New Brighton Library
BU 1330-004 EQ2
Detailed Engineering Evaluation
Quantitative Assessment Report

CHRISTCHURCH CITY COUNCIL

New Brighton Library

Detailed Engineering Evaluation Quantitative Assessment Report

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Detailed Engineering Evaluation
Quantitative Report - SUMMARY
Final

213 Marine Parade, Christchurch

Background
This is a summary of the quantitative report for the building structure at 213 Marine Parade, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group in May 2012, visual inspections on 16 June 2011, 2 January 2012, 1 February 2012, 9 May 2012, and 6 August 2012, available drawings and calculations.

Key Damage Observed
Key damage observed includes:
- Cracking of the ground floor slab;
- Spalling and cracking of the internal concrete columns;
- Cracking of the concrete walls;
- Flexural cracking to the precast beams about the weak axis;
- Corrosion of the mechanical plant on the roof platform.

Critical Structural Weaknesses
In the qualitative report, the following critical structural weaknesses were identified:

a) Variation in the lateral load resisting system and wall area in the north-south direction results in the building having some vertical irregularity;

b) The internal staircase is fixed at both ends and at midpoint, and therefore attracts seismic load;

c) The diaphragm collector elements to the southern walls and northern steel braces appear to be inadequate, and the first floor diaphragm relies only upon non-ductile mesh to distribute loads to the lateral load resisting elements.

These weaknesses have been assessed to determine their effect on the seismic capacity of the building. The assessment has shown that the lateral load resisting elements for the roof level are the limiting factor for the building. It will be necessary to retrofit a strengthening solution to provide sufficient capacity between the roof diaphragm and the south walls, to provide a robust solution and to bring the seismic capacity of the building up to 67% NBS.

Indicative Building Strength (from quantitative assessment)
Based on the information available, and from undertaking a quantitative assessment, the building's original capacity has been assessed to be in the order of 36% NBS as an Importance Level 3 building, and post-earthquake capacity in the order of 36% NBS under IL3. The building is therefore classed as earthquake risk, but not earthquake prone.

Recommendations

It is recommended that:

(a) The building has a seismic capacity of 36% NBS. In accordance with NZSEE guidelines, this relates to a relative failure risk of 5-10 times that of a building constructed to the New Building Standard, and is therefore considered to pose a moderate risk to occupancy.

(b) A strengthening works scheme is developed to increase the seismic capacity of the building to at least 67% NBS; this will need to consider compliance with accessibility and fire requirements;

(c) A quantity surveyor be engaged to determine the costs for strengthening the building.

Contents

1 Introduction

Opus International Consultants Limited has been engaged by Christchurch City Council (CCC) to undertake a detailed seismic assessment of the New Brighton Library, located on 213 Marine Parade, 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) in May 2012.

2 Compliance

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

2.1 Canterbury Earthquake Recovery Authority (CERA)

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

Section 38 – Works

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

Section 51 – Requiring Structural Survey

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

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

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

1. The importance level and occupancy of the building.

2. The placard status and amount of damage.

3. The age and structural type of the building.

4. Consideration of any critical structural weaknesses.

Christchurch City Council requires any building with a capacity of less than 34% of New Building Standard (including consideration of critical structural weaknesses) to be strengthened to a target of 67% as required under the CCC Earthquake Prone Building Policy.

2.2 Building Act

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

Section 112 - Alterations

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

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

Section 115 – Change of Use

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

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

Section 121 – Dangerous Buildings

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

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

Section 122 – Earthquake Prone Buildings

This section defines a building as earthquake prone 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 October 2011 following the Darfield Earthquake on 4 September 2010.

The policy includes the following:

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

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

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

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

- Where an application for a change of use of a building is made to Council, the building will be required to be strengthened to 67% of New Building Standard or as near as is reasonably practicable.

2.4 Building Code

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

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

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

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

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

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

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

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

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

3 Earthquake Resistance Standards

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

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

Description	Grade	Risk	%NBS	Existing Building Structural Performance	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
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.

Table 1: %NBS compared to relative risk of failure

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

3.1 Minimum and Recommended Standards

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

a) Occupancy

The Canterbury Earthquake Order[i] in Council 16 September 2010 modified the meaning of "dangerous building" to include buildings that were identified as being Earthquake Prone Buildings. As a result of this, we would expect such a building would be issued with a Section 124 notice by the Territorial Authority, or CERA acting on their behalf, once they are made aware of our assessment. Our understanding, based on information received from CERA, is that this notice would prohibit occupancy of the building (or parts thereof), until its seismic capacity is improved to the point that it is no longer considered an Earthquake Prone Building.

b) Cordoning

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

c) Strengthening

Industry guidelines (NZSEE 2006 [3]) strongly recommend that every effort be made to achieve improvement to at least 67%NBS. A solution to anything less than 67% would not provide an adequate reduction to the level of risk.

It should be noted that full compliance with the current building code requires building strength of 100%NBS.

d) Our Ethical Obligation

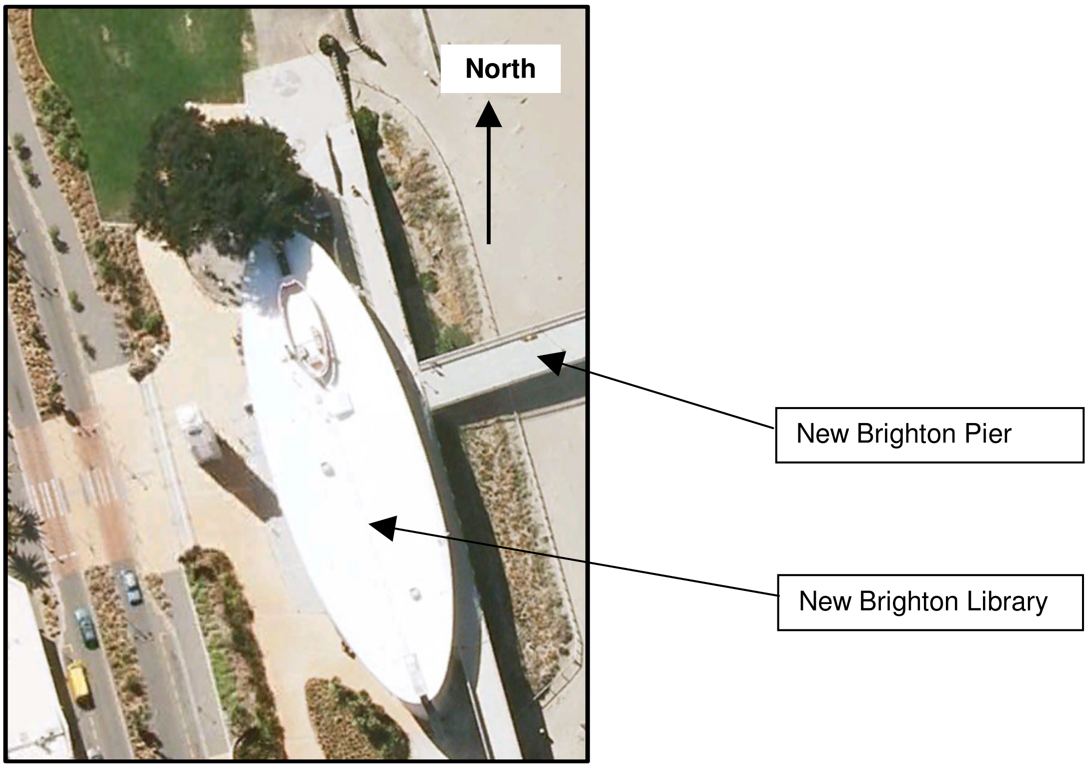
In accordance with the IPENZ code of ethics, we have a duty of care to the public. This obligation requires us to identify and inform CERA of potentially dangerous buildings; this would include earthquake prone buildings.

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

4 Building Description

4.1 General

The New Brighton Library is located along the waterfront of New Brighton, on Marine Parade. It forms the terminus of the New Brighton Pier, and was completed in 1999. It is a two storey structure, constructed from a combination of reinforced concrete frames, walls, and steel beams. The building has an elliptical shape, with a long dimension of 63m in the north-south direction, and a short dimension of 16m in the east-west direction. The ground floor also extends a further 3 – 5m beyond this on the east side to form a 40m long raised promenade. This is accessible from the north by a staircase, and from the south by a ramp, and it provides access to the pier on the east. There is a roof plant deck approximately 9m above ground level at the north end of the building, and a basement plantroom floor is approximately 2m below ground level at the southern end of the building.



