INDEPENDENT ASSESSMENT ON EARTHQUAKE PERFORMANCE OF

WESTPAC TOWER – 166 CASHEL STREET

FOR

Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes

Report prepared by Peter C Smith and Vaughan England
Spencer Holmes Limited

February 2012

Introduction

This report has been commissioned by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes to review the performance of the Westpac Tower at 166 Cashel Street, Christchurch during the Canterbury earthquake sequence.

The report is based on documentation provided by the Royal Commission of Inquiry into building failure caused by the Canterbury Earthquakes and a limited inspection of the building on the 15th September 2011.

No analytical work has been undertaken and only that compliance information that was forwarded by the Royal Commission has been reviewed. Only in the event of an anomaly arising in the review has further investigation been considered necessary. No level survey of the building was undertaken to establish the extent, if any, of differential settlement

Location of Building

The building is located at 166 Cashel Street, Christchurch, at the intersection of Cashel Street and High Street, Cashel Street to the North.

The location of the building in the Christchurch CBD is shown on an aerial photo of Christchurch included in Appendix 1, together with the direction from the epicentre of the main earthquakes.

Geotechnical Site Assessment

At the time of writing the report we have not had access to any geotechnical reports on the site of the building.

Description of Building

The Westpac Tower building at 166 Cashel Street is a 13-storey building with a basement. The tower is inter-connected with a 3-storey podium with basement. The basement of the podium inter-connects with the basement of the tower. The Westpac Tower is of a hexagonal form with the tower orientation offset from the essentially rectangular form of the podium. The obtuse angle between High Street and Cashel Street defines the hexagonal form of the tower. The basement area of the building is utilised for car parking. The ground floor and first floor are of retail space where as the second and upper floors are of office space.

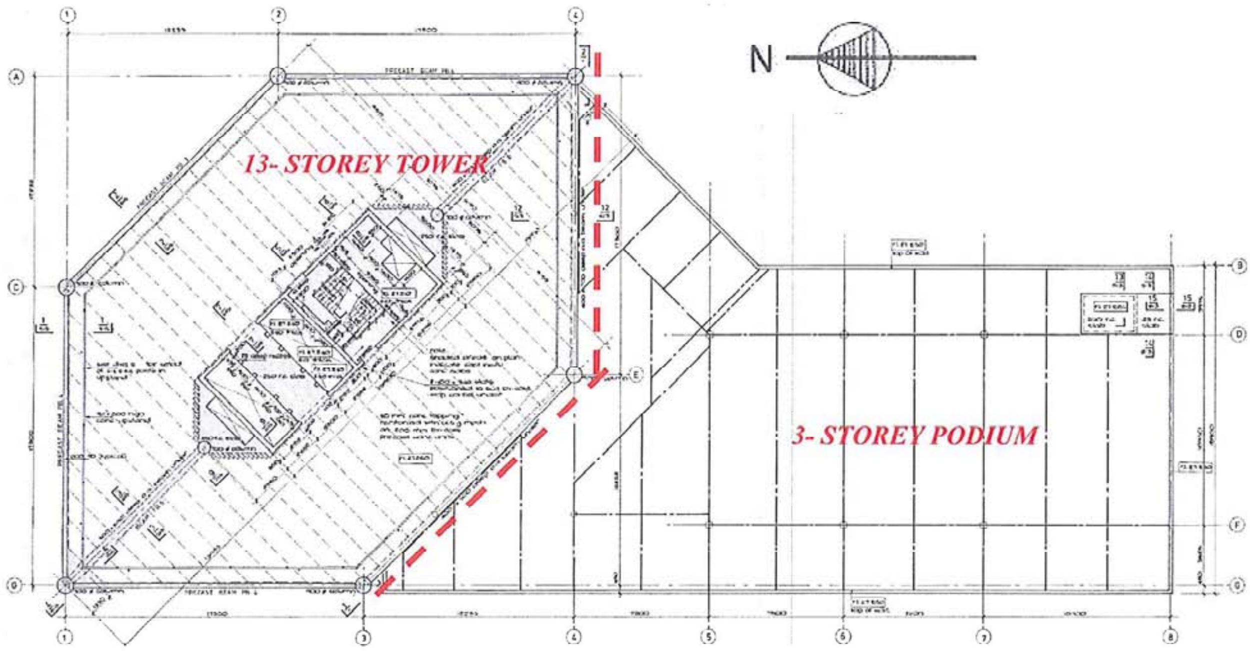
The building was designed in 1981 by Warren & Mahoney Architects, and Holmes Wood Poole & Johnstone Engineers. We have reviewed the engineering documentation set, a schedule of which is provided in Appendix 2.

Gravity System
The suspended floors are an early variety of 200mm thick Dycore pre-cast, pre-stressed units with a 50mm in-situ topping reinforced with 665 cold-worked mesh. The Dycore floor units are assumed to have been formed by setting up the pre-stressing strand in a stressing bed, placing the concrete for the bottom flange of the units, laying polystyrene fillers to form the cores of the Dycore units and then placing the concrete for the webs and top flange of the units.

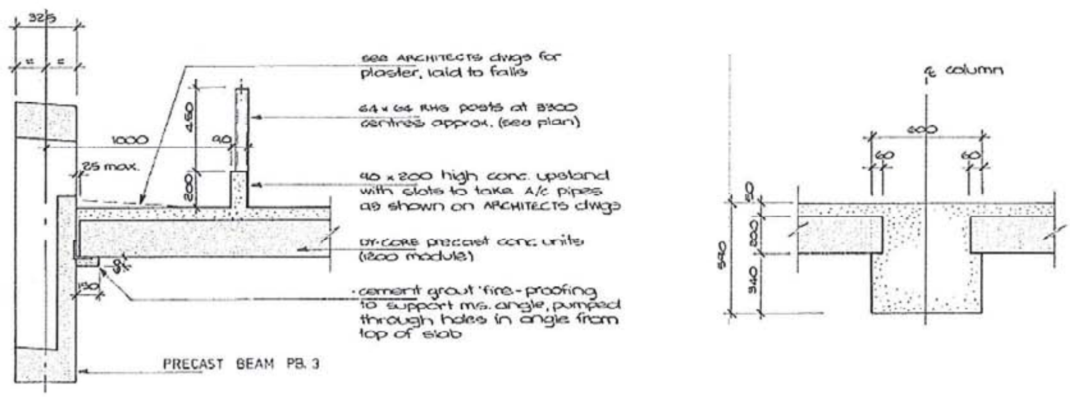
The Dycore floor units span from the corbels on the central shear core to feature long span pre-cast concrete trusses around the perimeter of the tower. These pre-cast truss units form a feature of the external elevation of the building. The Dycore units are seated onto the inner face of the pre-cast trusses using a 102mm by 102mm cast-in steel angle. The pre-cast concrete trusses are supported on 900mm diameter reinforced concrete columns at each corner of the hexagonal floor plate.

Gravity support of the Dycore floor units between the ends of the shear core and the external columns beyond the shear core at the northeast and southwest corners of the building, is provided by 60mm seating onto 590mm deep by 600mm wide beams. These beams are supported by 700mm diameter reinforced concrete columns at the ends of the shear core, and by the 900mm diameter external columns. The 700mm and 900mm diameter columns are reinforced with longitudinal steel and perimeter spiral reinforcement.

A zone of in-situ concrete floor is used between the 700mm diameter columns and the shear core. The Dycore floor units at these locations appear to be supported by in-situ concrete being cast inside the hollow voids of the Dycore.

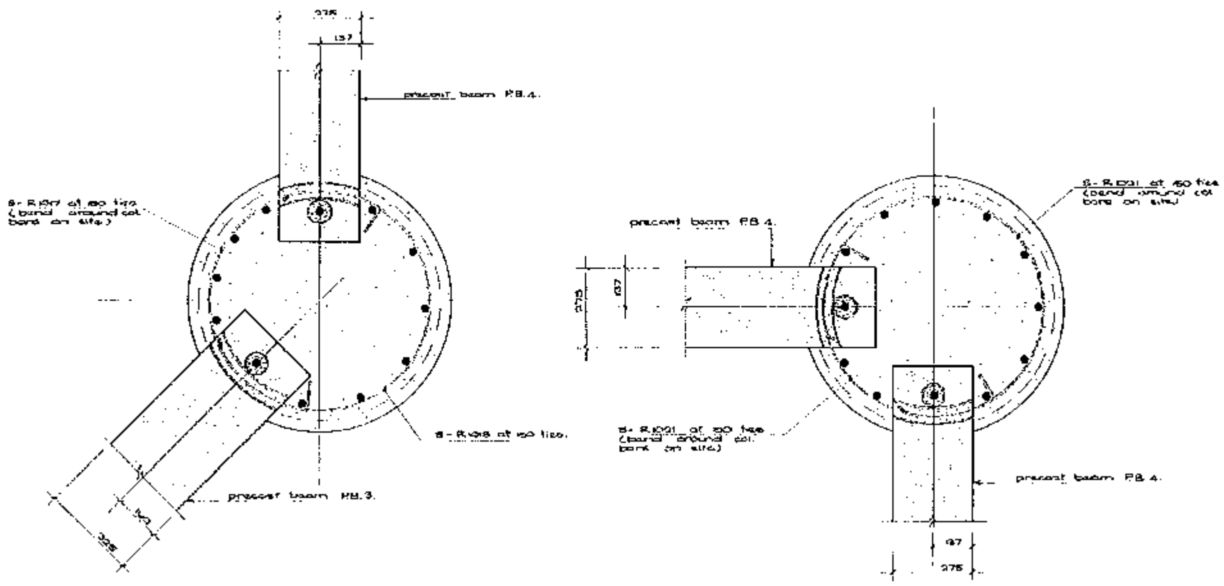


Typical Floor Plan Drawing 936 / S14



Perimeter Truss (left) and Internal Beam (right) Seating - Drawing 936 / S14

The external feature truss elements extend into the 900mm dia columns on the exterior of the building. Refer details below. As the seating for the pre-cast trusses extends through the spiral, the pre-cast truss units are detailed with curved ducts to allow the column spiral to pass through the end of the pre-cast truss sections. This reinforcement was placed once the truss units were in their final position. A longitudinal column bar is placed through a duct in the pre-cast truss and the spacing of column bars is increased at the precast truss locations.



Pre-cast truss to column details from drawing 936 / S48

Seismic System

The primary lateral load resisting system to the tower is provided in the reinforced concrete shear core, which is located centrally within the hexagonal floor plate of the building. The tower is seismically isolated from the podium with a seismic joint detailed for 25mm of movement at levels 1 and 2. The shear core lateral load-resisting system can be considered in two orthogonal directions, being longitudinal in the northwest-southeast direction, and transverse in the northeast-southwest direction. These walls are of the following thickness:

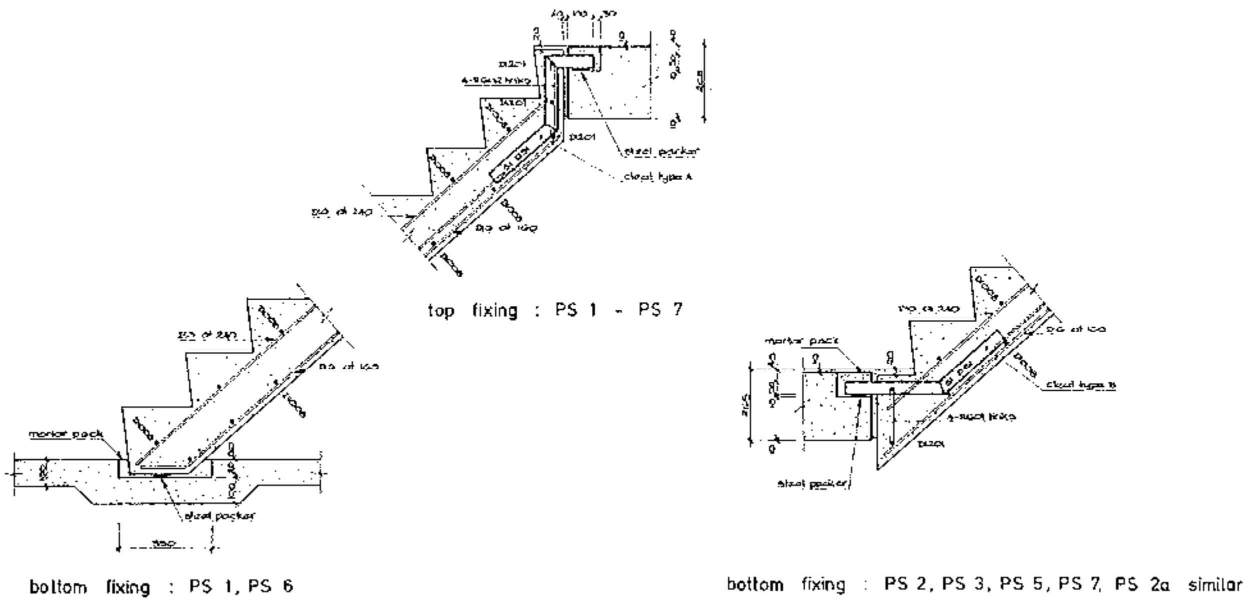
LOCATION	LONGITUDINAL	TRANSVERSE
Basement to Level 4	250 mm	300 mm
Level 4 to Level 8	250 mm	250 mm
Level 8 to Level 12	200 mm	200 mm

The perimeter frames incorporate the pre-cast truss elements. The trusses reduce in depth from 1500mm at mid span to 900mm at the column support locations and therefore it is reasonable to assume that these elements were not intended to contribute significantly to the seismic performance of the building.

