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Upper Riccarton Library

Quantitative Engineering Evaluation

Functional Location ID: BU 3036-001 EQ2

Address: 71 Main South Road

Reference: 227052

Prepared for:

Christchurch City Council

Revision: 2

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Approval			
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Title	Senior Structural Engineer	Title	Senior Structural Engineer

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Appendices

Executive Summary

This is a summary of the Quantitative Engineering Evaluation for the Upper Riccarton Library building and is based on the Detailed Engineering Evaluation Procedure document issued by the Engineering Advisory Group on 19 July 2011, visual inspections, available structural documentation and summary calculations as appropriate.

Building Details	**Name**	Upper Riccarton Library			
Building Location ID	BU 3036-001 EQ2			**Multiple Building Site**	Y
Building Address	71 Main South Road			**No. of residential units**	0
Soil Technical Category	NA	**Importance Level**	3	**Approximate Year Built**	2005
Foot Print (m²)	1870	**Storeys above ground**	1	**Storeys below ground**	0
Type of Construction	Lightweight roof, steel purlins, main building has moment resisting steel portal frames on isolated concrete foundation, the rest of the building has precast concrete and timber framed walls, concrete slab with thickening under load bearing walls.				
Qualitative L4 Report Results Summary					
Building Occupied	Y	The Upper Riccarton Library is currently in service.			
Suitable for Continued Occupancy	Y	The Upper Riccarton Library is suitable for continued use.			
Key Damage Summary	Y	Refer to summary of building damage Section 3.1 report body.			
Critical Structural Weaknesses (CSW)	N	No critical structural weaknesses were identified.			
Levels Survey Results	N	Floor level survey was undertaken in 16 December 2011. Floor issues have been rectified during the time of our quantitative investigation. Therefore, no further level survey is considered necessary.			
Building %NBS From Analysis	49%	Based on an analysis of capacity and demand (see Table 2 for Summary of Seismic Performance).			
Qualitative L4 Report Recommendations					
Geotechnical Survey Required	N	Geotechnical survey not required due to lack of observed ground damage on site.			
Proceed to L5 Quantitative DEE	N	There is no statutory requirement to strengthen the building.			
Approval					
Author Signature		Approver Signature			
Name	Luis Castillo	Name	Lee Howard		
Title	Senior Structural Engineer	Title	Senior Structural Engineer		

1 Introduction

1.1 General

On 23 September 2012 Aurecon engineers visited the Upper Riccarton Library to undertake a quantitative building damage and strength assessment on behalf of Christchurch City Council. Detailed visual inspections and further intrusive investigation undertaken on 5 October 2012 were carried out to assess the damage caused by the earthquakes on 4 September 2010, 22 February 2011, 13 June 2011, 23 December 2011 and related aftershocks.

The scope of work included:

- Re-assessment of the nature and extent of the building damage as stated in the Qualitative Assessment Report.
- Visual assessment of the building strength particularly with respect to safety of occupants if the building is currently occupied.
- Assessment of requirements for detailed engineering evaluation including any areas where linings and floor coverings need removal to expose connection details.

This report outlines the results of our Quantitative Assessment of damage to the Upper Riccarton Library and is based on the Detailed Engineering Evaluation Procedure document issued by the Engineering Advisory Group on 19 July 2011, visual inspections, available structural documentation and summary calculations as appropriate.

2 Description of the Building

2.1 Building Age and Configuration

The Upper Riccarton Library is a large rectangular single storey steel and precast concrete tilt panel structure built in 2005. The central part of the building has higher level roof while the extended sections along the north and south sides of the building have lower level roofs. The lower roof to the south extends the full length of the building. The lower roof to the north extends over two bays only. The roofs are lightweight metal supported on cold rolled steel sections.

For the upper level roof, cold rolled steel purlins span between heavy steel section rafters at 8.0m grid centres. These rafters are rigidly connected to internally exposed circular steel columns. At each end the rafters cantilever past the columns by approximately 3.0m.

Rafters for the lower level roof to the south span between supporting precast concrete wing walls that occur on the grids. Purlins for the smaller roof section to the north are supported on steel angles screwed to timber framed walls.

Rafters in the east and west end bays of the building span between steel beams that are supported by a circular steel column at one end and precast concrete tilt up wall panel on the other end.

The Library has a precast concrete perimeter foundation wall supported on approximately 1.0m wide in-situ concrete strip footings founded about 500mm below ground level. The finished level of the floor slab is approximately 600mm above ground level indicating that a significant depth of compacted hard fill was required to build up to level. 50mm thick polystyrene insulation has been installed below the floor slab and against the perimeter wall.

The building has an approximate floor area of 1870 square metres. Importance level 3 has been applied to this structure in accordance with City Solutions original assumptions and Christchurch City Council Guidelines.

2.2 Building Structural Systems Vertical and Horizontal

The north to south direction is referred to as the transverse direction and the east to west direction as the longitudinal direction. Vertical loads from the lightweight metal roof in the main building are resisted by cold formed steel purlins supported by moment resisting frames in both the transverse and longitudinal directions. The bottom flange of the main beam is welded on top of the column and runs in the transverse direction. The high roof diaphragm is then connected to the top of the main beam by welded metal cleats and bolted timber blocking (see Detail 2 in Appendix B). The secondary beam sits on top of the main beam and runs in the longitudinal direction. The top flange of the secondary beam has fixed connection and is bolted on top of the main beam (see Detail 1 in Appendix B). Metal cleats are welded on top of the secondary beam with bolted timber blocking to support the high level roof diaphragm (see Detail 1 in Appendix B).

The portal frames in the main building resist lateral loads in both the transverse and longitudinal direction. The east and west end bays of the main building have a steel portal frame at one end and a precast concrete tilt up panel on the other end. This system is resisting the lateral load in the transverse direction. The cantilevered precast concrete tilt-up panels at the end bays are not resisting any lateral loads. The lateral load in the longitudinal direction is transferred back to the moment resisting frame in the same direction.

Vertical loads from the lightweight metal sheeting from the southern end of the building are resisted by cold formed steel purlins supported, primarily, by precast concrete tilt up wall panels in the transverse direction. The precast concrete tilt-up wall panels also resist lateral loads in the longitudinal direction by cantilever action.

The steel framed roof in the northern end of the building supports lightweight roof sheeting that transfer loads to the load bearing walls. Purlins for the lower level roof are supported on steel angles screwed to timber framed walls that also restrain the roof transversely. Lateral loads are resisted by timber framed external walls lined with plywood sheets. The steel portal frame at one end and reinforced concrete masonry block wall on the other end resist the lateral load in the longitudinal direction.

