Wharenui Pool Building

BU 0533-002 EQ2

Detailed Engineering Evaluation

Stage Two Quantitative Report

Christchurch City Council

Christchurch City Council

Wharenui Pool Building

Detailed Engineering Evaluation Quantitative Report BU 0533-002 EQ2

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Contents

Executive Summary

Christchurch City Council appointed Opus International Consultants to carry out a detailed seismic assessment of the Wharenui Pool building in Riccarton, Christchurch. The purpose of this assessment was to ascertain the anticipated seismic performance of the structure and to compare this performance with current design standards.

Two critical structural weaknesses have been identified for the building. These are the lack of a load path for north-south seismic loads to be distributed to the western vertical cross bracing elements and the eastern reinforced masonry wall. The seismic loads in the north-south direction can be resisted by out of plane flexure in the glue-laminated timber portal frames, however this could lead to increased levels of damage in the building.

The seismic capacity of the building has been calculated as between 35-40% NBS including all critical structural weaknesses. The capacity is governed by the out of plane flexural capacity of the eastern reinforced masonry wall.

It is recommended that strengthening works are undertaken to restore the load paths to the bracing elements on the eastern and western walls. This would need to be developed in a strengthening options stage.

Intrusive investigation works undertaken on the eastern wall revealed that a number of bolt fixings supporting the plywood box gutter are heavily corroded. It is recommended that a structural condition survey of these and other hidden fixings is completed.

1 Introduction

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

The purpose of the assessment is to determine if the building is classed as being earthquake prone in accordance with the Building Act 2004.

The seismic assessment and reporting have been undertaken based on the qualitative and quantitative procedures detailed in the Detailed Engineering Evaluation Procedure (DEEP) document (draft) issued by the Structural Engineering Society (SESOC) on 19 July 2011.

A qualitative seismic assessment report for the building was issued on 9 November 2011.

2 Compliance

This section contains a brief summary of the requirements of the various statutes and authorities that control activities in relation to buildings in Christchurch at present.

2.1 Canterbury Earthquake Recovery Authority (CERA)

CERA was established on 28 March 2011 to take control of the recovery of Christchurch using powers established by the Canterbury Earthquake Recovery Act enacted on 18 April 2011. This act gives the Chief Executive Officer of CERA wide powers in relation to building safety, demolition and repair. Two relevant sections are:

Section 38 – Works

This section outlines a process in which the chief executive can give notice that a building is to be demolished and if the owner does not carry out the demolition, the chief executive can commission the demolition and recover the costs from the owner or by placing a charge on the owners' land.

Section 51 – Requiring Structural Survey

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

We understand that CERA will require a detailed engineering evaluation to be carried out for all buildings (other than those exempt from the Earthquake Prone Building 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 CCC Earthquake Prone Building Policy.

2.2 Building Act

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

Section 112 - Alterations

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

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

Section 115 – Change of Use

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

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

2.2.1 Section 121 – Dangerous Buildings

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

1. In the ordinary course of events (excluding the occurrence of an earthquake), the building is likely to cause injury or death or damage to other property; or
2. In the event of fire, injury or death to any persons in the building or on other property is likely because of fire hazard or the occupancy of the building; or
3. There is a risk that the building could collapse or otherwise cause injury or death as a result of earthquake shaking that is less than a 'moderate earthquake' (refer to Section 122 below); or
4. There is a risk that other property could collapse or otherwise cause injury or death; or
5. A territorial authority has not been able to undertake an inspection to determine whether the building is dangerous.

Section 122 – Earthquake Prone Buildings

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

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

Section 124 – Powers of Territorial Authorities

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

Section 131 – Earthquake Prone Building Policy

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

2.3 Christchurch City Council Policy

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

The 2010 amendment includes the following:

1. A process for identifying, categorising and prioritising Earthquake Prone Buildings, commencing on 1 July 2012;
2. A strengthening target level of 67% of a new building for buildings that are Earthquake Prone;
3. A timeframe of 15-30 years for Earthquake Prone Buildings to be strengthened; and,
4. Repair works for buildings damaged by earthquakes will be required to comply with the above.

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

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

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

2.4 Building Code

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

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

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

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	Improvement of Structural Performance: 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
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 Building Description

4.1 General

The Wharenui pool building is a single storey structure which forms part of a complex of buildings for the Wharenui Sports Centre. The centre 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 north to south direction and the direction parallel to Elizabeth Street as east to west direction.

From archive drawings we have deduced that the building was probably built in the late 1960s over an existing outdoor swimming pool and later modified in the 1990s. The existing building is a single level glue-laminated timber portal frame structure with a steel framed structure for changing areas along the western elevation. To the north end of the glue-laminated portal structure is a masonry and precast concrete panels structure incorporating the plant and administrative areas. This structure has been built around an existing brick substation owned by Orion. The south gable and east external walls of the pool building are constructed in reinforced masonry.

The building is approximately 50m long in the north-south direction and 26m wide in the east-west direction. The roof apex is approximately 6m above ground level.

4.2 Gravity Load Resisting System

Eight glue-laminated timber portal frames span in the east-west direction and are supported on concrete bases. The portal frames do not have any intermediate props. A lightweight roof consisting of insulated panels is supported on glue-laminated timber purlins which span between the portal frames. A box gutter is provided on the eastern and western sides of the building and is supported by a 300mm deep steel beam spanning between the portal frames.

The changing block to the west is a steel clad framed structure which dates from the period when the pool was an outdoor facility. There are no details of the changing block structure amongst the archive drawings however it appears that the structural system consists of insulated panels.