The longitudinal seismic system consists of two reinforced concrete shear walls approximately 13.2 metres in length. The two walls are of similar design, each being of the same geometry and having two large openings for access to the interior of the shear core at each level. There are deep concrete lintels over these openings, which are heavily reinforced with diagonally orientated bars indicating they are intended as coupling beams for the longitudinal shear wall system. The transverse walls form end region flanges giving each longitudinal wall a C-shaped geometry. The longitudinal shear walls are connected to the overall floor system via the 50mm topping slab at each level, although we were unable to find any specific detailing of reinforcement for this connection.

The transverse seismic system consists of three reinforced concrete shear walls approximately 5.8 metres in length. The longitudinal walls form end region flanges giving each transverse wall a C-shaped geometry. The faces of the transverse walls do not have any major openings although there are very large openings in the adjacent areas of in-situ concrete floor. We were again unable to find any specific detailing of the reinforcement for the connection of the adjacent floor slab to the walls.

Secondary Elements
The main stairs are located within the shear core. They are constructed of single flight pre-cast reinforced concrete. The stairs incorporate 51 x 4mm steel CHS legs embedded in the pre-cast unit and these legs are seated and mortared into pockets formed at the in-situ concrete slab edge. The details provide little provision for inter-storey deformations.



Details from drawing 936 / S70

Foundations
The gravity loads from the structure are supported on reinforced concrete foundation beams located beyond the internal face of the shear core walls. These foundation beams consist of a 3070 deep by 3200 wide beam at the northwest and southeast sides of the shear core. These beams extend to the external columns on the northeast and southwest sides of the hexagonal face of the tower. These beams are interconnected by a 2000 wide by 3070 deep beams on the northeast and southwest sides of the shear core. There are several 3070 deep by 300 wide beams and a 3070 deep by 1000 wide beam, which interconnect between these beams within the shear core.

The northwest and southeast columns to the hexagonal tower are supported on 3000 square offset pads at the northwest corner and a 3000 square by 2500 deep pad at the southeast corner respectively.

Compliance

The building was constructed for the Canterbury Savings Bank. A building consent for the structure was issued on 7th May 1981 with the building consent for the internal fit out being issued on 1st October 1982. The documentation provided is consistent with a building permit having been issued by the Christchurch City Council in May 1981.

The compliance documentation appears to be in order, with the following consent documents having been approved by Christchurch City Council;

7th May 1981	Architectural and Structural Drawings dated 16th April 1981
1st October 1982	Architectural Fitout Drawings dated 30th August 1982
26th April 2001	Architectural Fitout Drawings dated 30th March 2001

As the building predated the introduction of the Building Act 1991, the building was not required to have a Code Compliance Certificate.

Events Subsequent to 4th September 2010 Earthquake

The building suffered structural damage in the 4th September 2010 earthquake. Documentation provided includes Christchurch Earthquake Rapid Assessment Forms dated 15th September 2010, 26th December 2010 and 23rd March 2011.

Reports Following 4th September 2010 Event
The level 2 rapid assessment dated 15th September 2010 indicated spalling to the columns and damage to the floor diaphragms requiring repair. The form indicates the existing placard was *"restricted use"* yellow and was changed to an *"occupiable, repairs required"* green placard.

The Canterbury Earthquake Royal Commission has been forwarded calculations in respect of repairs and a report titled *"Canterbury Savings Bank Building Preliminary Seismic Evaluation of Existing Building"* by Holmes Consulting Group, assumed to have been undertaken following the September earthquake. The repair calculations relate to diaphragm connections to the shear core walls, tying the perimeter columns into the floor diaphragms, and Dycore pre-cast floor unit seating. A 3-dimensional computer analysis was undertaken that compares the performance of the building to the requirements of NZS 4203:1976, the current loadings standard at the time of the building's design.

The Holmes Consulting Group report commented on the building's design as follows:

- *Excluding a detailed evaluation of the floors, the building responded much as would be expected for a structure designed according to the capacity design principles in use at the time the building was designed, and still in use.*
- *Were it not for uncertainties about the floor performance, it would be concluded from this evaluation that the performance of the building is satisfactory under seismic loads, in terms of the objectives of designs to NZS 4203.*
- *Note that, not withstanding the objectives of NZS 4203, this building would be severely damaged under this level of earthquake. There would be large flexural cracks at the base of the shear walls and in the coupling beams, with probable loss of cover concrete. Some bars would be close to fracture limits. The building would not be able to be occupied post-earthquake and repair would be very expensive, and possibly not cost-effective.*

- *Assuming pinned trusses, the floors must act to transfer inertia forces from the floor itself plus the perimeter frames to the lateral load elements in the shear core. The floor must be able to transfer these loads while simultaneously being subjected to deformations from the displacements of the shear walls.*

A further report titled *"Canterbury Centre Seismic Repairs"* prepared by Holmes Consulting Ltd dated 4th November 2010 has also been provided. The report describes the significant areas of damage in detail, identifies inspected areas requiring repair, and provides construction drawings and specifications of structural repairs.

The Holmes Consulting Group report commented on the significant areas of damage as follows;

- *Flexural and shear cracking of the lower level shear walls and coupling beams*
- *Tearing of the floor slabs adjacent to the core walls*
- *Cracking of the floor slabs adjacent to the exterior beams*
- *Damage and spalling of seismic gaps*
- *Spalling of external columns (and minor rusting of reinforcing exposed)*
- *Destruction of level 13 non-structural cladding (glazing)*

The Holmes Consulting Group report details structural repairs on the following items;

- Fibre reinforced polymer overlay to the floor.
- Fibre reinforced polymer ties into external columns.
- Mortar reinstating spalled column concrete with fibre reinforced polymer overlay.
- RHS sections for Dycore support at in-situ concrete slab locations.
- Steel angle seating for Dycore at seismic gap.

Reports Following 26th December 2010 Event
A level 1 rapid assessment dated 26th December 2010 recorded moderate damage to walls and other structural elements with a comment that there were *"no obvious signs of new damage"*. The form indicates an *"occupiable"* green placard was applied with no further action recommended.

Reports Following 22nd February 2011 Event
A level 2 rapid assessment dated 23rd March 2011 recorded moderate damage to the external reinforced concrete columns at the pre-cast truss connections, which is consistent with the previous assessments. The form indicates the existing placard was *"unsafe"* red, and was changed to a *"restricted use, no entry to parts until repaired or demolished"* yellow placard.

It also records that the building was empty having been undergoing repairs following the September 2010 event, and recommends the building owner's engineer carry out a detailed reassessment of the repair methodology.

An interim report titled *"Canterbury Centre Seismic Repairs"* prepared by Holmes Consulting Ltd dated 8th August 2011 has also been provided.

The report describes the significant areas of damage in detail, identifies inspected areas requiring repair, and provides construction drawings and specifications of structural repairs.

The Canterbury Earthquake Royal Commission engaged Holmes Solutions to undertake some material testing. A report was prepared by Holmes Solutions titled *"Report 107267-1 v1.1, Materials Testing in Buildings of Interest, November 2011"* by Holmes Solutions, which is attached in Appendix 4.

The testing involved destructive testing of concrete cores and non-destructive Schmidt Hammer testing to determine the compressive and tensile properties of the concrete. Holmes Solutions comments on the building's concrete properties were as follows:

No significant variations in the concrete strengths were noted between the precast and in-situ concrete items at the Westpac Centre.

The pre-cast elements tested in the report did not include the concrete trusses on the exterior of the building. We are yet to receive the results of testing on these trusses, but it is expected that their concrete strength would be in the order of double that of the insitu columns.

Holmes Solutions also undertook testing of the reinforcing steel at selected areas of damage and commented on the building's reinforcing steel properties as follows:

The material properties of the reinforcing steel were investigated in zones of damage in the building, to determine the likely damage the earthquake has induced in the steel, and control samples in areas away from the damage. The use of Leab hardness testing has been shown to provide a strong correlation with the peak strain the steel has been subjected to during in-elastic loading cycles and is become increasingly adopted as a tool for assessing structural damage.

The results from the testing indicated that the reinforcing steel in the Westpac Centre had undergone previous inelastic strain cycles of between 2% and 8%.

Structural Performance

Design Standards

The Westpac Tower was built in 1983 and was designed to the loadings standard NZS 4203:1976. The building is now 28 years old, having reached approx 50% of its design life. The introduction of the Building Act 1991 and multiple significant revisions of the relevant loadings and material design standards have occurred since the building was designed.

A review of the performance of the Westpac Tower under the Canterbury Earthquakes must make allowance for advances in the seismic design of structures that have occurred since the building was designed.

A comparison of the NZS 4203:1976 and NZS 1170.5:2004 seismic design coefficients was made assuming a building period of 1.2 seconds, deep site subsoils, and limited ductile performance. The seismic coefficient for the 1976 standard was 0.065 using seismic zone B and the equivalent seismic coefficient for the current standard is 0.087 using a hazard factor of Z = 0.22. This means the building is likely to have been designed to 75% of the lateral load required by the current loadings standard.

While the building is likely to have achieved the performance objectives of the design codes prior to the introduction of the Building Act 1991, a review of the building performance identified the following design aspects that need to be enhanced to achieve resilient building performance.

Spalling and Compression Buckling of Shear Wall Reinforcement above Level 2

Inspection of the shear core walls establishes that the more significant inelastic deformation and spalling of the walls occurred above level 2. The walls were designed on the assumption that inelastic deformations would occur below level 1 where the end zones of the walls were detailed with greater confinement.

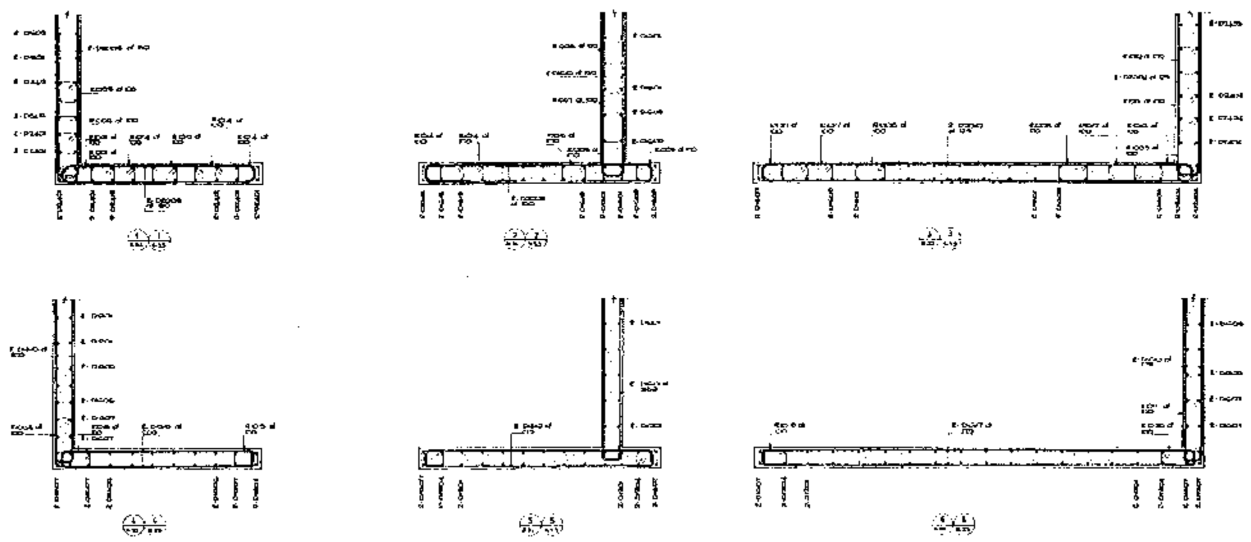
A review of the documentation of the shear walls establishes that the shear walls between the basement level and level 1 are detailed for inelastic deformation where as the shear walls above level 1 are detailed to a lower level of inelastic resilience.

The damage to the seismic joint at level 1 indicates that impact between the tower and the podium at level 1 may have occurred and could have contributed to causing the inelastic deformations at level 2.

