Canterbury Earthquakes
Royal Commission
Te Komihana Rūwhenua o Waitaha

H.2

FINAL REPORT

VOLUME 2
THE PERFORMANCE OF CHRISTCHURCH CBD BUILDINGS

A. The main auditorium of the Christchurch Town Hall (source: Marshall Day Acoustics)

B. The intersection of High, Manchester and Lichfield Streets in the Christchurch Central Business District after the 22 February 2011 earthquake (source: Photosouth)

C. People escaping from the 17-storey Forsyth Barr Building on 22 February 2011 (source: Fairfax Media/*The Press*)

D. The five-storey Pyne Gould Corporation Building collapsed as a result of the 22 February 2011 earthquake (source: ex eye witness)

ISBN: 978-0-478-39558-7
(Final Report web-quality PDF)

ISBN: 978-0-478-39560-0
(Volume 2 web-quality PDF)

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Section 1: Approach to the representative sample of buildings

The Royal Commission's Terms of Reference require that a reasonably representative sample of buildings be considered to analyse the performance of buildings in the Christchurch Central Business District (CBD) in the Canterbury earthquakes. The Commission is tasked with investigating why some buildings failed severely, why the failure of some buildings caused extensive injury and death, why buildings differed in the extent to which they failed and their failure caused injury and death, and whether there were particular features of buildings (such as age, location and design) that contributed to their failure.

1.1 Determining which buildings to include in the sample

The Christchurch CBD includes buildings with a wide range of ages, construction methods and sizes. Four buildings were required to be included by specific direction in the Terms of Reference:

1. The Pyne Gould Corporation (PGC) Building at 233 Cambridge Terrace. This was a five-storey, 1960s building that collapsed catastrophically killing 18 people (see section 2 of this Volume).
2. The Hotel Grand Chancellor at 161 Cashel Street. This was a 27-storey (12 of which were half-storeys) 1980s building in which a wall failed on the ground floor, leading to a near-collapse that would have caused loss of life both within the building and in the vicinity. Stairs also collapsed in this building, trapping people in the upper levels (section 3).
3. The Forsyth Barr Building at 764 Colombo Street (the corner of Armagh and Colombo Streets). This is an 18-storey 1980s building in which the stairs collapsed, trapping people in the upper levels (section 4).
4. The CTV building at 245 Madras Street. This was a six-storey 1980s building that collapsed catastrophically killing 115 people. The report on this building has been delayed so that it may be completed after the relevant hearings have been held.

It was left to the Royal Commission to decide what other buildings should be included in the representative sample. At an early stage the Commission decided that it would be appropriate to include all buildings that caused a fatality as a result of their failure. Five of these were outside the CBD, but were included in recognition of the effects of the failures on people's lives. All these buildings are dealt with in Volume 4 of this Report.

We considered that the objectives of this study should be to:

- determine what changes should be made to current design standards to improve the seismic performance of new buildings;
- determine changes that should occur in the approach that structural engineers take to the design of new buildings and/or the seismic assessment of existing buildings; and
- identify critical features in existing buildings that have led to poor performance in the Christchurch earthquakes so that attention can be drawn to these for the retrofit of existing buildings, both in Christchurch and elsewhere in the country.

To ensure that the buildings to be studied were truly a representative range, we considered many factors, including:

- age, particularly in relation to design standards that applied at the time of design;
- size;
- predominant construction materials;
- seismic resistance system;
- any seismic upgrades that had been carried out;
- the level of damage to the building from the earthquakes;
- the level of damage to the building due to land damage caused by the earthquakes; and
- the public interest in some buildings.

This led to a list of about 160 buildings that were of potential interest to the Commission. Information on these buildings was sought from a range of people and organisations, including the Canterbury Earthquake Recovery Authority (CERA), building owners and consulting engineers.

We express our appreciation to the Christchurch City Council (CCC) for its efforts in locating and providing detailed information on these buildings.

For a large number of buildings there was inadequate information available for the Royal Commission to consider them further. In many cases engineers' reports had been commissioned by the building owners in order to determine the level of damage, and once it had been established that repair was not economic, the assessment did not proceed to a detailed engineering evaluation. The Royal Commission is required to consider how and why a building failed. Once a building is demolished and the site cleared it is difficult to establish the reasons for the way it performed if full evaluations are not available.

1.2 Reports available

We commissioned a number of reports to assist us in our investigation. They should be of some assistance to structural engineers engaged in assessing the seismic performance of buildings. The reports are briefly described below.

Bull, D. K. (2011). *Report to the Royal Commission – Stairs and Access Ramps between Floors in Multi Storey Buildings*[1]. This report details the level of inter-storey drift required in the design of stairs and methods of assessing the drift required in existing buildings.

Carr, A. J. (2011). *Inelastic Response Spectra for the Christchurch Earthquake Records*, Report to Royal Commission, Sept. 2011[2]. This report gives details of the response spectra that can be used to assess the likely magnitudes of forces and displacements imposed on buildings during the Christchurch earthquakes.

MacRae, G., Clifton, C., and Megget, L. (2011). *Review of New Zealand Building Codes of Practice*, Report submitted to Royal Commission[3] August 2011. This report details changes that have occurred in standards for reinforced concrete buildings, structural steel buildings, and loadings over the last six decades as they apply to seismic loading on buildings and design standards.

Smith, P. C., and Devine, J. W. (2011). *Historical Review of Masonry Standards in New Zealand*, Report submitted to Royal Commission, 2011[4]. Describes changes that have occurred in masonry standards since 1948.

In the aftermath of the earthquakes, reports were also being independently prepared by other organisations and people. Those made available to us have been considered, and have assisted us in our understanding of the performance of many building types. These include:

Clifton, C., Bruneau, M., Fussell, A., Leon, R., MacRae, G., (2011). *Steel Building Damage from the Christchurch Earthquake Series of 2010 and 2011*. November 2011[5].

Dhakal, R., MacRae, G., Hogg, K., (2011). *Performance of Ceilings in the February 2011 Christchurch Earthquake*, report published in the Bulletin of the New Zealand Society for Earthquake Engineering (NZSEE), Volume 44, Number 4, December 2011[6].

Pampanin, S., Kam, Y. K., Akguzel, U., Tasligedik, S., and Quintana-Gallo, P. (2011). *Seismic Performance of Reinforced Concrete Buildings in the Christchurch CBD under the 22nd February Earthquake*, Report prepared for Christchurch City Council and University of Canterbury, Civil and Natural Resources Engineering, November 2011[7]. Some of the damage patterns observed in this report are summarised in section 5 of this Report.

Uma, S., Beattie, G., (2011). *Observed Performance of Industrial Pallet Rack Storage Systems in the Canterbury Earthquakes*, Bulletin of the NZSEE, Volume 44, Number 4, December 2011[8].

Structural Engineering Society New Zealand Inc. (SESOC) (2010), *Interim Design Guidance, Design of Conventional Structural Systems Following the Canterbury Earthquakes*, Draft version 6, 18 April 2012[9]. This document is intended to provide guidance to structural and geotechnical engineers and territorial authorities in the design of structures during the interim period between the Canterbury earthquakes and possible changes to the New Zealand Building Code. This document has no legal status but it is hoped that the issues raised will be considered. The document reflects the views of SESOC. The Royal Commission believes the development of the guide is very positive and we encourage engineers involved in the rebuild of Christchurch to consider the recommendations closely.

A range of details used in buildings is considered from the point of view of current requirements in design Standards but with additional recommendations, which are based on observed performance in the Canterbury earthquakes. In many cases the recommendations are aimed at increasing buildings' performance for the serviceability limit state. In some cases following the recommendations will involve extra cost and it will be necessary to weigh the added cost against the potential improvement in seismic performance. For example, in a number of situations it is recommended that walls be increased in thickness compared to the minimum thickness values used in current practice. While this change is likely to make only a small difference to the cost of the wall, the carry-over effects may in some cases be significant and this needs to be considered carefully. The carry-over effects may arise from the increased mass of the structure and the increased ultimate strength of the walls, which may apply increased forces to the foundations with a consequent increase in cost of the foundation structure.

1.3 Discussion

After considering all information available we were of the opinion that reinforced concrete buildings constructed within (about) the last 50 years would be the major group that would require independent analysis. Our main reasons are:

1. From the late 1960s to the early 1980s there were major changes in structural design for earthquakes. The MacRae et al report charts the change in approach by describing the change in design Standards that occurred during this time. This was the period when design for ductility and capacity design were being developed. From the late 1960s to the late 1970s different structural engineers followed widely different practices. The Ministry of Works led the way in the application of the new approach to design. The poor performance of many buildings built prior to the introduction of ductility and capacity design is well known and is only briefly discussed in this Report. The report by Pampanin et al gives some idea of the extent of the problem with these structures, most of which were built between 1935 and the mid-1970s.
2. Few buildings were constructed between 1935 (when unreinforced masonry (URM) building construction ceased), and the late 1960s, when more modern design methods became prevalent. This was largely due to the Second World War and the tighter financial times both before and after it. In addition, these buildings generally do not assist the understanding of the design of new buildings. Lessons learned from buildings in the representative sample designed and constructed prior to 1976 (especially the PGC building) will apply equally to buildings that are older than those chosen where critical structural weaknesses need to be identified. However, an overall impression of the performance of this group of buildings was also obtained from the Pampanin et al report, with a summary of typical damage patterns provided in section 5 of this Volume of this Report.
3. Steel frame buildings are not particularly numerous in the Christchurch CBD, and information available shows that they performed well as they were designed and constructed to modern standards. The Clifton et al report has assisted the Royal Commission with a general understanding of the performance of these buildings. We have investigated one building as a representative of this style, and comment on the findings of the Clifton et al report in section 8 of this Volume of the Report.

4. Small buildings such as houses and commercial buildings constructed using similar techniques are being extensively analysed by other agencies, primarily under the umbrella of the Engineering Advisory Group (EAG). The EAG is set up as a committee appointed by the Department of Building and Housing's Chief Executive, and comprises a small group of leading engineers and remediation specialists, including representatives from DBH, The Earthquake Commission, BRANZ, GNS Science, Structural Engineering Society New Zealand Inc. (SESOC), New Zealand Society for Earthquake Engineering and New Zealand Geotechnical Society.

   This work has been considered by the Royal Commission, and we have decided that it is not necessary for us to replicate it in regard to small buildings.

5. Owing to the economic boom of the 1980s, a significant number of the examples of larger buildings are from this era.

1.4 The representative sample

In addition to the buildings specified in the terms of reference and those that are included in Volume 4 of this Report, the following buildings were the subject of detailed consideration by the Royal Commission. The references are the location within this Report.

Pre 1976 – Buildings designed prior to the introduction of Loadings Code NZS 4203:1976[10] (see section 6.1)

- 6.1.1 48 Hereford Street: Christchurch Central Police Station
- 6.1.2 53 Hereford Street: Christchurch City Council Civic Offices
- 6.1.3 100 Kilmore Street: Christchurch Town Hall.

1976 to 1984 – Buildings designed to Loadings Code NZS 4203:1976[10] (see section 6.2)

- 6.2.1 166 Cashel Street: Westpac/Canterbury Centre building.

1984 to 1992 – Buildings designed to Loadings Code NZS 4203:1984[11] (see section 6.3)

- 6.3.1 90 Armagh Street: Craigs Investment House
- 6.3.2 20 Bedford Row: Bedford Row Car Park
- 6.3.3 79 Cambridge Terrace: Bradley Nuttall House
- 6.3.4 151 Worcester Street
- 6.3.5 78 Worcester Street: Clarendon Tower.

1992 to 2008 – Buildings designed to Loadings Standard NZS 4203:1992[12] (see section 6.4)

- 6.4.1 100 Armagh Street: Victoria Square apartment building.

2004 to 2011 – Buildings designed to Earthquake Actions Standard NZS 1170.5:2004[13] (see section 6.5)

- 6.5.1 62 Gloucester Street: Gallery Apartments
- 6.5.2 2 Riccarton Avenue: The Christchurch Women's Hospital
- 6.5.3 224 Cashel Street: IRD building
- 6.5.4 166 Gloucester Street: Pacific Tower
- 6.5.5 52 Cathedral Square: Novotel Hotel.

The study of sample buildings included analyses to reveal likely actions that have caused the observed damage. To assist in the inquiry the consulting engineers Spencer Holmes Limited, Compusoft Engineering Limited, and Rutherford and Chekene (California) were employed by the Royal Commission to assess the performance of a number of the buildings. We have recorded our opinions on the behaviour of structures that has resulted in the damage observed.

Given the time constraints, the number of buildings that were fully assessed was limited. We are, however, satisfied that we have considered a sufficiently representative sample of buildings in the course of the inquiry. Currently CERA is requiring owners to obtain detailed engineering assessment of a wide range of buildings. As these become available the information gained should add to the available knowledge about building performance in the earthquakes. In addition, the EAG is investigating a full range of buildings and the findings of that group are expected to be complementary to the Royal Commission findings.

1.5 Summary of buildings analysed (not URM)

This table sets out the buildings considered, grouping them in accordance with their age, structural type and other relevant considerations.

	1935 – 1976	NZS 4203:1976[10]	NZS 4203:1984[11]	NZS 4203:1992[12]	NZS 1170.5: 2004[13]
Address of building	233 Cambridge Tce 48 Hereford St 53 Hereford St 100 Kilmore St	166 Cashel St	90 Armagh St 20 Bedford Row 161 Cashel St 79 Cambridge Tce 764 Colombo St 70 Kilmore St 151 Worcester St	100 Armagh St	224 Cashel St 52 Cathedral Square 62 Gloucester St
Moment resisting frame	53 Hereford St		764 Colombo St 151 Worcester St		224 Cashel St
Structural walls	233 Cambridge Tce 100 Kilmore St	166 Cashel St	20 Bedford Row 161 Cashel St 79 Cambridge Tce 70 Kilmore St	100 Armagh St	224 Cashel St 52 Cathedral Square 62 Gloucester St
Precast floors	100 Kilmore St		20 Bedford Row 161 Cashel St 79 Cambridge Tce 764 Colombo St 151 Worcester St	100 Armagh St	224 Cashel St 62 Gloucester St
Precast beams or columns or walls			20 Bedford Row 79 Cambridge Tce 764 Colombo St 151 Worcester St	100 Armagh St	224 Cashel St 62 Gloucester St
Minor land damage	233 Cambridge Tce 53 Hereford St		20 Bedford Row 79 Cambridge Tce		224 Cashel St
Significant land damage	100 Kilmore St		90 Armagh St 70 Kilmore St	100 Armagh St	

	1935 – 1976	NZS 4203:1976[10]	NZS 4203:1984[11]	NZS 4203:1992[12]	NZS 1170.5: 2004[13]
Minor to moderate building damage	48 Hereford St 53 Hereford St		764 Colombo St	100 Armagh St	224 Cashel St 52 Cathedral Square
Significant building damage	233 Cambridge Tce 100 Kilmore St	166 Cashel St	90 Armagh St 161 Cashel St 79 Cambridge Tce 151 Worcester St 70 Kilmore St		62 Gloucester St
Regular structural form		166 Cashel St	90 Armagh St 20 Bedford Row 764 Colombo St 151 Worcester St	100 Armagh St	224 Cashel St
Irregular structural form	233 Cambridge Tce 100 Kilmore St		161 Cashel St 79 Cambridge Tce 70 Kilmore St		224 Cashel St 62 Gloucester St
Shallow foundations	233 Cambridge Tce 48 Hereford St 53 Hereford St 100 Kilmore St	166 Cashel St	90 Armagh St 79 Cambridge Tce	100 Armagh St	
Deep foundations			90 Armagh St 20 Bedford Row 161 Cashel St 79 Cambridge Tce 764 Colombo St 151 Worcester St	100 Armagh St	

1.6 The Christchurch earthquakes

In assessing the seismic performance of buildings in Christchurch it is important to be able to relate their performance to the characteristics of the earthquakes. These characteristics are described in detail in the seismicity section in Volume 1 of this Report. Details of the ground motion and acceleration and displacement response spectra are given in the Carr report[2]. For convenient reference in this section of this Report the displacement spectra obtained by averaging the recorded ground motion at the four principal CBD seismic measuring sites (locations shown in Figure 1) are reproduced in Figures 2–5.

The averaged spectral displacements at the four sites are shown for the September earthquake in Figures 2 and 3 for the east–west and north–south directions respectively. Corresponding spectra for the February earthquake are shown in Figures 4 and 5. The design displacement spectrum for the 500-year return earthquake is shown on the figures.

The sites are:

- Resthaven Retirement Home (REHS) site near Peacock Street in the north-west.
- Christchurch Catholic Cathedral College (CCCC) site near Barbadoes Street in the south-east.
- Christchurch Botanic Gardens (CBGS) site near the rose garden in the west.
- Christchurch Hospital (CHHC) site near Antigua Street in the south-west.

Spectral values have been obtained assuming five per cent equivalent viscous damping for elastic response, and for structures with displacement ductility values of 2, 4 and 6, for the September and February earthquakes (see the Carr report[2]).

We note that spectral displacements in the period range of 2.5–4 seconds were particularly high relative to the spectral values in the north–south direction in the September earthquake and in the east–west direction in the February earthquake. These high values would have generated particularly severe conditions for the Hotel Grand Chancellor, Forsyth Barr, Clarendon Tower and Gallery Apartments buildings.

The Canterbury earthquakes have tested CBD buildings in excess of their ultimate limit state.

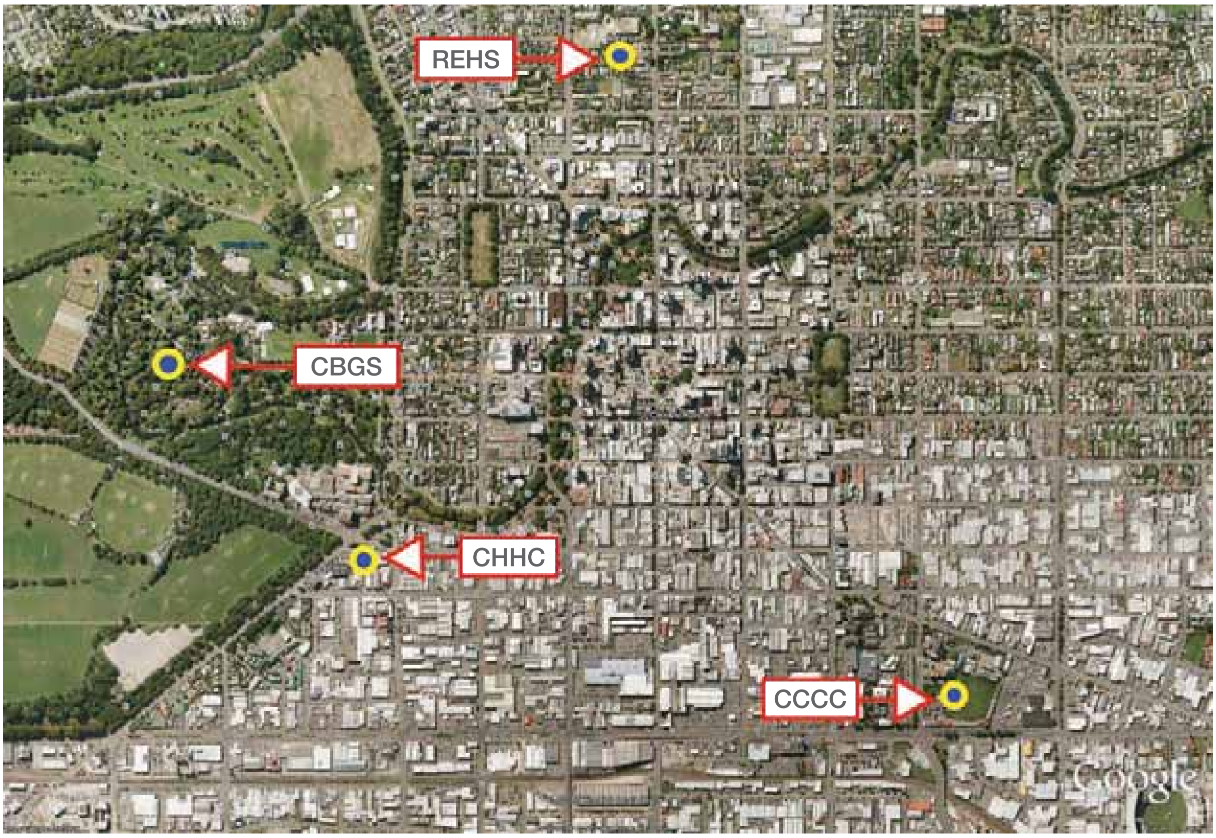

Figure 1: Location of seismic measuring stations

Displacement spectra averaged from ground records obtained at stations CHHC, CBGS, CCCC and REHS (Carr report[2])

Note that the Code displacements for periods greater than 0.7 seconds are the same for all ductility levels.