The Upper Riccarton Library has a concrete floor slab with a precast perimeter foundation supported on in-situ concrete strip footings founded about 500mm below ground level. The portal frame columns are supported on isolated concrete foundation pads. Load bearing walls are supported on a concrete floor slab with thickening.

2.3 Reference Building Type

A general overview of the reference building type, construction era and likely earthquake risk is presented in the figure below. The Upper Riccarton Library, according to the figure below shows it is probably not earthquake prone.

Figure 1: Timeline showing the building types, approximate time of construction and likely earthquake risk.
(From the Draft Guidance on DEEs of non-residential buildings by the Engineering Advisory Group)

2.4 Building Foundation System and Soil Conditions

The Upper Riccarton Library has a concrete floor slab with thickening under load bearing walls. Additional isolated concrete pads support loaded columns.

The land and surrounds of the Upper Riccarton Library does not have a technical classification. It is of note however, that the closest suburb consists primarily of Technical Category 1 (TC 1) land. According to CERA, TC1 land is “unlikely to have future land damage due to liquefaction”.

2.5 Available Structural Documentation and Inspection Priorities

Structural drawings of the Upper Riccarton Library were available when we prepared the Qualitative Assessment Report in January 2012. However, we had concerns regarding a number of structural issues particularly the lateral restraint provisions in the longitudinal direction. This was raised in the report as one of the main issues and is the reason we recommended further investigation in the form of this quantitative assessment.

Additional drawings, calculations and documents were provided by the Christchurch City Council on 13 September 2012. From this new information, we have been able to identify numerous revisions made to the structure subsequent to the original drawings that we reviewed. One vital revision which answered our query regarding the lateral restraint provisions in the longitudinal direction was the additional plate that is welded and bolted on top of the secondary steel beam in the longitudinal direction which converted the structure into a moment resisting frame.

Our inspection priorities were to confirm how the building is resisting lateral loads in both the transverse and longitudinal directions. This applies to the main building, northern and southern end of the building and the east and west end bays.

2.6 Available Survey Information

A floor level survey was undertaken as part of our qualitative report. The issues identified in our qualitative report had been rectified/covered prior to our quantitative assessment being undertaken.

3 Structural Investigation

3.1 Summary of Building Damage

The Qualitative visual assessment of the exterior and interior of the Upper Riccarton Library in 21 October 2011 indicated that apart from increased crack widths in the floor slab, the damage observed was of a minor nature. The damage noted in the 10 January 2012 Qualitative Assessment Report had been rectified prior to our quantitative building damage assessment on 23 September 2012.

The Upper Riccarton Library was in use at the time of our internal and external visual damage assessment. It has performed well and there are no significant visible signs of damage that can be attributed to seismic actions.

3.2 Record of Intrusive Investigation

As per our recommendation in our 10 January 2012 Qualitative Assessment Report, an intrusive investigation was undertaken on 5 October 2012.

And as part of our intrusive investigation, linings have been removed to expose connection details on the following sections (see Appendix A – Floor Plan and photos):

- Welded and bolted cleats on the top flange of the secondary beam in the longitudinal direction (see Detail 1A in Appendix B);
- Welded cleat on top of the main beam in the transverse direction and timber blocking to support roof diaphragm (see Detail 2 in Appendix B);
- Welded cleat on top of the secondary beam in the longitudinal direction and timber blocking to support roof diaphragm (see Detail 1B in Appendix B);

- Bolted angle cleat on top of the transverse precast concrete tilt up wall panel (P6) which is connected to the steel rafter running in the longitudinal direction; and
- Metal cleat cast in precast concrete tilt up wall panel (P7) which is bolted to a steel lintel beam in the longitudinal direction (see Appendix B page 16).

We are now able to confirm that the proposed additional plates, as mentioned in Section 2.5 of this report, are in place and connection details comply with original design intent. Details of the additional plates can be found in Detail 1 in Appendix B.

Vertical and horizontal reinforcement spacing detected along precast concrete tilt up wall panel P2L in Grid 3 complies with what has been specified in the drawings (see Appendix B page 17).

3.3 Damage Discussion

Damage to the building had been rectified prior to our quantitative investigation. Therefore, there was no observed damage to the building as a result of the seismic actions prior to our inspection.

4 Building Review Summary

4.1 Building Review Statement

We used the original and remedial drawings, calculations and documentations provided by the Christchurch City Council to verify and confirm the structural system and elements, investigate the potential critical structural weaknesses (CSW) and any concerns raised in our qualitative report.

The original calculations by City Solutions showed that the building was designed, predominantly, for wind loads. They used the current loading standard NZS1170.2:2002 for calculating the wind loads but NZS4203:1992 Part 4 was used for calculating the seismic coefficients. The current loading standard for seismic, NZS1170.5:2004, has increased the seismic load by around 50% over that used by City Solutions, mainly as a result of the increased hazard factor for Christchurch. We have analysed the structure using the current seismic coefficient. Wind loads still governed in most of the analyses but the capacities of some of the structural elements resisting seismic loads were found to be less than 50%NBS.

Note: Our assessments have been based on CCC drawings and calculations, our own calculations and intrusive investigation. We reviewed these information using current NZ standards (NZS1170:2004, NZS3404:2009, NZS3101:2006 and NZS3603:1993). For our dead loads, we used Cory Bedford and Rolando Castillo of City Solutions' values. We also used their assumed allowable bearing capacity of 200kPa.

4.1.1 CCC drawings reviewed:

We reviewed the following drawings:

1. Structural drawings. File Reference: 254 / 25496 – Sheet S1-1 to S3-25.
2. Base plate. CCC drawing number sd 033301 showed 2 different versions. In Sheet S1-8, 2-HD16 bars are cast-in while in Sheet S3-18 the base plate has 2-M16 Hilti HSA anchors.

3. The vertical and horizontal reinforcement bar sizes and spacing noted in Appendix B (pages 17 to 20) are not noted or different to 02.02.05 drawings (Drawing number sd 033301, Sheet S2-1 to S2-3).

4.1.2 Independent Calculations:

We carried out the following independent calculations:

1. Assessed the probable strength (flexural and/or axial as appropriate) and rotation capacity available from the individual members and connections of the seismic-resisting system.
2. Assessed the probable strengths of the components and members to obtain the strength hierarchy of the system, making allowance for foundation strength and stiffness limitation on the strength hierarchy.
3. Determined the actual ductility demand on the system that is required to match the seismic actions generated by the required strength assessment limit (from Section 5 for moderate risk determination) with the first yield strength available from the system.
4. Determined the deflection limit for the system.