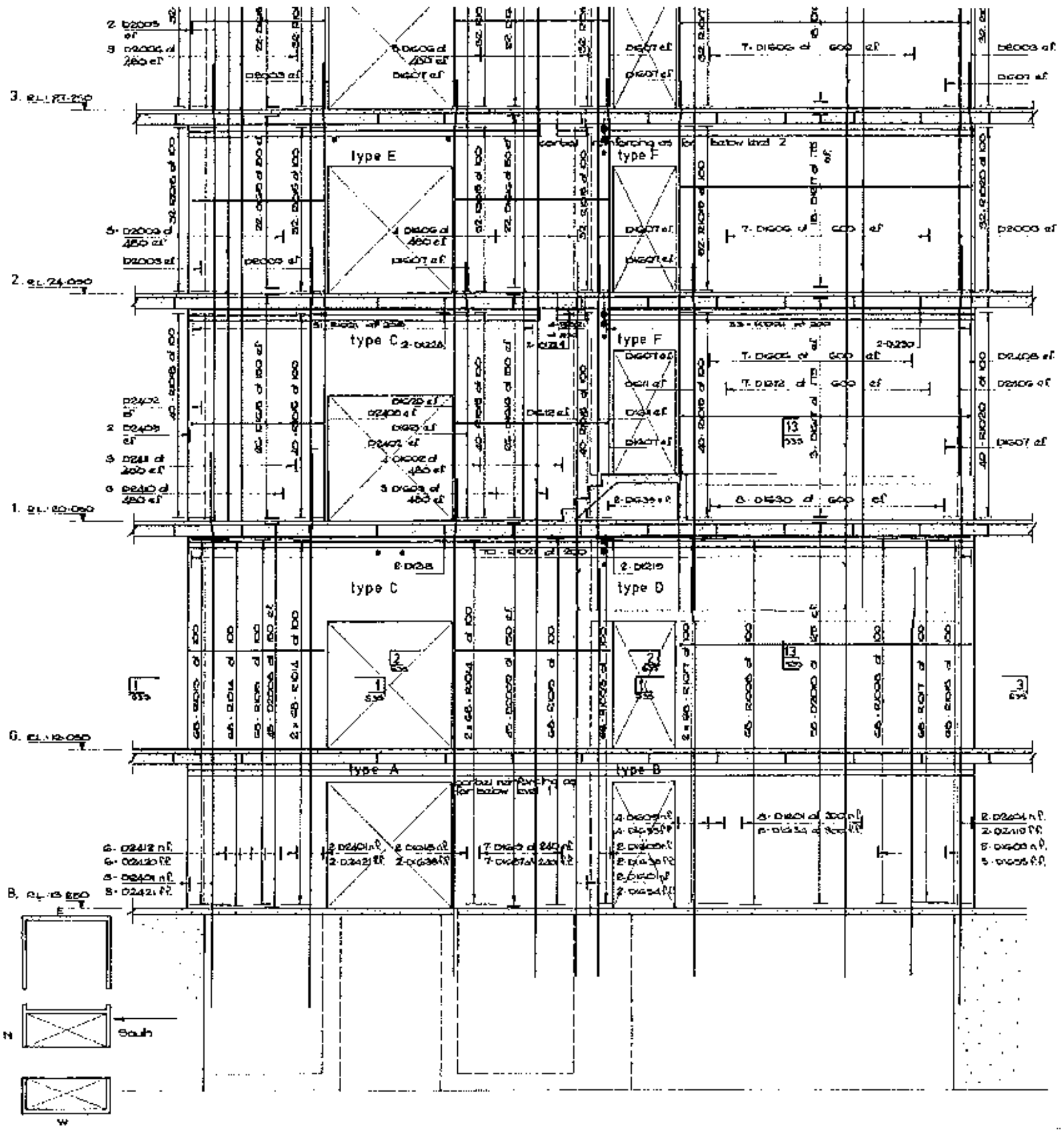
The current standard NZS 3101:2006 clause 11.4.3 states:

Potential plastic hinge regions in walls shall be taken as the length of the wall L_w or one-sixth of the height of the wall, whichever is larger, measured from the section at which the first flexural yielding is expected. The height of the end region need not exceed 2 L_w.

Buckling of the main tensile reinforcement occurred in the areas of greatest inelastic deformation. (Refer Photos in Appendix 3). The importance of maintaining stirrup spacing at distances that prevent buckling of longitudinal reinforcement is obvious. The spacing of stirrups in the area of main reinforcement buckling was measured as 120mm. The longitudinal reinforcement size that buckled was 16mm deformed bar (D16), giving a relative spacing of 7.5 diameters. The spacing of lateral restraint reinforcement detailed on the plans was 100mm, or 6.25 diameters, which is close to the current code requirements of 6 diameters. Refer photo in Appendix 3. The difference of 20mm emphasises the importance of engineering inspection of those aspects of construction that are critical to achieving the inelastic performance necessary to achieve the design level of ductility. As the performance of the building under seismic loading is determined by the widest spacing of stirrups in the areas of inelastic demand, consideration should be given to all areas of expected inelastic deformation being inspected and approved to level CM4 construction monitoring.



Detail drawing 936 / S35



Detail drawing 936 / S33

Damage to Floor at Podium Seismic Joint

The concrete either side of the level 1 seismic joint is damaged indicating that combined inter-storey deflections at this location exceeded the as-constructed joint width. The joint is detailed as having 25mm of movement between the tower and the podium at level 1 and 2.

Outward Movement of the Perimeter Columns

Inspection of the building establishes that the ends of the pre-cast trusses are effectively anchored in the insitu columns and that lateral displacements of the building under seismic loads have induced inelastic deformations in the precast trusses. These inelastic deformations have resulted in elongation of the trusses (beam- hinge elongation) which, through the orientation of the beams has induced sufficient force on the column to caused separation of the floor and the 900mm diameter circular columns at the column locations around the outer perimeter of the building. The outward movement of the columns was in the order of 25 to 30mm at most floors. (Refer Photos in Appendix 3).

A review of the plans established that the attachment of the columns to the floor relied on continuity of the main reinforcement in the pre-cast truss elements into the columns and 50mm long inserts at 400mm centres embedded in a 100mm thick concrete up-stand formed integrally with the pre-cast trusses.

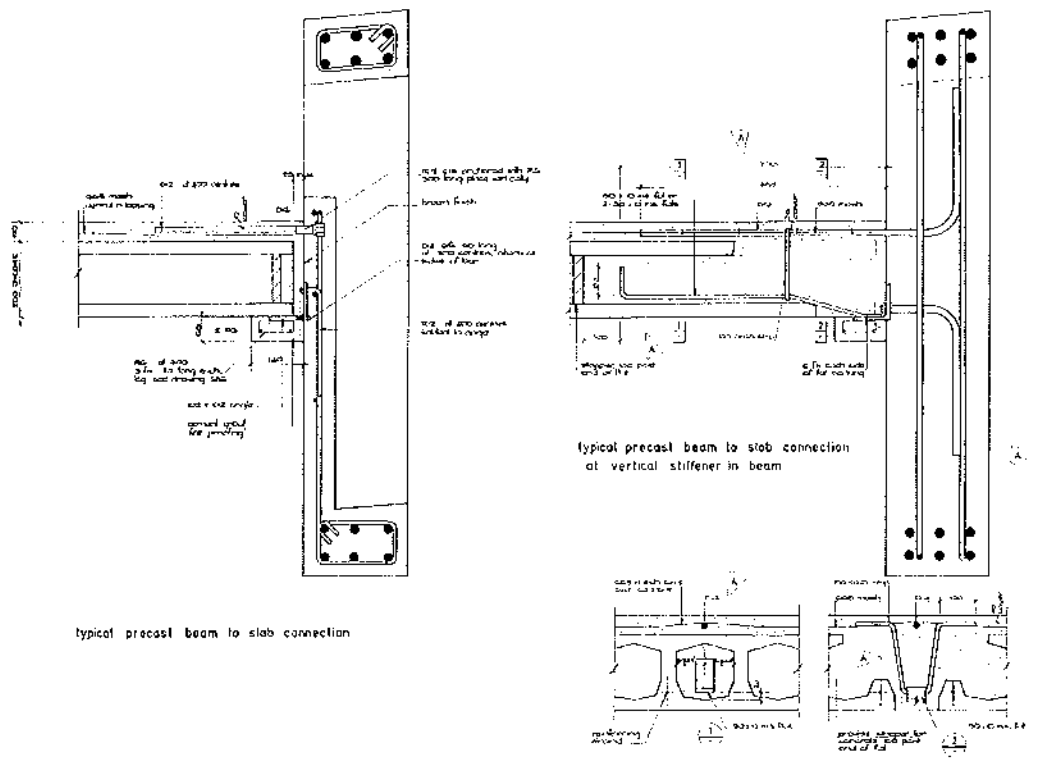
Damage to the pre-cast truss-column connection and the tearing of the floor from the pre-cast truss near the column demonstrated the care required in detailing such connections to accept the displacement-induced curvatures under moderate to severe seismic events if the joint is not truly pinned.

Clause 10.3.6 of NZS 3101:2006 states:

Columns at the perimeter of the floor to be tied into the floor by either reinforced concrete beams or reinforcement provided in the topping. The tie reinforcement should be effectively anchored perpendicular to the frame and capable of resisting the larger of 5% of the maximum total axial compression load on the column or 20% of the column shear force induced by lateral design forces in the storey below the load considered.

It is evident that inelastic deformations in the perimeter beams to a building which are arranged in plan other than in alignment will induce greater forces than set out in NZS 3101. Any attempt to restrain the column will develop high axial forces in the pre-cast trusses.

Separation of Floor from Pre-Cast Trusses in the Vicinity of the Perimeter Columns
Outward movement of the 900mm diameter columns due to inelastic deformations at the pre-cast truss-column interface has also caused outward movements between the truss and the floor over a distance of several metres either side of the columns. This movement has likely caused a failure of the 50mm deep inserts embedded in the 100mm thick up-stand of the pre-cast trusses, or failure of the starters threaded into the inserts under the forces induced through inelastic deformations at the truss-column connection. Furthermore, the use of shallow inserts or inserts embedded in thin singly reinforced concrete elements in such situations is not recommended.



Dycore seating on trusses from drawing 936 / S68

Spalling of the Perimeter Columns Initiated at the Pre-Cast Truss-Column Connection
A noticeable feature of the damage to the 900mm diameter columns to the perimeter of the Tower is the depth of cover and the steep angle of the interface of the spalled concrete, an interface that extended over half the height of some columns (Refer to photos in Appendix 3).

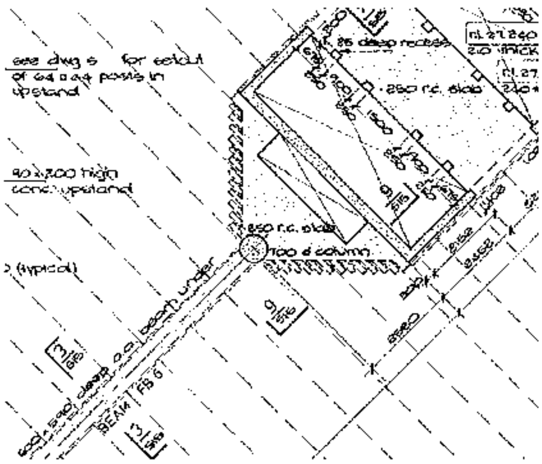
The use of cover concrete in excess of the minimum requirements not only creates a danger from falling concrete, but also results in a greater loss of strength when spalling occurs than where minimum code cover was provided.

The presence of construction joints within the columns at the pre-cast truss column junction created by seating the ends of the pre-cast trusses on the 900mm circular columns during construction of the building contributed to the extent of damage at the pre-cast truss- 900mm dia column connections. Continuing the spiral column reinforcement through the depth of the truss by use of spirals proved reasonably effective in protecting the column at the joint, but the weakness induced through the projection of the pre-cast truss into the column reduced the resilience of the joint. Testing of the concrete to the circular in-situ columns established concrete strength in the order of 20-25MPa. The concrete strength to the precast truss is yet to be tested but it is expected that the concrete strength of the trusses was approximately double that of the columns. The significant difference in strength combined with the projection of the truss into the column is assessed to have significantly contributed to the damage that occurred to the pre-cast truss-columns at the joint.

Loss of Glazing at Level 13
At the time of inspection the external glazing to level 13 was absent. We understand that the glazing failed in the September 2010 earthquake. Inspection of the structure and the structural drawings establish that the roof to the upper floor was fully supported off the shear core. To achieve a visual effect, the roof was not supported around the perimeter of the upper level. Deflections and rotations at the top of the shear wall are magnified by the distance to the external walls. Deflections and rotation of the shear walls under lateral load have created significant deflections of the roof at the end of the cantilever support off the shear walls. There is a need for a conservative approach to the provision of seismic movement during moderate to severe seismic events in such locations and the detailing for a suitable margin beyond the deformations calculated at the ultimate limit state.

In-situ Slab Supporting Dycore Units
A 590mm deep by 600mm wide reinforced concrete beam spans from the external columns at the northwest and southeast ends of the hexagonal floor plate, and terminates at the 700mm diameter internal columns at the northwest and southeast of the shear core. An in-situ wedge of 250mm thick floor provides the support of the Dycore over the span from the internal column to the shear core. Refer drawing 936/S15. The in-situ slab adjacent to the northwest and southeast ends of the shear core is penetrated by a 3200mm by 700mm services duct so that the Dycore is in effect supported by a strip of 250mm thick in-situ slab between the outer corners of the shear core and the internal 700m diameter column.

The presence of the duct opening and the displacements imposed on the in-situ slab due to displacements of the shear core have resulted in damage to the in-situ slab, the integrity of which is important to the support of the adjoining Dycore floor units. Areas of gravity support need to be detailed to accept displacement-induced deformations under severe seismic events.



Area of in-situ concrete slab adjacent to shear core on drawing 936 / S15

Issues Arising from Review

A review of the documentation supplied by the Canterbury Earthquake Royal Commission identifies that the Westpac Tower at 166 Cashel Street is a 13-storey reinforced concrete building inter-connected with a 3-storey podium. The building was designed in 1981.

Post-earthquake reports on the damage to the building following the 4th September 2010 and 22nd February 2011 earthquakes were prepared by Holmes Consulting Group.

1. **Concrete Shear Wall Detailing**
   Inspection of the shear core walls establishes that the more significant inelastic deformation and spalling of the walls occurred above level 2. The walls were designed on the assumption that inelastic deformations would occur below level 1 where the end zones of the walls were detailed with greater confinement.

   A review of the documentation establishes that the shear walls between the basement level and level 1 were detailed for inelastic deformation, but the shear walls above level 1 were detailed to a lower level of inelastic resilience.

   The current standard NZS 3101:2006 clause 11.4.3 states "*potential plastic hinge regions in walls shall be taken as the length of the wall L_w or one-sixth of the height of the wall, whichever is larger, measured from the section at which the first flexural yielding is expected. The height of the end region need not exceed 2 L_w*".

   - It is suggested that ductile detailing of shear walls should not end abruptly, but extend above expected zones of inelastic deformation and reduce gradually to prevent a brittle failure of upper levels.