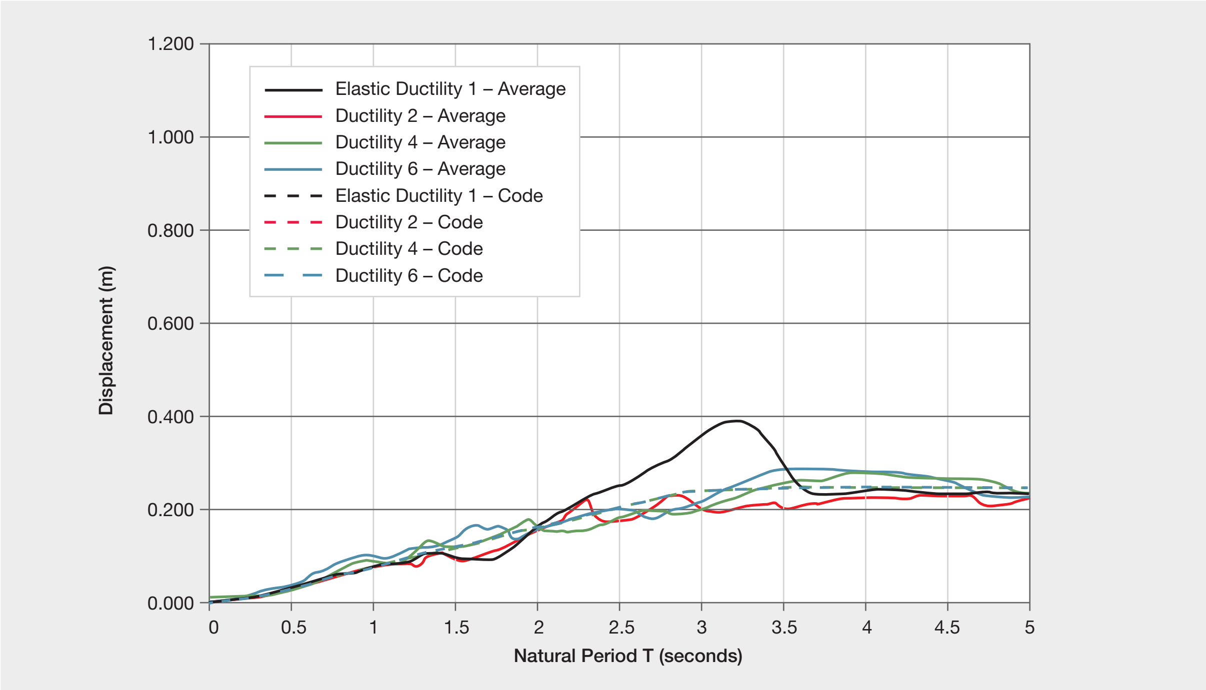

Figure 2: Average displacement spectra from four stations in the Christchurch CDB – September east–west direction (source: Carr report)

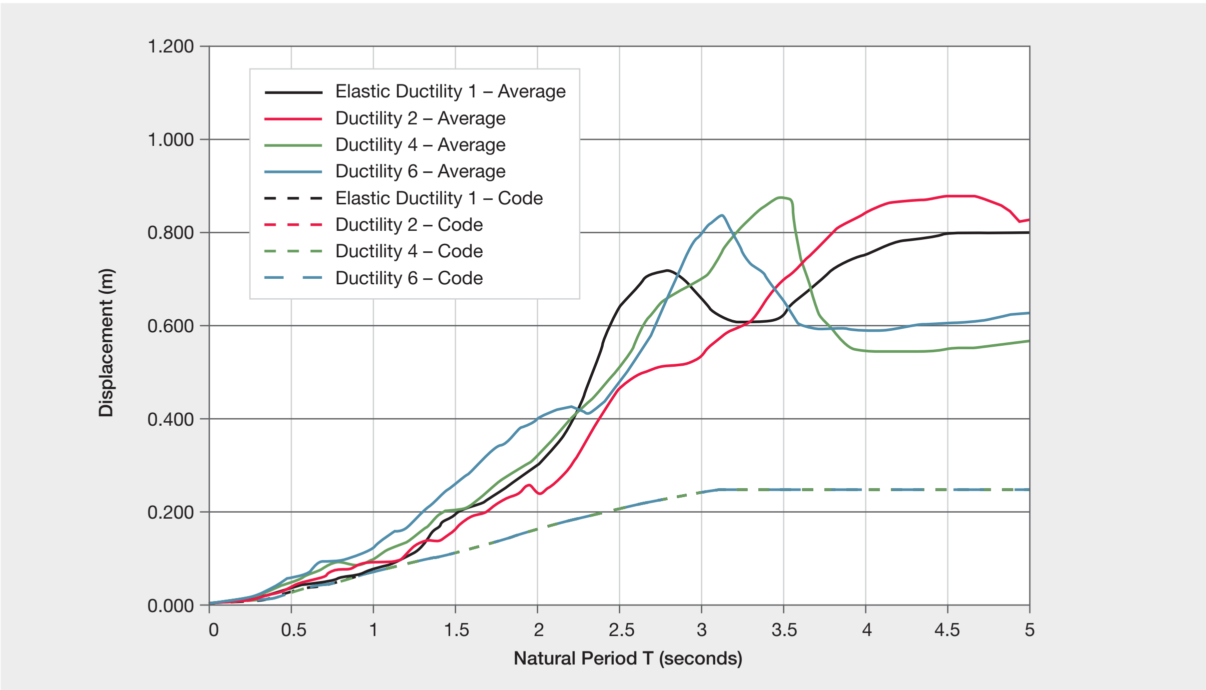

Figure 3: Average displacement spectra from four stations in the Christchurch CDB – September north–south direction (source: Carr report)

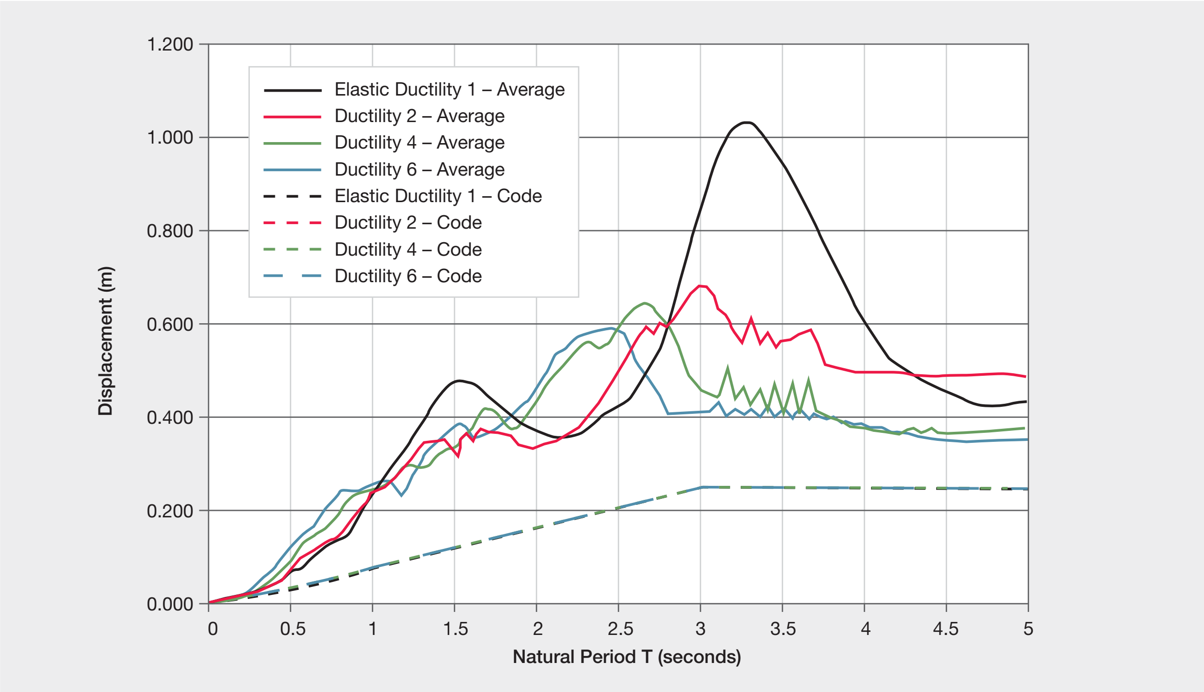

Figure 4: Average displacement spectra from four stations in the Christchurch CDB – February east–west direction (source: Carr report)

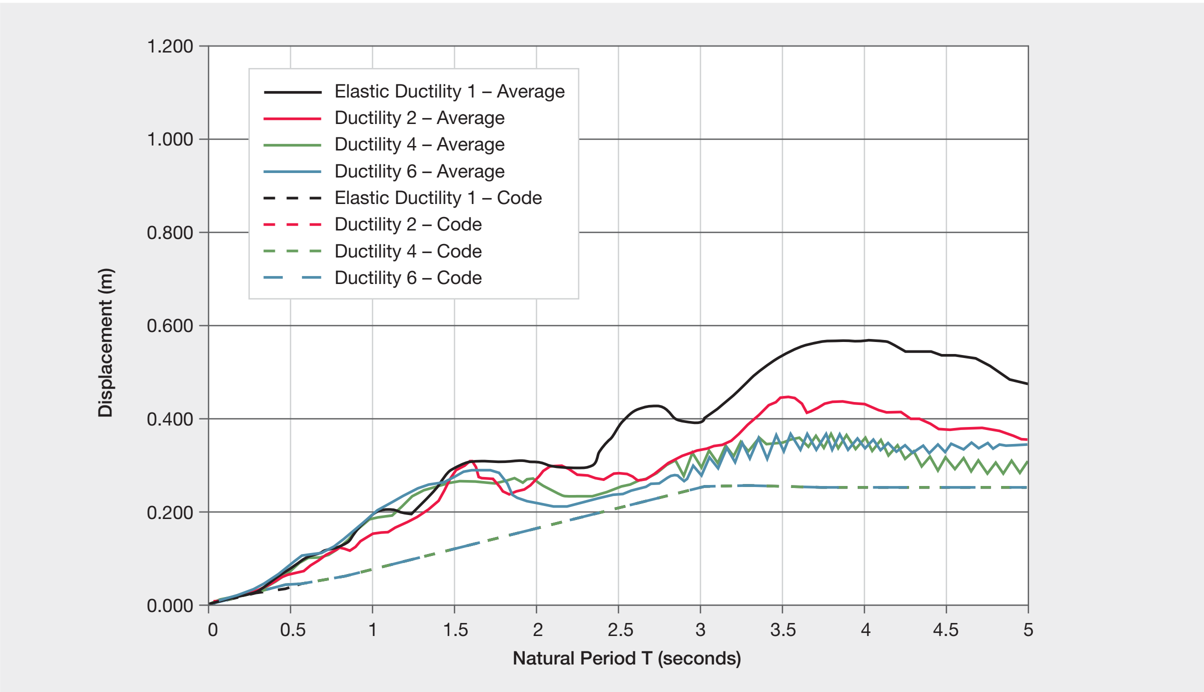

Figure 5: Average displacement spectra from four stations in the Christchurch CDB – February north–south direction (source: Carr report)

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7. Pampanin, S., Kam, W., Akguzel, U., Tasligedik, S. and Quintana-Gallo, P. (2011). *Seismic Performance of Reinforced Concrete Buildings in the Christchurch CBD under the 22nd February Earthquake*. Report prepared for Christchurch City Council and University of Canterbury, Civil and Natural Resources Engineering, November 2011.

8. Uma, S. and Beattie, G. (2011). Observed Performance of Industrial Pallet Rack Storage Systems in the Canterbury Earthquakes, *Bulletin of the New Zealand Society for Earthquake Engineering Inc*. (*NZSEE*) 44(4).

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Section 2:
Pyne Gould Corporation building

Shortly after the onset of the earthquake of 22 February 2011, the Pyne Gould Corporation (PGC) building at 233 Cambridge Terrace suffered a catastrophic collapse. As a result, 18 people lost their lives in the building.

Figure 6: View from south-east prior to the February 2011 earthquake

Pyne Gould Corporation building fatalities

As members of the Royal Commission we are conscious that our Report is largely of a technical nature. However, at the forefront of our minds have been those who lost their lives as result of the earthquake of 22 February 2011 and those left behind who loved them.

Our thoughts have also been with those who were injured and their families. We think particularly of Kate Barron and Brian Coker.

To honour those who died, we asked family members to tell us about their loved ones. The words that follow reflect what they said. We thank the families for their willingness to share this information publicly, given the personal nature of their grief.

The biographies below all relate to people who worked for companies that were tenants in the Pyne Gould Corporation building at 233 Cambridge Terrace, Christchurch. The companies referred to are Perpetual Trust Ltd, Leech and Partners Ltd, Marac Finance and Marsh Insurance Ltd. These people were all at work when the earthquake struck.

Biographies of others who died as a result of the earthquake are published elsewhere in this Report.

Jane-Marie Alberts

Ms Jane-Marie Alberts (known as JM), 44, was an account manager at Marac Finance.

She is remembered for her great love of life and her enthusiasm for so many things. She loved anything with style, anything French, top fashion, gardens with topiaries, glossy magazines, the beach and basking in the sun. If she could incorporate her favourite music and a glass of chardonnay with the preceding list she was even happier. She loved her partner Derek, her sons, and family and friends dearly and would jump at any opportunity to get them all together.

JM was an amazing mother to Jackson and Sam, always loving, supportive and interested in what they were doing. She was very proud of them and their achievements. Derek, who met JM 15 years ago at his first job straight out of university, describes her as just amazing, with a great personality, gorgeous, athletic and outgoing. JM and Derek were soul mates and were very happy together.

JM is survived by Derek Neal (her fiancé and partner of 13 years), Jackson Smith (son, aged 17), and Sam Neal (son, aged 10).

Carey Bird

Mr Carey Bird, 48, was a forensic accountant at Marsh Insurance in Australia but at the time of the February earthquake was working at Marsh’s Christchurch office in the PGC building on claims relating to the earthquake of 4 September 2010. Carey was originally from Dunedin but had lived in Sydney for almost 20 years.

Carey, who had a degree in philosophy in addition to his professional qualifications, is described as laid-back, reliable and dependable, with a dry sense of humour.

He had a keen interest in photography, in particular black and white large format landscape photography, and he displayed his photography on a website: http://members.iinet.net.au/~cbird/index.html. He was also an avid reader.

Carey is survived by Jan Bird (wife), Andrew (son, aged 20), Lauren (daughter, aged 16), Don Bird (father) and Fran Bird (mother).

Melanie Brown

Mrs Melanie Brown (known as Mel), 53, had been a broker support officer for Marsh Insurance for 13 years.

Mel enjoyed gardening, travelling, sewing, photography and arts and crafts. She made scrapbooks for the most important events in her life, including getting married to husband Steve and moving into their new home. Mel and Steve had been married for three years and had plans to pay off their house and go travelling.

Mel is described as modest, unassuming and quiet. Nothing was ever a problem for her. She was a very caring, loving person who always put other people first.

She is survived by Steve Brown (husband), Derek Gentle (father), Patrice, Deborah and Alison (sisters), Nicola, Blair, Sam, Scott, Josh, Michael and Todd (nieces and nephews) and Neve (great-niece).

Helen Chambers

Mrs Helen Chambers, 44, was a chartered accountant and held the position of Corporate Trust Risk Manager at Perpetual Trust.

Helen was conscientiously involved in her local school and parish communities as chairperson of the parent council and financial advisor to her church's parish finance committee, and she was a very enthusiastic and supportive parent. Before having a family she was involved in the Christchurch Marist Netball Club, first as a senior player then as a coach. She was later made a life member of the club. She had a great love of travel and shopping. She enjoyed playing the piano and encouraged her sons' love of music. She was extremely competitive and would confidently back herself in any task or challenge the boys would throw at her. She loved being involved with their sports as team manager, coach, scorer, taxi driver, or just cheering from the sidelines.

Helen is described as the most kind, generous, welcoming, fun-loving person one could ever hope to meet. She loved to laugh and had a wicked sense of humour. She had a huge circle of friends who very much valued her wisdom, sincerity, support and warmth.

Helen is survived by her husband of 20 years, Brett Chambers, two boys, Will and George aged 15 and 13 respectively, and Toby (a five-year-old Border Collie). Helen was the sixth of 10 children of Mr and Mrs Mervyn and Margaret Johnston. She was a much-loved favourite aunty to 36 nephews and nieces.

Patrick Coupe

Mr Patrick Coupe, 46, spent his childhood and teenage years in New Zealand and Australia. After graduating from Massey University he started working in foreign exchange, which ultimately led to his position as Financial Services Manager at Marac.

Patrick was a no-nonsense person who was passionate about life, his work and especially his children, whom he supported whole-heartedly in all of their endeavours. Through his big-heartedness and keen support for his children's interests, he had a real impact on Harewood Hockey Club and Canterbury Hockey Association where he spent years as a manager, administrator and amazing supporter.

Patrick's mother said, "He became everything I could have ever asked for." Patrick is deeply missed by family and friends and the huge support the family has gratefully received has been testament to his personality and the relationships he held with colleagues, friends and family members.

He is survived by Joanne (wife), Sean and Liam (sons), Allie (daughter), Sally (mother), Michael (father), Anna and Rachael (sisters).

Barry Craig

Mr Barry Craig, 68, was an insurance broker and risk advisor for Marsh Insurance Ltd.

Barry loved all sport. He played golf (10 handicap), was a long distance runner and an outstanding rugby league player who represented Canterbury many times. After his playing days ended Barry took up coaching. One of his proudest moments was coaching a Canterbury Under-19 side to victory over Auckland at a national tournament. In his later years Barry spent many happy hours salmon fishing at a secret spot up the Rakaia River and he kept his family well supplied with smoked salmon.