4.1.3 CCC calculations reviewed:

We reviewed the following CCC calculations:

1. Remedial drawings and calculation as shown in details 1 and 2 (see Appendix B pages 1 to 5).
2. Cleat and screws that supports high level roof diaphragm.
3. Welded and bolted cleat on the top flange of the secondary beam in the longitudinal direction.
4. Capacity of RHS posts in the northern and southern end of the building – including base plate.
5. End bay walls and cantilever.
6. Sliding doors at Grid D.
7. Capacities of cantilevered precast concrete wall panel.
8. Support between timber frame and precast concrete wall panel.

We specifically investigated how the building is laterally restrained in the longitudinal direction but we have also reviewed other aspects of the building. Through intrusive investigation, we are able to verify and confirm that the additional connection details to the building issued subsequently to the main drawings have been followed.

4.2 Critical Structural Weaknesses

No specific critical structural weaknesses were identified during our quantitative assessment of the building.

5 Building Strength (Refer to Appendix C for background information)

5.1 General

The Upper Riccarton Library is of steel construction. With lightweight roof, moment resisting steel frame and roof diaphragm, the building has performed well in the Canterbury earthquake sequence as evidenced by the lack of noted damage in section 3 above.

Structural drawings for the building were available and these were reviewed as part of the building assessment. The moment resisting frames in the central part of the building are resisting the lateral loads in both the transverse and longitudinal direction. This type of structure normally performs well and in this particular building there is no significant visible damage that is obviously related to recent earthquakes due to its lightweight metal roof sheeting and wall cladding which allows it to be flexible and ductile. The concrete precast tilt up wall panels in the transverse direction also performed well due to adequate reinforcement. However, the cantilevered concrete precast tilt up wall panel does not have adequate capacities and restraints to resist lateral loads in the longitudinal direction.

5.2 Strength Assessment In Relation To New Building Standard (NBS)

The Upper Riccarton Library has been subject to specific engineering design and the lateral load capacity has been calculated by adopting values for the strengths of existing materials and calculating the capacity of existing precast concrete tilt up wall panels and moment resisting steel frames in both the transverse and longitudinal directions.

Selected seismic parameters used in our assessment are tabulated in the tables below.

Table 1: Parameters used in the Seismic Assessment

Seismic Parameter	Quantity	Comment/Reference
Site Soil Class	D	NZS 1170.5:2004, Clause 3.1.3, Deep or Soft Soil
Site Hazard Factor, Z	0.30	DBH Info Sheet on Seismicity Changes (Effective 19 May 2011)
Return period Factor, R_u	1.30	NZS 1170.5:2004, Table 3.5, Importance Level 3 Structure with a Design Life of 50 years
Ductility Factor in the Longitudinal Direction, μ	1.25	Moment resisting frame
Ductility Factor in the Longitudinal Direction, μ	2.00	Precast concrete tilt up wall panels
Ductility Factor in the Transverse Direction, μ	1.25	Moment resisting frame and precast concrete tilt up wall panels

The seismic demand for the Upper Riccarton Library was designed based on NZS 4203:1994 but we have used the current loading code NZS 1170.5:2004 to analyse the structure. Use of the current standard loading has increased the seismic load by around 50%. However, wind loads still dominate in most cases.

The capacity of the moment resisting steel frames was calculated from the strengths of existing materials present in both the transverse and longitudinal directions. The strength of the precast concrete tilt up wall panels was also calculated based on the strength of the materials and the number and length of walls present for both the transverse and longitudinal directions. The seismic demand was then compared with the building capacity in these directions.

The seismic capacity of the concrete precast tilt up wall in the north side of the building has been calculated at >100%NBS in the transverse direction and 59%NBS in the longitudinal direction; which makes capacity in the longitudinal direction the weaker element.

However, the bolts used for the spliced beam in the longitudinal direction have been undersized. The calculated capacity of the bolts is **49%NBS** which make this the weakest element (i.e. a 'moderate risk' building according to NZEE Guidelines).

Table 2: Summary of Seismic Performance

5.2.1 Transverse Direction		
Structural Element/System	Failure mode, or description of the limiting criteria based on displacement capacity of critical element	%NBS – Based on calculated capacities
Moment-resisting Steel Framed System (pinned base)		**>100%**
Member strength	Probable nominal flexural strength	
Beam	An I-section beam not responding inelastically under moment will deliver axial forces through the flanges (tension and compression) and vertical shear through the web. Yielding in flexure of the major beam	>100%
Column	The incoming axial forces from an I-section beam flange connected to a column must be transferred through the circular column. Yielding of the column about the major axis. The columns are designed as 'pinned' base.	>100%
Connection strength	The flexural strength is governed by the weakest of the individual elements. In many instances, this will be the connections. The probable strengths of the components and members are assessed to obtain the strength hierarchy of the system. The force transfer through the connection assessed, weak links determined and their strength evaluated.	>100%
Beam to column connection - welded	Brittle failure of the weld of the beam to column connection. The weld cannot develop the flexural capacity of the column. So no column yielding can occur without brittle failure of the weld.	>100%
Stiffeners	If there are no tension and compression stiffeners in columns adjacent to incoming beam flanges in a moment-resisting beam to column connection, then tensile distortion of the column flange or compression buckling of the column web are likely to occur before the beam can develop its section moment capacity.	>100%
Base plate	Excessive drift can lead to premature collapse due to P-delta effect which can also cause damage to non-structural elements.	>100%
Plate strength	Ductile failure mode includes yielding of the gross area of the base plate.	>100%
Bolt capacities	Relatively brittle failure mode includes shear fracture of bolts	>100%
Foundation		
Isolated pads	If bearing capacity of ground or compacted fill suitable for foundation.	>100%

Structural Element/System	Failure mode, or description of the limiting criteria based on displacement capacity of critical element	%NBS – Based on calculated capacities
Moment-resisting Steel Framed System (pinned base)		**>100%**
Displacement	The extent of deflection to which the system is allowed to displace has been checked and referred to the building height limit given by NZS1170.	>100%
Precast Concrete Tilt-up Wall Panel		**>100%**
In-plane flexural capacity	Yielding of the HD20 starters of the cantilevered precast concrete tilt-up wall panels. The yielding mechanism itself is not brittle. Panel P6 Panel P1.L or P1.R	>100% >100%