   - For thin walls subject to inelastic deformations, further consideration needs to be given to the resilience of the concrete core of the wall, and in particular the ability to confine an adequate end region to resist compression loads following spalling under repeated inelastic deformations.

   - Consideration needs to be given to the confinement and lateral restraint of horizontal reinforcement following spalling of cover concrete under repeated inelastic deformations. It is suggested that both the horizontal reinforcement and vertical reinforcement should be enclosed with ties, thereby restraining both the vertical and horizontal bars against buckling following the loss of cover concrete. It is also noted that the loss of cover concrete can compromise the bond to the outer reinforcement which will also alter the structural behaviour of the wall.

- Further emphasis needs to be placed on construction monitoring in areas requiring ductile performance to ensure reinforcement providing lateral restraint and confinement is placed correctly with minimal tolerance of variations from the design details.

2. Seismic Joints

The damage to the seismic joint at level 1 indicates that impact between the tower and the podium at level 1 may have occurred and could have contributed to causing the inelastic deformations of the shear core at level 2.

- Seismic joints need to be sized to provide for an adequate margin above the combined elastic and inelastic deformations of adjacent buildings under the ultimate limit state.

- It is suggested that even where sufficient seismic gaps have been provided for an ultimate limit state event, robust detailing be specifically required in areas where impact of adjacent structures may cause critical damage. This will provide resilience of the structure under the occurrence of higher than expected displacements, and under the maximum credible earthquake load case.

3. Connection of Perimeter Columns to Floor Diaphragm

A review of the documentation established that the attachment of the columns to the floor relied on continuity of the main reinforcement in the pre-cast truss elements. The current standard NZS 3101:2006 clause 10.3.6 requires "*tie reinforcement shall be effectively anchored perpendicular to the frame*".

- Consideration needs to be given to the adequacy of NZS 3101:2006 clause 10.3.6 for resisting the load exerted on a column by inelastic deformation of the interconnecting beams where the beams connecting into the column are other than in alignment on each side of the column.

- When providing tie reinforcement within the topping slab, the minimum thickness required for the topping slab stated in NZS 3101:2006 clause 10.3.6 should be specifically addressed to accommodate the size of reinforcement bar used.

- The feasibility of restraining a column when the beams are other than in alignment and the beams are subject to the effects of inelastic beam elongation in seismic resisting frames or gravity frames subjected to induced building deformations needs to be investigated.

4. Perimeter Beam Connection to Floor Diaphragm

Outward movement of the columns has also caused separation between the pre-cast truss and the floor over a distance of several metres either side of the columns. This movement has likely caused a failure of the 50mm deep inserts embedded in the 100mm thick up-stand of the pre-cast trusses, or failure of the starters threaded into the inserts under the forces induced through inelastic deformations at the truss-column connection.

- The use of shallow anchors embedded into singly reinforced thin concrete members for transferring seismic loading does not provide an adequate level of resilience under ultimate limit state seismic loads, in areas subjected to inelastic deformation.

- Furthermore, it is suggested that all concrete elements with embedded inserts used for transferring seismic loading shall be detailed for ductile behaviour to ensure resilience of the connection when subjected to inelastic deformations of the structure. Designers shall not rely on the load capacity of inserts stated by suppliers without properly considering the resilience of the concrete embedment area under seismic loads.

5. Concrete Spalling at Truss-Column Connection

The embedment of the precast beams within the columns appears to have accentuated spalling of the concrete cover to the perimeter columns. The particularly steep and extensive spalling that occurred, frequently over in excess of half the height of the columns was of concern..

The presence of complex construction joints created by embedment of the ends of the precast truss elements within the columns is assessed as having contributed to the extent of damage. The projection of the trusses into the columns, and the termination of a column bar above and below the truss appear to have reduced the resilience of the joint. Although it is yet to be confirmed, an expected significant difference in concrete strength between the pre-cast trusses and the in-situ columns is also assessed to have contributed to the damage.

- Embedment of the ends of structural members in joints of concrete frames that are likely to experience significant inelastic deformations, whether primary seismic frames or gravity frames subject to seismic induced building deformations, should be avoided. Construction joints create undesirable planes of weakness detrimental to maintaining structural integrity under severe earthquake loading.

- The detailing of end connections for concrete elements that carry gravity loads but that do not form part of the seismic resisting system must be capable of accepting the expected building deformation without loss of load capacity and without adversely affecting the seismic resisting system or the overall stability of the structure in an ultimate limit state event.

- Further emphasis needs to be placed on detailing the construction methodology and carrying out the construction monitoring for concrete construction joints and grouted ducts. The preparation of the joint surfaces and the methodology for grouting ducts is often critical to achieving structural reliability.

6. Deformation Compatibility of Non-Seismic Structural Elements

The deformations imposed on the in-situ slab due to inter-storey displacements and inelastic deformation of the shear core have resulted in damage to the slab, the integrity of which is important to the support of the adjoining Dycore floor units.

The deformations at the outer edge of the roof induced by the rotation of the shear walls have exceeded the detailed clearances, causing failure of the glazing. NZS 3101:2006 clause 2.6.1.1 states *"the structure and its component parts shall be designed to have adequate ductility at the ultimate limit state for load combinations including earthquake actions"*.

- Structural elements required to carry gravity loads that do not form part of the seismic resisting system should be detailed to accept the deformations imposed as a result of the displacements of the seismic resisting system under the ultimate limit state. The provision of deformation capacity must be inclusive of an adequate margin over the expected elastic and inelastic deformations of the primary seismic resisting elements, and the distortions of any other building component attached to the element under consideration.

- There is a need for an adequate margin beyond the provision of seismic movement under the ultimate limit state event around non-structural elements subject to brittle failure.

- This provision for seismic movement must also consider vertical movement concurrently, especially in the case of structural elements that are particularly susceptible to vertical displacements such as cantilevers.

Report Prepared By:-

Vaughan England
BE(Hons) GIPENZ
Senior Structural Engineer

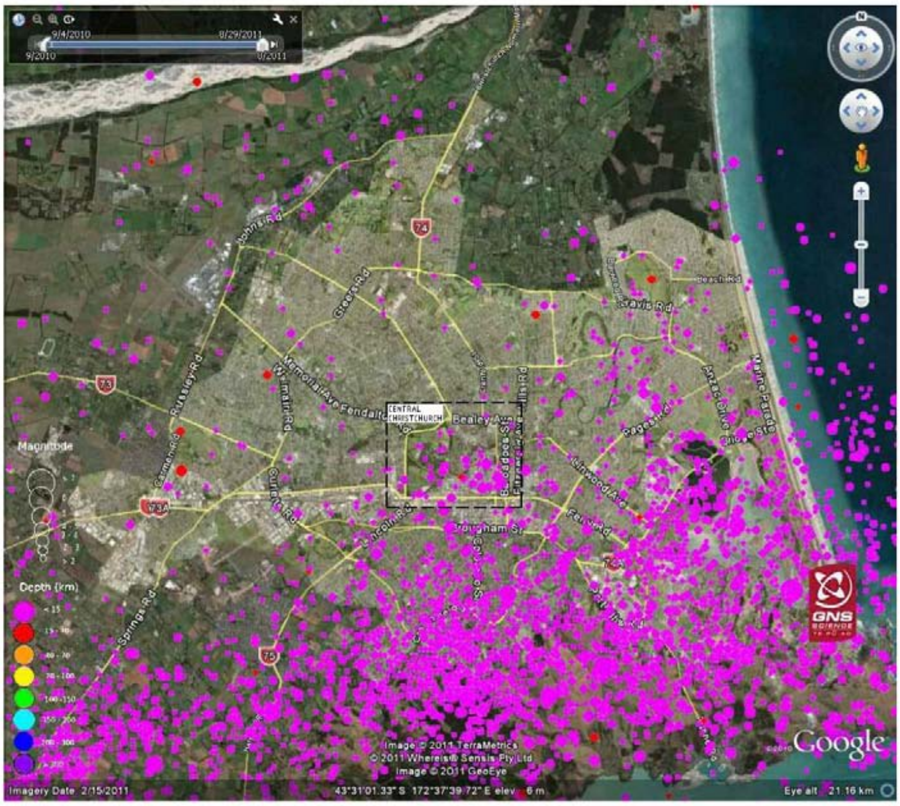
Report Reviewed By:

Peter C Smith
BE FIPENZ CPEng IntPE
Director

G:\2011jobs\110600-19\110604\166 Cashel Street - Westpac\FINAL REPORT\110604 166 Cashel St - Report FINAL.doc

APPENDIX 1

Site Plan


Magnitude
Depth (km)
Google
CENTRAL CHRISTCHURCH
Bealey Ave
Papanui Rd
Travis Rd
Russley Rd
Riccarton Rd
Springs Rd
Imagery Date 2/15/2011
Eye alt 21.16 km


NORTH
Colombo St
High St
National Mutual
Tattersalls Ln
Mw7.1 – 4th SEP 2010
No.166
Mw6.2 – 22nd FEB 2011
High St
Cashfields Arcade
Kivers Ln
Lichfield St

APPENDIX 2

List of documentation for report

The following documents (electronic file names listed) have been reviewed as part of this report:

1. 166 Cashel Street City Scanned Documents, April 1981
2. 166 Cashel Street City Scanned Documents(2), May 1981
3. 716 166 Cashel Street City Scanned Documents, October 1982
4. 166 Cashel Street City Scanned Documents(3), April 2011
5. ABA10107610 Application – Structural Calculations, October 2010
6. ABA10107610 Processing – Mid-Floor Column Points (Processed), November 2010
7. ABA10107610 Consent – Specification & Supporting Documents – DRAFT, June 2010
8. CDB75001631 EQ Rapid Assessment 2011-03-23, March 2011
9. CDB75001631, April 2011
10. EQ Rapid Assessment 1&2 Level Cashel Street 166 – 2010-09-05, December 2011
11. EQ SBP 44, Westpac Tower, 166 Cashel Street – Appendices 2011-06-08, June 2011
12. 105400 BORP R4 10-11-05, November 2010
13. 106356 Detailed Seismic Assessment Report Rev 3, August 2011
14. 107261-1(v1.1) – Materials Testing in Buildings of Interest, Holmes Solutions, November 2011

APPENDIX 3

Specific photographs of damage following 22nd February and 13th June 2011 earthquakes (Taken on 15th September 2011)

Spalling of shear wall level 1

Buckling of vertical reinforcement level 1

Spalling of shear wall level 2

Buckling of vertical reinforcement level 2

Variation in spacing of restraint at location where reinforcement has buckled

Buckling of reinforcement at corner of shear wall coupling beam

Spalling of circular external column

Spalling of circular external column at junction with pre-cast truss

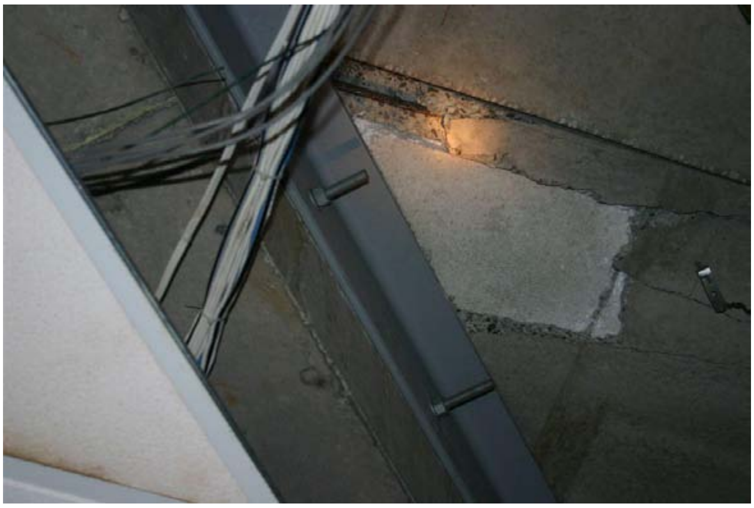
Spalling of circular external column

Spalling of circular external column at junction with pre-cast truss

Tearing between pre-cast truss and floor

Damage to seismic joint

Damage to seismic joint



Retrofitted steel angles for seating of Dycore floor units

Spalling of Dycore concrete has exposed the pre-stressing strand

View of roof structure cantilevering out from the shear core

APPENDIX 4

Holmes Solutions Report "107267-1 v1.1, Materials Testing in Buildings of Interest, November 2011"

HOLMESSOLUTIONS

REPORT 107267-1 (v1.1)

PREPARED FOR ROYAL COMMISSION

NOVEMBER 2011

MATERIALS TESTING IN BUILDINGS OF INTEREST

GALLERY APARTMENTS
WESTPAC CENTRE
IRD BUILDING

HOLMESSOLUTIONS

DISCLAIMER

This document was prepared by Holmes Solutions Ltd (HSL) under contract. The information presented in this document relates to non-destructive structural load testing and does not address any other related or un-related issues, including but not limited to environmental durability of the product, nor applications for the tested product. It is the responsibility of the user to assess relevant performance of the product and determine suitable applications.

This document does not constitute a standard, specification, or regulation. In undertaking the testing described in this report, Holmes Solutions have exercised the degree of skill, care, and diligence normally expected of a competent testing agency. The name of specific products or manufacturers listed herein does not imply endorsement of those products or manufacturers.