The plant block at the northern end of the pool building comprises an extended plant area and a substation (owned by Orion). The plant area has been altered and extended by building a small steel portal framed structure around the original plant area with external precast concrete panel walls. From the record drawings the plant block and substation appear to be structurally independent from the pool building in terms of gravity loads.

4.3 Seismic Load Resisting System

There are four distinct parts to the building, each having different seismic load resisting systems.

The seismic load resisting system in the east-west direction for the main pool building is provided by the glue-laminated timber portal frames and the southern gable masonry wall. In the north-south direction the seismic loads were intended to be provided by in-plane shear action of the eastern full length reinforced masonry wall and by two vertically cross braced steel frames on the western wall. There is no visible lateral bracing in the roof and it has been assumed that the insulated panel roof is providing a form of diaphragm action in distributing seismic loads to the load resisting elements.

The changing block on the western side of the main pool building pre-dates both current and previous pool buildings and no structural details have been located in the archive drawings. The changing block is formed from insulated panels, and it has been assumed that these panels resist the seismic loads in each direction.

The plant block at the north-eastern end of the pool building has been altered and extended in the past. Originally a masonry structure, a steel frame has been constructed over the existing footprint incorporating an additional area to create a larger building. The new external walls are constructed from precast concrete tilt panels. According to the record drawings there is no roof bracing although Villaboard has been specified as a ceiling lining. In the north to south direction stability is achieved though steel portalised bays and precast concrete panels. In the east to west direction precast concrete panels provide stability against seismic loads.

The administration area at the north-western end of the pool building comprises reinforced masonry walls to the north and west and two internal steel portal frames in the east-west direction. Seismic loads in the east-west direction are resisted by the northern masonry wall and the steel portal frames, and loads in the north-south direction are resisted by the western wall.

From the archive drawings the substation appears to be structurally independent of the pool building, even though it is adjacent and forms a party wall with the pool. The substation is rectangular on plan with masonry walls providing lateral stability in both directions.

5 Survey

A survey of the building was undertaken on 1 November 2011 by Opus International Consultants. Intrusive opening works were undertaken to several structural elements in the pool building on 20 January 2012 to ascertain details of structural connections.

The pool building currently has a green placard (not issued as part of this inspection and authorised by an engineer working for a company other than Opus International Consultants).

Copies of the following archive drawings were referred to as part of the assessment:

- A set of R & A Design architectural drawings in relation to the extension and alteration of the plant block dated 1997.
- Drawings for a new club room dated 1962 but are no longer representative of the current structure.
- A set of drawings by Bill Lovell-Smith dated 1968 for the original steel portal framed building erected over the pool.
- A set of Christchurch City Council Drawings dated 1990 for the construction of a new gallery (these drawings show a small part of the building under evaluation).

No copies of the design calculations have been obtained as part of the documentation set.

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

Structural drawings have not been located for the building in its current form.

6 Damage Assessment

The following damage has been noted:

6.1 Glue-laminated portal frames

Clamps have been put in place at the ridge locations on some of the rafters of the glue-laminated portal frames. There are cracks in these locations between the laminates.

6.2 Perimeter block masonry walls

The east wall of the pool building appears to be out of plumb but there are no signs of cracking or other damage.

7 General Observations

Overall the building has performed well under seismic conditions which would be expected for a modern single storey structure. The building has sustained little damage and continues to be fully operational.

Due to the non-intrusive nature of the original survey, many connection details could not be ascertained. However a limited “opening up” exercise has now been carried out, and most of the critical details at junctions and interfaces between load bearing elements have been determined and their capacity assessed, particularly at the head of the east external wall where apparent displacement has occurred.

8 Detailed Seismic Assessment

8.1 Critical Structural Weaknesses

As outlined in the Critical Structural Weakness and Collapse Hazards draft briefing document, issued by the Structural Engineering Society (SESOC) on 7 May 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.

We have identified the following potential CSW’s for the building:

8.1.1 Seismic load path to the eastern reinforced masonry wall

No vertical bracing system is apparent on the eastern side of the main pool building to resist seismic loads in the north to south direction, and it has been assumed that the existing 190mm thick reinforced masonry wall is intended to act as a shear wall.

The reinforced masonry wall is strutted back to the portal frames just below the roof level with two 140x45mm timber members. This detail does not provide any means for the north-south seismic loads to be transferred to the masonry wall.

The box gutter appears to be formed from plywood which could potentially provide some diaphragm action between the masonry wall and the longitudinal steel beam supporting the box gutter, however the bolted connections between the timber runner and masonry wall are extremely corroded and have limited capacity.

8.1.2 Seismic load path to the western wall vertical bracing elements

Two bays of steel vertical cross bracing are provided on the western wall in order to resist seismic loads in the north to south direction.

The opening up works have revealed that there is no viable load path to transfer north to south direction seismic loads from the main roof level and into the cross bracing elements. There does also not appear to be an adequate collector beam running along the top of the wall to transfer the seismic forces into the frames.

8.2 Seismic Coefficient Parameters

The seismic design parameters based on current design requirements from NZS1170.5:2004 and the NZBC clause B1 for this building are:

- Site soil class D, clause 3.1.3 NZS 1170.5:2004
- Site hazard factor, Z=0.3, B1/VM1 clause 2.2.14B
- Return period factor R_u = 1.0 from Table 3.5, NZS 1170.5:2004, for an Importance Level 2 structure with a 50 year design life.