5.2.2 Longitudinal Direction

Structural Element/System	Failure mode, or description of the limiting criteria based on displacement capacity of critical element	%NBS – Based on calculated capacities
Moment-resisting Steel Framed System (pinned base)		**49%**
Member strength	Probable nominal flexural strength	
Beam	An I-section beam not responding inelastically under moment will deliver axial forces through the flanges (tension and compression) and vertical shear through the web. Yielding in flexure of the minor (secondary) beam. Effective length (Le) of 2.20m Effective length (Le) of 5.80m	>100% 86%
Column	The incoming axial forces from an I-section beam flange connected to a column must be transferred through the circular column. Yielding of the column about the strong axis. The columns are designed as 'pinned' base. Yielding of the column about the minor axis. The columns are designed as 'pinned' base. When the connection fails, a collapse mechanism can form.	>100%
Connection strength	The flexural strength is governed by the weakest of the individual elements. In many instances, this will be the connections. The probable strengths of the components and members are assessed to obtain the strength hierarchy of the system. The force transfer through the connection assessed, weak links determined and their strength evaluated.	49%
Major beam to column weld connection	Brittle failure of the weld of the major beam to column connection. The weld cannot develop the flexural capacity of the column. So no column yielding can occur without brittle failure of the weld.	>100%
Splice bolt connection	**The location of these weakest elements will be the yielding regions. They are the primary elements at this level. Relatively brittle failure mode includes shear fracture of flange bolts.**	**49%**
Splice plate	Relatively brittle failure mode includes yielding of the gross area of the web splice plate due to combined shear and bending.	82%

Structural Element/System	Failure mode, or description of the limiting criteria based on displacement capacity of critical element	%NBS – Based on calculated capacities
Moment-resisting Steel Framed System (pinned base)		**49%**
Stiffeners	If there are no tension and compression stiffeners in columns adjacent to incoming beam flanges in a moment-resisting beam to column connection, then tensile distortion of the column flange or compression buckling of the column web are likely to occur before the beam can develop its section moment capacity.	>100%
Base plate	Excessive drift can lead to premature collapse due to P-delta effect which can also cause damage to non-structural elements.	>100%
Foundation	A dependable load path has been selected for transmitting the earthquake plus associated gravity design actions between structure and ground. The bearing pressure of the foundation has been determined on the basis of its pinned status.	
Isolated pads	If bearing capacity of ground or compacted fill suitable for foundation.	>100%
Displacement	The extent of deflection to which the system is allowed to displace has been checked and referred to the building height limit given by NZS1170.	>100%
Structural Element/System	Failure mode, or description of the limiting criteria based on displacement capacity of critical element	%NBS – Based on calculated capacities
Precast Concrete Tilt-up Wall Panel		**59%**
Out-of plane flexural capacity embedded at bottom	**The panel acts as a cantilever off the ground floor. Yielding of the HD20 starters from the thickened concrete floor slab to the precast concrete tilt-up wall panels. The yielding mechanism itself is not brittle. Face loading capacity of the following concrete precast tilt up wall panels:** **Panel P6** Panel P1.L or P1.R	**59%** >100%
Cantilevered concrete floor slab	Yielding of the HD16 from the precast concrete tilt-up wall panels. The yielding mechanism itself is not brittle. Panel P1.L or P1.R	>100%
Foundation	A dependable load path has been selected for transmitting the earthquake plus associated gravity design actions between structure and ground. If bearing capacity of ground or compacted fill suitable for foundation. Panel P6 Panel P1.L or P1.R	56% 100%

Notes:

1. Reference: New Zealand Society for Earthquake Engineering – Assessment and Improvement of the Structural Performance of Buildings in Earthquakes – June 2006, NZS1170:2004, NZS3404:2009, NZS3101:2006 and NZS3603:1993.
2. Dead loads based on Rolando-Castillo of City Solutions' calculation.
3. Allowable bearing pressure based of City Solutions' assumption of 200 kPa.
4. Assessment based on drawings and calculations provided by the Christchurch City Council.
5. Member capacities are based on Australian Steel Institute (ASI): Design Capacity Tables for Structural Steel.
    a. Open Sections (Volume 1): 4th Edition 2009
    b. Hollow Sections (Volume 2): 2nd Edition 2004
6. Stiffener check based on AISC Structural Connections: 4th Edition

5.3 Results Discussion

Based on the intrusive investigation undertaken in 5 October 2012, our independent calculations and reviewing City Solutions' various drawings and calculations, we can conclude that the Upper Riccarton Library has a strength of **49%NBS**.

This is due to the bolts used in the spliced secondary beam connection in the longitudinal direction being undersized. The cantilevered concrete precast tilt up wall panels in the north side of the building are also inadequately reinforced and lack capacity to resist the lateral loads out of plane. The contributing factor to this inadequacy was due to City Solutions using the seismic coefficient based on NZS4203:1994. The current standard loading NZS1170:5:2004 has increased the seismic load by around 50% mainly due to the increased hazard factor for Christchurch.

6 Conclusions and Recommendations

Given the good performance of the Upper Riccarton Library in the Canterbury earthquake sequence, the lack of foundation damage, **a geotechnical investigation is currently not considered necessary**.

The Upper Riccarton Library is currently occupied and the building has suffered no loss of functionality and in our opinion the Upper Riccarton Library **is suitable for continued occupation**.

However, the seismic capacity of the building is governed by the bolts used to connect the spliced secondary beams in the longitudinal direction which have been calculated in this quantitative assessment as **49%NBS**. The building is considered a moderate earthquake risk. There is no statutory requirement to strengthen the building (unless there is a change in use). We recommend strengthening the building to 67%.

7 Explanatory Statement

The inspections of the building discussed in this report have been undertaken to assess structural earthquake damage. No analysis has been undertaken to assess the strength of the building or to determine whether or not it complies with the relevant building codes, except to the extent that Aurecon expressly indicates otherwise in the report. Aurecon has not made any assessment of structural stability or building safety in connection with future aftershocks or earthquakes – which have the potential to damage the building and to jeopardise the safety of those either inside or adjacent to the building, except to the extent that Aurecon expressly indicates otherwise in the report.

This report is necessarily limited by the restricted ability to carry out inspections due to potential structural instabilities/safety considerations, and the time available to carry out such inspections. The report does not address defects that are not reasonably discoverable on visual inspection, including defects in inaccessible places and latent defects. Where site inspections were made, they were restricted to external inspections and, where practicable, limited internal visual inspections.

To carry out the structural review, existing building drawings were obtained (where available) from the Christchurch City Council records. We have assumed that the building has been constructed in accordance with the drawings.

While this report may assist the client in assessing whether the building should be repaired, strengthened, or replaced that decision is the sole responsibility of the client.

This review has been prepared by Aurecon at the request of its client and is exclusively for the client's use. It is not possible to make a proper assessment of this review without a clear understanding of the terms of engagement under which it has been prepared, including the scope of the instructions and directions given to and the assumptions made by Aurecon. The report will not address issues which would need to be considered for another party if that party's particular circumstances, requirements and experience were known and, further, may make assumptions about matters of which a third party is not aware. No responsibility or liability to any third party is accepted for any loss or damage whatsoever arising out of the use of or reliance on this report by any third party.

Without limiting any of the above, Aurecon's liability, whether under the law of contract, tort, statute, equity or otherwise, is limited as set out in the terms of the engagement with the client.