Report Produced by:

Dr Chris Allington, B.E (Hons), PhD (Civil)
MANAGING DIRECTOR

Report Reviewed by:

Wouter von Toor, B.E (Hons)
SENIOR MECHANICAL ENGINEER

Rev No.	Date	Revision
V1.0	15/11/11	Issued for client review

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

1.0 EXECUTIVE SUMMARY

Following the recent Christchurch earthquakes significant structural damage was noted in a large number of buildings in the Christchurch CBD. In particular, a number of buildings appear to have undergone greater damage than previously expected. The Royal Commission appointed an engineering team to review the damage in a number of building in the CBD in an effort to gain a greater understanding of the buildings behaviour under the induced seismic loads. From this investigation, a series of three buildings were identified as requiring materials testing to be completed, namely the Gallery Apartments on Glouster St, the Westpac Centre on Cashel St, and the IRD building on Cashel St. Holmes Solutions was commissioned to undertake the required materials testing.

All three buildings requiring investigation are reinforced concrete, with a mixture of precast concrete and in-situ cast concrete elements. The Royal Commission requested a series of destructive and non-destructive testing to be completed on the concrete and reinforcing steel used in the buildings. Furthermore, Holmes Solutions was independently engaged by external third parties working for the owners of the building to undertake additional testing on the reinforcing steel in the Westpac Centre and IRD building.

Testing of the concrete elements included the removal of concrete cores for destructive testing to determine the tensile and compressive properties of the concrete. Additional non-destructive testing was completed using Schmidt Hammer testing in the buildings.

The material properties of the reinforcing steel were investigated in zones of damage in the building, to determine the likely damage the earthquake has induced in the steel, and control samples in areas away from any noted damage. The use of Leeb Hardness testing has been shown to provide a strong correlation with the peak strain the steel has been subjected to during in-elastic loading cycles and is become increasingly adopted as a tool for assessing structural damage.

The results from the testing indicated that the reinforcing steel in the Westpac Centre had undergone previous inelastic strain cycles of between 2% and 8%. The reinforcing steel testing in the IRD building showed significant reduction in strain capacity with only 2% strain capacity remaining.

Concrete strength results for the Gallery Apartments indicated that the walls had compressive strengths of 46 MPa to 56 MPa, with associated tensile strengths ranges from 3.4 MPa to 2.6 MPa respectively.

No significant variations in concrete strengths were noted between the precast and in-situ concrete items in the Westpac Centre.

Concrete results from the IRD indicated that the precast concrete was stronger than the in-situ concrete elements by approximately 10 MPa.

2.0 TEST METHODOLOGY

2.1. CONCRETE CORE TESTING

A series of concrete core samples, approximately 100 mm in diameter, were removed from elements in the Gallery Apartment and the Westpac Centre. The cores were removed using a diamond tipped drilling head. Wherever possible, samples were taken from areas showing no physical damage and remote from reinforcing steel embedded in the concrete. If a reinforcing bar was impacted by the drilling head, the sample was discarded and an alternative sample taken from a nearby position. Prior to removing the core, the orientation of the sample was clearly identified to allow the subsequent testing to be undertaken in the correct orientation.

The concrete cores were subjected to either tensile splitting tests or compression testing. All tensile splitting tests were performed to the specific requirements of NZS 3112: 1986, Pt 2, Clause 8. Care was taken to ensure the samples were oriented as per location in the building. All samples were prepared in accordance with the standard prior to completion of the testing.

All concrete cores subjected to compression testing were firstly capped, in accordance with the requirement of NZS 3112: Part 2: 1986, clause 4. Once the capping material had achieved the required hardness the samples were tested in accordance to NZS 3112: Part 2: 1986, Clause 6.

2.2. TENSILE STEEL TESTS

A series of steel samples, approximately 500 mm long were removed from the Westpac Centre and the IRD building. Steel samples from the Westpac centre were obtained from zones of noted damage in the building and additional samples collected from areas that appeared to be free of visual damage to act as control samples and provide a true measure of the stress-strain properties of the parent steel. Prior to their removal from the Westpac Centre, all steel bars were subjected to Leeb Hardness testing in-situ.

2.3. LEEB HARDNESS

Leeb hardness is a direct measure of a materials dynamic hardness and is considered to be accurately measuring the materials elastic and plastic hardness characteristics. Leeb hardness is obtained by firing an impact body containing a permanent magnet and a very hard indenter sphere towards the surface of the test material and measuring the velocity of the impact body. The velocity is measured in three main test phases;

- Pre-impact phase, where the impact body is accelerated by spring force towards the surface of the test piece.

- Impact phase, where the impact body and the test piece are in contact. The hard indenter tip deforms the test material elastically and plastically and is deformed itself elastically. After the impact body is fully stopped, elastic recovery of the test material and the impact body takes place and causes the rebound of the impact body.

- Rebound phase, where the impact body leaves the test piece with residual energy, not consumed during the impact phase.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

The Leeb hardness is determined by calculation, relating the three recorded velocities. The velocities are measured in a contact-free means via the induction voltage generated by the moving magnet through a defined induction coil mounted on the guide tube of the device. The induced voltage is directly proportional to the velocity of the magnet and therefore used to determine the hardness of the steel sample.

Recent research has shown that hardness can be used as an indicator of the current strain state of steel samples [G1, L1, M2, N2, N3]. Relating the hardness of steel samples to the stress-strain properties of the base material allows an understanding of likely damage (or loss of strain capacity) that the steel sample has sustained and therefore to determine how much residual strain capacity the sample retains. This form of direct comparison can only be achieved if suitable correlations are developed between the measured hardness and the strain state of the specific steel sample.

Holmes Solutions has completed extensive research into the correlation between Leeb hardness and the steel samples strain state for a range of different reinforcing steels. The results from the research have been developed into a series of multi-dimensional correlation factors. When combined with a series of normalisation techniques we can use the measured Leeb hardness results to provide an indication as to the current strain state of the tested steel sample. The degree of uncertainty in the recorded measurements is decreased through the physical testing of a control section of the steel to a uniaxial tension test and undertaking hardness measurements at a series of predefined stress and strains. The resulting correlation is used, in conjunction with the normalisation techniques derived from obtaining numerous hardness readings in the area surrounding the expected zone of damage, to determine the value of strain in the steel from the recorded Leeb measurements. These results are then directly compared to the properties of the parent material to estimate the potential reduction in strain capacity that has been sustained by the steel sample.

Leeb readings are collected from in-situ reinforcing bars. The surface of the bars is carefully prepared to specific requirements prior to testing. Readings are obtained at critical locations along the length of the reinforcing bar to allow the strain profile of the steel to be determined and to assist in the normalisation procedures.

The overall estimation of strain degradation for the tested steel samples is achieved by using the derived strain damage from the Leeb testing in conjunction with engineering knowledge of the particular application.

All in situ hardness testing is completed in accordance with ASTM A959-06 Standard Test Methods for Leeb Hardness Testing of Steel Products [A2]. For all locations, a minimum of 6 individual hardness tests were completed with the results averaged to obtain the recorded Leeb value [A1]. All recorded values were then normalised using the derived multi-dimensional correlation factors.

2.4. CONCRETE REBOUND HARDNESS

Concrete hardness is often used as a non-destructive means of determining the compressive strength of concrete. The most common method employed is the rebound hardness, obtained from a portable Schmidt Hammer. The Schmidt hammer works using a similar principle to the Leeb Hardness measurements, whereby a weight is impacted on the surface of the material and the change in velocity between the impact speed and rebound speed is determined. Correlations are then applied to convert the change in speed to hardness and compressive strength.

As with the Leeb Hardness measurements, increased accuracy in the obtained results is achieved if the hardness measurements can be directly correlated against

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

the specific material being tested, by completing destructive materials testing on samples of the material. This s typically achieved by removing core samples from the structure and subjecting them to compressive testing. However, if no materials testing is completed, standard conversion tables can be used to form the correlations, with an associated reduction in accuracy.

The correlations for the Gallery Apartments and Westpac Centre were completed using the results from the physical testing of concrete core samples removed the buildings. No cores could be removed from the IRD building and as such the standard lower 10 percentile strength curves specifically developed for the instrument used in the testing. The curves were derived from testing of over 2,300 discrete locations. Use of the lower 10 percentile curve is recommended by the leading Standards, EN 13791 and ASTM C805/ACI 228.1.

In each tested location, a grid of readings were recorded. The results from the grid of readings were then averaged to provide the concrete hardness and associated concrete strength of that location. This testing method is endorsed by most International Testing Standards, and the manufacturers of the test equipment.

Steel samples from the IRD building were supplied to HSL by the engineers who designed the building. The steel samples were taken from a damaged zone in the central core of the building. Leeb Hardness testing was completed on the steel samples prior to the completion of the physical tensile testing.

All tensile testing was completed to the requirements of ASTM E8/ E8M:08.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

3.0 TEST EQUIPMENT

3.1. LEEB HARDNESS TESTER

A Proceq Equotip 3 portable hardness tester was used to collect all material hardness values. The device is generally acknowledged as the industry standard for the determination of Leeb hardness. The hardness tester was installed with a DL impact device, allowing measurements on smaller diameter steel samples than the conventional D device.

The Equotip 3 has a reported accuracy of ±4 HL and is traceably calibrated to NIST standards.

3.2. SILVERSCHMIDT HAMMER

A Proceq Silverschmidt Rebound Hammer was used to undertake all field based concrete hardness testing for concretes of compressive strength ranging from 10 to 100 MPa. This device and methodology generally accepted as the industry leading device for determining the compressive strength of concrete in-situ.

The Proceq Silverschmidt was fitted with the N-Type rebound hammer providing test impact energy of 2.207 Nm.

3.3. UNIVERSAL TEST MACHINE

A UH600 Shimazu servo-controlled Universal Test Machine (UTM) with a 600 kN capacity was used to undertake all laboratory based materials testing. The UTM has a maximum stroke of 250 mm and a peak table velocity of 150 mm/min.

Steel Elongation was recorded using a strain gauge based digital extensometer with a gauge length of 50 mm. Applied loads were recorded directly using the internal pressure transducer of the Shimazu control system.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

4.0 GALLERY APARTMENT RESULTS

4.1. CONCRETE RESULTS

A series of four concrete cores were removed from the concrete shear wall elements towards the front of the Gallery apartments. Two cores were subjected to uniaxial compression testing whilst the remaining two cores were subjected to split cylinder testing in order to determine the tensile properties of the concrete. The results from the physical testing on the cores are presented below.

Table 1 Compressive Cylinder results for the Gallery Apartment

Specimen Name		**RWRC**	**FWRC**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	94.1	93.9
Average core length (upon receipt)	(mm)	255.6	254.8
Average core length (after docking)	(mm)	190.0	187.6
Mass of core prior to capping	(g)	3191	3098
Density	(kg/m^3)	2421	2387
Height diameter ratio		2.02	2.0
Conditioning		Air dried	Air dried
Load at Failure	(kN)	388.8	322.1
Compressive Strength	**(MPa)**	**56.0**	**46.5**
Type of fracture		column	Shear

Table 2 Split Cylinder results for the Gallery Apartment

Specimen Name		**RWLC**	**FWLC**
Date Tested		11 Nov 2011	11 Nov 2011
Age	(days)	Unknown	Unknown
Defects in cylinder		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.6	94.0
Average length	(mm)	189.5	167.5
Mass of cylinder in air	(g)	3133	2742
Density	(kg/m^3)	2400	2380
Height diameter ratio		2.02	1.78
Conditioning		Air dried	Air dried
Tensile Strength	**(MPa)**	**2.4**	**3.4**

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

In addition to physical testing, a series of Schmidt hammer tests were completed in additional locations surrounding the noted zones of damage in the building. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe. The results indicate that the normalised correlation curves typically overestimated the actual concrete strength when compared to the actual concrete strength information obtained from the concrete cores that were tested.

Table 3 Schmidt Hammer test results for Gallery Apartments

location: Front Wall - Left Side

	1	2	3	4
A	73	71.5	72	
B	67	73.5	77	72
C	72.5	72.5	71.5	70
D		60	70.5	72

Correct Average:	71.8	
Cube Strength:	87.1	MPa
Cylinder Strength, f'c:	**70.0**	**MPa**

location: Front Wall - Right Side

	1	2	3	4
A	68.5	70.5	67.5	
B	73.5	71	71	71.5
C	72	75.5	66.5	70.5
D		72	70.5	73.5

Correct Average:	70.8	
Cube Strength:	82.8	MPa
Cylinder Strength, f'c:	**66.0**	**MPa**

location: Rear Wall - Left Side

	1	2	3	4
A	75	73	62.5	
B	65	70.5	75	68.5
C	67.5	67.5	69.5	62.5
D		68.5	70.5	68.5

Correct Average:	70.0	
Cube Strength:	80.2	MPa
Cylinder Strength, f'c:	**63.0**	**MPa**

location: Rear Wall - Right Side

	1	2	3	4
A	73	65	72	
B	69.5	65	65	61
C	69.5	64	65	64.5
D		58.5	74	63

Correct Average:	66.2	
Cube Strength:	67.4	MPa
Cylinder Strength, f'c:	**54.0**	**MPa**

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

HOLMESSOLUTIONS

Figure 1 *Drilling concrete core from Gallery Apartments*

Figure 2 Core removed from Gallery Apartment Wall

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

HOLMESSOLUTIONS

Figure 3 *Test locations on Front Wall of Gallery Appartments*

Figure 4 Schmidt Hammer test location GAFLS

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

5.0 WESTPAC CENTRE RESULTS

5.1. CONCRETE RESULTS

A series of 6 concrete cores were removed from the concrete elements, all of which were subjected to compression testing. Two of the cores were removed from precast beams, two from column elements, and the remaining two were extracted from the in-situ walls. The results from the physical testing on the cores are presented below.