Barry is described as a gentleman, widely known for his tremendous integrity and values. He believed that tomorrow is always an opportunity and he refused to let yesterday's disappointments put an end to his dreams for the future.

He is survived by Val Craig (wife), Mark and Andrea (children), Amanda (daughter-in-law), Vanessa and Jacob (grandchildren).

Estelle Cullen

Ms Estelle Cullen, 32, was a client administration manager for Perpetual Trust.

Estelle's hobbies included Rosie (her bulldog), music, travel, socialising and home renovations. She is described as intelligent, funny, loyal, insightful, meticulous, caring and compassionate; a very beautiful person.

She is survived by Melissa Blackler and Hayley Cullen (sisters), Jacob Orchard (partner), Lloyd Cullen (father), Jocelyn Cullen (mother) and Rosie (her dog).

Adam Fisher

Mr Adam Fisher, 27, worked as a financial advisor for Perpetual Trust.

Adam enjoyed playing soccer and also played indoor netball with his fiancée, Becky. He loved watching all sport, especially rugby, and was a Crusaders fan. He was admired and well liked by others in the finance industry who commented that he was sympathetic, empathetic and cared dearly for his clients.

Adam was a loving, kind, happy and positive person throughout his life. Everyone who knew him loved him. He was funny, supportive and an amazing big brother to Simon and Sarah. He loved to tease Sarah. He adored and respected Becky, his fiancée.

He was a humble man and was caring and considerate to everyone he met.

He was besotted with his son Jack and loved being his dad. Adam's family was very important to him and he talked constantly about his son Ashton's arrival, which he was eagerly awaiting. He loved life and most of all his family.

Adam is survived by Gaye Fisher (mother), Steve Fisher (father), Simon (brother), Sarah (sister), Becky Gane (fiancée), Jack (son, aged four) and baby Ashton, who was born 10 days after his father died.

Amanda Hooper

Mrs Amanda Hooper, 30, was an account manager at Marac Finance.

Amanda had represented New Zealand as a member of the Black Sticks, gaining 40 international caps between 2001 and 2003.

She also participated in the 2002 Commonwealth Games in Manchester, the 2002 Women's Hockey World Cup and Champions Trophy and was nominated for World Junior Player of the Year in 2002. With 77 caps as a Canterbury representative, Amanda also played locally for Carlton Redcliffs.

Amanda completed the Coast to Coast race in 2005 and also ran a number of half marathons.

Her greatest achievement in life, however, was becoming a mum, which she loved immensely; she is described as an awesome mum to her daughters, Aimee and Keily.

She was an organised, very friendly, outgoing, loving, caring, motivated, committed, diligent, respectful, devoted, giving and sharing person.

Amanda is survived by Richard Hooper (husband), Aimee (daughter, aged 4) and Keily (daughter, aged 2).

Catherine Lunney

Mrs Catherine Lunney, 62, was a credit officer for Marac Finance.

She was a very strong Scottish woman who had a great sense of humour. She is described as the best friend and mum that anyone could ask for: amazing, funny, loved and deeply missed.

Catherine loved shopping weekends and looking for bargains, but what she really enjoyed was doing everything for her daughters – making them happy made her happy.

Her family are Romaine (daughter, aged 29), Ailsa (daughter, aged 25) and the late Edward Lunney (husband). Catherine had recently adopted two little Schnoodles, Shadu and Wookie, because she was unable to choose between the two and separate a brother and sister.

Kelly Maynard

Mrs Kelly Maynard, 43, worked part-time at Perpetual Trust as an estate administrator.

Kelly enjoyed walking and watching sport. She was a hard-working person who would always put others before herself. She had a lovely smile.

She is survived by Mark Maynard (husband), Molly (daughter, aged five), Matilda (daughter, aged three) and Don and Pam Hlaca (parents).

Philip McDonald

Mr Philip McDonald, 57, was a partner at the accountancy firm, Leech & Partners. He was a Mid Canterbury Rugby Union director and a Crusaders Canterbury Rugby Union Board director. He was also a keen sailor and skier.

Philip is described as being enthusiastic, supportive and loving; he was an achiever.

Philip is survived by Sharon McDonald (wife), Chantelle (daughter, aged 28 at the time of the earthquake), Andrea (daughter, aged 23 at the time) and Michael (son, aged 22 at the time).

Adrienne Meredith

Ms Adrienne Meredith, 36, was an investment support administrator for Perpetual Trust.

Adrienne loved tramping and the outdoors. She also had a dream of becoming a full-time clothes designer and sold her clothes at the Lyttelton market on Saturdays. Adrienne had returned to New Zealand three years ago after spending eight years working in the United Kingdom and travelling extensively throughout Europe.

Adrienne was very loyal to her many friends. She was funny, witty, thoughtful, and very talented in everything she did. She had an affinity with the sea.

Adrienne is survived by Anita Meredith (mother) and Paul Meredith (father).

Blair O’Connor

Mr Blair O’Connor, 34, was a managed fund accountant at Perpetual Trust.

Blair is described as being very generous and hard-working, a devoted family man and a true gentleman. Blair learned the piano and skied when younger, but more recently enjoyed spending time with his family and children, camping and reading. He was involved in helping his local church with the children’s liturgy and was on the church’s finance committee.

He is survived by Bryan and Jan O’Connor (parents), Marie O’Connor (wife), Charlotte (daughter) and Caleb (son).

John O’Connor

Mr John O’Connor, 40, was a senior investment accountant at Perpetual Trust.

John was an Irishman, born in Tralee, County Kerry. He graduated as an accountant from Trinity College and the College of Commerce in Dublin. John met his New Zealand wife, Sarah, in London in 1999 and they moved to Christchurch in October 2010 with their baby, Dan.

John is described as a family man who was charismatic and quick-witted, with a great sense of humour. He had an amazing knowledge of most sports and always followed the fortunes of the Irish national teams as well as his favourite premiership football team, Manchester United. Sport was one of his great passions in life.

John’s family are Sarah O’Connor (wife), Dan (son, aged three), Sean (son, aged 10 months; John did not get to meet Sean as he was born in May 2011), Sheila O’Connor (mother), the late Donal O’Connor (father), Marie O’Connor (sister), Thomas O’Connor (brother), Don O’Connor (brother) and Anne O’Connor (sister).

Emma Shaharudin

Ms Emma Shaharudin, 35, was an accountant for Perpetual Trust.

Emma was a loving and caring partner, daughter, sister and sister-in-law, and a proud and devoted aunty to her two young nephews, Jacob and Leo. She was a fun-loving friend and respected work colleague. She is described as always being in the hearts and thoughts of those who knew her; they will cherish the wonderful memories forever.

Emma is survived by Paul Winter (partner), Miranda Cahn (mother), Ahmad Shaharudin (father), Melanie Shaharudin (sister) and David Shahar-Yu (brother).

Michael Styant

Mr Michael Styant, 41, was a business development consultant at Perpetual Trust. Before this he had been the South Island regional manager, corporate trust division, at Perpetual.

Michael was passionate about mountain biking, snowboarding, travel, hunting and working outdoors on the family's lifestyle property. He was especially committed to his family; loved spending all his spare time with family and friends and being a hands-on dad.

He is described as unfailingly loyal and trustworthy and endlessly kind, caring and generous. Michael was solid as a rock, intelligent and practical, clever with his hands, building and DIY. He was beautiful inside and out.

Michael is survived by Rachel Fairweather (wife of 15 years), Gabriel (son, aged 11), Zachary (son, aged nine), Isabella (daughter, aged seven), Alexandra (daughter, aged five), Patricia Brooker (mother) and Alan Martin (father). Michael was one of four boys born to Patricia and Alan.

Julie Wong

Mrs Julie Kathryn Wong, 37, was an accountant at Perpetual Trust.

Julie was an active Christian, had a keen sense of adventure and loved exploring new cultures. Her personality is described as patient, gentle, mischievous and incredibly accepting of other people's faults.

She is survived by David Wong (husband), Ethan Wong (son) and Robin and Eunice Johnston (parents).

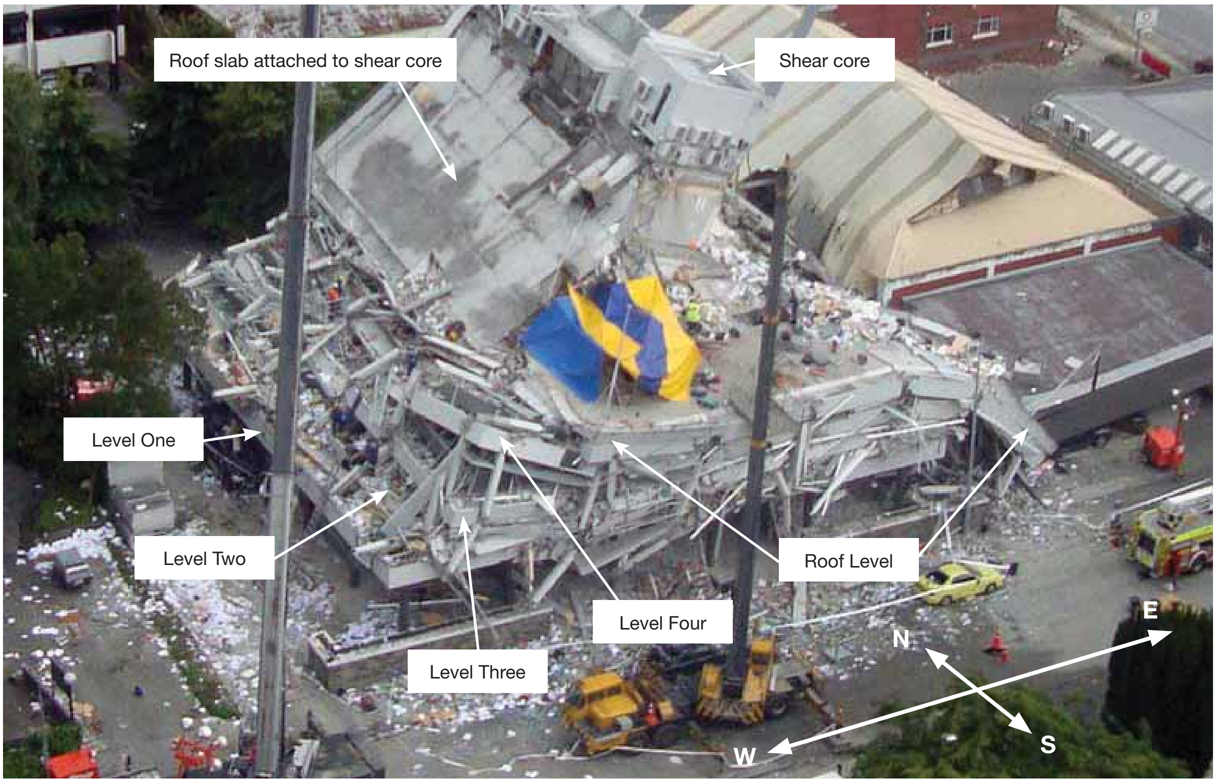

Figure 7: Aerial view from north-east after February earthquake

At the time of the February earthquake Pyne Gould Corporation Ltd (PGC) occupied the ground floor of the building, and related companies Perpetual Group Ltd (Perpetual) and Marac Finance Ltd (Marac Finance) occupied the first and second floors (levels 1 and 2). An unrelated company, Leech and Partners Ltd, also occupied part of level 1. The third floor (level 3) was occupied by the Education Review Office (ERO), and the fourth floor (level 4) by Marsh Ltd (Marsh).

The building had been purchased by PGC from the Christchurch City Council (CCC) in 1997 but was sold to Cambridge 233 Ltd in 2009. Notwithstanding the sale, the building continued to be known as the PGC building.

The discussion below covers:

- the history of the PGC building prior to the September earthquake;
- the performance of the building in the September earthquake and the Boxing Day aftershock, and the assessments of the building after those events;
- the February earthquake and the reasons the building failed; and
- lessons the Royal Commission considers should be learned from this failure.

It reflects information gathered from a variety of sources including:

- the CCC as both the regulatory authority administering building controls in Christchurch, and also as a former owner of the building;
- PGC as a former owner, and then as a tenant up until 22 February 2011;
- Cambridge 233 Ltd, the owner at 22 February 2011;
- the Beca Carter Hollings and Ferner Ltd (Beca) investigation into the failure for the Department of Building and Housing (DBH) (the Beca report)[1];
- the DBH Expert Panel review of the Beca investigation (the Expert Panel report)[2];
- a review of the Beca and Expert Panel reports carried out on behalf of the Royal Commission by Mr William T. Holmes of Rutherford and Chekene;
- evidence given and submissions made to the Royal Commission at the hearing held on 28, 29 and 30 November and 5 and 6 December 2011; and
- witnesses to the collapse.

2.1 Original construction of the PGC building

The building was originally designed as the main administration office building for the then Christchurch Drainage Board (CDB). Architectural plans for the building were prepared by the firm Paul Pascoe and Linton Architects in 1963. Structural plans (reference 691/180, S1 to S17) were prepared by I.L. Holmes Structural Engineers. These plans are available and have been considered by the Royal Commission. They are variously dated 29/10/63 and 5/12/63, and all stamped as approved by the CCC on 18 or 19 March 1964. Although a copy of the building permit has not been located, the CCC's electronic records indicate that a building permit (reference PER 63400604) was issued under the CCC's Building Bylaw No. 44 on 25 March 1964.

The book "Christchurch – Swamp to City: A Short History of the Christchurch Drainage Board 1875-1989"[3] states that construction commenced at the end of March 1964 and was completed in 1966. The building was designed, approved and constructed during the period when Building By-law No. 44[4] (in force from 1 December 1962 to 1 September 1969) applied. This particular bylaw was relatively self-contained in that it included provisions relating to design without significant reference to New Zealand Standards. It was not, however, inconsistent with design Standards of the time, although there were minor wording differences from the New Zealand Standard that could have allowed an engineer to apply lower loadings than if the Standard was used.

The Beca report refers to Part IV of NZSS 95[5] applying at the time of design and approval. However, the CCC bylaw did not in fact specify those design standards as a means of compliance. This is of little consequence to the building as constructed, as analysis by the Beca report indicates that it would have met the higher standards of NZSS 1900: Chapter 8[6], which superseded NZSS 95[5] in July 1964.

The PGC building had a plan area of 28m by 28m, which in both directions was built up from five bays of 5.08m with an additional 1.32m strip around the perimeter of the structure. The building had five floors, with housing for lift machinery and services on the roof.

Figures 8 and 9 show the general arrangement of structural walls and columns in the ground and elevated levels respectively. The lateral force resistance was provided by a shear core of structural walls, which were centred on the north–south centre line of the building but offset towards the northern side of the east–west centre line.

The arrangement was such that the eastern and western walls of the shear core, which ran in the north–south direction, were three times as long as the transverse walls labelled W1, W2, W3 and W4 in Figure 9. The figure shows two internal walls to the shear core, W2 and W3, that linked the eastern and western walls. This structural arrangement gave the building a greater lateral strength and stiffness for lateral forces acting in the north–south direction than the corresponding actions in the east–west direction. The wall thickness was 203mm throughout.

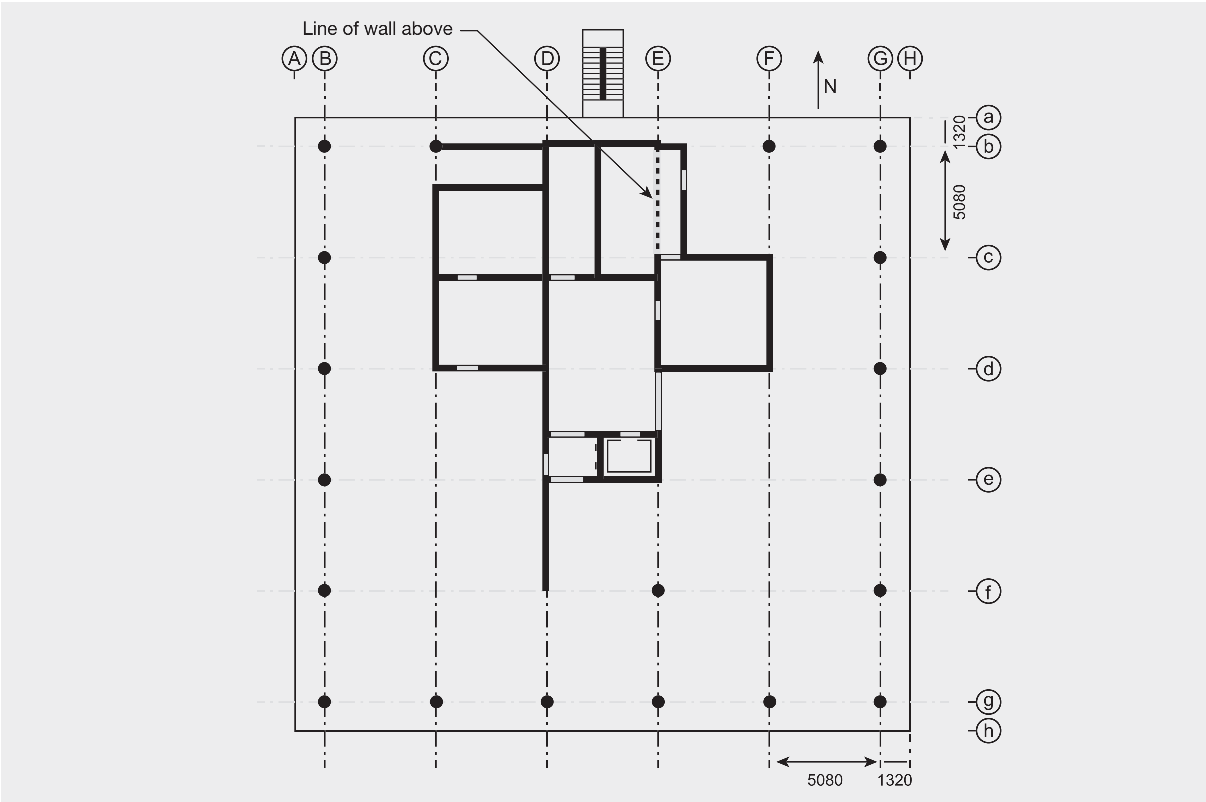

Figure 8: Ground floor plan

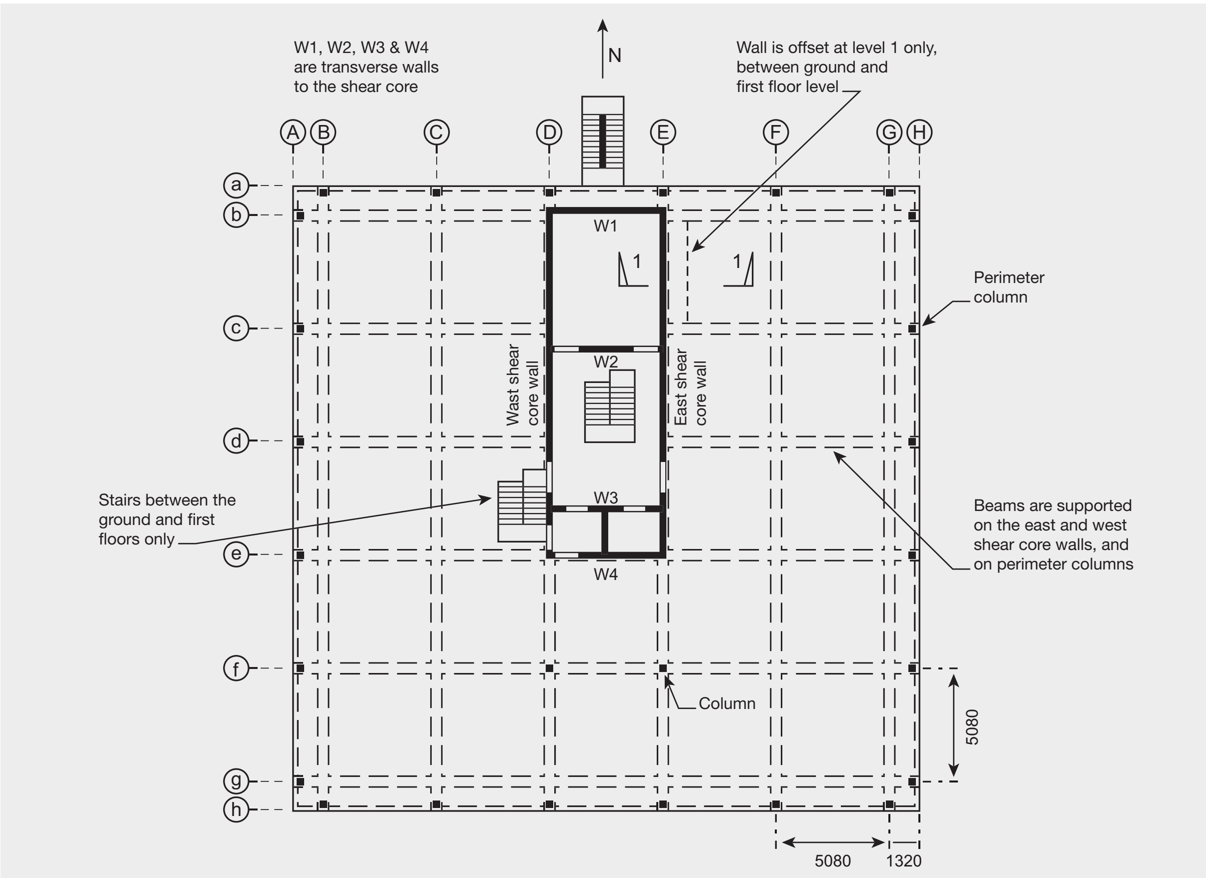

Figure 9: Upper level plan (typical)