Appendices

Appendix A
Site Map, Floor Plan and Photos

23 September 2012 – Upper Riccarton Library Site Photographs

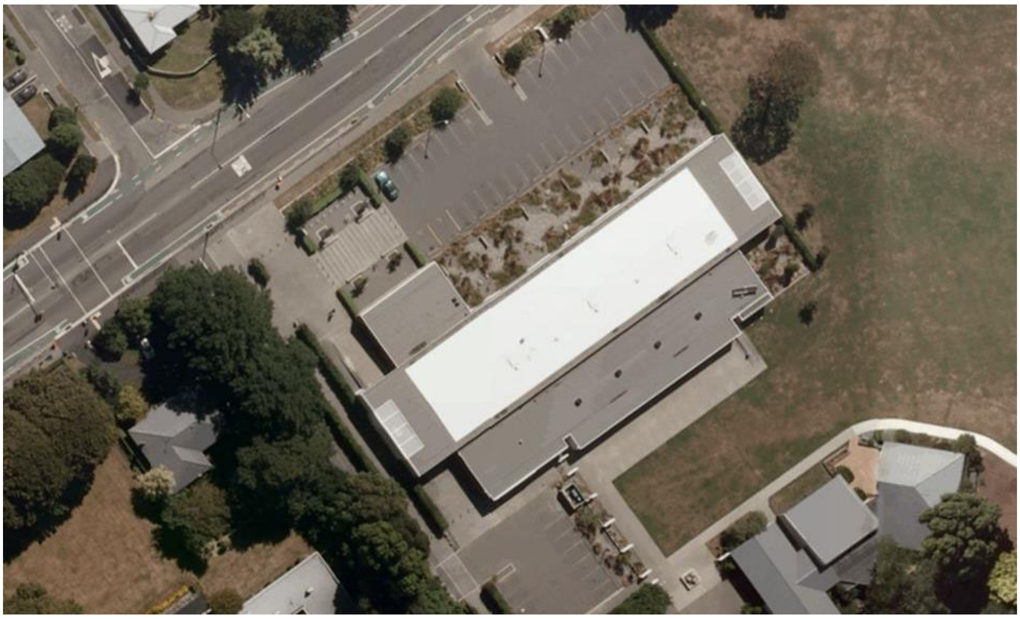
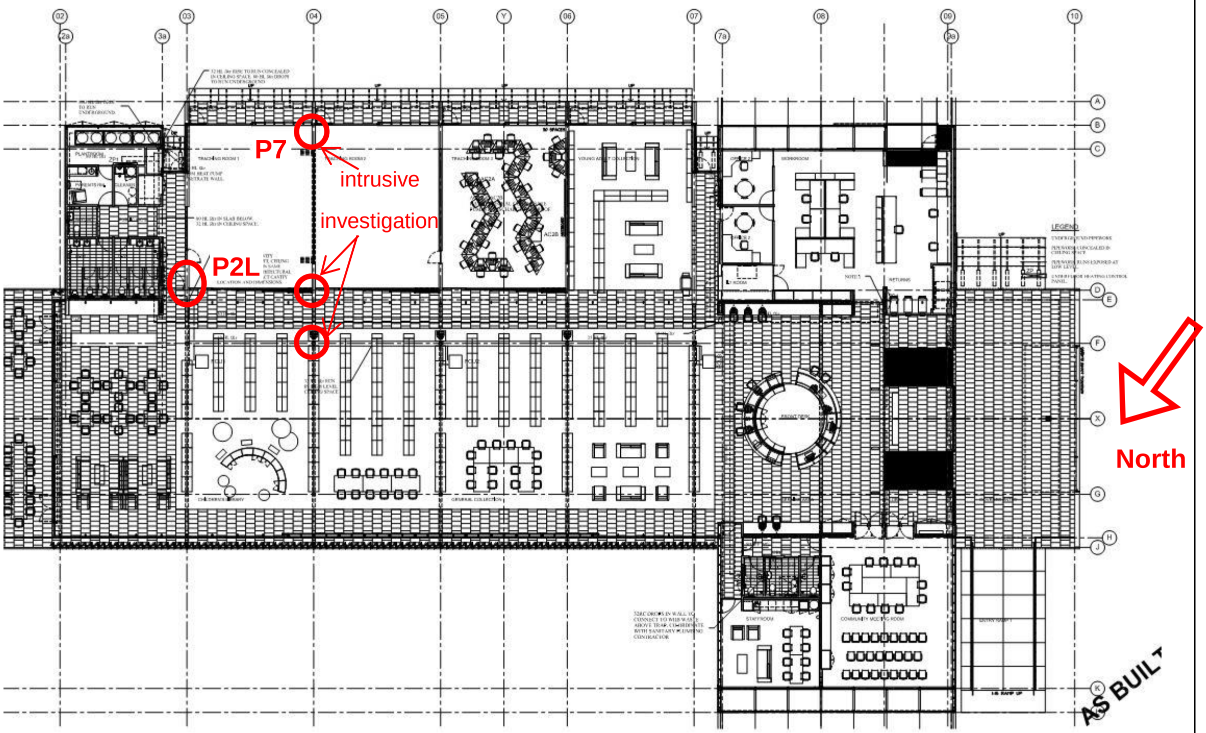
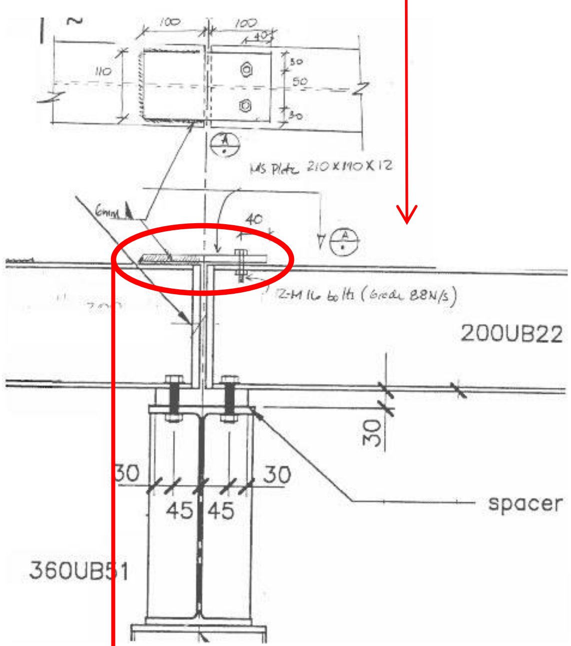
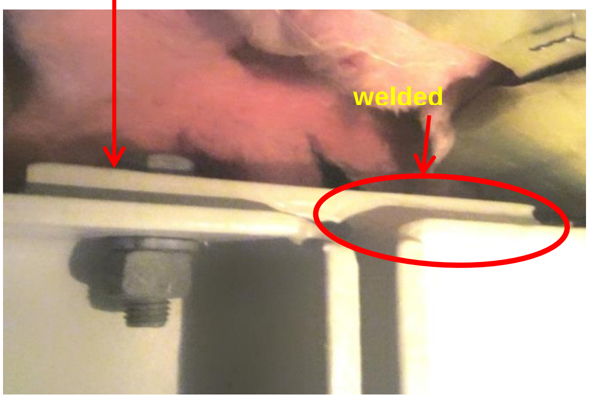