Table 4 Compressive Cylinder results for the Precast beams in Westpac Centre

Specimen Name		**Precast Beam 2**	**Precast Beam 3**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.8	93.9
Average core length (upon receipt)	(mm)	227.3	211.0
Average core length (after docking)	(mm)	192.0	188.1
Mass of core prior to capping	(g)	3032	2920
Density	(kg/m^3)	2311	2253
Height diameter ratio		2.05	2.00
Conditioning		Air dried	Air dried
Load at Failure	(kN)	158.4	149.5
Compressive Strength	**(MPa)**	**23.0**	**21.5**
Type of fracture		shear	shear

Table 5 Compressive Cylinder results for the In-situ walls in Westpac Centre

Specimen Name		**In-situ wall - Bottom**	**In-situ wall - Top**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	93.7	94.1
Average core length (upon receipt)	(mm)	234.5	218.5
Average core length (after docking)	(mm)	191.1	193.1
Mass of core prior to capping	(g)	3028	3068
Density	(kg/m^3)	2315	2305
Height diameter ratio		2.04	2.05
Conditioning		Air dried	Air dried
Load at Failure	(kN)	134.5	119.2
Compressive Strength	**(MPa)**	**19.5**	**17.0**
Type of fracture		column	shear

Table 6 Compressive Cylinder results for the Circular columns in Westpac Centre

Specimen Name		**Column 1**	**Column 2**
Date Tested		10 Nov 2011	10 Nov 2011
Age	(days)	Unknown	Unknown
Size & Position of any reinforcing		None	None
Visual description		Homogeneous	Homogeneous
Average core diameter	(mm)	94.1	94.2
Average core length (upon receipt)	(mm)	223.1	154.8
Average core length (after docking)	(mm)	185	123
Mass of core prior to capping	(g)	3074	1992
Density	(kg/m^3)	2394	2344
Height diameter ratio		1.97	1.31
Conditioning		Air dried	Air dried
Load at Failure	(kN)	158.4	224.2
Compressive Strength	**(MPa)**	**23.0**	**32.0**
Type of fracture		column	shear

Schmidt hammer tests were also completed on the various concrete elements in the building. All tests were completed in zones remote from where the concrete cylinders were extracted from the building. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe. The results indicate that the normalised correlation curves typically overestimated the actual concrete strength when compared to the actual concrete strength information obtained from the concrete cores that were tested.

Table 7 Schmidt Hammer results for the Precast beams in Westpac Centre

location: Precast Beam

	1	2	3	4
A	65.5	53	56	56.5
B	57	63	56.5	54.5
C	54	62	58.5	58
D	67	60	45.5	52

Correct Average:	56.8	
Cube Strength:	42.4	MPa
Cylinder Strength, f'c:	**34.0**	**MPa**

Table 8 Schmidt Hammer results for the Columns in Westpac Centre

location: Column Level 3

	1	2	3	4
A	57	64.5	58.5	56
B	56.5	63.5	63	54.5
C	61	58	56.5	60
D	58.5	56	64	57.5

Correct Average:	58.6	
Cube Strength:	46.2	MPa
Cylinder Strength, f'c:	**37.0**	**MPa**

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

Figure 5 *Core Drilling in concrete column*

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

HOLMESSOLUTIONS

Figure 6 *Core and Schmidt hammer location on Wall element*

Figure 7 *Core location on Wall element*

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

Table 9 Schmidt Hammer results for the In-situ Wall elements of Westpac Centre

location: Basement Wall

	1	2	3	4
A	68.5	64	60.5	60.5
B	67	620	65.5	64
C	58	57	68	64
D	61.5	58	65	62.5

Correct Average:	62.9	
Cube Strength:	56.9	MPa
Cylinder Strength, f'c:	**46.0**	**MPa**

location: Level 3 wall - RHS

	1	2	3	4
A	67	71	68	57
B	56	56	54	62
C	66	57.5	59.5	70.5
D	61	63.5	54	69.5

Correct Average:	62.1	
Cube Strength:	55.7	MPa
Cylinder Strength, f'c:	**45.0**	**MPa**

location: Level 3 wall - LHS

	1	2	3	4
A	57.5	61.5	66	66.5
B	59	52	66	73
C	55.5	61	52	55.5
D	53	60.5	58	57

Correct Average:	59.8	
Cube Strength:	49.6	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

5.2. STEEL RESULTS

Four 16 mm diameter reinforcing bars were removed from the insitu concrete walls of the structure and subjected to uniaxial tensile testing in the laboratory. Two of the bars were retrieved from areas in the building considered to have sustained little or no damage during the recent earthquakes. As such the material properties obtained from these sample can be assumed to have been unmodified from previous inelastic strain cycles. One of the bars was from the horizontal reinforcing and the other formed an element of vertical reinforcing in the wall

The obtained stress-strain responses of the two undamaged steel samples are shown in *Figure 10* below. The steel samples were subjected to unidirectional cyclic tensile testing rather than cycles of reverse cyclic loading to near equal values of tensile and compressive strain. In the structural element, under imposed lateral loads the neutral axis is likely to have been located near the location of the reinforcing steel during the compression load cycle, and as such the steel would have been subjected to very small induced compressive strains. During the reverse loading cycle the steel located at or near a crack in the concrete section is likely to have been subjected to disproportionately larger tensile strains, thereby significantly skewing the strain profile experienced by the reinforcing steel into the tension domain. Due to the skewed strain profile, it is believed that the unidirectional cyclic tensile test provides an adequate representation of the strains induced in the steel during a seismic event.

Leeb Hardness testing was also completed on the steel samples at various levels of applied strain, both with the load applied and with the load removed from the steel. The points of inspection can be observed in the recorded stress-strain response as areas of load cycling.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

HOLMESSOLUTIONS

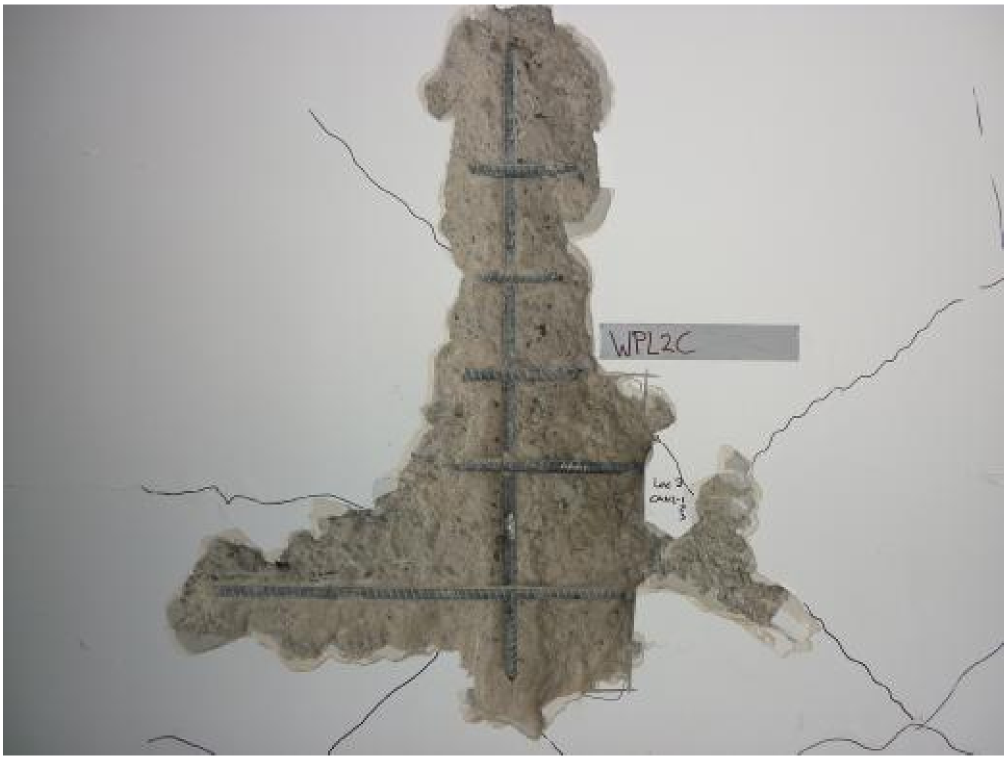
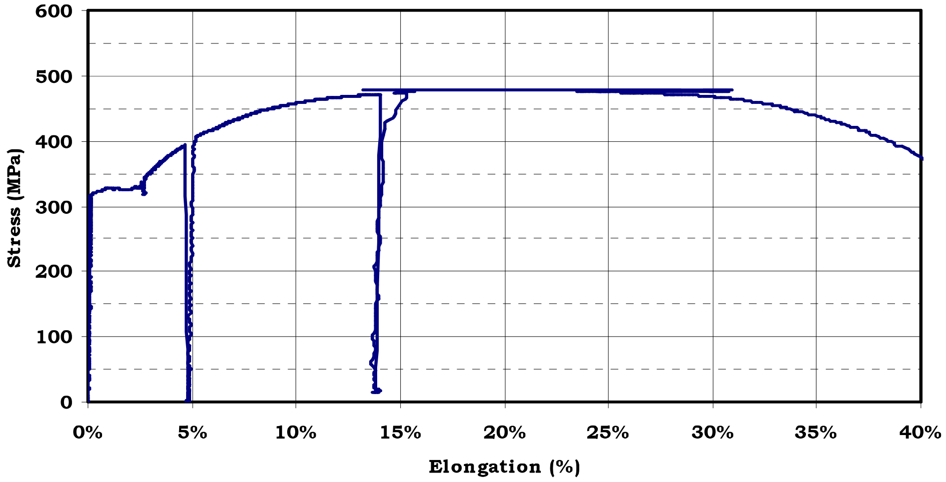
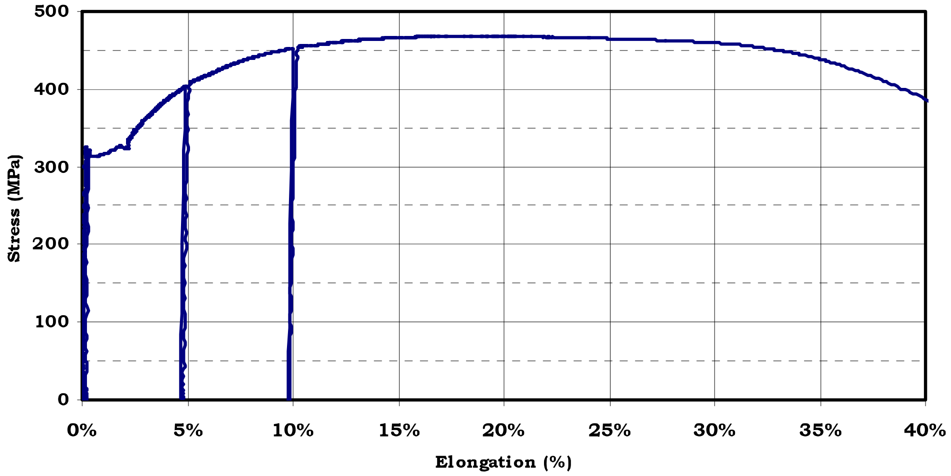


Figure 8 *Exposed reinforcing steel in zone of damage in wall element*

Figure 9 *Exposed reinforcing steel in zone of damage in wall element*



a) Horizontal reinforcing bar sample



b) Vertical reinforcing steel sample

Figure 10 Materials Test Result for the Steel test coupons obtained from undamaged area in the Westpac Centre

The steel samples had an average recorded yield stress (f_y) of 320 MPa and an average maximum recorded stress (f_u) of 472 MPa. The strain hardening ratio (f_u/f_y) of the tested steel sample was defined as 1.475. This value of strain hardening ratio indicates that the steel has a good likelihood of spreading the zone of yield along the bar, a beneficial property for limiting the potential damage at a localised zone of damage in a reinforced concrete member. It also indicates that the steel has a high plastic hardness and therefore it likely to provide suitable variation in Leeb hardness values for various levels of imposed strain.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

The recorded Leeb hardness for the steel samples, and the associated stress and strain at the point of testing are reported below. A series of 6 individual Leeb hardness test results were taken and averaged to produce the reported value of Recorded Average Leeb. The recorded Leeb values for the steel show a good variation across the stress range. This is a result of the relatively high plastic stiffness of the material, defined by the extent of strain hardening observed in the recorded stress-strain plot of the tested samples.

The reported values of Leeb hardness were derived for the steel sample supported in the universal testing machine. Additional hardness tests were also completed on the tested steel sample with the bar fully supported in a mortar matrix. Based on the Leeb Hardness results obtain, the reinforcing steel used in the building appears to have a base Leeb Hardness of 610 DLHL.

Table 10 Baseline Material Strength Results for Test Sample 1

Applied Load (kN)	Steel Strain (%)	Steel Stress (MPa)	Recorded Average Leeb (DLHL)
0.0	0.0	0	610
61.0	0.5	303	610
80.0	5.0	398	650
95.0	14.0	472	680

Table 11 Baseline Material Strength Results for Test Sample 2

Applied Load (kN)	Steel Strain (%)	Steel Stress (MPa)	Recorded Average Leeb (DLHL)
0.0	0.0	0	610
63.0	0.5	313	612
80.0	4.5	398	650
91.0	11.0	453	670

Leeb Hardness testing was completed on a further 2 horizontal bar and two vertical bar located in zones of heavy damage in the in-situ wall of the building. The results from the Leeb Hardness are presented below.

The Leeb hardness results for the Vertical Bar 2 shows a peak elevated hardness value of 660 DLHL approximately mid way along the length of tested steel. This zone of elevated hardness coincides with the location where the reinforcing bar crosses a significant crack in the wall element. The zone of elevated hardness occurs over a length of approximately 35-40 mm, equivalent to 2 times the diameter of the reinforcing bar. Based on the derived correlations obtained from the undamaged reinforcing bars, this level of Leeb Hardness indicates that the steel has previously been strained to approximately 10% strain. This level of induced strain indicates that the steel has lost approximately 75% of the available strain capacity, and can only undergo an additional 5% strain before fracturing. Based on the short zone observed to have an elevated hardness, this would equate to approximately 2 mm of elongation over a 40 mm length prior to fracture.