A number of structural details appear to have had important implications for the performance of the building:

1. There were considerably more structural walls on the ground floor (Figure 8) than in the elevated levels. This gave the ground floor greater seismic protection than the elevated levels.
2. The elevated floors at each level consisted of a 152mm thick reinforced concrete slab supported on a grid of beams that were spaced at 5.08m in each direction (Figure 9). The span of the beams in the east–west direction on grid lines b to g was close to 11.5m.
3. The two transverse walls W1 (on grid line b) and W2 (close to grid line c) are shown in elevation in Figure 11. W2 was penetrated by doors at each level while W1 was penetrated by windows. There were a number of less significant openings in the transverse walls W3 and W4 located between grid lines d and e.
4. The beams supporting the floor slabs were supported by the eastern and western shear core walls on grid lines D and E and by internal columns located at the intersections of grid line f with grid lines D and E, and by columns located close to the building perimeter.
5. On the ground floor the perimeter columns were located in grid lines B, G, b and g. In the elevated levels the perimeter columns were moved out to grid lines A, H, a and h. This was achieved by supporting the perimeter columns on beams at the first elevated level that cantilevered out from the columns on the ground floor (see Figures 8 and 9).
6. The shear core walls are continuous in elevation, with one major exception. In the eastern shear core wall there is a discontinuity at level 1, where the wall in bay b-c is offset by a distance of 1.17m from the ground floor wall. This offset is illustrated in Figure 10.
7. The building was designed to codes of practice used in the 1960s. This was before ductile detailing had been developed and consequently the ductile performance of the building was both poor and not representative of more modern buildings. In particular, in terms of ductile detailing compared with current practice, there was inadequate confinement of the columns, inadequate longitudinal reinforcement in the walls, no confinement in the walls, and inadequate connection between the beams and the walls. A building with this combination of features could not lawfully be constructed today.

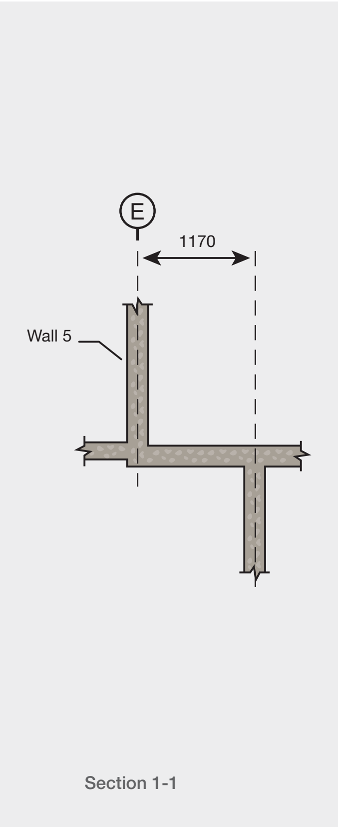

Figure 10: Cross-section 1–1

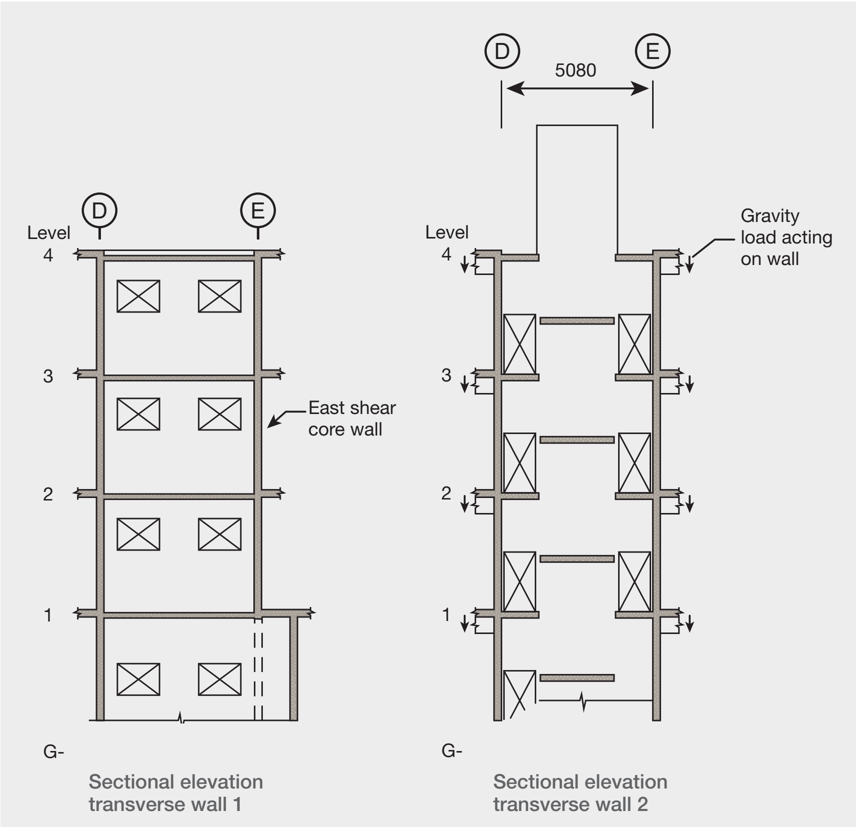

Figure 11: Elevations of transverse walls W1 and W2

2.2 Up until 4 September 2010

A number of the permits and consents issued by the CCC (including resource consents) were for work that had no relevance to the structural performance of the building. These approvals are not discussed in this Report.

In 1989 the CDB was abolished and its assets and liabilities were transferred to the new CCC established as a result of the nationwide reorganisation of local government implemented during that year.

In 1993 a prospective purchaser made an unsolicited offer to buy the building subject to a structural analysis being carried out by the CCC. The offer was rejected as the CCC considered that it should be the purchaser's responsibility to carry out any necessary investigations for its proposed use. However, in 1994 the CCC offered the building for sale by tender as it was surplus to requirements. The tender process was unsuccessful.

In 1996 feasibility studies for other uses were carried out by Arrow International Ltd on behalf of the CCC, preparatory to the sale of the building. Arrow presented a report to the CCC dated July 1996. This report included advice from a CCC senior structural engineer that no analysis of the building or structural upgrade would be required, unless there was a change of use or alterations to structural members were made. That advice was correct under the Building Act 1991: the building could not be defined as earthquake-prone because it was not constructed of unreinforced or predominantly unreinforced masonry (section 66), it could not be defined as dangerous because earthquake weaknesses were excluded from this definition (section 64), and the owner could not be compelled to upgrade the building unless specifically allowed for in the Act (section 8). There is no record of any structural analysis at this time.

The building was again offered for sale by tender in the latter part of 1996, with a closing date for tenders of 29 November. On 24 January 1997 a sale was confirmed to PGC, and the transfer was registered on the title to the land on 5 March 1997.

Mr Colin Hair, who was PGC's Company Secretary at the time of the acquisition (and remained in that role at the time of the Royal Commission hearing) gave evidence about events that occurred after the acquisition, beginning with a refurbishment of the building. PGC engaged Mr William Fox to project manage the refurbishment, and Architecture Warren and Mahoney Ltd (Warren and Mahoney) to provide architectural services. Holmes Consulting Group Ltd (HCG) was engaged by Warren and Mahoney for structural engineering services.

In relation to the refurbishment we record that:

1. On 17 February 1997, Mr Grant Wilkinson of HCG wrote to Mr Barry Dacombe of Warren and Mahoney discussing the proposed work. Among other things, Mr Wilkinson wrote:

> The building is now 34 years old and while it was designed and built to the structural standards of the day it cannot be expected to perform as well as more modern building [sic] designed and built to current standards. We recently made a preliminary study of the building and have found a potential for seismic damage to some of the columns and to the base of sections of some of the shear walls. The shear walls can be expected to "rock" in a major seismic event, so damage to secondary elements will be likely.

2. On 25 March Mr Wilkinson sent a fax to Mr Fox headed "Interim Report – Preliminary analysis". The fax included the following statements:

> The potential failure of the columns is a life safety issue, as it could result in the loss of support and consequential collapse of all or part of the building…
>
> The cracking and movement of the walls does not appear to carry any life safety implications. …
>
> Note that we consider the life safety issues above are essential, but the damage reduction measures are optional.

   There was no explanation putting the term "life safety" into context. The comment "the cracking and movement of the walls does not appear to carry any life safety implications" is a reasonably conclusive statement, and a person reading this might assume safety in any reasonably foreseeable event.

3. In April HCG produced a report titled "Seismic Evaluation of Existing Building". This report gave a detailed assessment of the potential seismic performance of the PGC building using an inelastic time history analysis. A detailed finite element model of the building was made. In the modelling, inelastic hysteretic deformation rules for both flexure and shear were defined for the individual structural elements. The shear stiffness degradation hysteretic rule was based on published results of tests made on walls.

The ground motion inputs used in the analyses were based on two ground motion records. The first was developed from the 1940 El Centro earthquake's north–south ground motion and the second was from the east–west ground motion in that event. In both cases the motion was modified so that the acceleration response spectra corresponded closely to the NZS 4203:1992[7] design response spectrum for Christchurch with deep alluvial soils. The two earthquake records were applied to the model at different levels of intensity so that the proportion of design level earthquake (as defined in NZS 4203:1992[7]) that could be sustained was able to be calculated.

The analyses indicated that the weakest link was in the performance of the perimeter columns in the elevated storeys. The concern was that these columns were highly loaded and not effectively confined, and consequently they could fail at relatively small inter-storey drifts. To ensure that they could act as props in the event of a major earthquake it was recommended that rectangular steel sections be added to the columns to maintain their axial load carrying capacity. This was assessed as a life-safety issue with potential failure occurring in an earthquake of one third or less of the design level given in NZS 4203:1992[7] if the props were not added.

The analysis indicated that some uplift of foundation pads could occur, flexural cracking could be anticipated in the walls and that there was a potential weakness in the transverse walls in the shear core. It was anticipated that extensive shear cracking could occur in these walls. None of this potential cracking was assessed as a life-safety issue but it was assessed as a potential problem in terms of serviceability. It should be noted that the analyses were made to the level of the design seismic loading in NZS 4203:1992[7], which were considerably less in magnitude than the actions associated with the February 2011 earthquake.

With the addition of the recommended rectangular steel props to the perimeter columns HCG rated the potential seismic performance of the building as being equivalent to 50 per cent of the seismic design loading given in NZS 4203:1992[7].

4. From 5 May 1997, building consent applications (the project was split into more than one application) were submitted by Mr Fox for the alterations. These included:
   - installation of steel props behind the external columns on the upper four floors to address the most critical issue identified by the HCG report;
   - removing and reinstating a stair flight;
   - infilling a stair void;
   - re-glazing;
   - removal of precast concrete sunscreens; and
   - removal of existing shell concrete roof projections.

   The work was completed and a code compliance certificate was issued by the CCC on 17 June 1998.

5. Warren and Mahoney provided a written report for the PGC Board meeting of 30 May 1997. The report dealt with the refurbishment and fitting out of the building, and discussed costs and options.

   Part of the report was headed "Structural Strengthening" and it referred to the advice previously provided by HCG. The report said:

> **Structural Strengthening**
>
> Prior to purchase of 233 Cambridge Terrace, Holmes Consulting Group provided preliminary structural comment on the structural adequacy of the building with respect to code obligations and anticipated performance of structural elements under seismic loading.
>
> While the building is a good one from a structural viewpoint it is 34 years old and cannot be expected to perform as well as a modern building built to upgraded structural standards.
>
> Since this report they have been commissioned and have prepared a more detailed structural analysis using computer modelling and have reported their findings and recommendations for strengthening work.
>
> Essentially their recommendations fall into two categories:
>
> 1. Those considered imperative to preserve life safety in the event of a major earthquake.
> 2. Those recommended as damage reduction measures.
>
> Cost estimates were prepared for the strengthening work recommended and these were subsequently evaluated in relation to risk and life cycle cost.
>
> The additional cost of damage reduction measures was estimated at $30,400.00.
>
> As a consequence only the strengthening work considered necessary to preserve life safety has been adopted and the documentation for this aspect is nearing completion.

We accept Mr Hair's evidence that there are no records to suggest HCG's written advice of February and April was provided directly to the Board. However, its substance was conveyed in the above passage from the report.

On 21 April 1998 an application (CCC reference CON98002794) was made for building consent for an office fit out on level 4. The proposal included penetrations in shear walls for which HCG provided structural drawings. The work was completed and a code compliance certificate was issued by the CCC on 3 November 1998.

On 29 January 2001 an application (CCC reference ABA10013069) was made for building consent for a roof support beam in the roof-level tearoom. Structural engineering services were provided by HCG. This was a small beam that was not of relevance to the overall structure. The work was completed and a code compliance certificate was issued by the CCC on 30 May 2001.

In 2007 PGC commissioned Warren and Mahoney to investigate the potential for further development of the site and building. A number of possible concepts were addressed, but according to Mr Hair, PGC management considered that none was economically viable and Warren and Mahoney's report was not presented to the Board. In the course of this process, Mr John Hare of HCG sent a memorandum dated 4 July 2007 to Mr Bisman of Warren and Mahoney headed "PGC Building Review-Study Findings". The memorandum stated:

> I have reviewed briefly the findings of our 1997 study when PGC purchased the building. At that stage we concluded that there were severe deficiencies with the exterior columns at the upper levels, but that the basic shear wall system was reasonably robust. Assuming the column failure were mitigated in all cases by placing secondary steel props behind them, the capacity of the building was judged at the time to be in excess of 2/3 of current seismic code loading at the time.
>
> The loading code has subsequently been updated, and probably represents a 10% increase for this building but this is not significant in the context of an existing building. It is certainly not considered earthquake-prone which is at a threshold level of 1/3 of current code loading.

Later in this memorandum Mr Hare referred to the building's "unusual structural form that may work to our benefit", noting that the columns "step across at the first floor to create the structural setback…" He referred to this as "a severe structural weakness seismically as this discontinuity has the potential for severe failure".

Mr Hare sent a handwritten fax to Mr Bisman on 4 September 2007 regarding a further development option under consideration, called Option D. This involved a low addition to the rear of the existing building. Mr Hare wrote:

> Potentially we may need to look a lot more closely at the existing exterior gravity structure as the walls may rock a long way even with the proposed new structure.

On 2 November 2007 an application (CCC reference ABA10081446) was made for building consent for a fit out on the ground floor. This fit out included structural alterations to walls, for which HCG provided the engineering design. The Project Information Memorandum (PIM) issued by the CCC for this work on 12 December 2007 included a statement on earthquake-prone buildings:

> Due to changes to the definition for Earthquake Prone Buildings in the Building Act 2004, Council's current records do not fully identify all buildings which may be potentially earthquake-prone.
>
> The [effect] of this change is that buildings built prior to 1976 may now need to be assessed to ascertain if they meet the standard of a third of current New Zealand Building Code as specified in the Building Act Regulations.
>
> Consent applicants may be asked to engage a structural engineer to assess the building to determine if the building is above the Earthquake Prone Standard as specified in the Building Act Regulations and to provide this information with any consent application to the Council.
>
> Note: Prior strengthening work may no longer be sufficient to comply with the Building Act 2004.

The application included a document prepared by HCG: "PGC Office Relocation-Project Features Report". It stated (referring to work carried out in 1997):

> At that time a full seismic assessment was carried out by Holmes Consulting Group, and it was determined that although the building does not conform with current codes, it is expected to behave reasonably well in an earthquake, provided that sufficient secondary supports were installed to provide back-up to the exterior precast column elements above the ground floor. The general lateral support system of the building comprises a system of structural walls on rocking foundations. These walls are also gravity load bearing, although the overall floor loads are not high.

The expression "reasonably well" is not quantified in the report but this was evidently accepted by the CCC and there is no evidence of further consideration of the building's seismic strength.

Under the CCC's Earthquake-Prone, Dangerous and Insanitary Buildings Policy 2006[8], as the value of the work was less that 25 per cent of the rateable value of the building, an assessment of the seismic strength of the building was not required, provided there was compliance with section 112 of the Building Act 2004. Relevantly, that meant that the building had to comply with the structural provisions of the Building Code to at least the same extent as before the alteration. Mr Hare's memorandum of 4 July 2007, discussed above, had effectively dealt with that issue. The relevant part of section 112 of the Act, and an extract from the CCC's 2006 policy are set out below.

The work was completed and a code compliance certificate was issued by the CCC on 30 October 2009.

Extract from Building Act 2004, Section 112, Alterations to existing buildings

(1) A building consent authority must not grant a building consent for the alteration of an existing building, or part of an existing building, unless the building consent authority is satisfied that, after the alteration, the building will—

(a) comply, as nearly as is reasonably practicable, with the provisions of the building code that relate to—

- (i) means of escape from fire; and
- (ii) access and facilities for persons with disabilities (if this is a requirement in terms of section 118); and

(b) continue to comply with the other provisions of the building code to at least the same extent as before the alteration.