The building is separated internally into two sides, with the northern end of the building housing a cafe on both floors, and the remainder of the building housing the library and entrance areas on both floors. The first floor of the library area is only a part floor, with the west side being open to ground floor, interrupted only by a steel gantry running north-south, and a steel staircase providing further access to the first floor from below.

The north access to the promenade consists of a staircase leading up to a 20m by 4m access ramp. The stair is composed of two precast stair units connected via a 3.5m long cast in-situ landing. Support is provided by precast wall panels.

The building is bounded by the ramp and toilet block to the south, Marine Parade to the west, open space to the north, and the seaside to the east. For the purposes of this report, we will refer to the direction parallel to Marine Parade as the north-south direction, and perpendicular to Marine Parade as the east-west direction.

The foundations consist of reinforced concrete pad footings and ground beams.

Refer to Appendix A for photographs of the building and Appendix C for a floor plan.

4.2 Gravity Load Resisting System

The gravity load resisting system consists of a combination of steel beams and columns, and concrete beams, columns and walls. The roof is constructed from a 17.5mm plywood diaphragm spanning between 200mm Dimond Hi-Span steel purlins at 600mm centres. These purlins span north to south between steel beams, which rest on steel posts, steel columns, and concrete walls around the perimeter of the building, and concrete columns within the building. The roof at the north end of the building is cantilevered approximately 6.5m beyond the external wall line. There is also an exposed plant platform above the roof, consisting of a 150mm reinforced concrete slab supported by steel framing above the roof framing, and cantilevering approximately 1.5m to the east and west.

The first floor is constructed from 75mm thick precast Unispan units with a 125-225mm thick insitu topping reinforced with D212 mesh throughout spanning between precast concrete beams, and in some cases a concrete wall. The seating length of the Unispan units is 50mm. The concrete beams typically span east to west between concrete columns and/or walls. On the north end of the building, in order to provide a clear storey at ground floor level, the shaped perimeter concrete beam is supported at approximately 1.6m centres by 150 x 100mm RHS posts.

The gantry at first floor level is constructed from 75mm thick precast Unispan units with a 50mm insitu topping reinforced with D147 mesh and H12 bars at 200 centres in the long direction. The Unispan units span between 380PFC beams hung from the roof at 7m centres and fixed into precast beams at either end.

The ground floor consists of a 125mm concrete slab on grade, reinforced with D212 mesh.

The northern ramp is constructed from 200mm Dycore Units spanning 7m, with a varying topping thickness of minimum 90mm, reinforced with D212 mesh and additional H10 steel over the supports to provide continuity. The seating provided to the Dycore units is 50mm.

4.3 Seismic Load Resisting System

a) North-South direction

The load from the plant platform is braced by two diagonal steel posts to roof level. At roof level the roof load is distributed by the timber diaphragm to two 200mm thick curved concrete walls on the south end of the building, and two 219x8.2 CHS steel braces to the north. The roof is connected to the southern walls via 12mm Trubolts wherever the rafters or purlins cross the wall, and to the northern steel braces via a 100x100mm timber plate bolted to M12 threaded studs welded to the steel tie beam.

Load from the first floor itself is distributed by diaphragm action in the slab to the walls on the south side of the building and to a wall at the north end of the building via H12 starters at 150mm centres. The load from the slab in the north-west corner of the building is transferred through the gantry to the walls on the south side of the building.

The internal steel staircase is fixed at both ends, and so may unintentionally provide some bracing to the first floor. The stringers are 230PFC members with 10mm plate welded to the open side. They span approximately 9m horizontally from the ends to a 250RHS in the centre, cantilevering 3.3m from the side of one of the columns in the middle of the building.

The northern access ramp is resisted by the main library building to the south, and strut action through the stairs to the foundation on the north side.

b) East-West direction

The load from the plant platform is braced by diagonal steel posts down to the steelwork at roof level. The roof load is distributed by the plywood roof diaphragm to the cantilevered concrete columns in the centre of the building. The connection between the two is at the top of the column, and consists of 8 M16 bolts cast into the top of the column.

Load from the first floor is distributed by the concrete floor diaphragm to the precast concrete walls at both ends of the building and the concrete frames distributed through the centre. The northernmost column is not part of a frame system, but, along with the mass further north is restrained by arch action in the floor plane through the floor slab and a deep spandrel beam back to transverse walls 16m from the northern end of the building.

The northern access ramp load is resisted in the east-west direction by spanning to the adjacent precast walls.

4.4 Survey

a) Post 22 February 2011 Rapid Assessment

An initial structural assessment of the building was undertaken on 16 June 2011 by Opus International Consultants. This assessment focused on cracking in the ground floor slab.

b) Further Inspections

Further investigations have since been undertaken by Opus International Consultants on 2 January 2012, 1 February 2012, 5 May 2012, and 6 August 2012.

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

4.5 Existing Documentation

Copies of the following drawings were provided by the CCC:

- Pier Terminus New Brighton Library for the Christchurch City Council, consent issue, structural drawings (Powell Fenwick Consultants Ltd.) dated 20 April 1998 and stamped for building consent.

- Pier Terminus New Brighton Library for the Christchurch City Council, construction set, architectural drawings (Barclay Architects Ltd.) dated 17 August 1998.

These drawings were used to confirm the structural systems, investigate potential critical structural weaknesses (CSW's) and identify details which required particular attention.

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

5 Damage Assessment

Some structural and non-structural damage has occurred. Repairs have been made to some structural and non-structural elements. Refer to Appendix A for photos of the damage. The following structural damage has been noted:

5.1 Ground floor slab

The ground floor of the library has cracks of up to 3mm wide extending down the centre of the building along a floor joint in the north-south direction from the lift pit. There are also other cracks in the floor slab not along a joint. These cracks typically do not extend very far. Slight differential vertical movement has also been observed across the cracks.

5.2 Central columns

The corners on the east side of most of the central columns in the library have begun to spall at floor level. The spalling pattern is very steep, and does not appear to extend below floor level. There is also a significant crack (up to 7mm wide) propagating down from a connection at the top of the column on gridline 4 on both sides, at the north end of the building.

5.3 External walls

The building sustained cracks to the external concrete walls around the building. Rust bleeding was observed from these cracks. These cracks have since been repaired by epoxy injection.

5.4 Precast beams

The visible portion of the precast beams running east-west across the building in the library show signs of flexural cracking on the vertical face of the beams due to weak-axis bending. These cracks are typically up to approximately 0.5mm wide, decreasing to nothing part-way across the underside of the beam. This may indicate that some softening of the floor diaphragm may have occurred. It will be necessary to remove the floor linings above to confirm whether the topping slab has been damaged.

5.5 Corrosion of plant

On the plant platform there has been extensive corrosion damage to the mechanical and electrical plant items. This is not seismic damage, but should be attended to before it affects the performance of the systems or causes a safety hazard.

6 General Observations

The building design is architecturally driven and incorporates non-standard design elements and a range of structural systems to resist applied loads. In order for this to work well, care must be taken with the detailing. The detailed quantitative assessment has shown that some of the detailing in the structural drawings lacks resilience and robustness.

Overall, the building has performed well during the recent earthquakes. The visible damage observed during our inspections was relatively minor. Of concern is the increased damage to the structure resulting from corrosion, as the building is very close to the sea. It is understood that this is why the repair programme was implemented before the full building analysis has been completed.

7 Detailed Seismic Assessment

The detailed seismic assessment has been based on the NZSEE 2006 [3] guidelines for the "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes" together with the "Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure" [4] draft document prepared by the Engineering Advisory Group in May 2012, and the SESOC guidelines "Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes" [6] issued on 21 December 2011.

