bracing comprises 60mm x 60mm angle diagonal tension braces and 89mm x 89mm x 3.6mm RHS compression struts. Both of these components are under-size for current design loads, particularly the struts which, being 8.6m long, have very limited buckling resistance. No contribution to the strut capacity has been included from the purlins because they are not in the plane of the bracing, being located on fly rafters.

The mezzanine floor structure comprises a hollowcore precast floor supported on precast concrete panels on the east and west sides. The in-plane strength of the panels provides good earthquake resistance in the north-south direction. However it lacks a reliable bracing system in the east-west direction. It appears that the design relied upon bolted connections of the mezzanine structure to the steel portal columns, which are very deficient for this purpose. There are substantial walls at the south end, and a nominal connection to the wall panels at the north end. However these are 32m apart, and the 7.5m wide, 65mm thick mesh-reinforced slab topping is ineffective as a diaphragm over this length, so the capacity of these end walls cannot be mobilised.

In summary, as a result of these deficiencies, the building is assessed to be earthquake prone in terms of the Building Act as it has a seismic capacity less than 34% NBS.

6.4 Limitations and Assumptions in Results

Our analysis and assessment is based on an assessment of the building in its undamaged state. Therefore the current capacity of the building will be lower than that stated.

The results have been reported as a %NBS and the stated value is that obtained from our analysis and assessment. Despite the use of best national and international practice in this analysis and assessment, this value contains uncertainty due to the many assumptions and simplifications which are made during the assessment. These include:

- Simplifications made in the analysis, including boundary conditions such as foundation fixity,

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

This analysis has focussed on potentially critical structural weaknesses identified during the engineering assessments. Other parts of the structure such as the portal frames, which were judged in the qualitative stage to be satisfactory, have not been analysed in detail.

7 Summary of Geotechnical Appraisal

7.1 Desk Study

A desk study of well logs in the area obtained from Environment Canterbury records identified four drill logs from boreholes located within 300m of the site.

The borehole logs indicate the area is underlain by a layer of sands and clay, which is underlain by gravel layer. The gravel layer is encountered between 8.5m and 13.7m below ground level.

7.2 Ground Damage

Aerial photographs taken on 24th February 2011 and 16th June 2011 show no evidence of surface rupture or liquefaction at the site. A walkover inspection of the exterior of the building and surrounding sites was completed on 10 January 2012. No evidence of liquefaction was observed during the site walkover and there was also no evidence of differential settlement. The footings for the western portal of the Sports Hall may have moved laterally west by less than 10mm. This movement is considered to be due to oscillatory ground shaking rather than liquefaction.

7.3 Liquefaction Hazard

The 2004 ECan Liquefaction study indicates that no liquefaction is predicted on the site.

The initial reconnaissance completed by Tonkin & Taylor on 24 Feb indicates the site is not in a liquefaction area.

The CERA land zone map released 23 June 2011 has classified the land as 'green', repair/rebuild process can begin.

The Department of Building and Housing (DBH) guidance document on residential house repairs and reconstruction indicates the residential areas surrounding the site are Technical Category 2. Technical Category 2 identifies the area may be subject to minor to moderate land damage from liquefaction in future significant earthquakes.

7.4 Summary

On the basis of the above observations, the existing foundations appear to have performed well under seismic loading. The existing foundations are considered to be suitable for the ground conditions. We do not believe any further geotechnical investigations are warranted at this site at this stage.

8 Remedial Work

The indicative scope of work required to repair the building and improve its seismic performance is listed below. It should be possible to achieve at least a 67%NBS standard.

1. Repair the cracked cladding panels.
2. Provide a more robust load-path from the roof bracing to the wall panels.
3. Provide additional support to the cladding panels to improve their out-of-plane strength.
4. Replace or enhance the roof bracing, including the diagonal braces and struts.
5. Brace the mezzanine floor structure.

We recommend that if remedial work is implemented, the other parts of the structure that have not been assessed should also be analysed to confirm that they meet an acceptable standard. The scope of remedial work could vary as a result of the further analysis associated with such an analysis and design of the remedial work.