Note that part 2 of this clause gives some exceptions that can be applied by a territorial authority in some limited situations.

Extract from Christchurch City Council Earthquake-Prone, Dangerous and Insanitary Buildings Policy 2006

1.2 Definitions
Significant alteration
Significant alteration, for the purpose of the Policy, is building work on the structural support of the building or building work that has a value of more than 25 per cent of the rateable value of the building.

1.7 Interaction between earthquake-prone building policy and related sections of the Building Act 2004
When an application for a consent for a Significant Alteration to a building is received and the building has an earthquake-prone strength of less than 10 per cent of the Code, the building will be required to be strengthened to at least 33 per cent of Code as part of the consent.

Owners of buildings with a strength between 10 per cent and 33 per cent will be given consent for alterations and will be formally advised that when the first review of the policy is completed and timeframes for action set, the owner is likely to be served formal notice requiring action to strengthen or demolish the building within the timeframe set in the policy review.

When an application for a consent involving a change of use is received, the requirements of the Building Act, section 115, for the building to be strengthened to as near as is reasonably practicable to the strength of a new building will be followed.

On 2 October 2008 a building consent application (CCC reference ABA10088473) was made to install a 12m telecommunications mast on the roof of the building. Engineering details were provided by Opus International Consultants Ltd. The PIM for the project contained the same information with regard to buildings that was provided by the 2 November 2007 building consent application. As with the ground floor fit out discussed previously, the value of the work was less than 25 per cent of the rateable valuation of the building, and the CCC's 2006 Earthquake-Prone, Dangerous and Insanitary Buildings Policy[8] did not require a consideration of the seismic strength provided there was compliance with section 112 of the Building Act 2004. There is no evidence that the Opus engineer or the CCC considered this provision of the Building Act in the context of this alteration to the structure. The work was completed and a code compliance certificate was issued by the CCC on 23 December 2008.

On 13 March 2009 Mr Hare wrote to Ms Golding at PGC. His letter was headed "Column Cracking Review" and recorded that he had been to the building on 9 March to inspect damage that had been reported to a column, towards the centre of the eastern face of the building. He expressed his view that the cracking was likely to be the result of plaster bond failure. However, he recommended that a specialist concrete repair contractor be engaged to assess the position, as in the worst case there could be a worsening problem with corroding reinforcing.

On 23 March Mr Hare wrote again to Ms Golding stating:

> I received your email last Friday instructing us to proceed with engaging Construction Techniques to complete the investigation and repair work so we will proceed on that as soon as possible.

On 26 April Mr Hare sent an email to Ms Golding in which he wrote:

> I now believe that almost all of the cracking that is visible on the columns (including most likely the one that we were looking at) is happening on the site of previous repairs. This makes it much more likely that the damage is indicative of corroded reinforcing… I am sorry that this looks like the worst case scenario from my earlier letter, but we will do our best through this process to control your costs and to keep you informed. We will look at alternative repair measures with Contech and present these if it makes sense from a whole of life perspective.

On 1 July 2009 an agreement for sale and purchase of the building was entered into between PGC and Mr Stephen Collins or nominee. Cambridge 233 Ltd was subsequently nominated as the purchaser. Mr Collins was the sole director of Cambridge 233 Ltd and a trustee of the trust that was its sole shareholder. At this stage it appears that NAI Harcourts Pty Ltd (Harcourts) was engaged to obtain a building condition report to help the purchaser carry out due diligence in respect of the purchase. Harcourts commissioned and obtained two reports on the building, one from Spotless Facilities Services (NZ) Ltd and the other from Plant & Building Safety Ltd. Harcourts was also to be wholly responsible for the management of the building, including arranging repairs where required throughout the period of Cambridge 233 Ltd's ownership.

A building condition report (which was undated) was prepared by Mr Scott Thompson of Spotless. The report included some information with regard to the structure, including details of where water had penetrated under plaster on the eastern side of the building, breaking away a section, and where steel had expanded "spalling off" concrete under a beam. Cost estimates were given for repair. The report referred to some outstanding items of deferred maintenance, and Mr Thompson wrote that "hopefully" the report would "give the prospective building owner a better insight into the current condition of the building and services".

The Plant & Building Safety Ltd report was prepared by Mr John Phillips. It focused on the building's warrant of fitness and also identified the potential for the building to be earthquake-prone, stating that remedial works might be required as a condition of future building consents. In doing so, it essentially repeated information that was set out in a land information memorandum (LIM) that was obtained by Chapman Tripp Sheffield Young Ltd, solicitors acting for Cambridge 233 Ltd as part of the due diligence process.

The LIM (Council reference LIM70108939) was issued on 21 July 2009. It included information about earthquake-prone buildings, which repeated the warnings given in the 2007 building consent. During the hearing Mr Collins claimed that he was not made aware of the potential of the building to be earthquake-prone. Mr Buchanan also confirmed that at no time had he told Mr Collins that the building had the potential to be earthquake-prone owing to its age.

On 15 September 2009 the purchase by Cambridge 233 Ltd was registered on the title to the land. Pyne Gould Corporation Ltd and Cambridge 233 Ltd entered into a lease of the ground floor.

2.2.1 Seismic analyses carried out by Beca

We break the narrative at this point to refer to some of the work that was carried out for the purposes of the Beca report[1], as it will assist the following discussion of the performance of the building in the earthquakes. To assess the cause of failure of the building in the February earthquake and its likely seismic performance in the 4 September and the Boxing Day 2010 earthquakes, Beca made a series of analyses. These included:

- a simple non-linear pushover analysis for forces in the east–west direction, but excluding torsion and ignoring the offset in the eastern wall in bay b-c at level 1 (see Figures 9 and 10);
- response spectrum analyses;
- a number of pushover analyses; and
- a number of non-linear inelastic time history analyses.

The conclusions about the structural performance of the building were determined from the time history analyses predictions. In these analyses the non-linear hysteretic response of reinforcement and the concrete were modelled. However, it is not clear from the report how shear strength and shear stress deformations were modelled.

The ground motion records used in the analysis were obtained from the Resthaven Retirement Home (REHS) site near Peacock Street in the north-west of the CBD, some 670m from the PGC site. These records were chosen as this was the closest site where ground motion during the earthquake was measured. However, it was noted in the Beca report that there were differences between the PGC building site and the REHS soil profiles, with the soils at the former being somewhat stiffer. Also, the REHS ground motion record is generally more energetic than the other records. The Royal Commission considers that owing to the sensitivity of predicted performance to the ground motion record, the use of at least one other record in a few analyses would have given more robust predictions.

Beca does not give an estimate of the initial fundamental period of the building. However, Professor Nigel Priestley gave an estimate of 0.7 seconds for this value, which the Royal Commission assumes applies to vibration in the east–west direction. Given the rectangular shape of the shear core, it could be anticipated that the fundamental period in the north–south direction would have been of the order of 0.35 seconds, although this value is not stated in the report.

2.3 The September earthquake

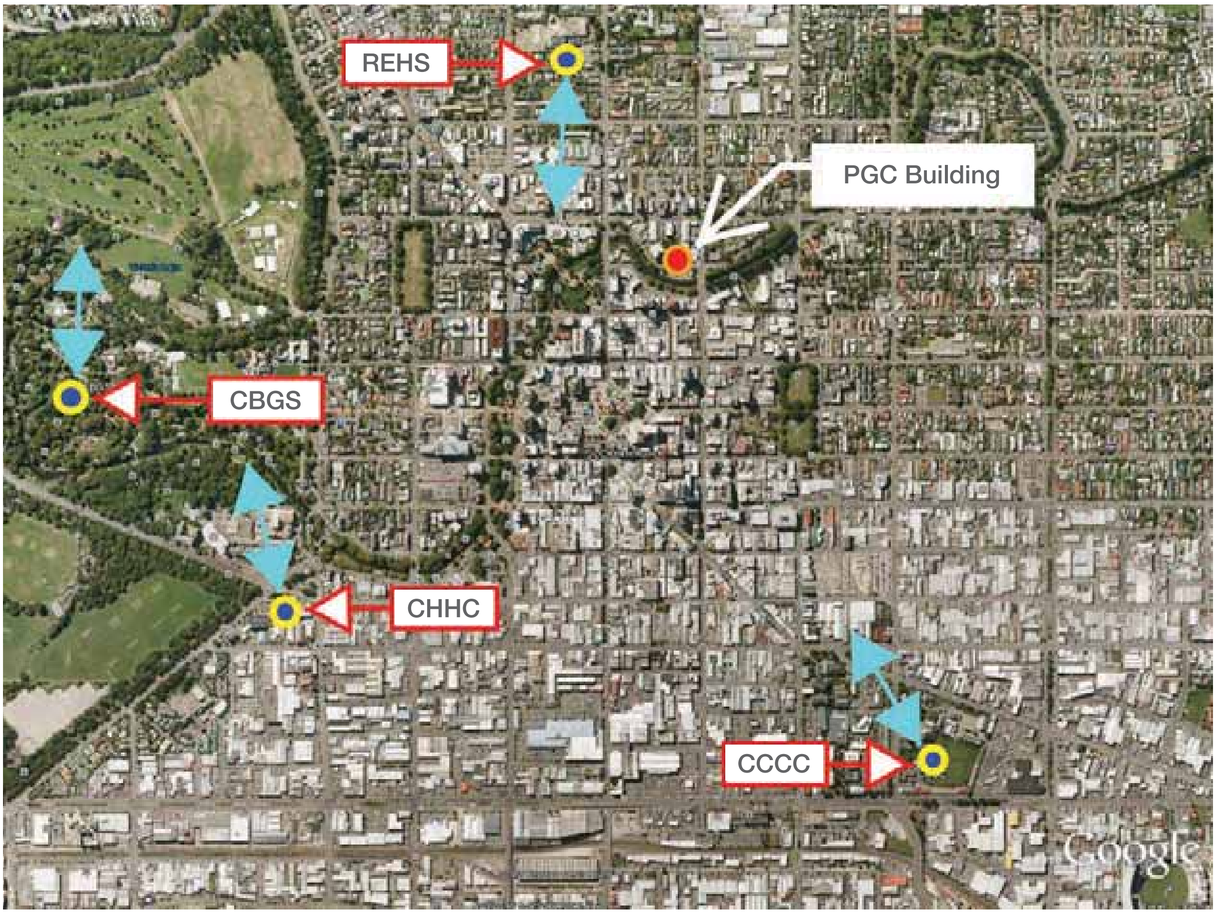

Figure 12: Location of seismic measuring stations and predominant direction of acceleration, September 2010

The nature and intensity of the September earthquake are described in section 2 of Volume 1 of this Report.

On the basis of available information "Inelastic Response Spectra for the Christchurch Earthquake Records"[9], and assuming the actual ground motion at the site was similar to that at the REHS site, the severity of the ground motion in the September earthquake was comparable to a design-level earthquake event for the ultimate limit state specified in NZS 1170.5:2004[10].

In this earthquake record the greatest shaking was in the north–south direction, which was the stronger direction of the building. In this direction the primary load resistance was by long shear walls on either side of the central core, symmetrically placed around the centre. The ground floor had a greater number of shear walls, making it significantly stronger than the floors above. Minimal torsional action was induced for ground motion in the north–south direction as the structure was symmetrical on this axis.

Under these seismic actions some damage could be anticipated, but as the spectral displacement in the north–south direction is of the order of 30mm, and the corresponding displacement in the east–west direction is of the order of 80mm, extensive damage would not be anticipated. It should be noted that the spectral displacement corresponding to the fundamental mode of an equivalent single degree of freedom structure is at a height of about 70 per cent of the height of the main part of the structure.

The Beca inelastic time history analysis for the September earthquake predicted that some minor yielding of reinforcement would have occurred in the structural walls but there would be no failure.
The predicted cracking in the walls was consistent with that observed during inspections of the building immediately after the September earthquake.

2.4 Between the September earthquake and the Boxing Day aftershock

As discussed elsewhere in this Report, soon after the September earthquake a state of local emergency was declared under section 68 of the Civil Defence Emergency Management Act 2002 and the CCC initiated a civil defence emergency management response. The state of emergency continued until midday on 16 September, when it lapsed.

Starting on the day after the earthquake, teams were sent out to all of the commercial parts of the central business district (CBD) to undertake a Level 1 Rapid Assessment. These teams included at least one CCC officer, who was usually accompanied by a Chartered Professional Engineer (CPEng). A Level 1 Rapid Assessment is an exterior inspection to look for obvious signs of damage that indicate immediate dangers, or to determine whether further investigations are required before the building can be used.

On the morning of 5 September such an inspection was made of the PGC building, resulting in the building being given a green placard in the standard form signifying that it had "No restriction on use or occupancy". The placard was placed on the main entrance door to the southern side of the building facing Cambridge Terrace. The standard form advised that the inspection was brief and no apparent structural or other safety hazards had been found. However, the form also encouraged the owner to obtain a detailed structural engineering assessment of the building as soon as possible. It will be recalled that a previous assessment had been carried out by HCG in 1997, which concluded that the building had 50 per cent of the design performance defined in NZS 4203:1992[7]. This would be less than 50 per cent of new building standard (NBS) in terms of the current Standard, NZS 1170.5:2004[10].

On the morning of the September earthquake, Mr Howard Buchanan of Harcourts contacted Mr Hare of HCG to request that an engineering assessment of Harcourts' entire portfolio of buildings be undertaken. There was also a telephone conversation between Mr Collins and Mr Buchanan, in which Mr Collins requested that immediate inspections be undertaken by a structural engineer to confirm that it was "safe to occupy" his buildings before the tenants were allowed to re-enter. This was after Mr Buchanan's instruction to HCG, and there is no indication that there were any monetary restrictions placed on obtaining this assurance. In fact, Mr Buchanan accepted that Harcourts had authority to spend money on the building in the order of "tens of thousands of dollars" without recourse to the owner. Such a sum would have allowed for the commissioning of a detailed structural analysis if that had been recommended by the engineers.

Mr Buchanan met Mr Richard Seville from HCG on 5 September to establish a procedure for the inspection of Harcourts' managed properties. A short form agreement prepared by HCG was signed by Mr Buchanan and Mr Seville at this time to provide initial earthquake inspection and securing measures as considered necessary.

There was no further elaboration in the contract of the services to be provided. Mr Seville was not called at the Royal Commission hearing but subsequently provided a statutory declaration, in which he stated:

> We discussed that HCG would be carrying out level 2 rapid visual inspections (external and internal). If further inspections or securing works were required to any building, to seek to upgrade a yellow placarded building to a green placarded building for example, we were instructed to recommend this. It was made clear that the initial inspections that HCG was instructed to carry out were not detailed evaluations and HCG was to report back to Harcourts if HCG recommended further, and potentially more intrusive, inspections or securing work.

On 7 September 2010, the first inspection by HCG was undertaken by Mr Mark Whiteside. Mr Whiteside had the qualifications Bachelor of Engineering (Civil) and Master of Engineering, was registered as a CPEng and was a member of the Institution of Professional Engineers New Zealand (IPENZ). He had 11 years' postgraduate experience in engineering at the time of his inspection. Mr Whiteside attended briefings on the requirements for Level 1 and 2 Rapid Assessments at both the CCC and HCG.

Mr Whiteside said in evidence that he was carrying out what he considered to be the equivalent of a Level 2 Rapid Assessment. He did not use the Level 2 assessment form, which may not have been widely available at that time, but prepared a brief written inspection report on HCG letterhead. The inspection report records the work he carried out as:

> Rapid Structural Assessment
>
> Walk around exterior, ground, first, fourth floors.

Under a heading "Observations & Comments" Mr Whiteside noted that he had carried out an "initial inspection" of the building, which he described as an "in situ concrete construction building with concrete shear wall to south side". He accepted in evidence that the reference to the shear wall being on the south side of the building was incorrect: the reference should have been to the north side. Mr Whiteside noted in his report:

> Cracks to ground floor and first floor level shear walls.
>
> Fourth floor ceiling grid bracing has failed, ceiling tiles have been removed, electrical and air conditioning services are exposed.

The report concluded:

> Confirming 'green placard' building okay to occupy (structurally)

In evidence Mr Whiteside stated that his assessment that the building was "okay to occupy (structurally)" was based on his opinion the building did not have "diminished structural capacity" as a result of the September earthquake. He considered that the extent of damage he had observed was "not indicative of a building under immediate distress or having any significant impaired resistance to earthquake shaking". He also stated that in carrying out the inspections he did not consider the possible magnitude of future aftershocks, concentrating only on the issue of whether the building showed signs of diminished seismic capacity. The possible limitations of that approach were not explained in writing to Harcourts, or to the tenants of the building.

Mr Whiteside had no knowledge of HCG's previous involvement with the building, and consequently no knowledge of the structural weaknesses previously identified. In cross-examination he expressed the view that such knowledge would not have been of assistance:

> Those previous reports were… addressing the capacity of the building. Our inspections were addressing whether the building had any diminished capacity. The building structural system was reasonably obvious and able to be observed and the reports confirmed that the system was a shear core wall so I don't believe they would have been of any benefit.

Mr Whiteside's opinion of the accuracy of this assessment had not changed by the time of the hearing before the Royal Commission. His assessment is also considered to have been accurate by Mr Hare, and the authors of the Beca and Expert Panel reports on the collapse. The Royal Commission notes that the shear core of the building (being the primary seismic resisting structure) was visible without removing linings. While we accept that viewing the existing drawings or previous structural analyses would not necessarily have led to a different decision about whether the building had diminished structural capacity as a result of the September earthquake, this information would have been of assistance had a detailed structural analysis been carried out.

As a result of the failure of the level 4 ceiling tiles, Harcourts contracted to remove the existing heavy tiles and replace them with a lighter system. The order for this work was placed on 7 September and the work was completed by 17 September.

On 10 September Ms Golding reported to Ms Louise Sutherland of Harcourts concerns expressed by Leech and Partners Ltd about cracks in the hallway leading to the car park. Ms Golding advised that the hallway was "very badly cracked in a number of areas including one key area that in fact according to Spotless holds up the building". On 15 September Ms Manawatu-Te Ra of Harcourts replied advising that an HCG engineer would be onsite that morning to investigate the cracks.

In fact it was on the morning of 16 September that a second HCG inspection was carried out, this time by Mr Alistair Boys. Mr Boys has the qualifications Bachelor of Engineering (Civil), and Master of Engineering (Structural). His specialist study area was the performance of poorly detailed reinforced concrete columns including reinforced concrete buildings and the performance of buildings in earthquakes. At the time of his inspection he had about two years' postgraduate experience. Mr Boys had attended briefings on post-earthquake assessments within HCG. He knew that there had been a previous HCG inspection, but he was not aware who had made it and did not rely on its conclusions. Rather, as he said to Mr Mills QC in cross-examination, he carried out the inspection in the same way he approached all inspections that he did, using the same methodology and "approaching it almost independently of the previous information using it as a verification at the end…against my own conclusions".