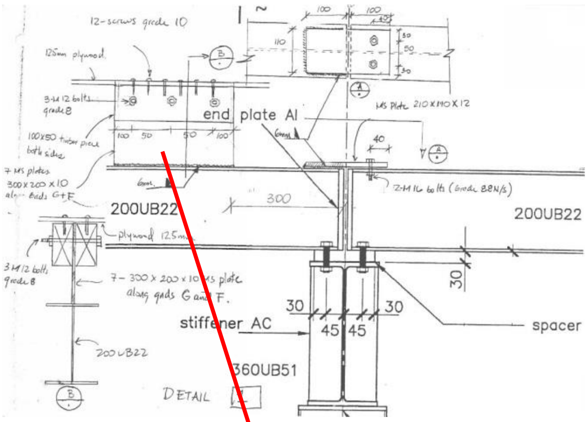
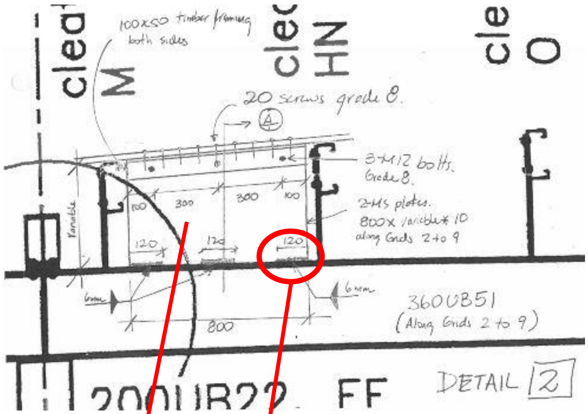
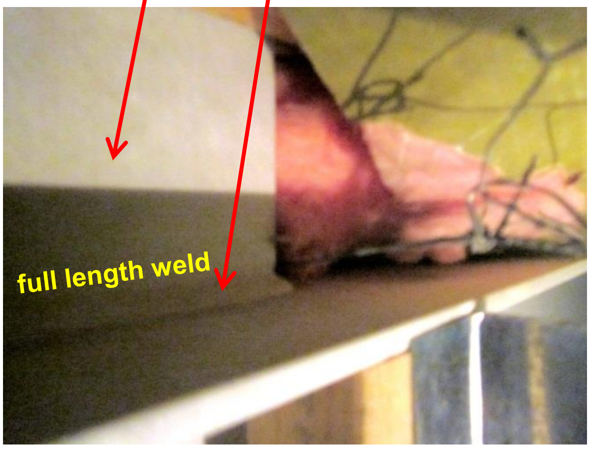
Image: Sourced from koordinates.con and LINZ. Crown Copyright Reserved



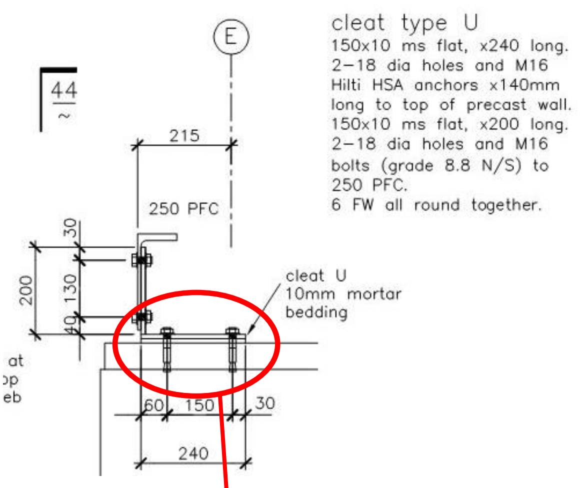
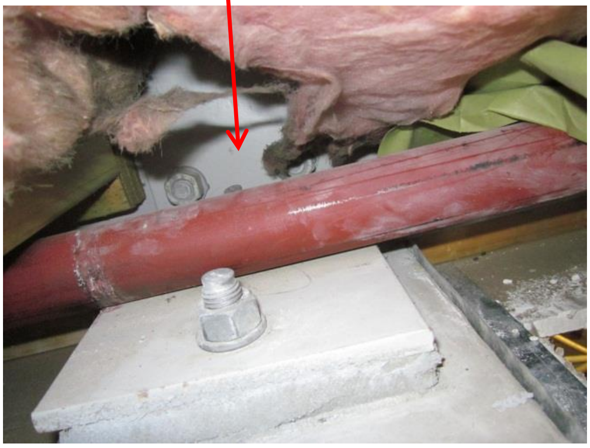
Upper Riccarton Floor Plan

Upper Riccarton Library – aerial photo.	
Internal view of the northern end of the building (along Grid J – see Floor Plan).	
Internal view of the central part of the building (Steel portal frames from Grid 2 to 9 between Grid E to H – see Floor Plan).	
Internal view of the precast concrete tilt up wall panels in the southern end of the building.	

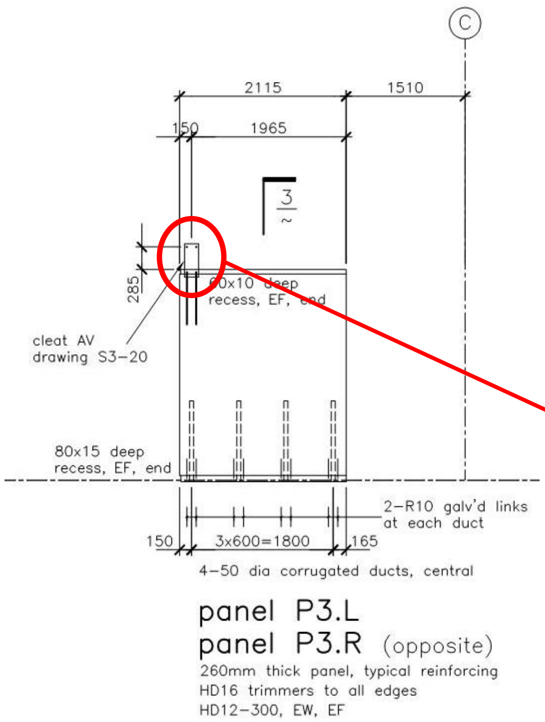
External view of the southern end of the building.	
Steel connection details of the moment resisting frame in the transverse direction.	
This welded plate and bolts has been added to the secondary frame in the longitudinal direction. Note: This connection detail is vital in establishing the lateral restraint in the longitudinal direction of the building.	
This welded plate and bolts have been added to the structure enabling the frame in the longitudinal direction to act as a moment resisting frame.	