8.3 Expected Ductility Factors

Based on our assessment of the structural details our estimates for the expected maximum structural ductility factors for the main seismic resisting systems are:

- μ_{max} = 1.25 for all reinforced concrete masonry walls and precast concrete panels.
- μ_{max} = 2.0 for the glue-laminated timber portal frames.
- μ_{max} = 2.0 for the steel portal frames and vertical cross bracing.

8.4 Detailed Seismic Assessment Results

A summary of the structural performance of the building is shown in the following tables. Note that the values given represent the worst performing elements in the building, as these effectively define the building's capacity. Other elements within the building may have significantly greater capacity when compared with the governing element.

Table 2: Summary of Seismic Performance

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Pool building glulam portals (east-west direction)	In-plane flexural capacity.	No	120%
Reinforced masonry wall (east elevation) – out of plane capacity	The wall is only reinforced with vertical reinforcement bars which have an out of plane flexural capacity of 73% NBS, however the limiting element is the bond beam at roof level which is required to span between the portal frame struts. The bond beam has a low level of reinforcement and limited flexural capacity, however is continually supported by the plywood forming the box gutter which provides an adequate load path. The box gutter fixings to the wall are in poor condition.	No	35-40%
Reinforced masonry wall (east elevation) – in plane capacity	The wall has an in-plane flexural capacity greater than 100% NBS however there is no viable load path for transferring seismic loads from the pool building into the wall.	No	<34%

Structural Element/System	Failure mode and description of limiting criteria	Critical Structural Weakness and Collapse Hazard	% NBS based on calculated capacity
Reinforced masonry wall (east elevation) – connection to glulam frame	Shear failure of the single M16 fixing between the horizontal timber struts and the steel brackets bolted to the masonry wall. The connection detail lacks redundancy.	No	79%
Reinforced masonry wall (south elevation) – out of plane capacity	Flexural out of plane failure of the wall. The wall reinforcement details are unknown so have been based on an assumed layout of D12 at 600mm centres.	No	41%
Purlin connection to southern reinforced masonry wall.	Shear failure of the bolted connection between the purlins and the masonry wall.	Yes	88%
Steel braced bays to the western elevation	The two sets of vertical cross bracing have a capacity greater than 100% NBS however there is no viable load path for transferring seismic loads from the pool building into the cross bracing elements.	No	<34%
Pool building glulam portals out of plane flexure (north-south direction)	Out of plane flexural capacity. Fixed base connection assumed.	No	80%
Diaphragm over main pool	Shear failure of the screw fixings along the eastern and western sides of the roof.	No	>100%
Plant room steel portal frames (north-south)	In-plane flexural capacity.	No	>100%
Plant room precast panels	Out of plane flexural capacity	No	>100%

8.5 Discussion of Results

The building has a calculated seismic capacity of around 35-40% NBS as limited by the out of plane capacity of the eastern reinforced masonry wall and is therefore not classified as an earthquake prone building.

The building contains two critical structural weaknesses in the lack of reliable load paths to transfer north-south seismic loads into the western wall braced steel frames and the eastern reinforced masonry wall. The calculated seismic capacity of these elements in their existing configuration is less than 34% NBS, however a secondary load resisting system is provided through out of plane flexure of the glue-laminated timber portal frames. This mechanism relies upon having a fixed connection at the base of the portal frames. While this is an adequate secondary load path to resist seismic loads in the north-south direction it is possible that this could lead to increased levels of damage within the building. It is therefore recommended that remedial works be undertaken to restore the load path to the western and eastern walls.

The bond beam at roof level in the eastern reinforced masonry wall has insufficient capacity to span horizontally between the portal frames. While some support from the plywood box gutter has been considered, the bolt fixings from the timber runner nailed to the plywood and bolted to the wall are extremely corroded and therefore have limited capacity. Two bolts were viewed during the opening up works and each had levels of corrosion greater than 50% section loss. The combination of the bond beam flexural capacity and support from the box gutter has been assessed as providing a seismic capacity of around 35-40% NBS which governs the overall capacity of the building.

8.6 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.

9 Geotechnical Appraisal

9.1 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.

9.2 Ground Damage

Aerial photographs taken on 24th February 2011 and 16th June 2011 show no evidence of surface rupture of 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.

9.3 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.

9.4 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.

10 Remedial Options

Any remedial options for increasing the seismic capacity above 34% NBS would need to address increasing the out of plane capacity of the eastern wall and the lack of adequate connection between the main pool building roof and the eastern and western wall bracing elements. These strengthening works would need to be specifically designed.

11 Conclusions

(a) The building has a seismic capacity of between 35-40% NBS and is therefore not considered to be earthquake prone.

(b) Two critical structural weaknesses have been identified in the building and these control the overall seismic capacity of the building. The critical structural weaknesses relate to inadequate loads paths for distributing seismic loads from the main pool building rood into the bracing elements along the eastern and western wall.

(c) 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.

(d) Intrusive investigation works undertaken on the eastern wall revealed that a number of bolt fixings supporting the plywood box gutter are heavily corroded. It is recommended that a structural condition survey of these and other hidden fixings is completed.

12 Recommendations

(a) Strengthening options be developed for increasing the seismic capacity of the building to at least 67% NBS and restoring the load paths to the eastern and western walls.

(b) Undertake a structural condition survey of all hidden fixings to check the levels of corrosion.

13 Limitations

(a) This report is based on an inspection of the structure with a focus on the damage sustained from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is mentioned but this is not intended to be a comprehensive list of 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 the time.