9 Conclusions

1. The seismic capacity of the building is approximately 11% of the current building code new building standard, based on the axial compressive strength of the roof level horizontal bracing. The failure of this bracing is unlikely to result in a collapse of the structure, however an increase in damage can be expected.
2. The precast concrete panel to portal frame connections and the mezzanine floor to portal frame connections also have a seismic capacity less than 34% of new building standard. The failure of these elements could result in partial collapse of some parts of the building.
3. The building is therefore classified as an earthquake prone building as it has a seismic capacity less than 34% NBS.
4. Remedial works could be undertaken to increase the seismic capacity of the building to at least 67% NBS by providing a more robust load path between the roof and wall panels, replacing or enhancing the roof level bracing and by bracing the mezzanine structure.

5. The existing foundations appear to have performed well under seismic loading and are considered to be suitable for the ground conditions. We do not believe any further geotechnical investigations are warranted at this site at this stage.

10 Recommendations

1. A strengthening works scheme be developed to increase the seismic capacity of the building to at least 67% NBS.
2. The parts of the building that have been assessed qualitatively to be satisfactory should be assessed quantitatively to confirm their ratings.
3. A quantity surveyor be engaged to determine the costs for either strengthening the building or demolishing and rebuilding.
4. The building should not be occupied, given its structural weaknesses.

11 Limitations

a) This report is based on an inspection of the structure of the building and focuses on the structural damage resulting from the 22 February Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

b) Our 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: 2006, Assessment and improvement of the structural performance of buildings in earthquakes, New Zealand Society for Earthquake Engineering.

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

Appendix 1 – Site Visit Report

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Tel +64 3 363 5400
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TO	Robert Davey, Mike Roys
COPY	Steve James
FROM	Josiah Thompson and Joel Stratford
DATE	7 December 2011
FILE	6-QUCCC.41/5SC
SUBJECT	Wharenui Sports Hall Site Visit

The following recommendations were outlined from the stage one qualitative assessment of the building:

1. An inspection survey is carried out to determine the damage sustained from the earthquakes and to review the damage to the external panels.

2. Further investigation is needed to assess and identify suitable load paths if any from the mezzanine floor to the shear walls.
.
3. A survey of the eaves is carried out to determine if there are tie beams between frames in the north-south direction.

These items were considered in a recent site visit and the findings are as follows:

1. The damage sustained to the external precast panels appears to be a combination of both shear cracking and out of plane failure. Some spalling/crushing of concrete was also noted near one of the ventilation grills, and internally at the top of one of the precast panel joints. Damage was observed primarily along the west and south walls, with cracks located in panels with access way cut-outs.

   a) We will first consider the damage observed along the west wall of the building, see over page:

Photo 1 - West wall elevation

Photo 2 – Diagonal cracks in the west wall precast panel

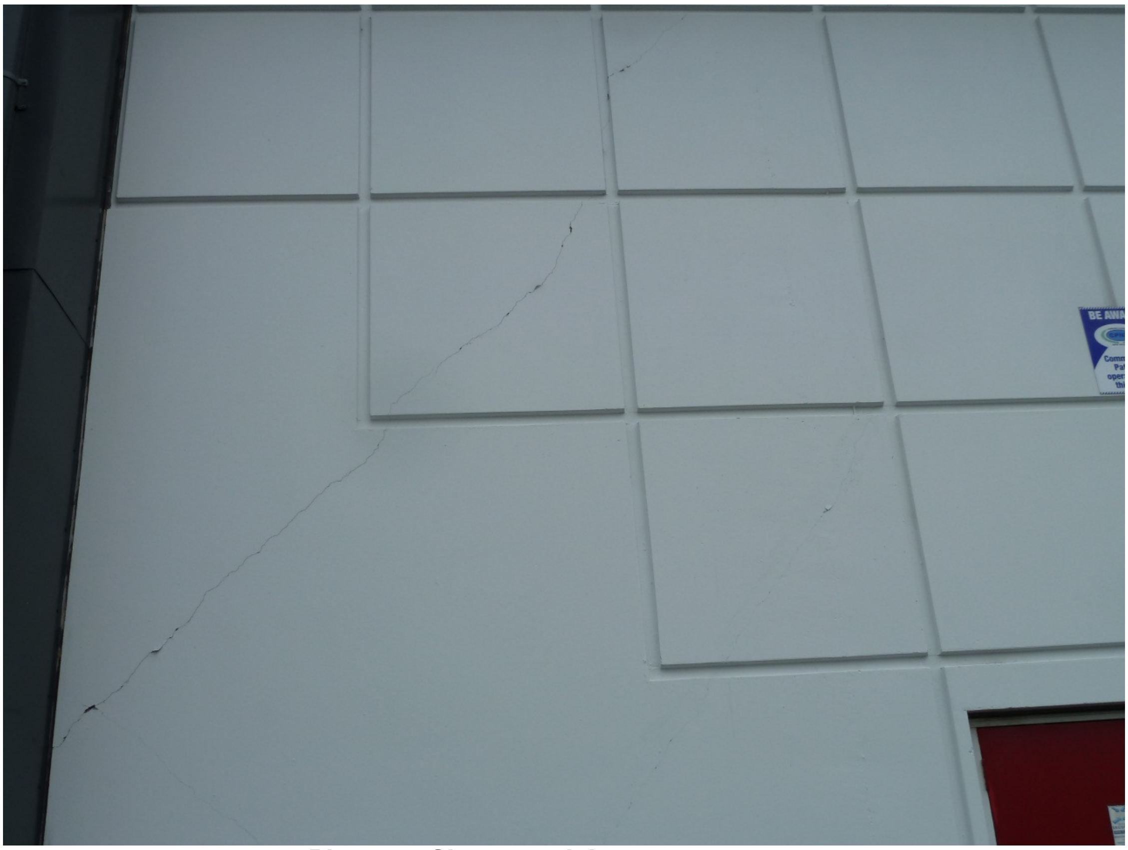

Photo 3 - Shear crack in west wall panel

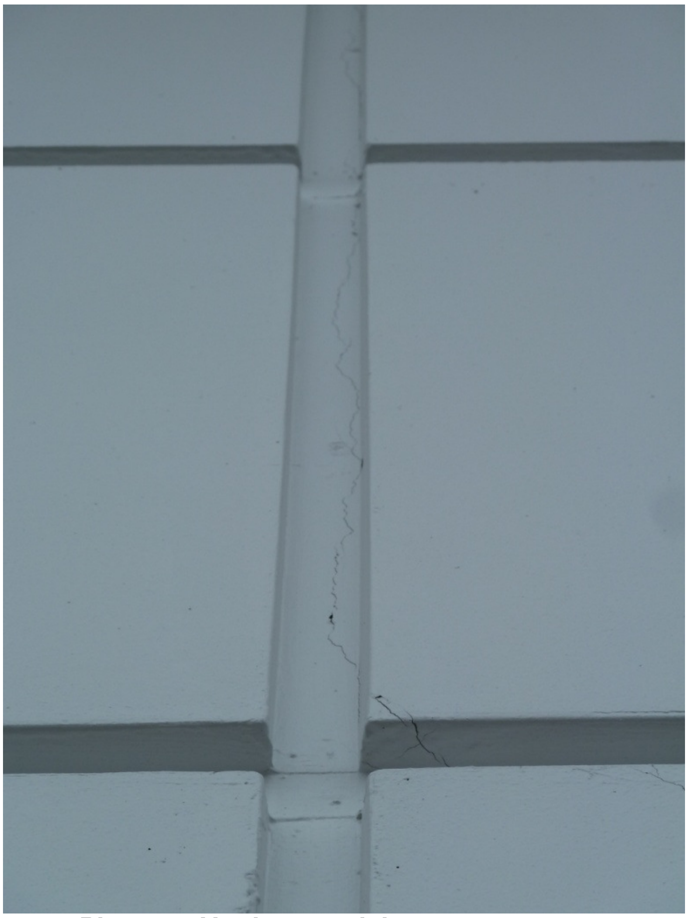

Photo 4 - Vertical crack in west wall panel

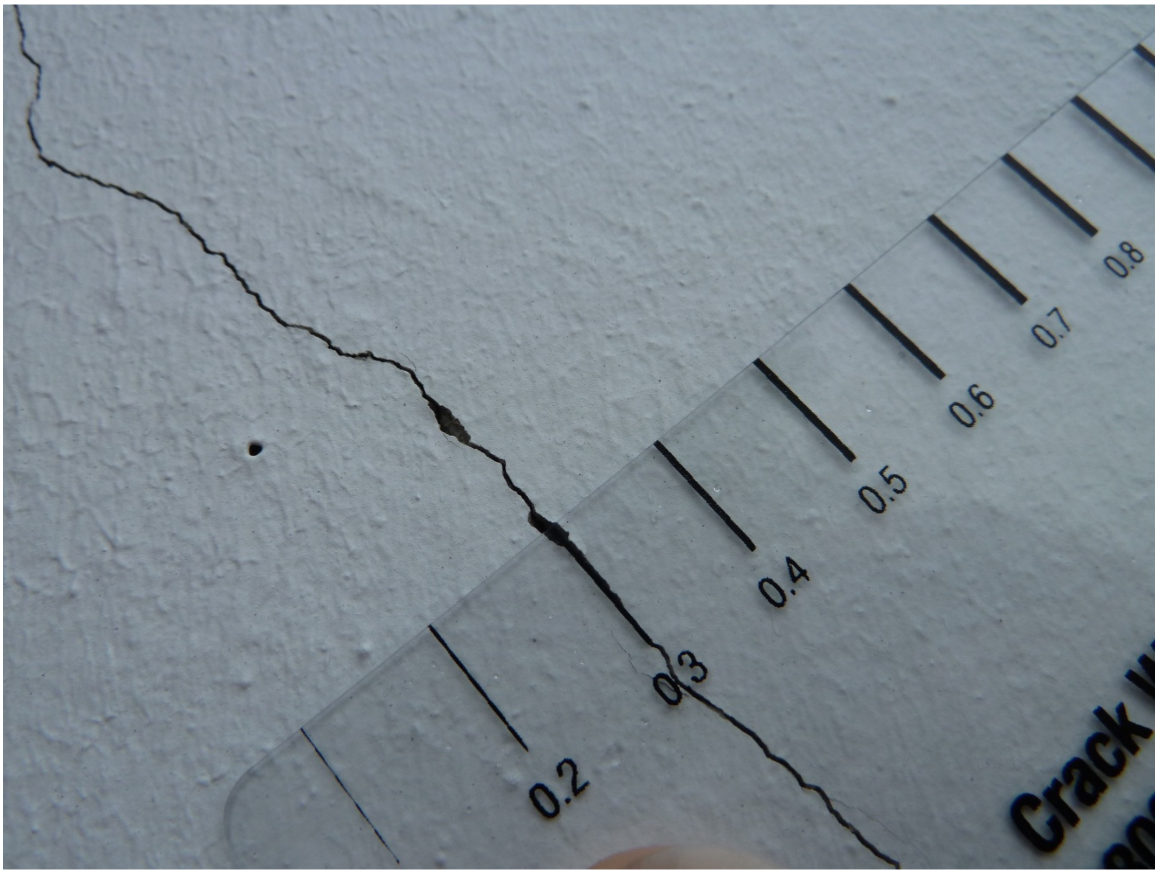

Photo 5 - Diagonal crack approximately 0.3mm wide on the west exterior panel

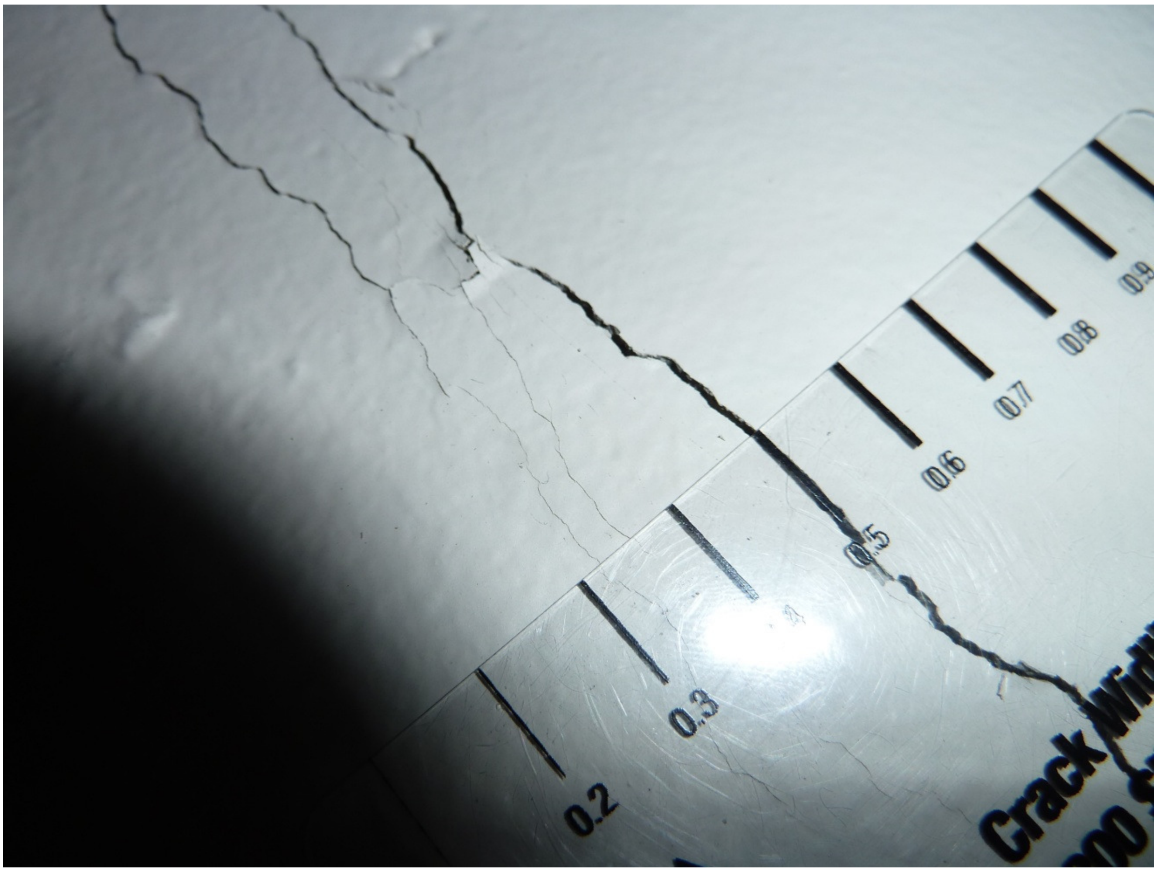

Photo 6 - Diagonal crack approximately 0.5mm wide on the west wall interior

b) Now considering the south wall:

Photo 7 - South wall elevation

Photo 8 - Cracks located around the large access opening

Photo 9 - Vertical crack propagating from the access way in the south wall panel approximately 5mm width

Photo 10 - Interior view of vertical crack on the south wall panel

Photo 11 - Diagonal crack along the south wall exterior approximately 0.9mm

Photo 12 - Cracking/Spalling at the base of the south wall

Photo 13: Cracking/Spalling at the top of the precast panel south wall, viewed internally to the east of the damaged panel shown previous

Note non-structural damage to the building was not considered or documented here.

2. The load paths from the mezzanine floor to the precast shear walls were investigated:

   D12 starter bars at 300 centres tie the precast panels to the east and west of the mezzanine into the topping slab of the floor. This detail should sufficiently collect lateral load from the mezzanine floor diaphragm in the north-south direction.