7.1 Critical Structural Weaknesses

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:

a) Lack of load collector elements to lateral load resisting elements

The drawings do not show sufficient load collector elements at the connection between the roof diaphragm and the curved concrete walls to the south, or to the steel braces to the north. We also note that the first floor diaphragm is relied upon to distribute the lateral loads at that level to the resisting elements, but it has not been specifically detailed to carry the loads. It relies upon non-ductile mesh, although it does have ductile starter bars between the wall elements and the slab.

b) Fixed staircase

The internal steel staircase is fixed at both first and ground floor level. This will cause it to take some load when the building moves during an earthquake. This will result in a concentrated eccentric load at all attachment points. This would also result in an eccentric load being applied part way up the column off which the mid-way landing is supported, and may result in hinging of the stringers. Being of steel construction, collapse of the staircase is not considered to be likely; however, levels of damage at and around the connection of the staircase may be increased.

c) Non-ductile precast panel connections

The precast cladding panels on the east side of the building and forming the balustrade to the northern stairs are attached to the building by non-ductile connections. If they are required to undergo large displacements due to the movement of the building during an earthquake, they may fail in a brittle manner and be a falling hazard.

d) Insufficient precast flooring unit seating length

The Dycore flooring units forming the ramp at the north end of the building have a specified seating length of 50mm. This is less than the minimum seating length of 75mm now required for such units. The reason for the larger seating requirement involves a number of factors, and if the building is designed for only nominal ductility in the critical direction and any construction tolerances were minimised, this may not be an issue.

e) Short columns

There are “short” columns on the ground floor level of the east face of the building. The lower 2.2m of the columns have infill cast in-situ wall fixed into the columns with H16 bars at 150mm centres. The upper part 1.4m of the columns have infill precast panels, fixed to the columns with two weld plates forming a 15mm gap on each side. The exposed column in the middle is 1.7m high, 32% of the total height of the column. This applies to seven of the eight columns on that face of the building at that level. The face is bounded by the pre-existing buttress to the north, and a wall to the south. Together, the buttress and wall make up 35% of the length of the face.

7.2 Intrusive Inspection

As part of the qualitative report previously completed for this building, it was recommended that linings on the first and ground floor be removed so that any potential cracks above the first floor beams and the known cracks around the central columns could be investigated. As a result of this investigation it was concluded that the damage in those locations was minimal.

In light of the results of the quantitative analysis, it was deemed necessary to undertake an invasive inspection of several critical structural elements. A summary of these invasive inspections is presented below.

a) South wall to purlin connection

We observed that there was not a purlin where expected from the structural drawings on the return section of the south-west wall. Instead, there was timber framing. It is unclear what connection the timber framing had to the wall.

On the south-west wall, the M12 Trubolts connecting the purlin to the wall had been installed on an angle to the purlins. Unlike on the structural drawings, where the purlins are shown flat across the top of the wall, the purlins are actually installed at an angle to the wall to align with the sloping roof. The Trubolts therefore were installed at an angle to the purlin onto the edge of the wall. It also appears that due to this the Trubolts have been installed closer to the edge of the wall, rather than installed in the middle of the wall, which will reduce their capacity. Three Trubolt fixings to the south-west wall were looked at, and all appeared the same. No damage to the fixings or other elements was observed.

The connection between the rafter and the south-east wall was also inspected. The nut at this location had not been fully tightened onto the bolt, and there was some rust on the fixing and steel. No damage was observed. The fixing on only one side of the rafter could be observed.

b) Top of gridline 4 central column

There is a crack propagating down the west side of the north face of the column. The crack is 7mm wide at the top of the column. At the top of the column, on the north side of the rafter, the nuts are not tight onto the bolts, but the bolts appear intact.

On the southern side of the column there was also a crack propagating down from the top, and of a similar size, but skewed off to the side of the column. We did not fully explore this crack or the connections above.

There is a small crack (approximately 0.1mm wide) across the south side of the concrete column on gridline 3 at approximately mind-height.

c) South-west steel brace to roof framing connection.

This connection differed to the consent issue structural drawings. The steel brace was fixed to the underside of the rafter via a fully welded connection rather than a bolted connection. There was no damage observed to any part of this connection or the horizontal collector elements.

Photos from the intrusive inspection are shown in Appendix A.

7.3 Quantitative Assessment Methodology

The assessment assumptions and methodology have been included in Appendix B due to the technical nature of the content. A brief summary follows:

A 3D model of the building was created in ETABS, a finite element structural analysis programme.

Static analyses were carried out using the spectral values established from NZS1170.5, with an updated Z factor of 0.3 (B1/VM1), and as an Importance Level 3 building. The CCC have advised that the building has a permitted occupancy of greater than 300 people which results in the building being classed as an Importance Level 3 structure. These analyses were used to establish the actions on the structural elements. Based on the actions determined from the analyses, an assessment of the building capacities was made.

Axial-moment analyses were carried out for the columns in Gen-Col, a computer analysis programme.

7.4 Limitations and Assumptions in Results

Our analysis and assessment is based on an evaluation 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. Approximations 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.

7.5 Quantitative Assessment

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

Table 2: Summary of Seismic Performance – μ = 2

Structural Element/System	**Failure mode or description of limiting criteria based on elastic capacity of critical element.**	**% NBS based on calculated capacity**
Central Concrete Columns	Softening of roof-framing to column connection (note that this is based on failure of the steel fixings not the concrete)	48%
	Flexural failure in the east-west direction	93%
	Flexural failure in the weak axis (assuming full failure of all south wall connections)	89%
	Flexural failure in the weak axis (assuming full failure of all south wall connections and the steel braces)	29%
Central Concrete Beams	Flexural failure in the strong axis	89%
Steel Braces	Axial compression and tension (with no failure of south walls)	>100%
	Assuming full failure of south-west wall connections	98%
	Assuming full failure of all south wall connections	36%
South Walls	Flexural failure at the base (north-south loads)	46%
South Walls	Collector elements to southern walls: • First failure: south-west wall • Second failure: south-east wall (with increased demand)	21% 34%
Roof Diaphragm	In plane flexure	>100%
First Floor Diaphragm	In plane flexure	>100%
East Wall	Shear	85%
East (Short) Columns	Flexure, shear	>100%
Other Walls	Shear (west wall on gridline 4)	48%
	Bending	93%
Non-ductile precast panel connections	Weld shear capacity (incorporating a factor of 2/1.25 on the demand in consideration of the nominally ductile nature of the weld connection)	54%
Precast flooring seating length	Loss of seating	67%

7.6 Discussion of Quantitative Assessment Results

From nearby seismic records it is apparent that the building would have been subjected to near full design level seismic shaking. Consistent with the analysis, the building performed well at that level of shaking.

In general, the analytical results are consistent with the site observations. The analysis found that that the building elements generally have sufficient capacity, and there was limited damage found to the building on site. The analysis indicated that there would be a high demand on the connections at the top of the column on gridline 4 (due to the stiffness provided to the lower half of the column from the adjacent wall), and this was the column that cracked. The analysis also showed that the roof diaphragm has enough lateral resistance without contribution from the purlin-to-wall fixings on the south-west concrete wall.

The model used did not predict some of the key damage observed on site: the cracking at the base of the central concrete columns; and the weak-axis bending of the precast concrete beams. A more detailed analysis, focusing specifically on those elements and the surrounding details, may enable these effects to be reproduced and the exact reason for the damage to be determined. This would then enable a suitable retrofit solution to be developed. This analysis is beyond the scope of this assessment, but we recommend that it be undertaken. In the interim, we recommend that the beam cracks be repaired to restore their stiffness and to provide corrosion protection.

The pier interaction with the building was checked specifically, but the deflections there from the building were found to be negligible in comparison to the available seismic gap.

The pier buttress was not modelled in this assessment. The drawings indicate that there is no structural tie between it and the building. On the sides, there is a 15mm gap filled with a flexible sealant between it and the adjacent structure, and above it there is a bearing pad on which the first floor sits.

The top of the central concrete columns are connected to the roof framing by pairs of M16 bolts. At first this was modelled as a full moment-resisting connection. However, the capacity of the fixings was inadequate to develop the full moment, so it was re-modelled as a pinned connection. The moment capacity of the columns, which was enough to resist the moment developed at the capacity of the fixings, is reported under the reduced moment resulting from the connection being modelled as pinned.

Initially, the south-west wall was modelled as shown on the structural drawings, with the top connected to the roof framing via the purlins. From this analysis, the capacity of that connection was shown to be 21% NBS. The connection was then remodelled as disconnected at that location to simulate what would happen if the connection failed. Under this model, the additional load was redistributed to the other lateral load resisting elements. The capacity of the fixings at the south-east wall was calculated to be 34% NBS, taking into account the higher demands. The building was then remodelled with the fixings on both the south-west and south-east walls disconnected. The roof was then relying on the steel braces and central concrete columns bending about the weak axis. From this analysis, the capacity of the steel braces was critical, at 36% NBS. The building was then remodelled with the fixings on the south walls disconnected and without the steel braces. The roof was then relying solely on the central concrete columns bending about the weak axis. From this analysis, the capacity of the building was at 29% NBS.