Plates, timber blockings, screws and bolts have been added to the steel moment resisting frame in both directions (see Appendix B – Calculation Sheet 1 for the location of these plates).	
This plate is along Grid 4 and close to the left of Grid F. Due to space limitations and numerous obstructions, we are unable to take photos of the bolts but we can confirm that there are bolts and timber blocking on the top part of this plate.	
This plate has been added to the main moment resisting frame in the transverse direction.	
This plate is along Grid F and closer to right of Grid 4. Note: Weld is applied to full length of the plate not hit-miss as drawn.	

Southern cantilevered end of the transverse moment resisting frame.	
Internal view along Grid D (see Floor Plan)	
Typical RHS (as shown in the previous photo) in the southern end of the building is not connected to the precast wall.	
Precast wall in the southern end of the building.	

<table>
<tr>
<td>Typical connection details between the precast wall in the transverse direction and 250PFC running in the longitudinal direction.</td>
<td>
</td>
</tr>
<tr>
<td>This is the plate and bolts on top of the precast wall (as shown above).</td>
<td></td>
</tr>
<tr>
<td>Top connection view of the precast concrete tilt up wall panel (Panel P2R along Grid 7 – see Appendix B page 1).</td>
<td>
</td>
</tr>
</table>

Internal view of precast wall P7.



Appendix B
References and List of Drawings

1. Department of Building and Housing (DBH), “Revised Guidance on Repairing and Rebuilding Houses Affected by the Canterbury Earthquake Sequence”, November 2011

2. New Zealand Society for Earthquake Engineering (NZSEE), “Assessment and Improvement of the Structural Performance of Buildings in Earthquakes”, April 2012

3. Standards New Zealand, “AS/NZS 1170 Part 0, Structural Design Actions: General Principles”, 2002

4. Standards New Zealand, “AS/NZS 1170 Part 1, Structural Design Actions: Permanent, imposed and other actions”, 2002

5. Standards New Zealand, “NZS 1170 Part 5, Structural Design Actions: Earthquake Actions – New Zealand”, 2004

6. Standards New Zealand, “NZS 3101 Part 1, The Design of Concrete Structures”, 2006

7. Standards New Zealand, “NZS 3404 Part 1, Steel Structures Standard”, 1997

8. Standards New Zealand, “NZS 3603, Timber Structures Standard”, 1993

9. Standards New Zealand, “NZS 3604, Timber Framed Structures”, 2011

10. Standards New Zealand, “NZS 4229, Concrete Masonry Buildings Not Requiring Specific Engineering Design”, 1999

11. Standards New Zealand, “NZS 4230, Design of Reinforced Concrete Masonry Structures”, 2004

Appendix C
Strength Assessment Explanation

New building standard (NBS)

New building standard (NBS) is the term used with reference to the earthquake standard that would apply to a new building of similar type and use if the building was designed to meet the latest design Codes of Practice. If the strength of a building is less than this level, then its strength is expressed as a percentage of NBS.

Earthquake Prone Buildings

A building can be considered to be earthquake prone if its strength is less than one third of the strength to which an equivalent new building would be designed, that is, less than 33%NBS (as defined by the New Zealand Building Act). If the building strength exceeds 33%NBS but is less than 67%NBS the building is considered at risk.

Christchurch City Council Earthquake Prone Building Policy 2010

The Christchurch City Council (CCC) already had in place an Earthquake Prone Building Policy (EPB Policy) requiring all earthquake-prone buildings to be strengthened within a timeframe varying from 15 to 30 years. The level to which the buildings were required to be strengthened was 33%NBS.

As a result of the 4 September 2010 Canterbury earthquake the CCC raised the level that a building was required to be strengthened to from 33% to 67% NBS but qualified this as a target level and noted that the actual strengthening level for each building will be determined in conjunction with the owners on a building-by-building basis. Factors that will be taken into account by the Council in determining the strengthening level include the cost of strengthening, the use to which the building is put, the level of danger posed by the building, and the extent of damage and repair involved.

Irrespective of strengthening level, the threshold level that triggers a requirement to strengthen is 33%NBS.

As part of any building consent application fire and disabled access provisions will need to be assessed.

Christchurch Seismicity

The level of seismicity within the current New Zealand loading code (AS/NZS 1170) is related to the seismic zone factor. The zone factor varies depending on the location of the building within NZ. Prior to the 22nd February 2011 earthquake the zone factor for Christchurch was 0.22. Following the earthquake the seismic zone factor (level of seismicity) in the Christchurch and surrounding areas has been increased to 0.3. This is a 36% increase.

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 new building standard load requirements have been determined in accordance with the current earthquake loading standard (NZS 1170.5:2004 Structural design actions - Earthquake actions - New Zealand).

The likely capacity of this building has been derived in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) guidelines 'Assessment and Improvement of the Structural Performance of Buildings in Earthquakes' (AISPBE), 2006. These guidelines provide an Initial Evaluation Procedure that assesses a buildings capacity based on a comparison of loading codes from when the building was designed and currently. It is a quick high-level procedure that can be used when undertaking a Qualitative analysis of a building. The guidelines also provide guidance on calculating a modified Ultimate Limit State capacity of the building which is much more accurate and can be used when undertaking a Quantitative analysis.

The New Zealand Society for Earthquake Engineering has proposed a way for classifying earthquake risk for existing buildings in terms of %NBS and this is shown in Figure C1 below.

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

Figure C1: NZSEE Risk Classifications Extracted from table 2.2 of the NZSEE 2006 AISPBE Guidelines

Table C1 below compares the percentage NBS to the relative risk of the building failing in a seismic event with a 10% probability 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% probability of exceedance in the next year.

Table C1: Relative Risk of Building Failure In A

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

Appendix D
Background and Legal Framework

Background

Aurecon has been engaged by the Christchurch City Council (CCC) to undertake a detailed engineering evaluation of the building

This report is a Qualitative Assessment of the building structure, and is based on the Detailed Engineering Evaluation Procedure document (draft) issued by the Structural Advisory Group on 19 July 2011.

A qualitative assessment involves inspections of the building and a desktop review of existing structural and geotechnical information, including existing drawings and calculations, if available.

The purpose of the assessment is to determine the likely building performance and damage patterns, to identify any potential critical structural weaknesses or collapse hazards, and to make an initial assessment of the likely building strength in terms of percentage of new building standard (%NBS).

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.