Mr Boys gave evidence that his inspection of the building took about 90 minutes. He first made a preliminary inspection of the exterior to provide an initial gauge of any damage that the building had sustained and to gain an appreciation of the building's form and primary load paths. He did not see any external evidence of damage. He ascertained that the building was of reinforced concrete with internal core walls (including a lift and stair core) and with a perimeter gravity frame at the exterior façade. Next, Mr Boys made a visual inspection of what he considered were the key accessible structural elements on the ground and first floors. These included the shear walls enclosing the lift and stair core and the perimeter frames of the building. The structural damage he observed was limited to cracking of the shear walls at the central core. He said in evidence that the cracks were typically about 0.2–0.3mm in width. One, however, located on the southern wall of the central core, "measured 0.5 and 0.6mm with minor spalling at the intersection of the opposing inclined cracks". This spalling was about 10mm deep and confined to the area immediately adjacent to the cracks. Mr Boys also looked at the central core walls and perimeter frames on levels 2, 3 and 4. He saw nothing of significance.

Mr Boys completed the "Christchurch Eq RAPID Assessment Form – Level 2". The status of the building shown on the form was confirmed as "Green G1", a category described on the form as signifying that the building was "Occupiable, no immediate further investigation required". Mr Boys also wrote a brief report in which he recorded, among other things:

> All cracks observed minor in shear walls – typically <0.5mm.
>
> One single crack 0.6mm and minor spalling initiated at intersection approximately 100x100x10mm max depth.
>
> Spalling in spandrel beams (outside) initiated by reinforcing corrosion – not significant.

As with Mr Whiteside, Mr Boys inspected the building for the purpose of ascertaining whether there was evidence that it had diminished capacity as a consequence of the earthquake. He confirmed under cross-examination by Mr Mills that he did not consider any issues relating to whether the building could have been considered as earthquake-prone before the earthquake, or what might have previously been known about any structural weaknesses. Such matters were not, in his view, relevant to the damage-based assessment he was carrying out. In cross-examination by Mr Elliott he confirmed that nothing he observed caused him to conclude that any further or more extensive investigation was required.

On 30 September Mr James West of Perpetual sent an email to Harcourts following up a verbal request said to have been made the previous week for an assessment by an engineer of new damage to the building after a series of aftershocks. Ms Sutherland responded for Harcourts on the same day, writing that the building had already been assessed by two structural engineers, and had been "classified as safe to occupy". Any damage seen was cosmetic. The cracks noted by Mr West, near Perpetual's storage area backing on to the liftshaft on level 1, would be "taken into account" when repair works were done. Ms Sutherland observed that as long as aftershocks were occurring new damage would appear, but that there was "little point in rushing into repair works until they had stopped".

On 14 October, Ms Golding of PGC made another request for a further engineering assessment as some external wall cladding appeared to have moved from the wall. Mr Whiteside returned to the site later that day to carry out the third inspection by HCG. On 15 October he wrote a brief report describing the "re-inspection of ground floor window frame gap and second floor partition crack". He stated:

> Ground floor – Window frames span from floor to floor. Aluminium mullions had moved internal cabinetry creating a gap (or enlarging).
>
> No structural issues. Gap should be addressed for weather proofing.
>
> Second floor – Partition crack at concrete interface.
>
> No structural issues.
>
> Building remains structurally okay to occupy on above observations.

On 20 October, following an aftershock the previous day, Ms Glenys Ryan of ERO sent an email to Ms Sutherland about movement of the ceiling tiles on the third floor. Mr Cambray of ERO followed up with an email on 22 October concerning the ceiling tiles and a crack in an internal wall. He also asked whether Harcourts had or planned to develop a full building evacuation plan.

On 5 November Ms Ryan sent an email to Ms Sutherland about a new crack observed between a partition wall and the liftshaft on the eastern side of the building. On 9 November Ms Sutherland replied that there was cracking similar to what Ms Ryan had described on other floors in the building. The cracks had been inspected several times by structural engineers and confirmed as superficial. She advised that Harcourts was working with the building owner's insurer and intended to appoint a project manager to oversee necessary repairs to the building, which would first be catalogued.

2.5 From the Boxing Day aftershock to the February earthquake

On Boxing Day 2010 an aftershock, described elsewhere in this Report, struck directly under the Christchurch CBD. A civil defence emergency was not declared.

Mr Tucker of ERO inspected its tenancy after the earthquake and contacted Ms Ryan. She went in on 27 December and saw that some tiles had fallen from the ceiling, while others were hanging down at an angle. Harcourts was advised and unsafe tiles were removed in time for the ERO office to re-open on 12 January.

On 20 January, following aftershocks on that day, Ms Sutherland sought that HCG carry out a further inspection of the building, after being advised by staff of Perpetual of a “new large crack that [had] appeared in a wall” and that there had been “damage sustained to the stairs (concrete come loose)”. As a consequence, Mr Whiteside carried out his third inspection of the building on 27 January.

Once again Mr Whiteside produced a written report of his inspection, which he described as a “re-inspection of previously observed damage and new cracks”.

His observations and comments recorded in the report were:

> Previous cracks have enlarged. Cracks to level 1 stationary wall now > .2mm, minor spalling also evident. General diagonal cracking to all shear walls.
>
> New cracks to stair connections at level 1 – spalled plaster. Hairline cracks to most landings (stairs appear tied to all floors).
>
> Building remains safe to occupy.
>
> Cracks to shear walls greater than 0.2mm will require epoxy injection repairs.
>
> Cracks to stairs should also be repaired where greater than 0.2mm.

A copy of this report was sent to Perpetual on 28 January.

Because of the high frequency of ground motion and the short duration, the Beca analysis did not predict significant further inelastic deformation for this earthquake.

2.6 The February earthquake

Figure 13: Location of seismic measuring stations and predominant directions of acceleration, February 2011 earthquake

The nature and intensity of the February earthquake are described in section 2 of Volume 1 of this Report.

For all four of the sites where earthquake ground motions were measured, the accelerations and displacement spectra in the February earthquake were appreciably greater in the east–west direction than in the north–south direction. With particular reference to the REHS site the spectral displacements in the north–south direction were of the same order as the NZS 1170.5:2004[10] design values at the corresponding fundamental period of 0.35 seconds, while in the east–west direction the corresponding values at a period of 0.7 seconds were about three times as high as the NZS 1170.5:2004[10] design values.

The earthquake resulted in the rapid catastrophic collapse of the PGC building. The reasons for failure and the likely sequence of events are addressed below.

2.7 The collapse of the building

The Royal Commission has been assisted in its understanding of the collapse of the building by the Beca report[1], the Expert Panel report[2] and a review of both by Mr Holmes, prepared at the request of the Royal Commission.

In addition, a number of witnesses (including some who were in the building at the time of the earthquake) gave evidence to the Royal Commission about their observations of the collapse. We refer to this evidence before turning to the experts' opinions.

2.7.1 The eyewitnesses

Mr Robert Wynn, an electrical engineer employed by Beca, observed the collapse of the building from his office on level 4 of the PricewaterhouseCoopers building at 119 Armagh Street. His view was partially obstructed by trees, which meant that he could only see the two top floors and the mechanical services housing on the top of the structure. He described this as falling very quickly, as if the building had been subjected to a controlled demolition. He said that the eastern side of the building collapsed more quickly than the western side, the former seeming to pull the latter around so that the building rotated as it fell. He thought that the collapse occurred between five to eight seconds after the commencement of the earthquake.

Mrs Helen Guiney was employed by Perpetual. When the earthquake struck she was at her desk on level 1, speaking on the telephone. She immediately dived under her desk. She said:

> The last thing I saw as I was getting under my desk was the front window which was to my left-hand side blowing out. The ceiling tiles were falling all around me but it seemed to be progressive from the reception area. The telephone connection was lost and power failed. Everything was dark and silent after the shaking stopped.
>
> I was not aware at that stage that the whole building had collapsed. All I knew was that I was trapped and my hand hurt. Fortunately there was also fresh air coming in. I could feel the draught. Every time I tried to reach my phone I had to give up. My cellphone was ringing at the time, obviously people trying to make contact. There was space around me to roll over onto my back because when I first got under the desk I was in pretty much a foetal position and I could extend my legs but I couldn't move apart from that. I tried yelling for help and eventually heard my colleague Jim Faithful calling out to me. He told me he was also under his desk, that a concrete slab was on top of him. We were both yelling for help and soon realised that nobody could hear us. The handset of my phone was near to me under the desk so I started tapping out SOS on steel frame of my desk.
>
> Eventually Jim and I heard drilling and hammering but it sounded very far away. There were several more shakes and every time I would hold my breath and pray that we would be safe. The rescuers finally made contact with Jim but they couldn't hear me, I presumed because I was further inside the building. Jim was able to relay to the rescuers that I was in the building near him.
>
> I was finally rescued about 9.30 am the following day, nearly 21 hours after the building collapsed. [Figure 14.]

Figure 14: Mrs Helen Guiney being assisted from the building (source: Helen Guiney)

Later she clarified that she in fact thought the ceiling tiles had fallen progressively from the northern side of the building towards the south, which would be consistent with Mr Wynn's description of a rotational collapse in an easterly direction.

The Royal Commission also heard evidence from Ms Glenys Ryan, who was in the ERO office on level 3 at the time of the earthquake. She was in the tearoom on the southern side of the building, with five colleagues. She remembered the shaking being in the west–east direction. She was able to move into a hallway, where she sat down before the building collapsed. She was rescued after about an hour. A colleague, Ms Ann Bodkin, waited 26 hours for her rescue.

Another who gave evidence about his experience in the building during the earthquake was Mr David Sandeman, who was employed by Marsh. At the time of the onset of the earthquake he was on level 4, talking to a colleague while looking west towards Mt Hutt. He described what happened:

> In less than 10 seconds from the violent shaking starting, and it was very definitely in a east–west direction, a Lundia filing system which was immediately on my right here ran on its rails in an easterly direction heading for Manchester Street. I don't recall it sort of crashing into its bump stops because by then the building had started to collapse and it was under my heels which were – I'd my back to Manchester Street to the east, I could feel it doing that and then the next moment we were – we were plunging down. I estimate it was approximately 40 feet because we ended up on the first floor as I subsequently discovered.
>
> Happily for all of us the floor was relatively horizontal where it – where it ended up but we were in a very confined space. We could all move, none of us happily were pinned but we were most assuredly trapped. I could lie on my tummy or I could turn onto my right-hand side on the floor and with my left shoulder jammed under some furniture. It was too dark to see any details, you couldn't tell the time on your wrist watch, it was – there was a glimmer of light in the distance I guess from where the floors had just pancaked together, so there were five of us in this small area here and one a bit further away, and after about an hour and a half, two hours, we heard an engine which I figured was the engine on a fire ladder, and indeed that's what it turned about to be, because after about 10 minutes of that there was a voice coming through the roof, "Anybody there?". We were able to confirm and give the names of the five of us and say we were stuck but we were not pinned, and they assured us that they would have us out within no more than six hours. Well happily it was significantly less than that. [Figure 15.]

Figure 15: Mr David Sandeman with Mr Jeff McLay celebrating their rescue (source: John Kirk-Anderson/*The Press*)

> The retrieval took place by them sledgehammering a hole through the concrete roof and then getting a big saw that would chomp through the steel reinforcing rods to create a hole big enough for us to be extracted. The lady who was closest to the hole was rescued first and they made it a little larger and a rescuer got in and pulled debris out of the way for the remainder of us, the other four of us to commando crawl across to the opening that had been made. We were assisted onto the roof by someone pulling our hand, but the collapse was such that we literally stepped onto the roof, they didn't need to bung a ladder down or anything, we just, a big step and we were on the roof, it was then that I realised it was sloping from the centre down here, not dangerously because you could comfortably walk across to here and eventually be laddered, the fire ladder was here which we were all able to climb down and make our way to safety.

We record our appreciation of the evidence from the eyewitnesses, and acknowledge the fortitude of those who were in the building in re-living their ordeals of 22 February. For present purposes we note that we heard nothing from them that would be inconsistent with the key conclusions reached by the experts: that the building was subject to violent shaking in a west–east and east–west direction and quickly collapsed in an easterly and downward movement.

2.7.2 The Beca report[1]

The findings of the Beca investigation were presented to the Royal Commission at the hearing on 5 December 2011 by Mr Robert Jury (author of the Beca report) and Dr Richard Sharpe, both from Beca.

The findings of the Beca investigation were set out in the executive summary of the Beca report:

> **Original Design**
>
> - The structure when built met the 1963 design requirements of that time for the prescribed earthquake loads, both in terms of the level of strength and the level of detailing provided.
> - Testing of concrete and reinforcing steel from some elements after the collapse did not indicate that they were less strong than required by the design.
>
> **Modifications**
>
> - Modifications made to structural elements (addition of perimeter steel props and insertion/deletion of doorways in the core walls) during the life of the building were not material with respect to the collapse on 22nd February 2011.
>
> **Comparison with Current Code**
>
> - Pre-September 2010, the building achieved between 30 and 40%NBS (new building standard) when assessed against the New Zealand Society for Earthquake Engineering Guideline recommendations.
>
> **Damage prior to 22nd February 2011**
>
> - Damage to the structure was observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes to the:
>   - tops and bottoms of the perimeter columns
>   - core walls (cracking)
>   - stairs (cracking).
> - This damage was relatively minor and not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking.
> - The proposed method of repair at that time of grouting the cracks appeared reasonable.

Mode of Collapse

- The building collapsed when the east and west reinforced concrete walls of the core between Level one and Level two failed during the earthquake.
- The west wall yielded in vertical tension, and then the east wall failed catastrophically in vertical compression.
- The ground floor structure stayed intact and virtually undamaged as it was significantly stronger and stiffer than the structure above.
- Torsional response (i.e., twisting of the building about a vertical axis) was not a significant factor.
- Once the west wall had failed, the horizontal deflections to the east increased markedly.
- The perimeter columns and/or joints between the columns and the beams, and the connections between the floor slabs and the shear core, failed consequentially at some levels, causing the floors to pancake.

Reasons for Collapse

- The damage observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes did not significantly weaken the structure with respect to the mode of collapse on 22nd February 2011.
- The shaking experienced in the east–west direction was almost certainly several times more intense than the capacity of the structure to resist it.
- The connections between the floors and the shear core, and between the perimeter beams and columns were not required at the time of design to take, nor were capable of taking, the distortions associated with the core collapse.

Commentary

- Neither foundation instability nor liquefaction was a factor in the collapse.
- Extensive studies undertaken in 1997 for a previous owner confirmed that the structure was below the current standard at that time with respect to earthquake resilience for new buildings.
- The capacity of the building in 1997, after the addition of the steel props behind the perimeter columns, was judged, at that time, to be in excess of 50% of the then current new building standard.

The collapse scenario that Beca inferred is shown in Figure 16.

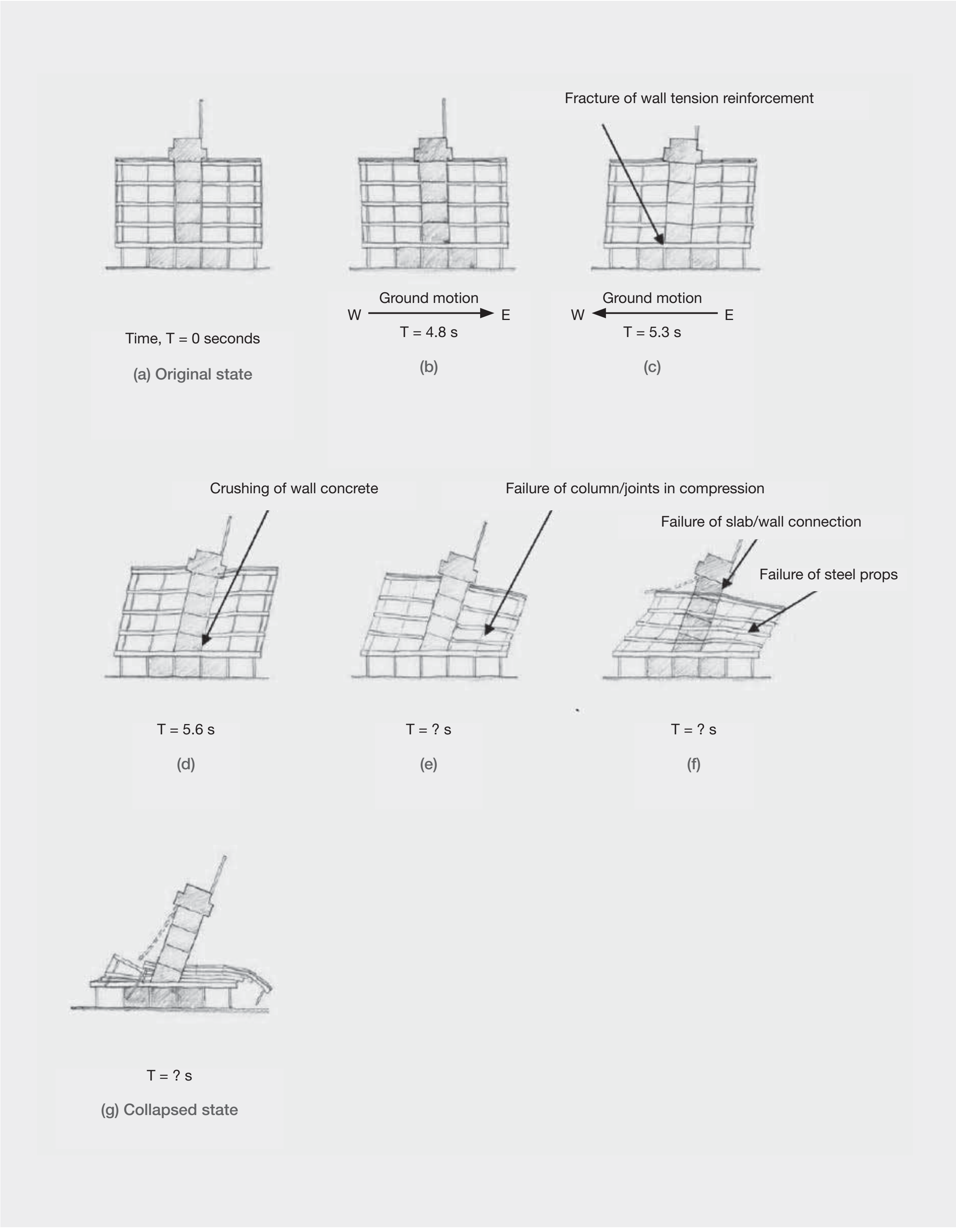

Figure 16: Inferred collapse scenario (source: Beca report)

2.7.3 DBH Expert Panel report[2]

The Expert Panel report concurred with the conclusions of the Beca report.

The findings were addressed at the Royal Commission hearing on 5 and 6 December 2011 by Professor Nigel Priestley, one of the members of the Expert Panel.