The lack of collectors between the roof diaphragm and the south walls is not a robust design and we recommend that this be addressed.

The building has been found to have a seismic capacity of 36% NBS, based on the capacity of the steel braces at the north end of the building assuming failure of the diaphragm connection to the two southern walls.

The building is therefore not considered as being earthquake prone in accordance with the Building Act 2004. The building has a relative risk of failure of between 5-10 times that of a building designed and constructed to the New Building Standard. It is therefore recommended that the building be strengthened to at least 67% NBS.

8 Summary of Geotechnical Appraisal

A geotechnical desktop study was completed as part of the qualitative stage of the DEE of this building. It concluded that while there is some risk of liquefaction at the site the current foundations are considered to be adequate. No further investigation was considered to be required for the quantitative assessment. It also recommended that the floor slab cracking be subject to further inspection once the carpets have been lifted.

Refer to Appendix E for the full geotechnical report.

9 Conclusions

(a) The seismic performance of the building is governed by the capacity of the lateral load resisting elements of the roof, which have an expected capacity of 36% NBS. The building is therefore not considered to be earthquake prone in accordance with the Building Act 2004.

(b) Strengthening the building to at least 67% NBS is recommended.

10 Recommendations

(a) The building has a seismic capacity of 36% NBS. In accordance with NZSEE guidelines, this relates to a relative failure risk of 5-10 times that of a building constructed to the New Building Standard, and is therefore considered to pose a moderate risk to occupancy.

(b) Develop a strengthening works scheme to increase the seismic capacity of the building to at least 67% NBS; this will need to consider compliance with accessibility and fire requirements.

(c) A quantity surveyor is engaged to determine the costs for strengthening the building.

11 Limitations

(a) This report is based on an inspection of the structure of the buildings and focuses on the structural damage resulting from the 22 February 2011 Canterbury Earthquake and aftershocks only. Some non-structural damage is described but this is not intended to be a complete list of damage to non-structural items.

(b) Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time.

(c) This report is prepared for CCC to assist with assessing the remedial works required for council buildings and facilities. It is not to be relied upon or used out of context by any other party without further reference to Opus International Consultants.

12 References

[1] New Brighton Library, *Detailed Engineering Evaluation, Stage One Qualitative Report*, prepared by Opus International Consultants for Christchurch City Council, April 2012.

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

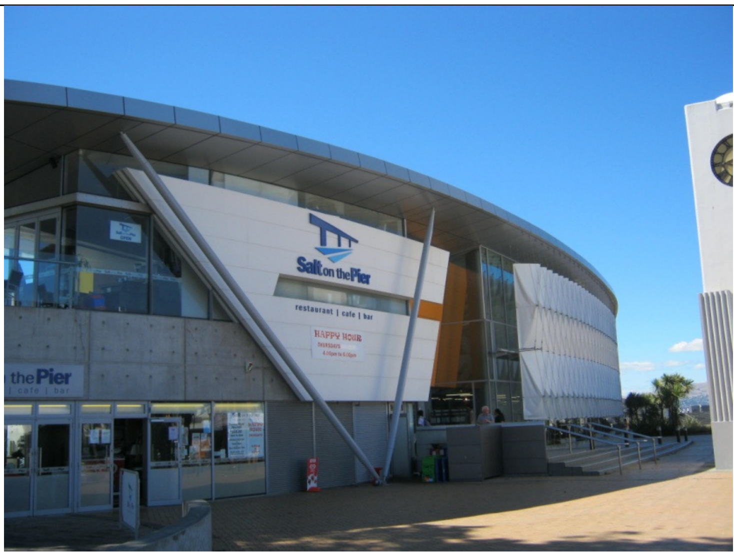
[3] NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.

[4] Engineering Advisory Group, *Guidance on Detailed Engineering Evaluation of Earthquake Affected Non-residential Buildings in Canterbury, Part 2 Evaluation Procedure*, Draft Prepared by the Engineering Advisory Group, Revision 7, May 2012.

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

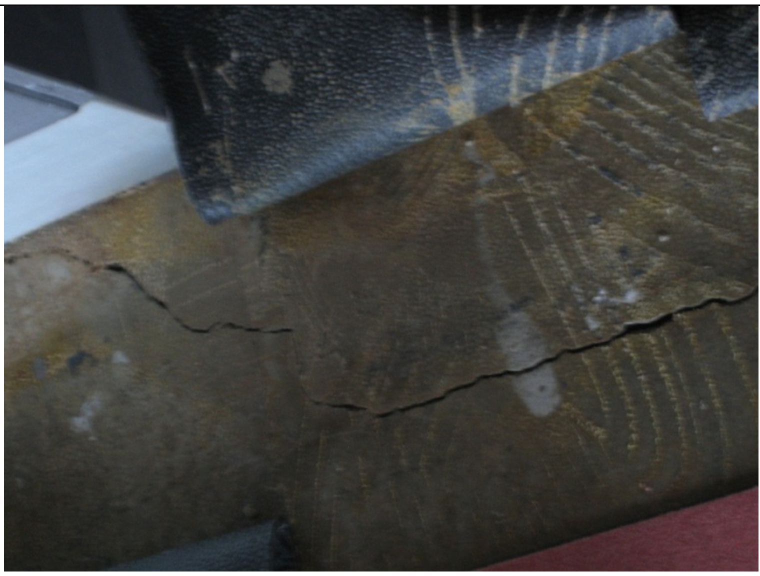
[6] SESOC, *Practice Note – Design of Conventional Structural Systems Following Canterbury Earthquakes*, Structural Engineering Society of New Zealand, 21 December 2011.

Appendix A - Photographs

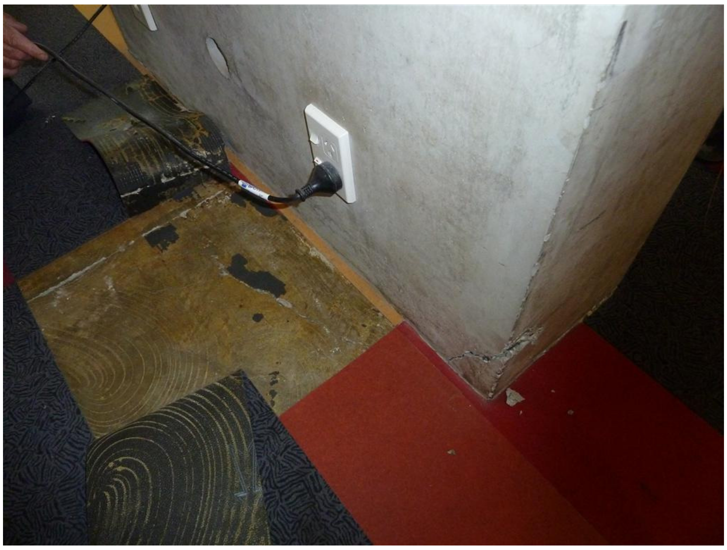
New Brighton Library Photographs		
No.	**Item description**	**Photo**
1.	West elevation	
2.	North elevation	

3.	East elevation	
4.	South elevation	

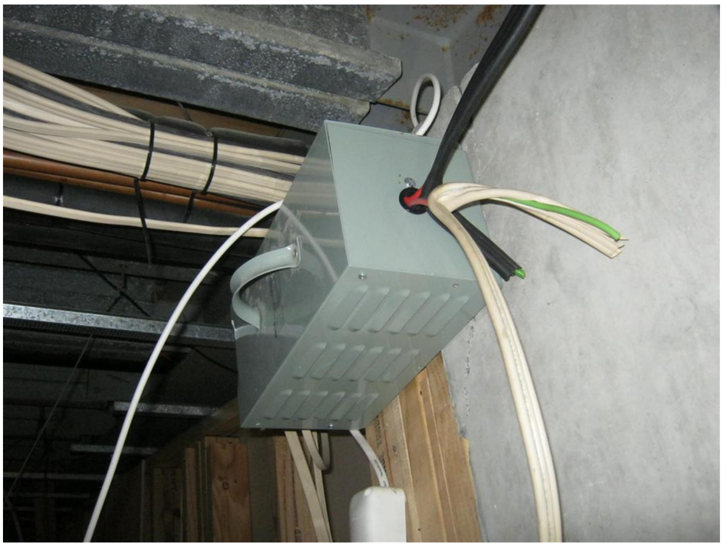
5.	Internal view of library showing cantilevered columns and internal staircase	
6.	Basement	