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

Appendix A – Photographs

Photo 1 – Eastern perimeter wall

Photo 2 – North elevation (east end)

Photo 3 – North elevation (west end)

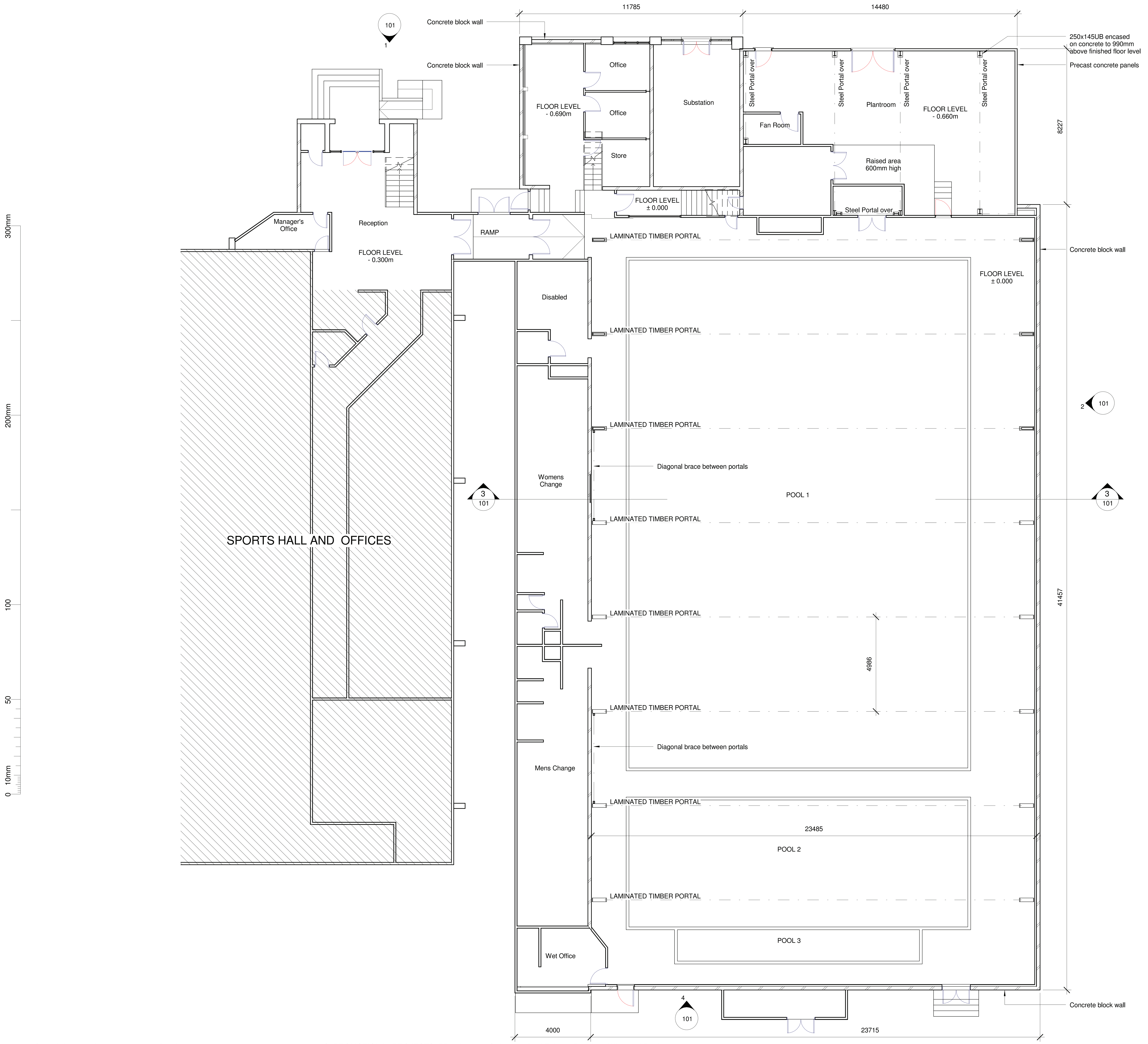
Photo 4 – South elevation

Photo 5 – Internal view of portals frames and cross bracing elements on western wall

Photo 6 – Connection between eastern wall and portal frame

Photo 7 – View of top of steel vertical cross bracing on the western wall

Appendix B – Floor Plan



Revision	Amendment	Approved	Revision Date

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OPUS ARCHITECTURE

Drawn	Designed	Approved	Revision Date

Project No.	Scale
6-QUCCC.41	1 : 100 1:200 @A3

Project
Wharenui Swimming Pool

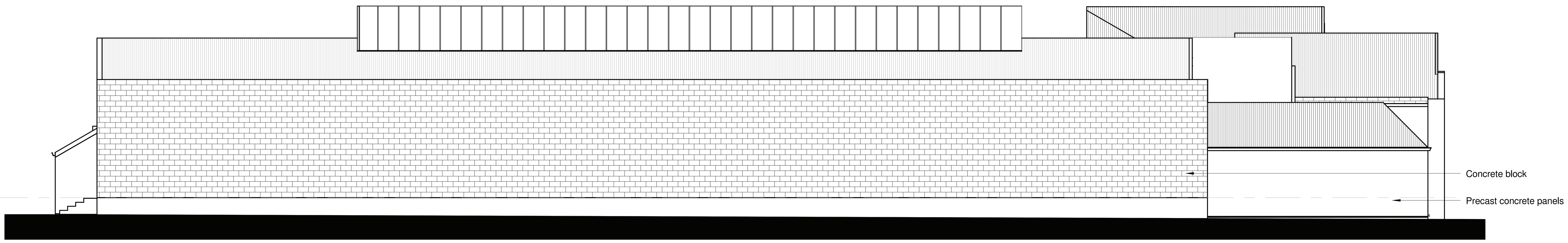
Title
Existing Floor Plan

Existing building as of 15 November 2011

Drawing No.	Sheet No.	Revision
	100	

Original sheet size A1 (841x594) Plot date: 16/11/2011 3:08:49 p.m. G:\PROMOTIONAL PROJECTS (PP)\Wharenui Swimming Pool\Existing Drawings\Wharenui Pool (2011).rvt