The principal conclusions of the Expert Panel were set out at paragraph 5.11 of the Stage 1 Expert Panel report dated 30 September 2011:

> **5.11. Conclusions**
>
> The PGC building structure was in accordance with the design requirements of the time (1963), both in terms of the level of strength and the level of detailing provided.
>
> Modifications made to structural elements (addition of perimeter steel props and insertion/deletion of doorways in the core walls) during the life of the building were not material with respect to the collapse on 22 February 2011.
>
> When compared to the current code for new buildings (NZS 1170.5: 2004[10], NZS 3101: 2006[11]), the PGC building would have achieved between 30 and 40 percent NBS (New Building Standard) prior to September 2010, when assessed against the New Zealand Society for Earthquake Engineering Guideline recommendations (NZSEE, 2006[12]).
>
> Testing of concrete and reinforcing steel elements retrieved from the collapsed building indicated that the strength and characteristics of those elements were consistent with those specified at the time of design.
>
> The damage to the building as a result of the 4 September 2010 earthquake and the 26 December 2010 aftershock was relatively minor, and was not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking. The proposed method of repair at that time, of grouting the cracks, appears reasonable.
>
> The investigation concluded that the damage observed and/or reported after the 4 September 2010 earthquake and the 26 December 2010 aftershock did not significantly weaken the structure with respect to the mode of collapse on 22 February 2011.
>
> Analyses and site observations indicate the following sequence of collapse [see also Figure 16]. The PGC building collapsed when the east and west reinforced concrete walls of the core between Level 1 and Level 2 failed during the aftershock. The west wall yielded in vertical tension, and then the east wall failed catastrophically in vertical compression. The ground floor structure stayed intact, virtually undamaged as it was significantly stronger and stiffer than the structure above. Torsional response (i.e., twisting of the building about a vertical axis) was not a significant factor. Once the west wall had failed, the horizontal deflections to the east increased markedly. The perimeter columns and/or joints between the columns and the beams, and the connections between the floor slabs and the shear core failed consequentially at some levels, causing the floors to collapse.
>
> The reason the PGC building collapsed was that the shaking experienced in the east–west direction was almost certainly several times more intense than the capacity of the structure to resist it. In addition, the connections between the floors and the shear core, and between the perimeter beams and columns, were not designed to take the distortions associated with the core collapse. Neither foundation instability nor liquefaction was found to be a factor in the collapse.
>
> Extensive studies undertaken in 1997 for a previous owner confirmed that the structure was below the current standard at that time with respect to earthquake resilience for new buildings.

A final report was released by the Expert Panel during February 2012. The conclusions were essentially the same, the above paragraph being renumbered as 6.11 but the last two paragraphs were removed and a new section, headed “Principal Findings and Recommendations” was added, within which paragraph 9.2.2 applied specifically to the PGC building:

9.2.2 PGC Building

The lack of ductility and strength inherent in the 1963 standards and the strong shaking combined to fail the eastern wall of the building's shear core. The resulting horizontal displacement of the floors led to the failure of the columns and beam-column joints, causing the floors to collapse on top of one another.

In reviewing the issues arising from the PGC Building investigation, the Panel concludes as follows:

a) Walls with centrally located and light reinforcement may be susceptible to failure when significantly overloaded. In such walls the concrete carrying compressive loads is not confined by reinforcement and will therefore behave in a brittle fashion.

b) Older buildings may lack redundancy and be vulnerable if they have only one lateral load resisting system or no alternative load path.

c) Columns and walls that are not regarded as contributing to earthquake resistance must be capable of sustaining the expected inelastic lateral displacements of the structure.

2.7.4 William T. Holmes review

The Royal Commission retained Mr Holmes to review both the Beca report and the Expert Panel report. Mr Holmes provided written advice dated 2 November 2011, which he amplified at the hearing on 6 December 2011.

Mr Holmes agreed that the failure mechanisms identified by Beca and the Expert Panel were likely to have resulted in the building's collapse, but also identified further possible weaknesses in the building that could equally have caused the failure. He summarised his views at the hearing in a series of bullet points which read:

- All agree that building collapsed due to failure of the central tower at floor 1-2
- The failure caused large movement of Level 2 downward and to the east (about 3m)
- Some girders supported by the tower pulled away and collapsed (in unknown sequence)
- Props placed behind perimeter columns as a retrofit were to provide supplemental support for the columns under excessive drifts (range of 5 cm), not meters. Exterior columns therefore collapsed (in unknown sequence). It is interesting to speculate if the "props" provided any assistance to the columns in September.

Level 1-2 had many seismic deficiencies

- Light central reinforcement. Weak in global flexure (overturning)
- Weak in EW shear (many openings, low R/F ratio, small trim bars. Piers in North Wall appear to be "shear critical"

Additional Seismic deficiencies

- Discontinuity at north end of east wall
- No confined "column" elements under floor girders
- Poor connection of girders to tower at all levels
- Displacement critical gravity columns at perimeter (retrofit props not intended to support gravity loads under very large displacements.)

Lessons for other "older" concrete buildings

- What conditions should be considered "Critical Structural Weaknesses"? Did it take a combination of the deficiencies to cause failure?
- Use of %NBS
  - Assessments of 33%-50% NBS but building was only slightly damaged in September, which, arguably, had shaking of the same order of magnitude as 100% NBS.
  - Brittle buildings of 100% NBS may be dangerous with only a small increase in shaking intensity.
    - However, it is unrealistic to evaluate buildings for very rare shaking (e.g. 2500 year return)
    - Brittle buildings examined for potential catastrophic failure modes at greater than 100% NBS?"

There was consensus among the expert witnesses that the building complied with the relevant standards at the time that it was built. We accept that is so. However, modern concepts of ductile design were then not well understood. While the design met the required strength of the time, the building was brittle beyond those limits.

2.8 Discussion

The principal issues that arise as a result of the Royal Commission's investigation, including the evidence given at the hearing, can be addressed by considering the building prior to the September earthquake, and the actions taken following the September earthquake and the aftershocks until 22 February. It will then be appropriate to address our findings in relation to the failure of the building in the February earthquake.

2.8.1 The building prior to the September earthquake

Between the time of construction and 4 September 2010 various alterations were made including the addition of steel supports behind the exterior columns to enhance the seismic performance of the building. In addition some maintenance work was undertaken to address corrosion of reinforcement. There was no legal requirement to upgrade the seismic strength of the building during this time, and the Royal Commission accepts that work undertaken did not detract from the overall strength of the building.

The Royal Commission also accepts that the building, when constructed, complied with the CCC's building by-law in force at the time when the CCC issued the building permit. We are also of the opinion that no works subsequently carried out on the building would have impaired its seismic strength.

However, it was recognised by the time of the HCG reports prepared for Warren and Mahoney in 1997 that the building would be at risk of collapse in a major earthquake. It was for that reason that the attempt was made to improve the building's ability to withstand earthquake actions by the installation in 1998 of steel props behind the columns above ground floor level. In 2007, HCG was able to revisit the issues concerning the building's seismic strength, and concluded that the building did not meet the requirements of the then current Loadings Standard. However, the building was not "earthquake-prone" under the CCC's policy adopted in 2006[8].

When the strength of the building was considered by HCG in both 1997 and 2007 it was in the context of possible development proposals. It appears that HCG's advice was not given directly to the PGC Board. However, the substance of HCG's advice was conveyed to the Board in 1997, and to PGC management in 2007. We do not consider that there was anything in the advice that should have caused PGC, acting responsibly, to have taken action beyond what was done in 1998 to strengthen the building.

The company was entitled to assume, on the basis of the advice received, that appropriate remedial action had been taken, in terms of the 1998 works, to remove weaknesses that posed life-safety issues.

By the time that application was made for consent to carry out the ground floor fit out in 2007, the CCC had adopted its 2006 buildings policy[8]. As we will discuss in more detail elsewhere in this Report, the policy was passive in nature and did not require any action to be taken in the context of the works proposed. We note in addition that HCG had in any event advised that the building was not below the threshold level of one third of current code loading at which it would have been regarded as earthquake-prone under the Building Act 2004. We heard no evidence questioning the correctness of that view, and we accept it.

When the building was purchased by Cambridge 233 Ltd in 2009, the due diligence process resulted in the issue by the CCC of the LIM, to which we have already referred. The LIM described the building, in very qualified language, as one that "may be potentially earthquake-prone". Mr McCarthy, the CCC's Environmental Policy and Approvals Manager at the time of the hearing, said in evidence that this was a standard notation applied by the CCC on land information memoranda issued with respect to all buildings built prior to 1976. The Plant & Building Services Management Ltd report, to which we have already referred, simply repeated the information about the potential status of the building set out in the LIM.

Mr Collins gave evidence that he was not aware of the advice about the building's potential status, and we have no reason to doubt that evidence. We also accept Mr Buchanan's evidence that he was not made aware by Chapman Tripp of the contents of the LIM, and that he did not advise Mr Collins of the relevant comment in the Plant & Building Services Ltd report. Mr Buchanan explained in cross-examination that he had been instructed to obtain a condition report on the building, and that he had not been asked to obtain a report on its structure.

Although at the hearing counsel assisting the Commission thoroughly tested those involved in decision making about the building prior to the September earthquake, we are satisfied that no criticism can properly be made of any action or omission on their part. Despite references to seismic weaknesses in the reports and correspondence emanating from HCG, the advice of Mr Hare at the relevant times was that the building was not earthquake-prone. We have no reason to doubt the correctness of that advice.

2.8.2 Actions taken following the September earthquake and the aftershocks up to 22 February 2011

2.8.2.1 Harcourts

Harcourts was required to manage the building for the owner. On 4 September, Mr Buchanan of Harcourts promptly requested an inspection by HCG. Soon after he had done so, Mr Collins independently confirmed that he wished the buildings in which he was interested to be checked to ascertain whether they were safe to occupy. Harcourts also requested that HCG carry out inspections on three other occasions as a result of questions raised by tenants concerned about visible cracks to the central shear walls. Harcourts did not request a further inspection as a result of every concern raised, owing to consistent advice from the engineers that the cracks that were visible were superficial.

Harcourts relied on the expertise of HCG engineers to advise whether further work or investigations were required. There were no limitations placed on time or costs. The replacement of ceiling tiles on the fourth floor was promptly arranged in order to reduce falling object hazards in aftershocks. Tenant concerns with regard to the heavy ceiling tiles on the third floor were eventually dealt with during January. Delays appear to have been the result of discussions with the insurer being prolonged by the volume of claims.

It is clear that Harcourts relied on HCG to carry out the necessary assessments and to advise whether anything observed indicated that a more detailed inspection of the building was required. We accept that the work HCG agreed to perform was effectively the carrying out of Level 2 Rapid Inspections. However, we are equally of the view that Harcourts would have expected to be told if it was HCG's opinion that a more detailed inspection was required. There was no advice to that effect. Rather, after each inspection, the advice given was, successively, to the effect that the building was "okay to occupy (structurally)", "occupiable, no immediate further investigation required" (this, by use of the standard form classifying the building as "Green G1" on 16 September), "[n]o structural issues. Building remains structurally okay to occupy" and "building remains safe to occupy". We consider that Harcourts was entitled to rely on the advice received and convey the advice to the building's tenants that the building could be safely occupied.

As previously noted, at the time of assessments of the PGC building after the September earthquake and until its collapse on 22 February, buildings in Christchurch were being checked to ensure that they were not of "diminished structural capacity" as a result of the earthquake sequence. The assumption made was that the aftershocks would generally follow a decaying sequence and that if a building was considered safe to occupy prior to 4 September and its structural strength had not been adversely affected by the earthquake, then continued occupation would be acceptable. What this assumption did not account for was the location of the building with regard to the epicentre, duration and depth of any potential aftershock.

The initial standard form green "INSPECTED" placard that was placed on the PGC building using emergency civil defence powers noted that it was the result of "a brief inspection only". It stated that while no *apparent* structural or other safety hazards had been found, a more comprehensive inspection of the exterior and interior might reveal such hazards. The form "encouraged" owners to obtain a "detailed structural engineering assessment of the building" as soon as possible. It is likely Harcourts considered that in instructing HCG it was acting prudently and in accordance with what had been recommended on the form.

In the course of questioning Mr Buchanan of Harcourts, Mr Elliott put it to him that Harcourts had placed the tenants of the building at the risk of injury or death by not requesting a full detailed structural assessment of the building. The premises of the question included the existence of the two HCG reports of 1997 and 2007, as well as the instruction by Mr Collins to obtain advice that the building was safe to occupy. There is, however, no evidence that Harcourts was aware of the HCG reports, and even if there were such evidence, it would still have been appropriate for Harcourts to rely on HCG to recommend a more detailed inspection of the building if it thought that was required on the basis of the damage observed.

2.8.2.2 HCG

Although Mr Whiteside and Mr Boys were privately instructed, and were not volunteers acting as part of the emergency civil defence response, they carried out inspections to a Level 2 standard, which is the terminology used in the New Zealand Society for Earthquake Engineering publication "Building Safety Evaluation During a State of Emergency: Guidelines for Territorial Authorities" (August 2009)[12]. Those guidelines have been endorsed by DBH. They provide for a Level 1 Rapid Assessment and a Level 2 Rapid Assessment. Table 1 (page 9) in the Guidelines states that the purpose of these inspections is to ascertain the level of structural damage to individual buildings, to assess building safety, decide on the appropriate level of occupancy and to recommend security and shoring requirements. The Guidelines state that Level 1 Rapid Assessments are based on exterior inspection only. Table 1 of the Guidelines refers to the Level 2 Rapid Assessment process as follows:

> Formal system based on inspection of interior and exterior of the building plus reference to available drawings. Calculations not envisaged. May result in revised placards posted on buildings…unsafe areas cordoned off, urgent work recommendations.

Both Mr Whiteside and Mr Boys observed cracks, including cracks in the shear walls, and both concluded that the resilience of the building had not been impaired. Both had been briefed on the inspection process that should be followed, and there is no suggestion that the standard of inspection that they undertook varied from the standard of other engineers in the city at that time. The Beca report also concluded, as set out above, that the damage observed was "relatively minor and not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking". As Mr Jury emphasised in his evidence to the Royal Commission, the inspections after the September earthquake were designed to establish whether the building's condition had seriously changed to the point that in any future shaking it might be detrimentally affected.

The observations made by Mr Whiteside and Mr Boys did not lead them to the conclusion that a more detailed assessment of the building was necessary. They appreciated that the shear core wall that failed in the February earthquake was the primary lateral load resisting element of the building's structure. They did not consider the cracks observed were significant. The evidence before the Royal Commission would not justify a finding that these conclusions were incorrect.

We do not doubt that had there been observation of damage with more serious implications they would have raised the issue with a principal of Holmes to consider, together with Harcourts and the building owner, whether a more comprehensive inspection and assessment was needed. Mr Elliott questioned Mr Whiteside about the ethical obligations of engineers to take reasonable steps to safeguard the health and safety of people in the course of their activities as engineers. The suggestion was that he might have been ethically obliged to recommend a more detailed inspection be carried out. We should record our view that this is not a case where there was any ethical shortcoming or failure to meet professional standards.

However, this was not a building designed with ductile detailing, and it is characteristic of brittle buildings that they may give little evidence of structural damage prior to collapse. In the circumstances, reliance only on visual inspection of such buildings after a major earthquake may be problematic, and the issue of how such buildings should be assessed after a significant earthquake is a subject to which we will return in another part of this Report.

It should also be noted that there are inherent limitations in the damage-based assessment approach in cases where a building has critical structural weaknesses. Particularly where the building is also brittle, surviving one earthquake may not mean surviving another of similar or greater intensity. This is another issue to which we will return in another part of the Report, dealing with building assessments after earthquakes.

We noted further that we are satisfied from the evidence we heard in this and other cases that there is a mismatch between the engineering profession's understanding of the rapid assessment process and that of the clients for whom the assessments are made. For the former, the limitations are well understood and there are strong practical considerations that dictate that in many situations there will be a need for the rapid assessment process to be all that is carried out. However, the phrases "ok to occupy" or "safe to occupy" are likely to convey the meaning to those without engineering knowledge that the building is safe, when in fact all that is intended to be conveyed is that the building does not appear to have been weakened as a result of the earthquake that prompted the assessment. We have encountered a number of cases where this difference was not appreciated by the occupants of buildings, and we consider that it was so in this case too.

2.8.3 Why the building failed

The analysis of any building in the Christchurch CBD is fraught with difficulties owing to uncertainties that exist with regard to the seismic actions at a particular site. Specific uncertainties arise from the lack of knowledge of the actual forces imposed on the building. From bore holes on the site there was no evidence of liquefaction under the building, so this is not considered in the evaluation, although it is acknowledged that assumed ground stiffness may have affected the response of the building. The actual seismic accelerations and displacements on the site are assumed from measuring sites that are a minimum of 670m away. There is no way to know with any great accuracy the actual loadings that were placed on the building.

The Commissioners raised a number of questions concerning the failure mechanism described in the Beca report and further expanded on by Mr Jury and Dr Sharpe during the hearing. Several of these questions were also addressed in the evidence of Professor Priestley and by Mr Holmes. Many of them had been raised in advance of the hearing. The questions and answers are summarised below. We record that at the hearing, Mr Jury and Dr. Sharpe were affirmed and gave evidence together. They were followed by Professor Priestley and Mr Holmes. All four witnesses then participated in a panel discussion.

The Royal Commission questioned why, in the Beca analyses, wall stiffness values had been taken as 0.4 of the stiffness values calculated from the gross section properties. Mr Jury responded that this was a generally accepted value, which was adopted to allow for flexural cracking. Commissioner Fenwick asked whether this was realistic given the apparently very limited crack formation away from the critical section at level 1. Mr Jury expressed the view that it did not appear to significantly affect the predictions obtained in the analyses. In response to a further question, Mr Jury agreed that the low wall stiffness assumed to apply above level 1 could have led to an underestimate of the inelastic deformation induced in the wall close to the critical section at level 1.

Questions were also posed about the significance of the offset in the eastern shear core wall in bay b-c. This offset, which is shown in Figure 10, page 21, was not mentioned in the Beca report. Mr Jury was asked whether this offset could have had any significant influence on the seismic performance of the building. He responded that this offset would cause stress concentrations to occur at or close to grid lines b and c at each end of the offset wall. When asked if the combined shear and compression stress in the wall at these locations could have initiated failure in the concrete, Mr Jury's response was that the analysis was not able to predict shear stresses in this location. He agreed that when the drawings of the building were considered this could be a critical weakness, which might have been a fatal weakness in the structure. In subsequent evidence both Professor Priestley and Mr Holmes stated that in their opinion the offset in the wall was a potential critical weakness that could have initiated failure.

A number of questions were posed about the cracking in the shear core walls. Mr Jury agreed that the critical section for the shear core wall was at level 1. The structural drawings showed that the walls had a thickness of 203mm and were reinforced with 16mm bars spaced at 380mm centres. Tension force that can be transmitted across a crack is limited by the strength of the reinforcement. Mr Jury agreed it was unlikely that sufficient tension could have been transmitted to initiate a secondary crack in the concrete. Commissioner Fenwick noted that the tensile force that could be resisted by the reinforcement could only induce tensile stresses in the concrete of the order of one half to one third of the expected direct tensile strength of the concrete.

In the finite element model a fibre length of 400mm was assumed for the reinforcement between points where it was coupled to the concrete. With this assumption the displacement of reinforcement crossing a crack would induce uniform strains in a length of 400mm. Given the usual assumption of linearly varying strain over the plastic region this implies a plastic hinge length of 800mm. Mr Jury agreed with this but noted that when this assumption was tested a smaller length did not appear to make a difference to the analytical predictions. Commissioner Fenwick pointed out that in the Beca report it was indicated that yielding could have been limited to a length of about six bar diameters, giving a length of about 80mm, which is an order of magnitude lower than that assumed in the analysis. The question was whether this would have had a significant influence on the predicted behaviour.