7.	Damage to concrete columns	
8.	Damage to floor slab	

9.	Damage to concrete beams	
10.	First floor diaphragm above precast concrete beam	

11.	Ground floor cracking around concrete beams angling back to central movement joint	
12.	Extensive corrosion of plant on roof	
13.	12mm Trubolt purlin connection to south-west wall	

14.	12mm Trubolt connection from rafter to south-east wall	
15.	Crack propagating from top of central concrete column on gridline 4	

16.	16mm bolt connection from which the crack propagates on the central concrete column on gridline 4	
17.	Loose nut on bolt rafter to central concrete column connection	
18.	Crack in top corner of concrete column	

19.	Steel brace to roof framing connection	
20.	Plywood diaphragm with timber collector element to horizontal steel brace	

Appendix B - Quantitative Assessment Methodology and Assumptions

A2.1. Referenced Documents

- AS/NZS 1170.0:2002, *Structural design actions, Part 0: General principles,* Standards New Zealand.
- AS/NZS 1170.1:2002, *Structural design actions, Part 1: Permanent, imposed and other actions,* Standards New Zealand.
- NZS1170.5:2004, *Structural design actions, Part 5: Earthquake actions* – New Zealand, Standards New Zealand.
- NZS 3101: Part 1:2006, *Concrete Structures Standard, The Design of Concrete Structures,* Standards New Zealand.
- NZS3101: Part 2:2006, *Concrete Structures Standard, Commentary on the Design of Concrete Structures,* Standards New Zealand.
- NZSEE: 2006, *Assessment and improvement of the structural performance of buildings in earthquakes,* New Zealand Society for Earthquake Engineering.
- 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.

A2.2. Analysis Parameters

The following parameters are used for the seismic analysis:

- Site Soil Category D (deep and soft soil);
- Seismic Hazard Factor Z = 0.3;
- Return Period Factor R_u = 1.3 (Importance Level 3 structure, 50 year design life);
- Ductility Factor $\mu = 2$;
- Structural Performance Factor S_p = 0.7.

A2.3. Material Properties

Table A3.1: Analysis Material Properties

Concrete nominal compressive strength, f'_c (MPa) (1)	45
High strength reinforcing nominal yield strength, f_y (MPa) (2)	464

Notes:
1. Based on guidance from *NZSEE 2006,* probable concrete compressive strength is based on a value of 1.5 times the nominal compressive strength (Cl. 7.1.1)
2. Based on guidance from *NZSEE 2006,* probable reinforcement yield strength is based on a value of 1.08 times the nominal yield strength (Cl. 7.1.1)

A2.4. Effective Section Properties

Table A3.2: Effective Section Properties from NZS 3101:2006

Type of member	Ultimate limit state		Serviceability limit state		
	f_y = 300 MPa	f_y = 500 MPa	μ = 1.25	μ = 3	μ = 6
1 Beams					
(a) Rectangular ¶	0.40 I_g (use with E_{40}) §	0.32 I_g (use with E_{40}) §	I_g	0.7 I_g	0.40 I_g (use with E_{40}) §
(b) T and L beams ¶	0.35 I_g (use with E_{40}) §	0.27 I_g (use with E_{40}) §	I_g	0.6 I_g	0.35 I_g (use with E_{40}) §
2 Columns					
(a) $N^*/A_g f'_c > 0.5$	0.80 I_g (1.0 I_g) ‡	0.80 I_g (1.0 I_g) ‡	I_g	1.0 I_g	As for the ultimate limit state values in brackets
(b) $N^*/A_g f'_c = 0.2$	0.55 I_g (0.66 I_g) ‡	0.50 I_g (0.66 I_g) ‡	I_g	0.8 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.40 I_g (0.45 I_g) ‡	0.30 I_g (0.35 I_g) ‡	I_g	0.7 I_g	
3 Walls ¶					
(a) $N^*/A_g f'_c = 0.2$	0.48 I_g	0.42 I_g	I_g	0.7 I_g	As for the ultimate limit state values
(b) $N^*/A_g f'_c = 0.1$	0.40 I_g	0.33 I_g	I_g	0.6 I_g	
(c) $N^*/A_g f'_c = 0.0$	0.32 I_g	0.25 I_g	I_g	0.5 I_g	
4 Diagonally reinforced coupling beams	0.6I_g for flexure Shear area, A_{shear}, as in text		I_g 1.5 A_{shear} for ULS	0.75 I_g 1.25 A_{shear} for ULS	As for ultimate limit state

NOTES –
(§) With these values the E value should be the elastic modulus for concrete with a strength of 40 MPa regardless of the actual concrete strength.
(‡) The values in brackets apply to columns which have a high level of protection against plastic hinge formation in the ultimate limit state.
(¶) For additional flexibility, within joint zones and for conventionally reinforced coupling beams refer to the text.

A2.5. Assessment Methodology

Equivalent Static Analysis

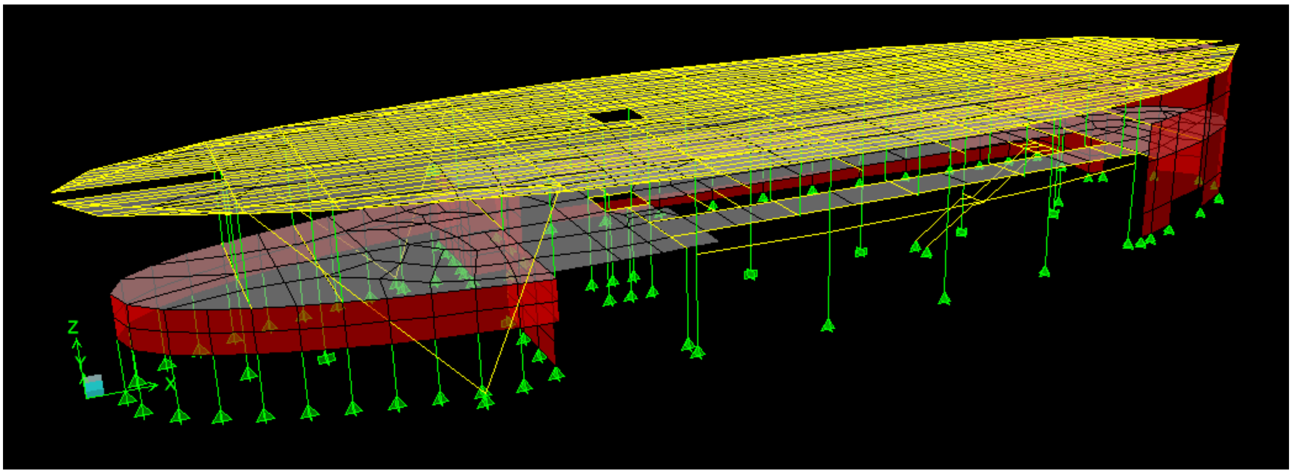


Figure A3.1: ETABS Model of the New Brighton Library

The more significant building modes of free vibration outputted from ETABS are:

T_3 = 0.22 seconds (torsional mode);

T_{10} = 0.12 seconds (longitudinal mode);

T_{11} = 0.12 seconds (transverse mode).