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 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 document (draft) issued by the Structural Advisory Group on 19 July 2011. This document sets out a methodology for both qualitative and quantitative assessments.

The qualitative assessment is a desk-top and site inspection assessment. It is based on a thorough visual inspection of the building coupled with a review of available documentation such as drawings and specifications. The quantitative assessment involves analytical calculation of the buildings strength and may require non-destructive or destructive material testing, geotechnical testing and intrusive investigation.

It is anticipated that factors determining the extent of evaluation and strengthening level required will include:

- The importance level and occupancy of the building
- The placard status and amount of damage
- The age and structural type of the building
- Consideration of any critical structural weaknesses
- The extent of any earthquake damage

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 any 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)) be satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'. Regarding seismic capacity 'as near as reasonably practicable' has previously been interpreted by CCC as achieving a minimum of 67%NBS however where practical achieving 100%NBS is desirable. The New Zealand Society for Earthquake Engineering (NZSEE) recommend a minimum of 67%NBS.

Section 121 – Dangerous Buildings

The definition of dangerous building in the Act was extended by the Canterbury Earthquake (Building Act) Order 2010, and it now defines a building as dangerous if:

- 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
- 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
- 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
- there is a risk that that other property could collapse or otherwise cause injury or death; or
- 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 ground shaking 33% of the shaking 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.

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 of the 4th September 2010.

The 2010 amendment includes the following:

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

We anticipate that any building with a capacity of less than 33%NBS (including consideration of critical structural weaknesses) will need to be strengthened to a target of 67%NBS of new building standard as recommended by the Policy.

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.

Building Code

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

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

- Hazard Factor increased from 0.22 to 0.3 (36% increase in the basic seismic design load)
- Serviceability Return Period Factor increased from 0.25 to 0.33 (80% increase in the serviceability design loads when combined with the Hazard Factor increase)

The increase in the above factors has resulted in a reduction in the level of compliance of an existing building relative to a new building despite the capacity of the existing building not changing

Detailed Engineering Evaluation Summary Data V1.11

Location

Building Name:	Upper Riccarton Library	Reviewer:	Lee Howard
Building Address:	Unit: No: 73 Street: Main South Road, Christchurch	CPEng No:	1008889
Legal Description:	Lot 2 DP 16894	Company:	Aurecon NZ Ltd
		Company project number:	227052
		Company phone number:	03 366 0821
GPS south:	Degrees 43, Min 32, Sec 5.69	Date of submission:	Oct-12
GPS east:	Degrees 172, Min 33, Sec 45.72	Inspection Date:	Sep-12
		Revision:	1
Building Unique Identifier (CCC):	BU 3036-001 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):	
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):	10.00

Building

No. of storeys above ground:	1	single storey = 1	Ground floor elevation (Absolute) (m): 10.50
Ground floor split?	no		Ground floor elevation above ground (m): 0.50
Storeys below ground	0		
Foundation type:	strip footings		if Foundation type is other, describe: with isolated concrete pads
Building height (m):	4.70		height from ground to level of uppermost seismic mass (for IEP only) (m): 4.5
Floor footprint area (approx):	1870		
Age of Building (years):	6		Date of design: 1992-2004
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):	library		
Importance level (to NZS1170.5):	IL3		

Gravity Structure

Gravity System:	frame system		
Roof:	steel framed	rafter type, purlin type and cladding	steel purlins and rafters with precast concrete walls in the transverse directio
Floors:			100
Beams:	steel non-composite	beam and connector type	360UB51 bolted and welde
Columns:	structural steel	typical dimensions (mm x mm)	219x4.8 CHS
Walls:	partially filled concrete masonry	thickness (mm)	140

Lateral load resisting structure

Note: Define along and across in detailed report!

Lateral system along:	welded and bolted steel moment frame			
Ductility assumed, μ:	1.25		note typical bay length (m)	
Period along:	0.40	0.43 from parameters in sheet	estimate or calculation?	estimated
Total deflection (ULS) (mm):			estimate or calculation?	
maximum interstorey deflection (ULS) (mm):			estimate or calculation?	
Lateral system across:	welded and bolted steel moment frame			
Ductility assumed, μ:	1.25		note typical bay length (m)	
Period across:	0.40	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:			
Wall cladding:	other light	describe	timber, precast concrete and masonry blockwork
Roof Cladding:	Metal	describe	
Glazing:	aluminium frames		
Ceilings:	strapped or direct fixed		suspended ceiling
Services(list):			

Available documentation

Architectural	partial	original designer name/date	City Solutions 2/2/2005
Structural	full	original designer name/date	City Solutions 2/2/2005
Mechanical	partial	original designer name/date	City Solutions 2/2/2005
Electrical	none	original designer name/date	
Geotech report	none	original designer name/date	

Damage

Site:
(refer DEE Table 4-2)

Site performance:	good	Describe damage:	none noted
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:	46%	Describe how damage ratio arrived at:	
	Describe (summary):			
Across	Damage ratio:	46%		
	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 e'quakes:	90%	90% %NBS from IEP below	If IEP not used, please detail assessment methodology:
	Assessed %NBS after e'quakes:	49%		
Across	Assessed %NBS before e'quakes:	90%	90% %NBS from IEP below	
	Assessed %NBS after e'quakes:	49%		

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: 4.5m

Seismic Zone, if designed between 1965 and 1992: B

not required for this age of building: C shallow soil

Design Soil type from NZS4203:1992, cl 4.6.2.2: b) Intermediate

	along	across
Period (from above):	0.4	0.4
(%NBS)nom from Fig 3.3:	23.0%	23.0%
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		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:	**23%**	**23%**

2.2 Near Fault Scaling Factor

		along	across
	Near Fault scaling factor, from NZS1170.5, cl 3.1.6:		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):	3
Return Period Scaling factor from Table 3.1, **Factor C:**	0.90

2.5 Ductility Scaling Factor

	along	across
Assessed ductility (less than max in Table 3.2)	1.25	1.25
Ductility scaling factor: =1 from 1976 onwards; or =kμ, if pre-1976, fromTable 3.3:	1.14	1.14
Ductiity Scaling Factor, **Factor D:**	1.00	1.00

2.6 Structural Performance Scaling Factor:

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

2.7 Baseline %NBS, (NBS%)b = (%NBS)nom x A x B x C x D x E

	along	across
%NBSb:	**60%**	**60%**

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
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 value =1.5, no minimum	1.5	1.5
Rationale for choice of F factor, if not 1		

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

	along	across
3.7. Overall Performance Achievement ratio (PAR)	**1.50**	**1.50**
4.3 PAR x (%NBS)b: PAR x Baselline %NBS:	**90%**	**90%**
4.4 Percentage New Building Standard (%NBS), (before)		**90%**

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