   The southern end of the mezzanine is tied into a 150mm thick concrete block wall and the precast wall panels with starter bars as above. This may provide sufficient load path to the southern end of the mezzanine floor, for the east-west direction of loading.

   The northern end of the mezzanine poses the only concern for potential load paths. There appears to be no sufficient collector element to transfer lateral loads from the floor diaphragm in the east-west direction of loading. There are starter bars from one of the precast panels tied into the insitu concrete landing, but these are only over a short length of approximately 500mm (4 D12 bars).

   There is still potential for an alternative load path from the precast floor diaphragm to the perimeter portal frames on the east via the mid height panel connection to the portal column. This is one M16 bolt in tension per panel connection (refer to sheet S4, insert ‘a’) and also relies on the anchorage of the insert which appears to be insufficient as noted further in 4(a). It was also noted there are no stiffeners to the portal column at this location where the concentration of load would occur, so it appears this load path was not intended.

   For further details refer to the Mezzanine floor mark-up attached.

3. Tie beams between frames in the north-south direction were not visible from both an exterior and interior view of the precast panels. We can therefore assume these are not present in the structure. See figures below:

Photo 14 - External view of west wall precast panel

Photo 15 - Interior view of west wall precast panel

Photo 16 - Diagonal ties above west wall panel

Note the vertical crack in the west wall is located in the panel with diagonal steel ties shown above. With the diagonal ties acting in tension, and without tie beams between portals, the precast panel will need to carry the resultant axial compression. The compression forces in the panel could contribute to the out of plane failure of the panel.

4. Other items noted from site visit and drawing review which are potential critical structural weaknesses and may need consideration in assessment:

    a. Precast panel inserts/weld plates in general have insufficient length of reinforcing into the panels. For example sheet S4 inserts a-f have typically around 200m of reinforcing length extending from weld plates, and therefore will not be able to develop their full capacity and are likely to fail in a brittle manner.

    b. The capacity of the perimeter precast walls will be reduced by the ventilation cut-outs. The location of these vents reduces the cross sectional area of the walls, and subsequently increases the stress concentrations at the bottom of the precast panels.

    c. The detailing of the precast stair reinforcing may need to be reviewed. In particular, the upper flight (sheet S3) is anchored both top and bottom, and the detailing around the stair to landing junction in the centre of the unit may need to be checked. Although we are aware the displacement demand may not be significant.

    d. The precast base connection does not appear to be well detailed (see sheet S1, section 10-19).

Appendix 2 – CERA DEE Spreadsheet

Detailed Engineering Evaluation Summary Data V1.10

Location

Building Name:	Wharenui Recreation Centre	Reviewer:	Alistair Boyce
Building Address:	Unit: / No: 73 / Street: Elizabeth Street	CPEng No:	209860
Legal Description:		Company:	Opus International Consultants
		Company project number:	6QUCCC.41
		Company phone number:	+64 3 363 5400
GPS south: (Degrees / Min / Sec)		Date of submission:	13-Sep-12
GPS east: (Degrees / Min / Sec)		Inspection Date:	
		Revision:	Final
Building Unique Identifier (CCC):	BU 0904-001 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	0
Soil type:	sandy silt	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):	

Building

No. of storeys above ground:	1 (single storey = 1)	Ground floor elevation (Absolute) (m):	
Ground floor split?	no	Ground floor elevation above ground (m):	0.38
Storeys below ground	0		
Foundation type:	driven precast piles	if Foundation type is other, describe:	Strip footing to the precast walls
Building height (m):	10.51	height from ground to level of uppermost seismic mass (for IEP only) (m):	
Floor footprint area (approx):	1024		
Age of Building (years):	21	Date of design:	1976-1992
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):	other (specify)		
Use notes (if required):	Viewing Gallery - 125mm thick Dycore precast system		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	Steel portal frame, steel purlins, metal roofing
Floors:	timber	joist depth and spacing (mm)	150 and 600
Beams:	steel non-composite	beam and connector type	
Columns:	structural steel	typical dimensions (mm x mm)	603 x 228
Walls:	load bearing concrete	#N/A	