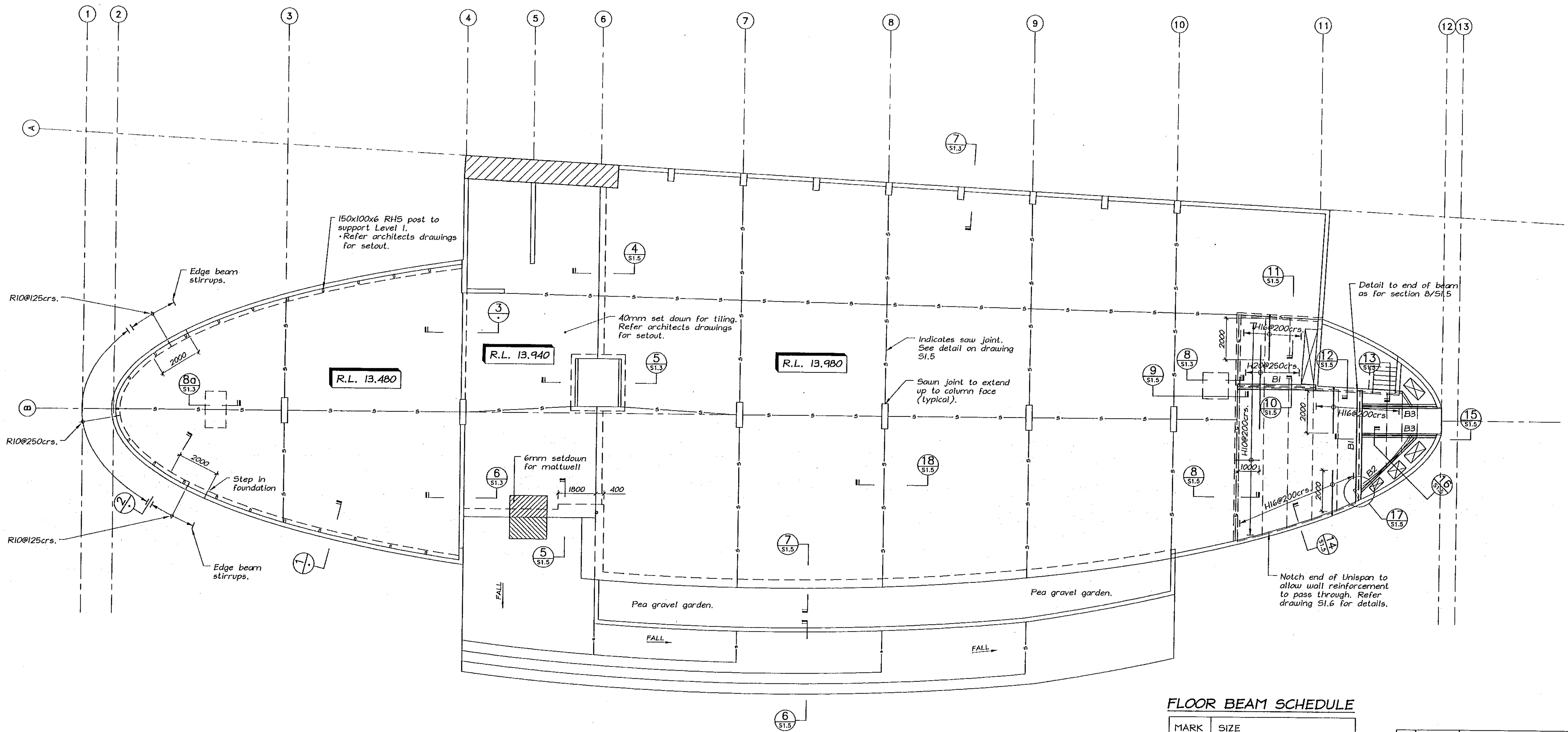
T_{12} = 0.10 seconds (second torsional mode).

The building was analysed as being ductile (μ = 2.0)

Element force demands were extracted from the equivalent static analysis and compared to calculated capacities based on the material properties assumed in Table A3.3. The results of these demand to capacity checks are summarised in further detail in the report and presented as %NBS.

The pier interaction with the building was checked specifically, but the deflections there from the building were found to be negligible in comparison to the available seismic gap.

Appendix C - Floor Plans



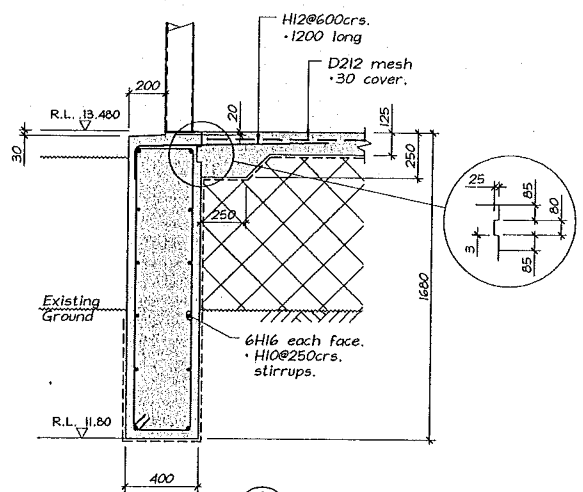
GROUND FLOOR PLAN

1:100

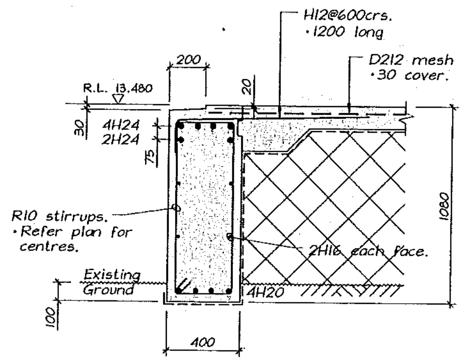
- Refer architects drawings for all setout dimensions.
- 125mm slab on hardfill generally. 75mm Unispan with 160mm topping over basement.
- Reinforce slab with D212 mesh throughout.
- Stop mesh 50mm either side of sawn joints.
- Additional reinforcement shown on plan and in section.
- 30mm cover to reinforcement internally 45mm cover to reinforcement externally
- Lay top bar of mesh parallel to span direction.
- ——5——5—— Indicates saw cut location. Refer S1.5 for details.

FLOOR BEAM SCHEDULE

MARK	SIZE
B1	250UC73
B2	250x90 channel
B3	250x90 ch. & 100x100x10 an.



1 1:20



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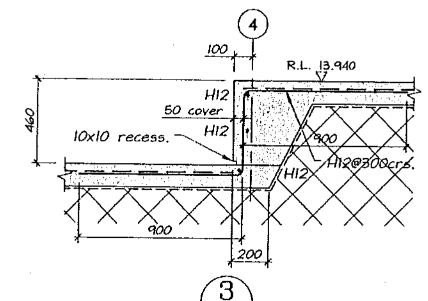

3 1:20

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2	06.04.98	Coordination Issue	
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GROUND FLOOR PLAN