The Leeb hardness for the Horizontal Bar 2 shows signs of moderately increased strain hardening over lengths of approximately 75-100 mm. Based on the correlations between Leeb Hardness and strain obtained previously, it is suggested that this steel sample has been previously strained to 2%.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

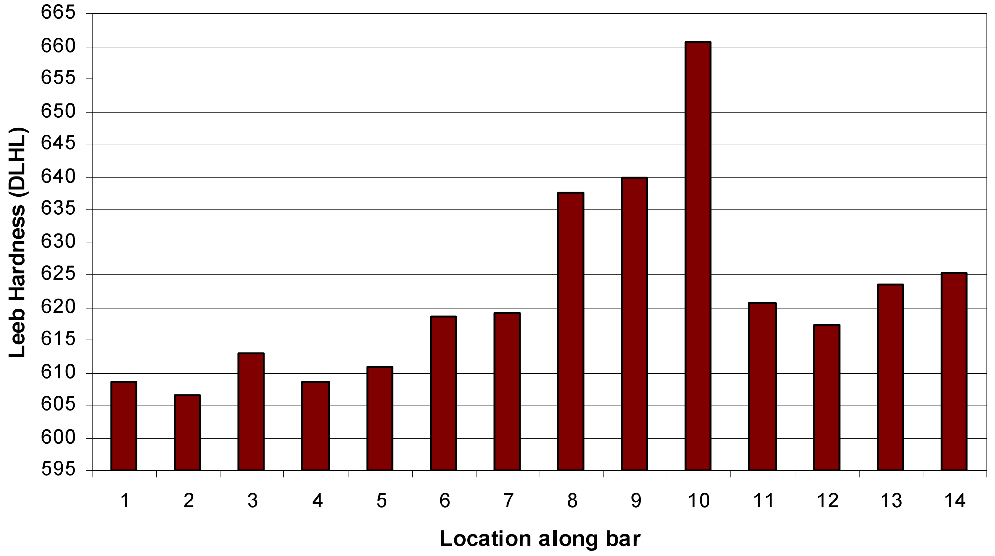


Figure 11 Leeb Hardness result for Vertical Bar 2 in zone of damage

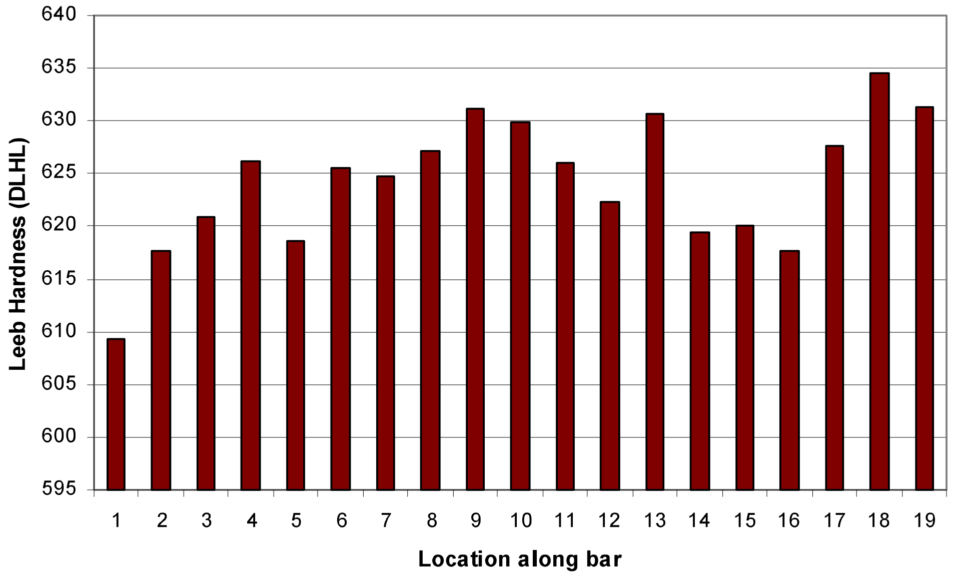


Figure 12 Leeb Hardness result for Horizontal Bar 2 in zone of damage

Vertical reinforcing Bar 3 shows two zones of increased hardness, corresponding to two cracks observed to cross the steel in the wall element. The first zone of elevated hardness is relatively wide, indicating that any yielding of the steel occurred across a relatively long length on the bar. The second zone of elevated hardness has a maximum recorded Leeb value of 640 DLHL and appears to occur over a relatively short distance. This level of hardness indicates that the steel was previously strained to approximately 5%. The results for Horizontal Bar 3 are similar to the previous horizontal bar with Leeb hardness values suggesting the steel was subjected to inelastic strains of approximately 2% over a relatively long length of the steel.

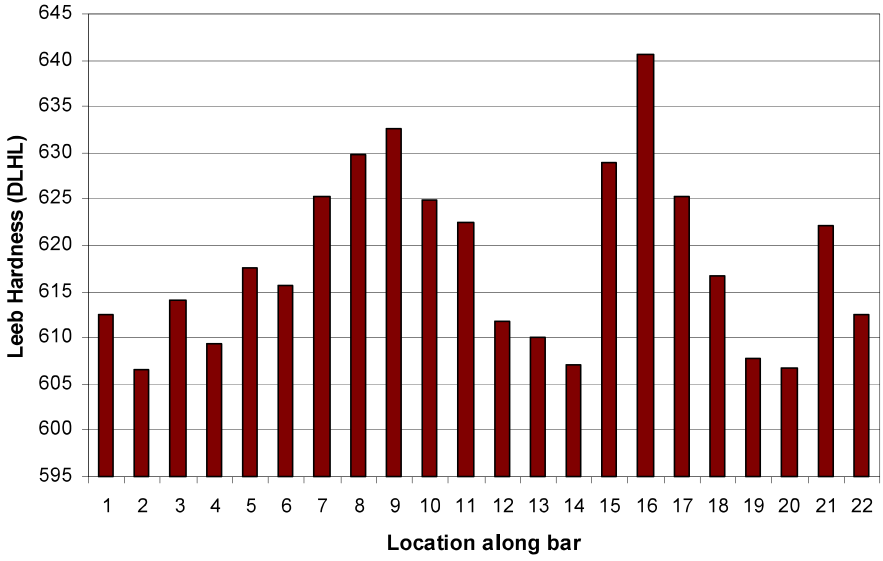


Figure 13 Leeb Hardness result for Vertical Bar 3 in zone of damage

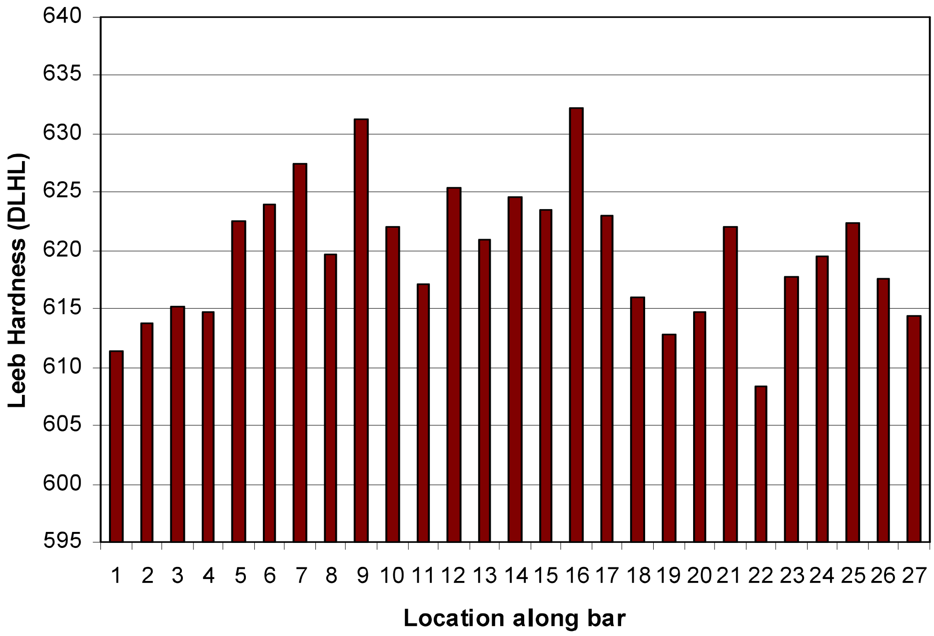


Figure 14 Leeb Hardness result for Horizontal Bar 3 in zone of damage

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

A further 4 reinforcing bar samples were removed from the building and subjected to destructive tensile testing. The results from the testing are shown below. The results indicate that the horizontal steel remained undamaged during the earthquake, with recorded uniform strain capacities in excess of 33%. The yield strength of the tested horizontal steel samples was found to be 314 MPa and 315 MPa respectively.

The vertical steel sections were found to have considerable lower uniform elongation capacity when compared to the horizontal steel section, with actual elongation capacities between 11% and 13%. This result indicates that the steel has lost strain capacity due to being exposed to previous cycles of inelastic loading. The yield strength of the vertical steel sections was found to be 319 MPa and 330 MPa respectively.

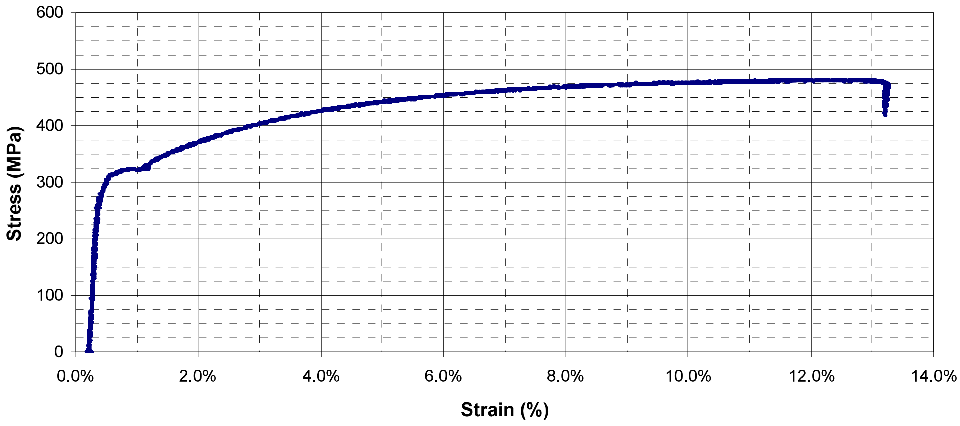


Figure 15 Stress-strain response for vertical steel section located in damaged zone

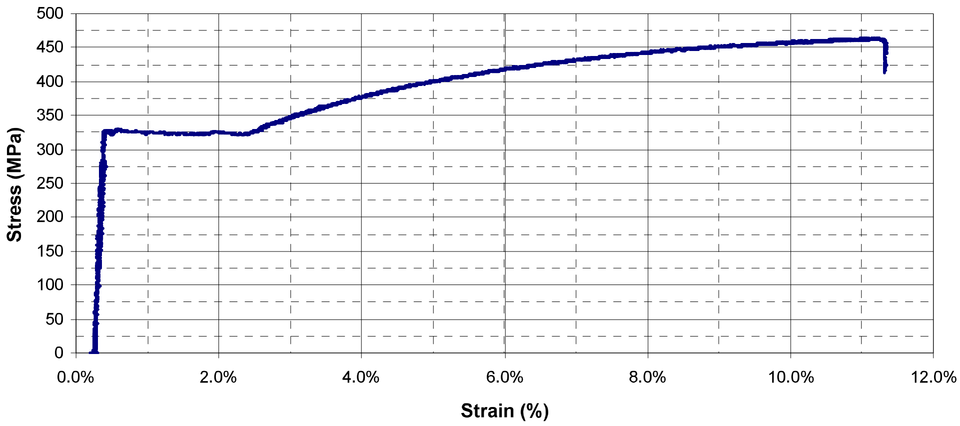


Figure 16 Stress-strain response for vertical steel section located in damaged zone

HOLMESSOLUTIONS

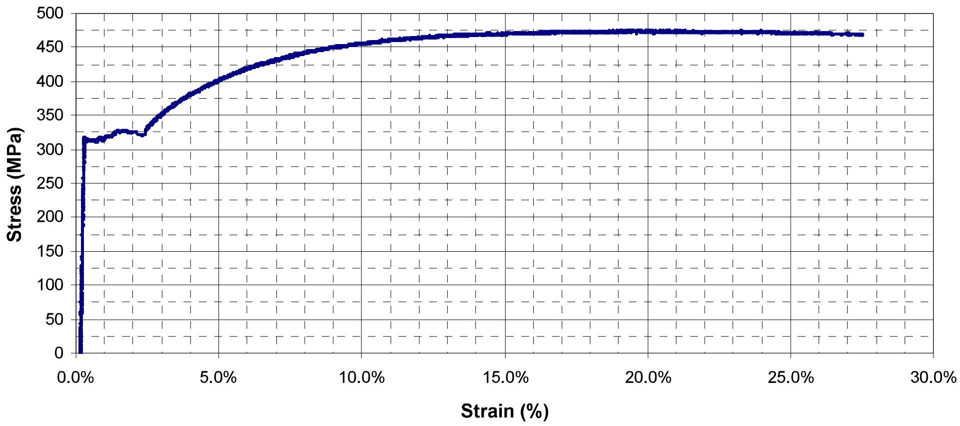


Figure 17 Stress-strain response for horizontal steel section located in damaged zone

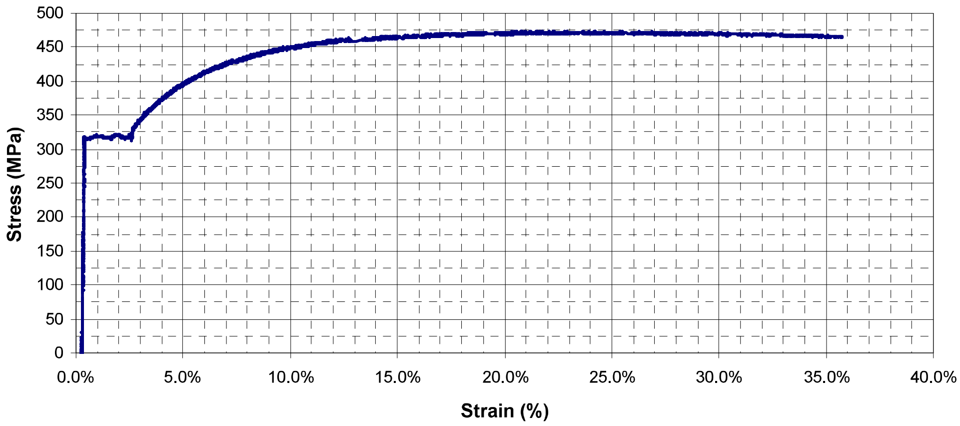


Figure 18 Stress-strain response for horizontal steel section located in damaged zone

6.0 IRD BUILDING RESULTS

6.1. CONCRETE RESULTS

No concrete cores were extracted from the IRD building. As a result, all concrete material information was obtained from Schmidt hammer tests. All tests were completed near the zones of damage in the in-situ and precast concrete shear walls. The results from the Schmidt hammer tests are presented below.