1 / 100 **Existing North Elevation**
1 : 100



2 / 100 **Existing East Elevation**
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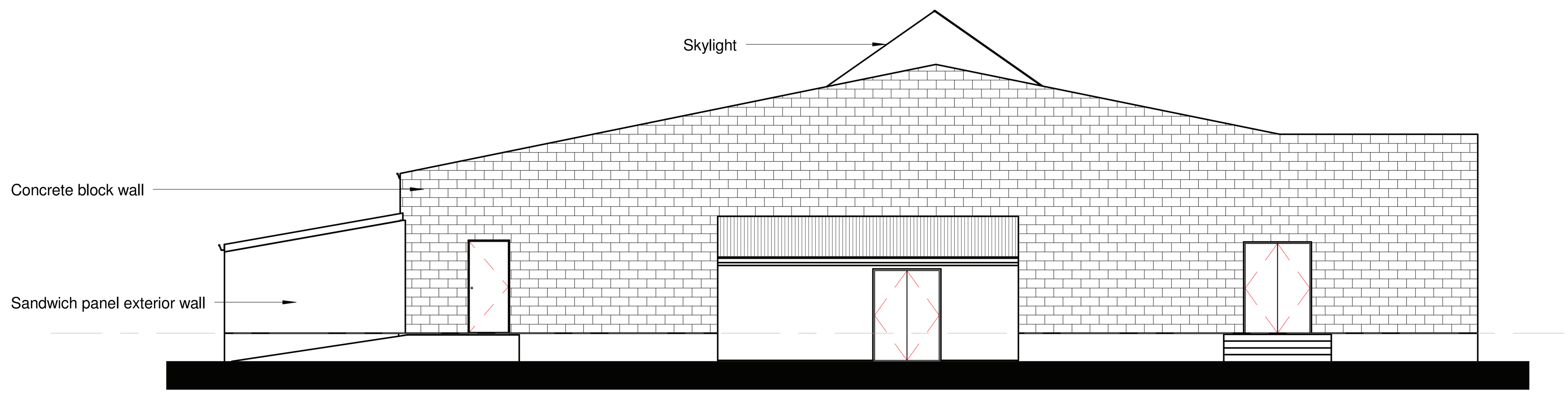
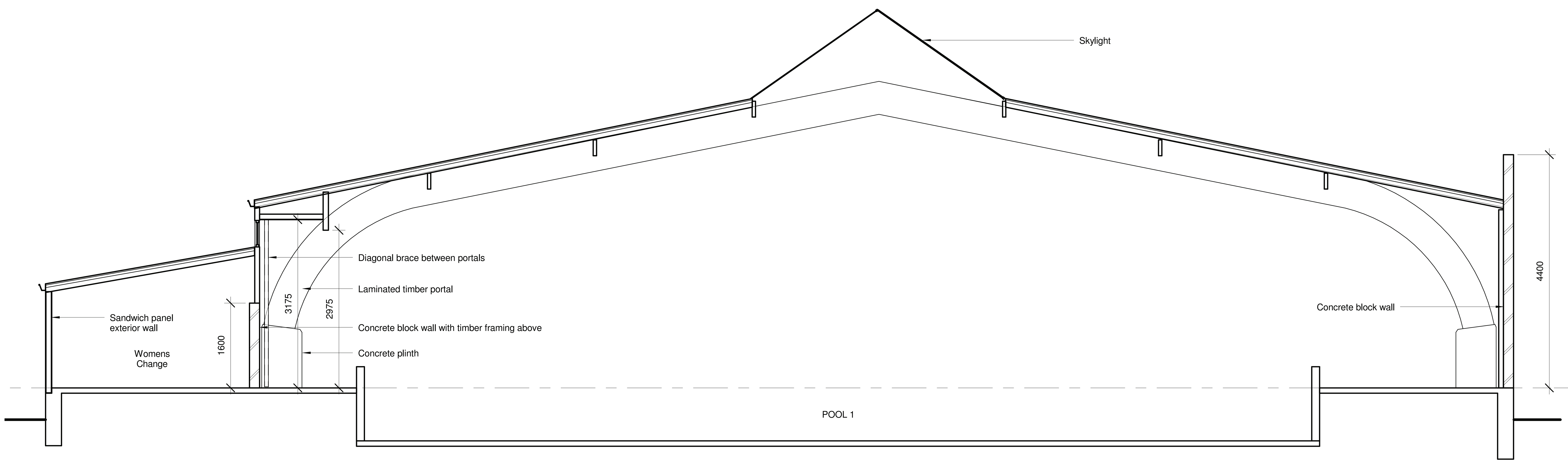


4 / 100 **Existing South Elevation**
1 : 100



3 / 100 **Section 1**
1 : 50

Existing building as of 15 November 2011

Revision	Amendment	Approved	Revision Date

OPUS ARCHITECTURE

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Drawn	Designed	Approved	Revision Date