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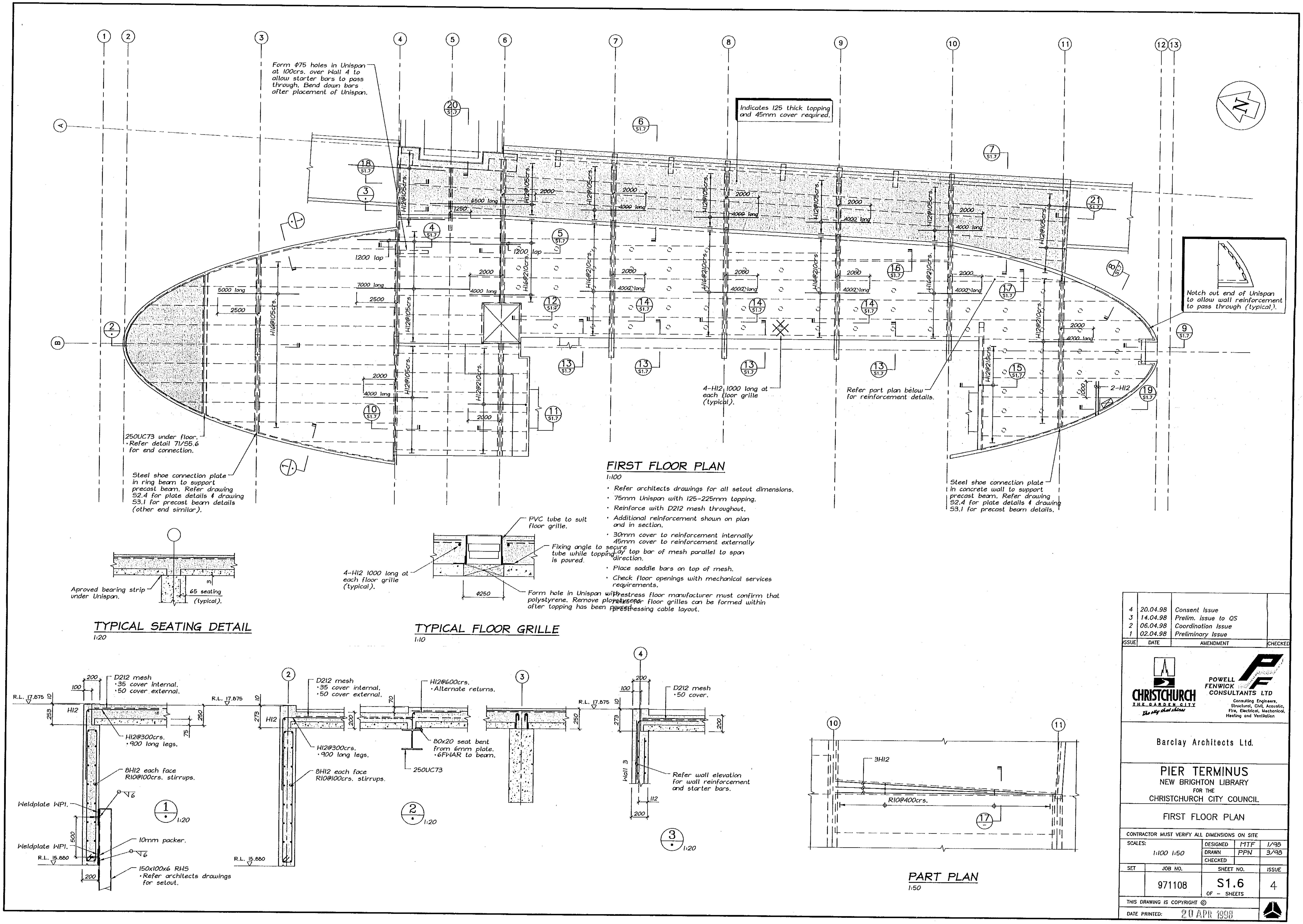
FIRST FLOOR PLAN
1:100
• Refer architects drawings for all setout dimensions.
• 75mm Unispan with 125-225mm topping.
• Reinforce with D212 mesh throughout.
• Additional reinforcement shown on plan and in section.
• 30mm cover to reinforcement internally
45mm cover to reinforcement externally
• Lay top bar of mesh parallel to span direction.
• Place saddle bars on top of mesh.
• Check floor openings with mechanical services requirements.
• Prestress floor manufacturer must confirm that holes for floor grilles can be formed within prestressing cable layout.
Form Ø75 holes in Unispan at 100crs. over Wall 4 to allow starter bars to pass through. Bend down bars after placement of Unispan.
Indicates 125 thick topping and 45mm cover required.
Notch out end of Unispan to allow wall reinforcement to pass through (typical).
250UC73 under floor. Refer detail 71/S5.6 for end connection.
Steel shoe connection plate in ring beam to support precast beam. Refer drawing S2.4 for plate details & drawing S3.1 for precast beam details (other end similar).
4-H12, 1000 long at each floor grille (typical).
Refer part plan below for reinforcement details.
Steel shoe connection plate in concrete wall to support precast beam. Refer drawing S2.4 for plate details & drawing S3.1 for precast beam details.
H12@105crs.
H16@105crs.
H12@210crs.
H16@210crs.
1200 lap
6500 long
7000 long
5000 long
4000 long
2000
2500
1250
2-H12
TYPICAL SEATING DETAIL
1:20
Aproved bearing strip under Unispan.
65 seating (typical).
TYPICAL FLOOR GRILLE
1:10
PVC tube to suit floor grille.
Fixing angle to secure tube while topping is poured.
4-H12 1000 long at each floor grille (typical).
Form hole in Unispan with polystyrene. Remove polystyrene after topping has been poured.
Ø250
D212 mesh
•35 cover internal.
•50 cover external.
R.L. 17.875
R.L. 15.880
H12@300crs.
•900 long legs.
8H12 each face
R10@100crs. stirrups.
Weldplate WP1.
10mm packer.
150x100x6 RHS
•Refer architects drawings for setout.
1:20
H12@600crs.
•Alternate returns.
80x20 seat bent from 6mm plate.
•6FWAR to beam.
250UC73
D212 mesh
•50 cover.
Wall 3
Refer wall elevation for wall reinforcement and starter bars.
3H12
R10@400crs.
PART PLAN
1:50
4 | 20.04.98 | Consent Issue
3 | 14.04.98 | Prelim. issue to QS
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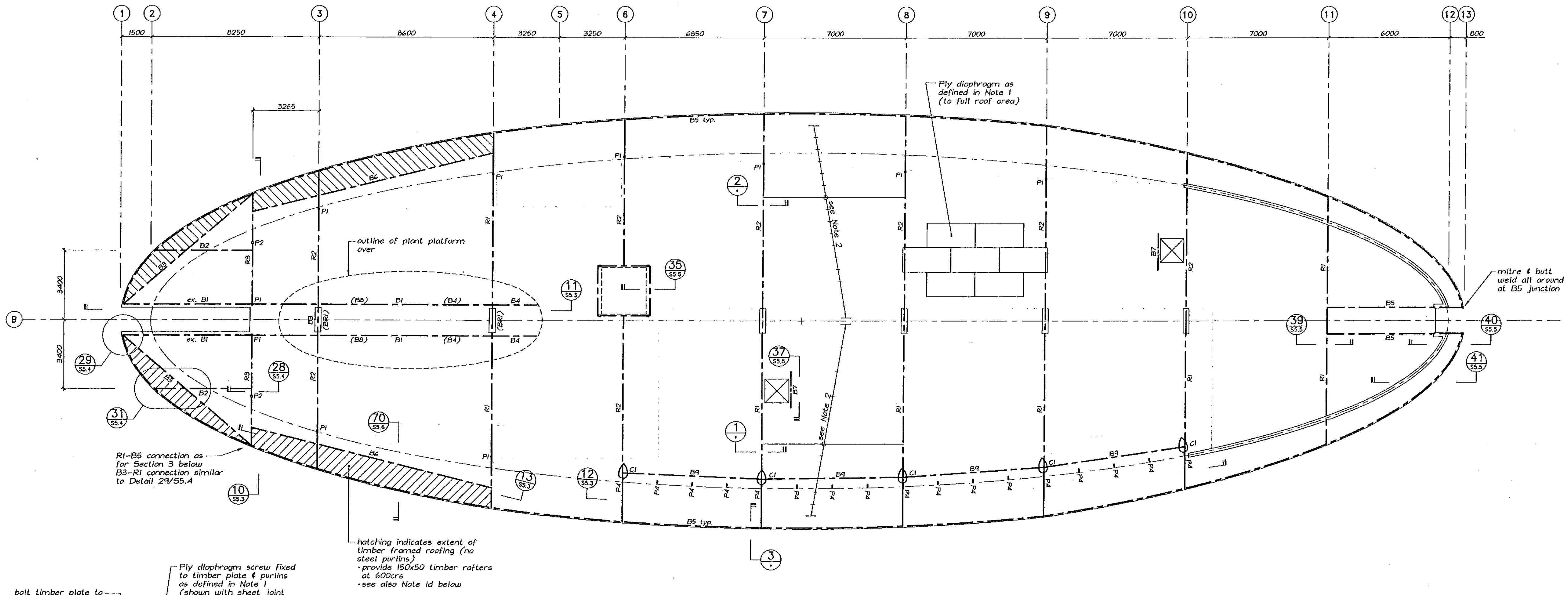
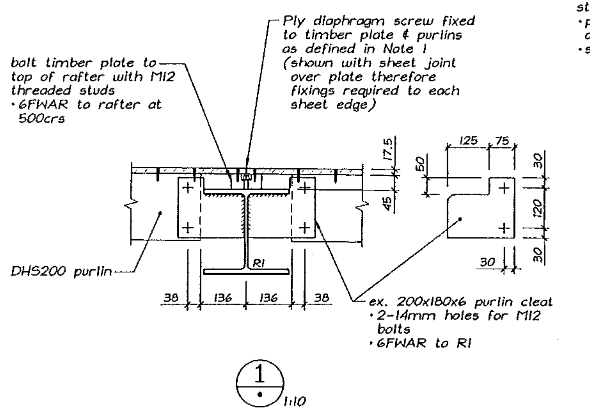
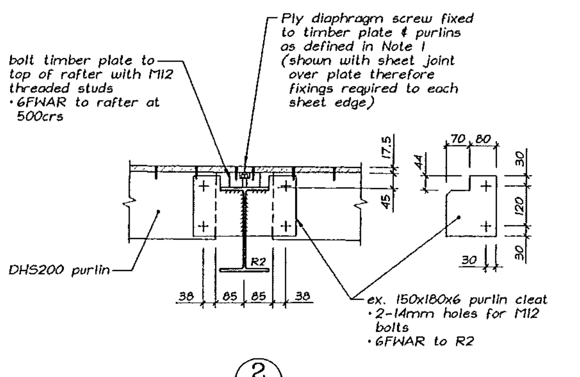