The conversion from hardness information into concrete cylinder compressive strength is presented utilises the standard conversion factors typically use with Schmidt hammers, which has been derived from extensive testing on concrete samples in Europe.

Table 12 Schmidt Hammer results for the Precast Walls in the IRD Building

location: Precast Wall section -1

	1	2	3	4
A	69.5	63.5	61.5	60.5
B	66	56.6	65.6	62.1
C	57	68.5	68	62
D	67	68	63	61.5

Correct Average:	64.1	
Cube Strength:	60.1	MPa
Cylinder Strength, f'c:	**48.0**	**MPa**

location: Precast Wall section -2

	1	2	3	4
A	72	63.5	68	69
B	68.5	63	58.5	61,5
C	56.5	63.5	59.5	71
D	63	65.5	58.5	72

Correct Average:	64.3	
Cube Strength:	61.2	MPa
Cylinder Strength, f'c:	**49.0**	**MPa**

location: Insitu Wall section -1

	1	2	3	4
A	58	59.5	63	63.5
B	65	63.5	67	71
C	53.5	55	52.5	53.5
D	55.5	65	57	60

Correct Average:	59.8	
Cube Strength:	50.1	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

location: Insitu Wall section -2

	1	2	3	4
A	65	61	59.5	55.5
B	55.5	65	56	63.5
C	65.5	66	55.5	59.5
D	58.5	59	62	55

Correct Average:	60.3	
Cube Strength:	50.2	MPa
Cylinder Strength, f'c:	**40.0**	**MPa**

6.2. STEEL RESULTS

HSL was commissioned independently to undertake materials testing on two steel samples extracted from the concrete walls of the IRD building. Two deformed reinforcing bars, 10 mm in diameter, were supplied for testing. The location of the steel in the building nor the origins of the steel were provided.

Prior to undertaking uniaxial tension testing on the steel, the samples were subjected to Leeb Hardness testing. The obtained results are presented below.

Both steel samples showed significant reduction in Leeb hardness readings at the location marked on the bars as corresponding with the crack in the concrete member. Reduction in Leeb hardness typically only occurs in steel bars immediately prior to the onset of necking, where micro alloy steel has been found to strain soften.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

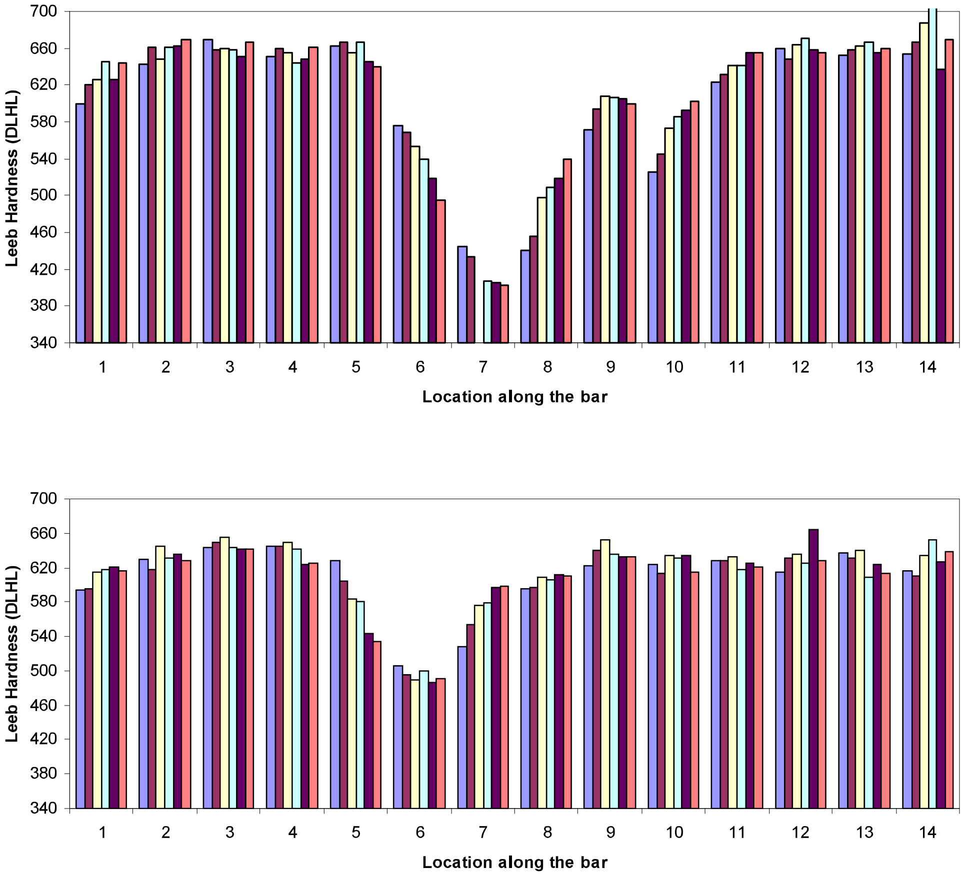
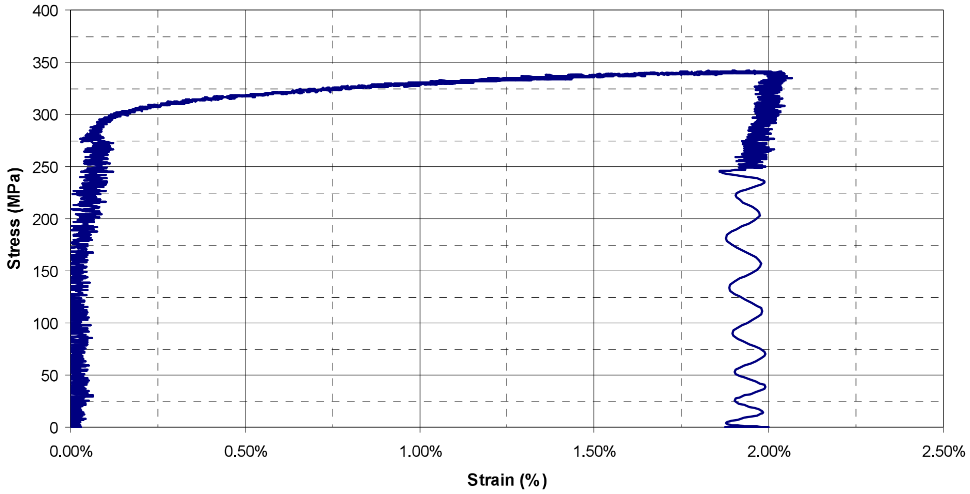


Figure 19 Leeb Hardness result for steel samples provided from IRD Building

The steel samples were then subjected to uniaxial tensile testing, with the obtained stress-strain responses shown in *Figure 20*. From the obtained stress-strain responses it would appear that the parent material was Grade 300E reinforcing steel. Grade 300E reinforcing steel has a lower characteristic yield strength of 300 MPa and is required in the New Zealand manufacturing Standard (AS/NZS 4671) to have a minimum uniform elongation capacity in excess of 15%. The results obtained for the two samples show they have an elongation capacity of 2% and 0.9% indicating that they have undergone significant inelastic deformation and are close to fracturing. This correlates with the observed Leeb Hardness results, showing significant strain softening at the cracked region.

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM



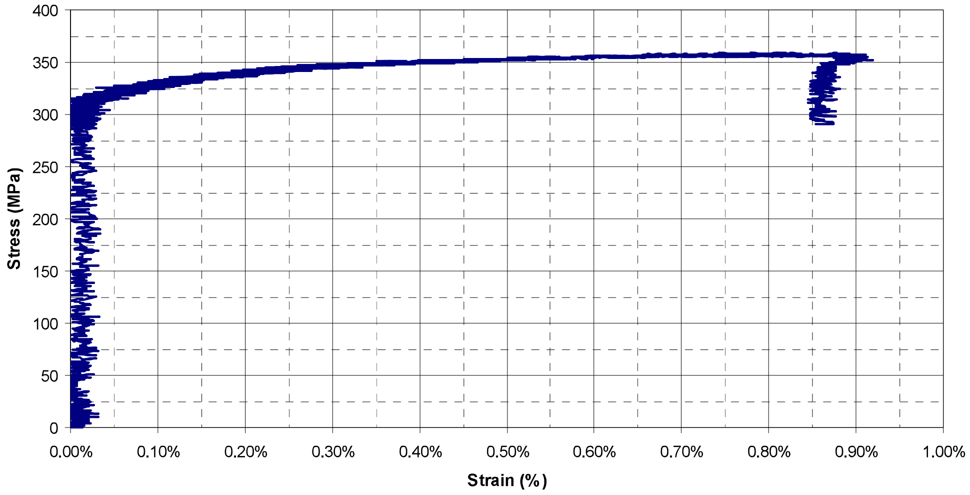


Figure 20 Stress-Strain responses for steel samples provided from IRD Building

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM

Figure 21 Tensile testing of steel sample

7.0 BIBLIOGRAPHY

[A1] ASTM International, "*Standard Test Method for Indentation Hardness o Metallic Materials by Portable Hardness Testers*", ASTM Designation E110-10, 2010

[A2] ASTM International, "*Standard Test Method for Leeb Hardness Testing of Steel Products*", ASTM Designation A956-06, 2006

[B1] Bonora, Gentile, Pirondi, Newaz, "*Ductile damage evaluation under triaxial state of stress: theory and experiments*", International Journal of Plasticity 21, 2005, pg 981-1007

[B2] Bonora, Newaz, "*Low cycle fatigue life estimation for ductile metals using a nonlinear continuum damage mechanics model*", International Journal of Solids and Structures, Vol 35, No 16, pp 1881-1894, 1998

[G1] Gasko, Rosenburg, "*Correlation between Hardness and Tensile Properties of Ultra High Strength Dual Phase Steels*", Materials Engineering, 18, pg 155-159, 2011

[K1] Kapoor, "*A re-evaluation of the life to rupture of ductile metals by cyclic plastic strain*", Fatigue and Fractures of Engineering Materials and Structures, Vol 17, No 2, pp 201-219, 1994

[L1] Lemaitre Dufailly, "*Damage Measurements*", Engineering Fracture Mechanics, Vol 28, pp 643-661, 1987

[M1] Mkaddem, Gassara, Hambli, "*A new procedure using micro-hardness technique for sheet material damage characterisation*", Journal of Material Processing Technology, 178, 2006, pg 111-118

[M2] Matsumoto, Y, "*Study on the residual deformation capacity of plastically strained steel*", Yokohama National Library, 2009

[N1] Nishikawa, Soyama, "*Two step method to evaluate equibiaxial residual stress of metal surface based on micro-indentation tests*", Materials and Design 32, 2011, 3240-3247

[N2] Nakane, Kanno, Kurosaki, Takagi, Komotori, Ogawa, "*Accuracy of Plastic Strain Estimated by Hardness to assess remaining Fatigue Live*", Material Propertied Measurement, 2010.

[N3] Nomoto, " *Report on the Integrity Assessment f Structures, Systems, and Components of the KK-NPP by the JANTI Committee*", 20th International Conference on Structutal Mechnaics in Reactor, Technology, 2009

[P1] Pirondi, Bonora, "*Modelling ductile damage under fully reversed cycling*", Computational Materials Science 26, 2003, pg 129-141

[T1] Tasan, Hoefnagels, "*A critical assessment of indentation based ductile damage quantification*", Acta Materialia, Volume 57, Issue 17, October 2009, Pages 4957, 4966

[T2] Tasan, Hoefnagels, Geers, "*Indentation-based damage quantification revisited*", Scipta Materialia 63, 2010, pg 316-319

UNIT 5, 295 BLENHEIM RD, PO BOX 6718, RICCARTON CHRISTCHURCH, NEW ZEALAND
T+64 3 363 218 F+64 3 379 2169 WWW.HOLMESSOLUTIONS.COM