Project No.: 6-QUCCC.41
Scale: As indicated

Project: Wharenui Swimming Pool

Title: Existing Elevations and Section

Drawing No. | Sheet No.: 101 | Revision

Original sheet size A1 (841x594) Plot date: 16/11/2011 3:11:48 p.m. G:\PROMOTIONAL PROJECTS (PP)\Wharenui Swimming Pool\Existing Drawings\Wharenui Pool (2011).rvt

300mm 200mm 100 50 10mm 0

Appendix C – CERA DEE Spreadsheets

Detailed Engineering Evaluation Summary Data V1.10

Location

Building Name: Wharenui Main Pool Building
Building Address: Unit: | No: 80 | Street: Elizabeth Street
Legal Description:
GPS south: Degrees 43 | Min 32 | Sec 8.00
GPS east: Degrees 172 | Min 35 | Sec 50.00
Building Unique Identifier (CCC): BU 0533-002 EQ2

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International Consultants
Company project number: 6-QUCCC.41
Company phone number: +64 3 363 5400
Date of submission: 27-Sep-12
Inspection Date:
Revision: Final
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: sandy silt
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m):
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):

Max retaining height (m): 0
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m):

Building

No. of storeys above ground: 1 (single storey = 1)
Ground floor split? no
Storeys below ground: 0
Foundation type: isolated pads, no tie beams
Building height (m): 8.70
Floor footprint area (approx): 1323
Age of Building (years): 17

Ground floor elevation (Absolute) (m):
Ground floor elevation above ground (m): 0.00
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m): 8.7
Date of design: 1992-2004

Strengthening present? no
Use (ground floor): public
Use (upper floors):
Use notes (if required):
Importance level (to NZS1170.5): IL2

If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Gravity Structure

Gravity System: frame system
Roof: other (note) — describe system: Glulam portals with glulam purlins and lightweight steel roof.
Floors: — Block Masonry perimeter walls
Beams: none — overall depth x width (mm x mm):
Columns: other (note) — typical dimensions (mm x mm):
Walls: non-load bearing — 0

Lateral load resisting structure

East-west
Lateral system along: other (note) — describe system: Glulam portal frames
Ductility assumed, μ: 1.25
Period along: 0.30 (0.00) — estimate or calculation? estimated
Total deflection (ULS) (mm): 75 — estimate or calculation? estimated
maximum interstorey deflection (ULS) (mm): 75 — estimate or calculation? estimated

North-south
Lateral system across: fully filled CMU — note total length of wall at ground (m): 48
Ductility assumed, μ: 1.25 — wall thickness (m): 0.1
Period across: 0.40 (0.04 from parameters in sheet) — estimate or calculation? estimated
Total deflection (ULS) (mm): 50 — estimate or calculation? estimated
maximum interstorey deflection (ULS) (mm): 50 — estimate or calculation? estimated

Separations:

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

Non-structural elements

Stairs: — no stairs
Wall cladding: other heavy — describe: unknown
Roof Cladding: Metal — describe: lightweight
Glazing: aluminium frames
Ceilings: light tiles
Services(list): Separate plant area

Available documentation

Architectural: partial — original designer name/date: Plant/amenity block alterations and extension
Structural: partial — original designer name/date: Drawings for original steel portal building
Mechanical: partial — 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: none observed — notes (if applicable):
Differential settlement: none observed — notes (if applicable):
Liquefaction: none apparent — notes (if applicable):
Lateral Spread: none apparent — notes (if applicable):
Differential lateral spread: none apparent — notes (if applicable):
Ground cracks: none apparent — notes (if applicable):
Damage to area: none apparent — notes (if applicable):

Building:
Current Placard Status: green

Along — Damage ratio: 5% — Describe how damage ratio arrived at: site observations
Describe (summary): Cracks in glulam and external wall displaced

Across — Damage ratio: 0%
Describe (summary):

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

Diaphragms — Damage?: no — Describe:
CSWs: — Damage?: no — Describe:
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: no — Describe:

Recommendations

Level of repair/strengthening required: minor structural — Describe:
Building Consent required: no — Describe:
Interim occupancy recommendations: full occupancy — Describe:

Along — Assessed %NBS before: 62% (0% %NBS from IEP below); Assessed %NBS after: 62%
Across — Assessed %NBS before: 84% (0% %NBS from IEP below); Assessed %NBS after: 84%

IEP

Age of Building (from above): 1992-2004
h_n from above: 8.7m
Seismic Zone, if designed between 1965 and 1992: B
not required for this age of building: D soft soil
Design Soil type from NZS4203:1992, cl 4.6.2.2: b) Intermediate

	along	across
Period (from above):	0.3	0.4
(%NBS)nom from Fig 3.3:	0.0%	0.0%
Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25		1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2		1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)		1.0
Final $(\%NBS)_{nom}$:	**0%**	**0%**

2.2 Near Fault Scaling Factor

	along	across
Near Fault scaling factor, from NZS1170.5, Table 3.3):		1.00
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3: 0.30
Z_{1992}, from NZS4203:1992: 0.8
Hazard scaling factor, **Factor B:** 2.666666667

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**: 1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.3)	1.00	1.25
Ductility scaling factor (if pre-1976):	1.00	1.00
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.925	0.700
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.428571429

2.7 Baseline %NBS, $(NBS\%)_b = (\%NBS)_{nom} \times A \times B \times C \times D \times E$ — $\%NBS_b$: 0% | 0%

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

3.1. Plan Irregularity, factor A: significant — 0.7
3.2. Vertical irregularity, Factor B: insignificant — 1
3.3. Short columns, Factor C: insignificant — 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 insignificant — 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	0.0	0.0

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: CMU Wall diplaced out-of-plane