ROOF PLAN

1:100



1 1:10



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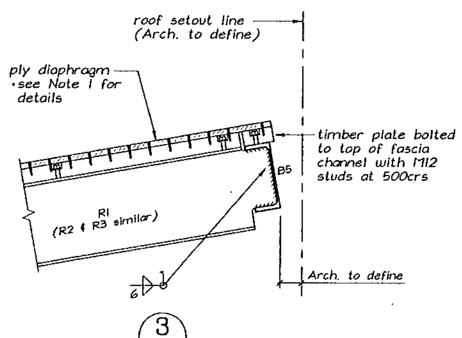

3 1:10

NOTES

1) Plywood Diaphragm
   a) Provide a 17.5mm Plywood diaphragm to full area of roof
   b) Ply sheets are to be staggered to form a checker-board pattern. Note sheet orientation shown on plan i.e. long edges parallel to Grid B
   c) All ply sheets are to be screw fixed with 10 gauge x 40mm long self drilling screws as follows:
      i) at 60mm centres to full perimeter of each ply sheet (provide timber blocking between purlins at joints between sheets to ensure diaphragm continuity)
      ii) at 60mm centres to timber plate bolted to main rafters (R1 & R2), fascia channel (B5) and RHS bracing chords (B6)
      iii) at 120mm centres to all intermediate supports
   d) At areas where roof structure is timber framed (shown hatched on Plan), ensure solid blocking is provided at all Ply sheet joints for diaphragm continuity

2) Purlins
   a) All purlins are to be DHS 200/12 at 600crs except in bay 3-4 where purlins are DHS 200/18 at 600crs.
   b) Provide one row of standard rod & channel bracing at midspan of purlins in all bays.
   c) Provide a double purlin each side of the Mech. Serv. openings in bays 7-8 & 9-10 (i.e. at each end of steel member B7). The double purlins are to be bolted together, back to back, with M12 bolts at 800crs. Provide a 6mm packing washer between the purlins at each bolt to maintain a gap for the support cleats at each end.

3) Refer to Architect for setout and orientation of all roof support posts. Setout to be approved by Engineer prior to any fabrication. Refer to Architect for defintion of roof plan shape.

MEMBER SCHEDULE

MARK	DESCRIPTION	SIZE
R1	Rafter	250UC73
R2	Rafter	250UB31
R3	Rafter	310UC97
B1	Beam	610UB125
B2	Beam	250UB37
B3	Beam	250UC73
B4	Beam	310UB40
B5	Fascia Beam	200x75 channel (contin.)
B6	Bracing Chord	150x150x6 RHS
B7	Beam	125x65 channel
B8	Beam	150x75 channel
B9	Beam	150UC30
C1	Column	see S5.6 for details
P1	Post	125x75x6 RHS
P2	Post	100x100x6 RHS
P3	Post	75x75x5 RHS
P4	Post	150x50x5 RHS
P5	Post	150x100x6 RHS
P6	Post	150x150x6 RHS
P7	Post	89x89x5 RHS
BR1	Brace	75x75x5 RHS
BR2	Brace	219Ø x 8.2mm CHS

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1	02.04.98	Preliminary Issue	

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Appendix D - DEEP Assessment Spreadsheet

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	New Brighton Library	Reviewer:	Alistair Boyce
	Unit / No: 213 / Street: Marine Parade	CPEng No:	209860
Building Address:	213 Marine Parade	Company:	Opus International Consultants
Legal Description:		Company project number:	6-QUCCC.67
		Company phone number:	(03) 363 5400
GPS south:	Degrees 43, Min 30, Sec 24.84	Date of submission:	11/09/2012
GPS east:	Degrees 172, Min 43, Sec 52.63	Inspection Date:	2-Jan-12
		Revision:	Final
Building Unique Identifier (CCC):	BU 1330-004 EQ2	Is there a full report with this summary?	yes

Site

Site slope:	flat	Max retaining height (m):	
Soil type:	mixed	Soil Profile (if available):	clay and SAND to 43m, GRAVEL below
Site Class (to NZS1170.5):	D		
Proximity to waterway (m, if <100m):	50	If Ground improvement on site, describe:	
Proximity to clifftop (m, if < 100m):			
Proximity to cliff base (m,if <100m):		Approx site elevation (m):	1.50

Building

No. of storeys above ground:	2 (single storey = 1)	Ground floor elevation (Absolute) (m):	
Ground floor split?	yes	Ground floor elevation above ground (m):	0.3 - 1.4
Storeys below ground	1		
Foundation type:	isolated pads, no tie beams	if Foundation type is other, describe:	
Building height (m):	9.00	height from ground to level of uppermost seismic mass (for IEP only) (m):	9
Floor footprint area (approx):	780		
Age of Building (years):	9	Date of design:	1992-2004
Strengthening present?	no	If so, when (year)?	
		And what load level (%g)?	
Use (ground floor):	commercial	Brief strengthening description:	
Use (upper floors):	public		
Use notes (if required):	library and cafe		
Importance level (to NZS1170.5):	IL2		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	250UB, 250UC, DHS200, Duromastic AC Monolithic cladding
Floors:	precast concrete with topping	unit type and depth (mm), topping	75mm Unispan125-225mm cladding
Beams:	precast concrete	overall depth (mm)	200-300mm
Columns:	cast-insitu concrete	typical dimensions (mm x mm)	1200x300mm
Walls:	load bearing concrete		#N/A

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	other (note)		describe system	219CHS steel braces + concrete shear walls
Ductility assumed, μ:	2.00			
Period along:	0.20	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	other (note)		describe system	concrete shear walls + concrete moment frame
Ductility assumed, μ:	2.00			
Period across:	0.10	0.00	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	

Separations:

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

Non-structural elements

Stairs:	steel	describe supports	fixed
Wall cladding:	other light	describe	glass
Roof Cladding:	Other (specify)	describe	Duromastic AC Monolithic
Glazing:	aluminium frames		
Ceilings:	strapped or direct fixed		
Services(list):	electrical, mechanical		

Available documentation

Architectural	full	original designer name/date	Barclay Architects Limited
Structural	full	original designer name/date	Powell Fenwick Consultants Limited
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:	minor damage	Describe damage:	minor cracking to ground surfacing
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:	slight	notes (if applicable):	

Building:

	Current Placard Status:	green		
Along	Damage ratio:	0%	Describe how damage ratio arrived at:	
	Describe (summary):	minor cracking of concrete walls		
Across	Damage ratio:	0%		
	Describe (summary):	some spalling of central concrete columns		
Diaphragms	Damage?:	no	Describe:	
CSWs:	Damage?:	no	Describe:	
Pounding:	Damage?:		Describe:	
Non-structural:	Damage?:	yes	Describe:	movement, cracking at joins

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

Recommendations

	Level of repair/strengthening required:	significant structural and strengthening	Describe:	repairs to cracked concrete, upgrade of collector elements
	Building Consent required:	yes	Describe:	
	Interim occupancy recommendations:	full occupancy	Describe:	
Along	Assessed %NBS before:	36% ##### %NBS from IEP below	If IEP not used, please detail assessment methodology:	full quantitative assessment
	Assessed %NBS after:	36%		
Across	Assessed %NBS before:	48% ##### %NBS from IEP below		
	Assessed %NBS after:	48%		

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

Period of design of building (from above): 1992-2004 — h_n from above: 9m

Seismic Zone, if designed between 1965 and 1992: — not required for this age of building

Design Soil type from NZS4203:1992, cl 4.6.2.2:

	along	across
Period (from above):	0.2	0.1
(%NBS)nom from Fig 3.3:		

Note:1 for specifically design public buildings, to the code of the day: pre-1965 = 1.25; 1965-1976, Zone A =1.33; 1965-1976, Zone B = 1.2; all else 1.0
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)

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

2.2 Near Fault Scaling Factor

Near Fault scaling factor, from NZS1170.5, cl 3.1.6:

	along	across
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 3.2)		
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:		
Ductiity 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 — **%NBSb:** **#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; Height Difference effect D2, from Table to right

Therefore, Factor D: 0

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

Detail Critical Structural Weaknesses: (refer to DEE Procedure section 6)
List any: — Refer also section 6.3.1 of DEE for discussion of F factor modification for other critical structural weaknesses

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

Appendix E - Geotechnical Appraisal

OPUS