Lateral load resisting structure

East-west	Lateral system along:	welded and bolted steel moment frame		note typical bay length (m)	8.6
	Ductility assumed, μ:	3.00			
	Period along:	0.54	0.00	estimate or calculation?	estimated
	Total deflection (ULS) (mm):	33		estimate or calculation?	estimated
	maximum interstorey deflection (ULS) (mm):	40		estimate or calculation?	estimated
North-south	Lateral system across:	welded and bolted steel moment frame		note typical bay length (m)	8.6
	Ductility assumed, μ:	1.25			
	Period across:	0.23	0.00	estimate or calculation?	estimated
	Total deflection (ULS) (mm):			estimate or calculation?	estimated
	maximum interstorey deflection (ULS) (mm):			estimate or calculation?	estimated

Separations:

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

Non-structural elements

Stairs:			
Wall cladding:	precast panels	thickness and fixing type	175mm thick
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	none		
Services(list):			

Available documentation

Architectural	partial	original designer name/date	John Warren and Associates Ltd. Architects July 1990
Structural	partial	original designer name/date	Evans Douglas Consulting Engineers August 1990
Mechanical	none	original designer name/date	
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:		Describe damage:	
Settlement:	0-25mm	notes (if applicable):	
Differential settlement:	0-1:350	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:	5%	Describe how damage ratio arrived at:	Estimated repair cost/replacement cost
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	yes	Describe:	Panels have failed in yielding
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	yes	Describe:	cracks in precast panels

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

Recommendations

Level of repair/strengthening required:	significant structural and strengthening	Describe:	
Building Consent required:	yes	Describe:	
Interim occupancy recommendations:	do not occupy	Describe:	

Along	Assessed %NBS before:	11%	##### %NBS from IEP below
	Assessed %NBS after:	11%	
Across	Assessed %NBS before:	11%	##### %NBS from IEP below
	Assessed %NBS after:	11%	

IEP

Age of Building (from above): 1976-1992

h_n from above: m

Seismic Zone, if designed between 1965 and 1992:

not required for this age of building
not required for this age of building

	along	across
Period (from above):	0.54	0.23
(%NBS)nom from Fig 3.3:		
Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25		
Note 2: for RC buildings designed between 1976-1984, use 1.2		
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)		
Final (%NBS)$_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, Table 3.3):		
Near Fault scaling factor (1/N(T,D), **Factor A**:	#DIV/0!	#DIV/0!

2.3 Hazard Scaling Factor

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

2.4 Return Period Scaling Factor

Building Importance level (from above):	2
Return Period Scaling factor from Table 3.1, **Factor C**:	

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.3)		
Ductility scaling factor (if pre-1976):		
Ductility Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

Sp:		
Structural Performance Scaling Factor **Factor E**:	#DIV/0!	#DIV/0!

2.7 Baseline %NBS, (NBS%)$_b$ = (%NBS)$_{nom}$ x A x B x C x D x E — **%NBS$_b$:** **#DIV/0!** | **#DIV/0!**

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

3.1. Plan Irregularity, factor A: 1

3.2. Vertical irregularity, Factor B: 1

3.3. Short columns, Factor C: 1

3.4. Pounding potential
Pounding effect D1, from Table to right 1.0
Height Difference effect D2, from Table to right 1.0

Therefore, Factor D: 1

3.5. Site Characteristics 1

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

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

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	1.0	1.0
Rationale for choice of F factor, if not 1	NZS 1170 soil class D but likely to have been designed assuming Intermediate subsoil ((NZS 4203 1992	

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

3.7. Overall Performance Achievement ratio (PAR)	**1.00**	**1.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**#DIV/0!**	**#DIV/0!**
4.4 Percentage New Building Standard (%NBS), (before)		**#DIV/0!**

OPUS