3.7. Overall Performance Achievement ratio (PAR) — **0.00** | **0.00**

4.3 PAR x (%NBS)b: — PAR x Baselline %NBS: **0%** | **0%**

4.4 Percentage New Building Standard (%NBS), (before) — **0%**

Detailed Engineering Evaluation Summary Data V1.10

Location

Building Name:	Wharenui Pool Changing Block	Reviewer:	Alistair Boyce
Building Address:	Unit: / No: 80 / Street: Elizabeth Street	CPEng No:	209860
Legal Description:		Company:	Opus International Consultants
		Company project number:	6-QUCCC.41
		Company phone number:	+64 3 363 5400
GPS south:	Degrees 43, Min 32, Sec 8.00	Date of submission:	27-Sep-12
GPS east:	Degrees 172, Min 35, Sec 50.00	Inspection Date:	
		Revision:	Final
Building Unique Identifier (CCC):	BU 0533-002 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):	0.00
Storeys below ground	0		
Foundation type:	isolated pads, no tie beams	if Foundation type is other, describe:	
Building height (m):	8.70	height from ground to level of uppermost seismic mass (for IEP only) (m):	8.7
Floor footprint area (approx):	1323		
Age of Building (years):	17	Date of design:	1965-1976
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	public	Brief strengthening description:	
Use (upper floors):			
Use notes (if required):			
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	Steel beams, lightweight cladding
Floors:			
Beams:	cast-insitu concrete	overall depth x width (mm x mm)	
Columns:	structural steel	typical dimensions (mm x mm)	
Walls:	non-load bearing	0	

Lateral load resisting structure

East-west

Lateral system along:	other (note)		describe system	Unknown
Ductility assumed, μ:	0.00			
Period along:	0.00	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	0		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	0		estimate or calculation?	estimated

North-south

Lateral system across:	steel concentric braced frame		note typical frame sizes and bay length (m)	6
Ductility assumed, μ:	1.25			
Period across:	0.10	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):	20		estimate or calculation?	estimated
maximum interstorey deflection (ULS) (mm):	20		estimate or calculation?	estimated

Separations:

north (mm):
east (mm):
south (mm):
west (mm):

leave blank if not relevant

Non-structural elements

Stairs:			no stairs
Wall cladding:	profiled metal	describe	
Roof Cladding:	Metal	describe	lightweight
Glazing:	other (specify)		
Ceilings:	none		
Services(list):	Separate plant area		

Available documentation

Architectural	partial	original designer name/date	R&A Design 2004
Structural	none	original designer name/date	
Mechanical	partial	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:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:

Current Placard Status: green

Along	Damage ratio:	1%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	0%		
	Describe (summary):			
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:	no	Describe:	
Non-structural:	Damage?:	no	Describe:	

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

Recommendations

Level of repair/strengthening required:	none	Describe:	
Building Consent required:	no	Describe:	
Interim occupancy recommendations:	full occupancy	Describe:	

Along	Assessed %NBS before:	49%	0% %NBS from IEP below
	Assessed %NBS after:	49%	
Across	Assessed %NBS before:	67%	0% %NBS from IEP below
	Assessed %NBS after:	67%	

IEP

Age of Building (from above): 1965-1976

h_n from above: 8.7m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building: D soft soil
not required for this age of building: b) Intermediate

	along	across
Period (from above):	0	0.1
(%NBS)nom from Fig 3.3:	0.0%	0.0%

Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25	1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2	1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0)	1.0

	along	across
Final (%NBS)nom:	**0%**	**0%**

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, Table 3.3): 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3: 0.30
Z_{1992}, from NZS4203:1992: 0.8
Hazard scaling factor, **Factor B:** 3.333333333

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**: 1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.3)	1.00	1.25
Ductility scaling factor (if pre-1976):	1.00	1.00
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.925	0.700
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.428571429

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

	along	across
$\%NBS_b$:	**0%**	**0%**

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

3.1. Plan Irregularity, factor A:	insignificant	1
3.2. Vertical irregularity, Factor B:	insignificant	1
3.3. Short columns, Factor C:	insignificant	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

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

3.5. Site Characteristics: insignificant | 1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	0.0	0.0

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: No roof bracing, lack of vertical bracing, possible inadequate connections to masonry walls

	along	across
3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**0%**	**0%**

4.4 Percentage New Building Standard (%NBS), (before): **0%**

Detailed Engineering Evaluation Summary Data

V1.10

Location

Building Name: Wharenui Pool Plant Block

	Unit	No:	Street
Building Address:		80	Elizabeth Street

Legal Description:

	Degrees	Min	Sec
GPS south:	43	32	8.00
GPS east:	172	35	50.00

Building Unique Identifier (CCC): BU 0533-002 EQ2

Reviewer: Alistair Boyce
CPEng No: 209860
Company: Opus International Consultants
Company project number: 6-QUCCC.41
Company phone number: +64 3 363 5400

Date of submission: 27-Sep-12
Inspection Date:
Revision: Final
Is there a full report with this summary? yes

Site

Site slope: flat
Soil type: sandy silt
Site Class (to NZS1170.5): D
Proximity to waterway (m, if <100m):
Proximity to clifftop (m, if < 100m):
Proximity to cliff base (m,if <100m):

Max retaining height (m): 0
Soil Profile (if available):
If Ground improvement on site, describe:
Approx site elevation (m):

Building

No. of storeys above ground: 1 — single storey = 1
Ground floor split? no
Storeys below ground 0
Foundation type: isolated pads, no tie beams
Building height (m): 8.70
Floor footprint area (approx): 1323
Age of Building (years): 17

Ground floor elevation (Absolute) (m):
Ground floor elevation above ground (m): 0.00
if Foundation type is other, describe:
height from ground to level of uppermost seismic mass (for IEP only) (m): 8.7
Date of design: 1992-2004

Strengthening present? no
If so, when (year)?
And what load level (%g)?
Brief strengthening description:

Use (ground floor): public
Use (upper floors):
Use notes (if required):
Importance level (to NZS1170.5): IL2

Gravity Structure

Gravity System: frame system
Roof: steel framed — rafter type, purlin type and cladding: steel beams, cold rolled purlins and sheet cladding
Floors:
Beams: none — overall depth x width (mm x mm): unknown
Columns: structural steel — typical dimensions (mm x mm): unknown
Walls: non-load bearing — 0

Lateral load resisting structure

East-west
Lateral system along: single level tilt panel — note total length of wall at ground (m): 14.5
Ductility assumed, μ: 1.25 — wall thickness (m): 0.12
Period along: 0.14 — 0.00 — estimate or calculation? estimated
Total deflection (ULS) (mm): 15 — estimate or calculation? estimated
maximum interstorey deflection (ULS) (mm): 15 — estimate or calculation? estimated

North-south
Lateral system across: welded and bolted steel moment frame — note typical bay length (m): 8.4
Ductility assumed, μ: 3.00
Period across: 0.44 — 0.00 — estimate or calculation? estimated
Total deflection (ULS) (mm): 25 — estimate or calculation? estimated
maximum interstorey deflection (ULS) (mm): 25 — estimate or calculation? estimated

Separations:

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

Non-structural elements

Stairs: — no stairs
Wall cladding: precast panels — thickness and fixing type: unknown
Roof Cladding: Metal — describe: lightweight
Glazing: aluminium frames
Ceilings: strapped or direct fixed
Services(list): plant included in area

Available documentation

Architectural	partial	original designer name/date	R&A Design 1997
Structural	partial	original designer name/date	R&A Design 1997
Mechanical	partial	original designer name/date	R&A Design 1997
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:	none observed	notes (if applicable):	
Differential settlement:	none observed	notes (if applicable):	
Liquefaction:	none apparent	notes (if applicable):	
Lateral Spread:	none apparent	notes (if applicable):	
Differential lateral spread:	none apparent	notes (if applicable):	
Ground cracks:	none apparent	notes (if applicable):	
Damage to area:	none apparent	notes (if applicable):	

Building:
Current Placard Status: green

Along — Damage ratio: 1% — Describe how damage ratio arrived at:
Describe (summary):

Across — Damage ratio: 0%
Describe (summary):

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

Diaphragms — Damage?: no — Describe:
CSWs: — Damage?: no — Describe:
Pounding: — Damage?: no — Describe:
Non-structural: — Damage?: no — Describe:

Recommendations

Level of repair/strengthening required: none — Describe:
Building Consent required: no — Describe:
Interim occupancy recommendations: full occupancy — Describe:

Along — Assessed %NBS before: 62% — 0% %NBS from IEP below
Assessed %NBS after: 62%

Across — Assessed %NBS before: 84% — 0% %NBS from IEP below
Assessed %NBS after: 84%

IEP

Age of Building (from above): 1992-2004
h_n from above: 8.7m

Seismic Zone, if designed between 1965 and 1992: B
not required for this age of building: D soft soil
Design Soil type from NZS4203:1992, cl 4.6.2.2: b) Intermediate

	along	across
Period (from above):	0.14	0.44
(%NBS)nom from Fig 3.3:	0.0%	0.0%

Note:1 for buildings designed prior to 1976 as public buildings, to code at time, use 1.25: 1.00
Note 2: for RC buildings designed between 1976-1984, use 1.2: 1.0
Note 3: for buildngs designed prior to 1935 use 0.8, except in Wellington (1.0): 1.0

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

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, Table 3.3): 1.00

	along	across
Near Fault scaling factor (1/N(T,D), **Factor A**:	1	1

2.3 Hazard Scaling Factor

Hazard factor Z for site from AS1170.5, Table 3.3: 0.30
Z_{1992}, from NZS4203:1992: 0.8
Hazard scaling factor, **Factor B:** 2.666666667

2.4 Return Period Scaling Factor

Building Importance level (from above): 2
Return Period Scaling factor from Table 3.1, **Factor C**: 1.00

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.3)	1.00	1.25
Ductility scaling factor (if pre-1976):	1.00	1.00
Ductiity Scaling Factor, **Factor D**:	1.00	1.00

2.6 Structural Performance Scaling Factor:

	along	across
Sp:	0.925	0.700
Structural Performance Scaling Factor **Factor E**:	1.081081081	1.428571429

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

	along	across
$\%NBS_b$:	**0%**	**0%**

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

3.1. Plan Irregularity, factor A: insignificant — 1

3.2. Vertical irregularity, Factor B: insignificant — 1

3.3. Short columns, Factor C: insignificant — 1

3.4. Pounding potential
Pounding effect D1, from Table to right: 0.7
Height Difference effect D2, from Table to right: 1.0
Therefore, Factor D: 0.7

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

3.5. Site Characteristics insignificant — 1

3.6. Other factors, Factor F

	Along	Across
For ≤ 3 storeys, max value =2.5, otherwise max valule =1.5, no minimum	0.0	0.0

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:

	along	across
3.7. Overall Performance Achievement ratio (PAR)	**0.00**	**0.00**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**0%**	**0%**

4.4 Percentage New Building Standard (%NBS), (before) **0%**

OPUS