Mr Jury responded that in testing, this fibre length was not found to have a significant effect because two thirds of the flexural strength came from the axial load acting on the walls. As a result of further questioning it became clear that the inelastic model of the walls could not predict actual crack widths and hence it was unable to predict when the crack width reached a few millimetres in width owing to either the bars yielding in tension or "more likely" their failure in direct tension. The Royal Commission notes that when crack widths of the order of a few millimetres are sustained, shear transfer across the crack by aggregate interlock action is lost and this results in a major loss of torsional resistance at this section.

As a result of answers to further questions it was clear that the analytical model could not predict either the loss of torsional resistance provided by the concrete, which was due to the opening up of the crack or the loss of torsional resistance provided by the reinforcement when the longitudinal reinforcement yielded in tension owing to flexural actions. For this reason the Royal Commission does not agree that the effective plastic hinge had no significant influence on the seismic behaviour of the building. We note that once a crack of the order of a few millimetres in width had formed in the eastern wall the torsional resistance contribution of both the eastern and western walls would have been lost, leaving only the transverse walls to resist any torsional moment. This is because the centre of resistance would have moved close to the western wall.

With this centre of rotation, torsion induces in-plane displacements and shear forces in the transverse walls, but the eastern wall twists out of plane and cannot significantly contribute to the torsional resistance. The loss in torsional resistance provided by the eastern and western walls results in a major loss in the strength of the structure as a whole.

The high shear forces induced in a transverse wall may result either in shear failure of the wall or in high shear stresses in the compression zone of the wall. As the high shear stresses act in and close to the intersection of the transverse wall and the eastern wall, the high lateral force may induce a local punching-type shear failure, which could lead to the collapse of the shear core. Professor Priestley referred to this failure mechanism in his evidence. Either of these mechanisms could result in collapse of the structure.

One of the conclusions of the Beca report was that the eastern shear core wall failed by crushing at level 1 as the core rocked over towards the east. Commissioner Fenwick asked questions about the shear stress levels induced in the transverse walls associated with their postulated failure mechanism. Interest in this aspect arose as the HCG analysis made in 1997, under seismic actions that were much smaller than those investigated by Beca, had predicted that diagonal cracking could be expected to occur in the transverse walls. No such cracking was predicted by Beca.

Mr Jury and Dr. Sharpe were asked to comment on the results of a conservative approximate hand calculation that indicated high shear stress levels would have been sustained in the transverse wall if the failure mechanism postulated by Beca had occurred. The basis of the hand calculation was as follows.

With reference to Figure 11, page 22, if a crack forms at level 1 the reinforcement at this location can sustain a force that is close to 2500kN. The beams on grid lines b, c, d and e apply gravity loads to both the eastern and western shear core walls. If the gravity load of the wall is included, these forces are of the order of 1250kN at each level on each wall. An assessment based on the locations of the walls and floor beams indicates that up to half the total forces applied to the western shear core wall would be likely to induce shear in the transverse wall W2. This would induce an average shear stress in the concrete above the doorways in excess of 3MPa. This and its associated bending moment could not be sustained by the wall as detailed. On this basis W2 could be expected to fail in a flexural shear mode.

Mr Jury was asked if he agreed with this assessment and replied that the Beca analysis gave a figure of 1.5MPa maximum shear stress. Despite subsequent communication with Mr Jury, the discrepancy in values has not been explained to our satisfaction. Professor Priestley subsequently suggested that the difference might be explained by redistribution of the shear forces in W2 to W1. Mr Holmes stated that a distribution of shear to W1 would have caused it to fail in shear, as in his assessment this wall was more critical in terms of shear strength than W2. Both Professor Priestley and Mr Holmes agreed that shear failure of the transverse walls was a possible failure mechanism.

Issues were raised about the influence of vertical ground motion on the performance of the building. In answer to questions about the representation of the soil in the Beca analytical model Mr Jury indicated that it was represented by elastic springs that disconnected (gapped) when subjected to tension. When asked about possible compaction of the soil in the repeated earthquakes he responded that they found changing the stiffness of the springs did not significantly affect the predicted performance of the building. Mr Jury agreed that changing the spring stiffness did not fully allow for possible compaction of the soil, which might have increased ground stiffness. However, inspection of the site did not indicate that any compaction had occurred in the foundation soils.

Professor Priestley raised a number of other issues that have not been discussed so far. We refer to three of these. First, it was his opinion that the PGC building lacked ductility and consequently there would have been little evidence of damage before the collapse state was reached. This has important consequences for the assessment of similar buildings after an earthquake. One particular point is that a small crack may be evident but owing to its small width it might be assumed not to have caused a significant loss in seismic performance. However, in a lightly reinforced structural wall, which was not designed for ductility, the reinforcement crossing the crack might have either extensively yielded or completely failed at the crack. After the earthquake, the crack, which might have opened to an appreciable width during the earthquake, might close owing to the gravity-induced axial load. This indicates that the visual inspection procedures after an earthquake for buildings such as the PGC building need to be reviewed. This should involve identifying buildings that are not ductile and using different criteria in their assessment from those for more modern ductile buildings.

Both Professor Priestley and Mr Holmes discussed the use of the capacity spectrum approach for assessing the potential failure of a building. This approach is briefly outlined in the "Introduction to seismic design of buildings" section in Volume 1 of this Report. In this approach the displacement spectrum is modified to allow for hysteretic damping and the fundamental period is based on the secant stiffness of the structure.

Finally, Professor Priestley suggested that bi-axial attack could have caused the compression zone to move towards a corner of the shear core. In such an event the reduced size of the compression zone and the increase in compression stresses could have caused a compression failure, leading to collapse of the building.

Three additional observations were made by Mr Holmes. First, he commented on the design of the support zone for the beams on the eastern and western shear core walls. Given the depth of the beams, about 380mm, and the thickness of the walls, 203mm (which supported the end of the beam), it is clear that the beams were not effectively anchored to the walls. In addition, only a small portion of the reinforcement in the beams was anchored into the walls. Because of the small thickness of the walls, anchorage of the bars would not have been fully effective. Furthermore, there was no additional reinforcement placed below the beam support zones. In order to tie the beam effectively into the wall, pilasters should have been used. This would have increased the robustness of the structural system. If the beams had been more effectively tied into the walls they might not have separated from them when collapse occurred. This could have resulted in a tepee shape forming, preventing the pancake-type collapse that occurred, thereby reducing the loss of life in the collapse.

Second, Mr Holmes's assessment of the drawings was that shear failure in the transverse walls was a likely cause of collapse, as the reinforcement did not appear to be adequate to suppress this mode of failure. He also noted that the transverse wall W1, which was at the northern end of the shear core, looked particularly brittle.

Third, Mr Holmes commented on the use of percentage of NBS as a measure of the potential seismic performance of buildings. He noted that assessed NBS values of the PGC building ranged from 35–60 per cent, but in fact the building had survived the September earthquake with minimal damage and this event was comparable to a design-level earthquake. On this basis perhaps it should have been assigned a rating of 100 per cent NBS. However, it should be pointed out that even a building with a rating of 100 per cent NBS can present a seismic hazard if it is of a non-ductile design.

2.9 Conclusions

The Royal Commission draws the conclusions given below from the investigation into the collapse for the PGC building.

2.9.1 Critical structural weaknesses

The building contained a number of critical structural weaknesses, which we list as follows:

1. The offset in the shear core wall at level 1, on grid line E and between bays b and c (as shown in Figures 9 and 10) resulted in local stress concentrations at the ends of the offset.
2. The vertical reinforcement content in the shear core walls was too low to initiate secondary cracks. This led to yielding of reinforcement being confined to a short length resulting in a single wide crack in the potential plastic region at level 1. The width of the crack induced in the west shear core wall necessary to accommodate the inelastic seismic displacement would have destroyed the capacity for shear to be transferred across the crack by aggregate interlock action. This would have led to a major decrease in torsional resistance and an increase in the lateral forces acting on the transverse walls. It is likely that the induced crack width was of sufficient magnitude to fail the reinforcement in tension, enabling the shear core to rock about the west wall.
3. The eccentric location of the shear core in the building greatly increased the torsional action applied to the shear core, which weakened the building's seismic performance.

4. The beams that were supported by the shear core walls were ineffectively tied into the walls. Pilasters should have been provided to enable the beam reinforcement to be effectively tied into the wall and to prevent localised flexural actions being induced in the walls.
5. The perimeter columns and associated beam column joints were inadequately confined to enable them to sustain significant inter-storey drift without failure. This shortcoming was partially overcome by the retrofit carried out in 1998, when rectangular steel props were attached to the columns to enable them to sustain axial loads in the event of an inter-storey drift of a few centimetres.
6. The building, designed in the 1960s, was based on the approach to seismic design current at that time. This was before the period when the importance of ductile behaviour was understood. Consequently the building did not contain ductile detailing that is a feature of more modern structures. A feature of non-ductile buildings is that they give little indication of structural damage prior to collapse, which is not the case with ductile buildings. This poses a major problem in assessing the seismic performance of non-ductile structures, such as the PGC building, by visual inspection after an earthquake that is large enough to damage a structure but not cause its collapse. Further guidance is required on how such assessments should be made for this class of structures for use in future earthquakes. We will address this issue in a subsequent part of this Report, which will deal with the assessment of buildings following earthquakes.

2.9.2 Analysing collapse mechanisms

The analysis of a building to determine its collapse mechanism is a difficult process. Of the different analytical techniques that are available, the inelastic time history method potentially gives the most accurate predictions. However, in the use of this approach it is important to be aware of aspects that may not be adequately treated in the analysis package. These are likely to include:

1. The location of wide individual cracks and the implications of these wide cracks on reinforcement strains and shear transfer across the cracks.
2. The significance of loss of shear transfer across cracks on shear and torsional strengths.
3. The significance of flexural torsional interaction, which causes torsion resisted by reinforcement to reduce when the longitudinal reinforcement yields owing to imposed bending moments.
4. The significance of localised forces in structural elements, such as the concentration of shear stresses in beams or walls in the compression zone when either the flexural tension reinforcement yields, or alternatively when the wall is subjected to axial load and the flexural tension reinforcement fails in tension.

2.9.3 Collapse mechanisms

There are a number of different failure mechanisms that individually or in combination may have caused the building to collapse in the February 2011 earthquake. They are:

1. Bi-axial attack could have induced high axial compression stresses in the corners of the shear core, potentially leading to compression failure of the walls. The north-eastern corner of the PGC building is particularly sensitive to such actions owing to the ineffective support of the eastern wall in bay b-c associated with the offset in the wall at level 1 at this location.
2. The transverse walls were inadequately reinforced to sustain high shear forces. It is likely that the additional shear forces applied to these walls, owing to the formation of the wide crack in the eastern wall and the associated loss of torsional resistance provided by the eastern and western walls, would have caused the transverse walls to fail in a shear or flexural shear mode.
3. If the vertical reinforcement in the western wall failed in tension at the crack at level 1, the shear force in the transverse walls would have been resisted in their compression zones. The high lateral force in these zones would have been applied as a concentrated force directly to the western wall at the junction with the transverse wall. The shear force from one or more of the transverse walls could have caused a local punching-type failure of the eastern wall, which would have initiated collapse of the shear core and of the building.
4. It is possible that the failure occurred as a result of a compression failure of the eastern wall due to axial load and flexure about the weak axis of the shear core, as suggested by Beca.

The Royal Commission concludes from the evidence of witnesses to the collapse, and from the analyses by experts, that failure of the eastern wall (see Figure 9) initiated the collapse. It was important to consider a variety of collapse scenarios in order to record the relevance of different seismic actions and how these might have initiated the collapse. Such possibilities may be relevant in future collapse studies and in the design of new structures.

References

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Section 3:
Hotel Grand Chancellor

During the February earthquake, a shear wall located on the ground floor of the Hotel Grand Chancellor (HGC) at 161 Cashel Street failed. The failure came close to causing a catastrophic collapse of the building.

Figure 17: A view from the south after the February earthquake; the podium is on the left side

Figure 18: A view from the north-east after the earthquake (source: Ross Becker Photography)

At the time of the 22 February 2011 earthquake there were about 50 staff members in the building and an unknown number of guests. Stair flights on one side of the scissor stairs in the tower collapsed, also collapsing the stairs in the upper section of the car park levels. About 30 people were trapped on the upper levels. Fortunately, there were two maintenance staff members with tools on level 26, who were able to force open doors that had become jammed and allow people down the stairs that remained and out on to the roof of the podium. They were lowered by crane down from there.

The following discussion covers:

- the history of the HGC building prior to the September earthquake;
- the September earthquake and Boxing Day aftershock, the performance of the building in these earthquakes, and the actions taken as a result;
- the February earthquake and the failure of the building; and
- lessons that the Royal Commission considers should be learned from this failure.

It reflects information gathered from a variety of sources, including:

- the Christchurch City Council (CCC) as the regulatory authority administering building controls in Christchurch;
- Grand Central (NZ) Ltd, the building owner on 22 February 2011;
- the Dunning Thornton Consultants Ltd investigation into the failure for the Department of Building and Housing (DBH); (the Dunning Thornton report)[1];
- the DBH Expert Panel review of the Dunning Thornton investigation (the Expert Panel report)[2];
- a review of the Dunning Thornton and Expert Panel reports carried out on behalf of the Royal Commission by Mr William T. Holmes; and
- evidence given and submissions made to the Royal Commission at a public hearing held on 17 and 18 January 2012.

Figure 19: Failed wall on ground floor

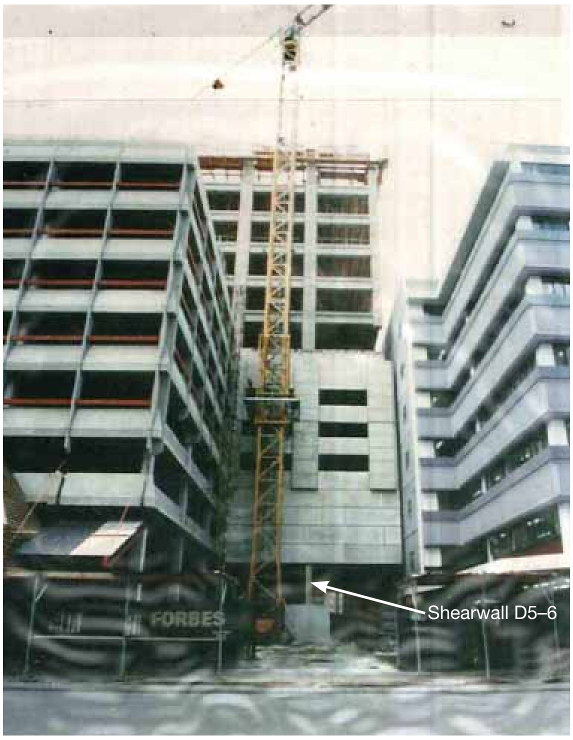


Figure 20: Photograph taken during original construction, looking from the south

3.1 Original construction of the HGC building

The building was originally designed as a car park building with an office tower above. There were 12 levels of car park decks (each being a half-floor) with 15 office floors above that. In effect it was a 21-storey building. To the front of the tower facing Cashel Street was a podium that consisted of parking up to level 12, with a conference room on top of that. There was no level 13, which meant that when the evacuees exited the tower from level 15 they were on the roof of the conference room.

The plan dimension of the tower was about 33m by 24m, with the podium being about 17m by 12m.

Foundations consisted of large pile caps and rafts supported on multiple driven bulb (Franki) piles. The depths of the piles varied from 5m to 13m. Above-ground structural elements were of reinforced concrete.

The ground floor to level 14 consisted of cast-in-situ flat slab concrete floors, with cast-in-situ reinforced concrete cantilever shear walls. The shear walls were not coupled and were arranged irregularly in the plan, accentuated by a right of way set back to accommodate right of access along Tattersalls Lane on the eastern side of the building. The wall that failed was at the ground level on grid line D, between grid lines 5 and 6 (wall D5–6). The failed wall can be seen in Figure 20, the view from Cashel Street when the HGC building was being constructed, and in Figure 21, the ground floor plan view showing the location.

The eastern bay (see elevation and photograph in Figure 22) was supported by an unusual structural arrangement consisting of deep transfer beams (see Figures 23–25), cantilevered over the right of way between levels 12 and 14 to support a series of tension hangers. The tension hangers can also be seen in Figure 24. The hangers, in turn, supported a long deep transfer beam along the eastern boundary above the first floor. Interspaced with the hangers were column struts supported by the long beam which, together with the hangers, supported the perimeter beams on the eastern boundary side of the tower (grid line E).

Of note are the deep cantilever transfer beams that lay on grid lines 5 and 6. These beams, which were part of the eastern bay hanging system, were both supported at the fulcrum of their cantilevers by the critical wall D5–6. The transfer beams were each a full floor in height and were tied into the concrete floor diaphragms at levels 12 and 14.

At level 14 a vertical irregularity occurred as the shear walls stopped and, from levels 14 to 28, seismic resistance was provided by ductile moment resisting frames on the perimeter to the north, west and south and offset by one grid on the eastern side. These upper floors were constructed using a proprietary precast prestressed rib and timber infill system with in situ topping. This flooring was supported on the seismic frames and on additional frames (beams and columns) not specifically designed as primary seismic-resisting elements.

In the upper structure, the eastern-most bay between grid lines D and E was cantilevered off the rest of the structure over Tattersalls Lane at each floor level. This cantilever can be seen in Figure 25.

There was a vertical separation at level 14 along the eastern boundary line (grid line E). This meant that the vertical loads accumulating along grid line E were not transferred directly down on to the system that existed in the lower structure along that grid line. However, the loads from the eastern bay, between grid lines D and E, did find a load path to wall D5–6 via the upper columns on grid line D. In particular, the columns at grid lines D5 and D6 were supported directly on wall D5–6.

The seismic frame lay on grid lines A, D, 5 and 11 (see Figure 28 on page 69). The internal columns of seismic frames do not typically carry additional axial (vertical) loads induced by seismic actions, but the end columns of seismic frames can attract large seismic axial loads in addition to their normal gravity loads. Column D5 was an end column for the frames on both grid lines D and 5, which meant that it could attract seismic-induced axial load from both axes. These loads fed directly onto the critical wall, D5–6.

Overall, the structure of the building was complex, with irregularities both horizontally and vertically.

The original building was approved by the issue of a series of building permits, all of which were issued to Don Forbes Construction Ltd. The architects were Architecture Warren and Mahoney Ltd, and the owner of the building was Cashel St Parking Building Ltd. The original engineering design was carried out by Holmes Wood Poole & Johnstone Ltd. Holmes Consulting Group was responsible for the later parts of the design.

The engineering design work occurred over an extended period, from 1985 to 1987, during which time there were several design changes relating to land and site usage. During the early construction of the building the original design had to be amended to remove structure from Tattersalls Lane. The developer had attempted to secure title, or rights, over the use of Tattersalls Lane but was prevented by legal action after the original design was completed and construction was under way. Consequently, the engineers were required to redesign parts of the structure in order to relocate the wall that was initially at E5–6 to the west of Tattersalls Lane so that it was at D5–6. This required additional structure, including transfer beams to cantilever the eastern bay between grid lines D and E over Tattersalls Lane.

The approval process can be traced through the building permits (the dates given are the dates of applications for the permits):

Approved under Christchurch City Council Building By-Law 105 (1979) (this bylaw applied until 30 November 1985):

- 10 September 1985 – Piles for car park building – Council reference 85/2412; and
- 29 October 1985 – Pile caps – Council reference 85/3043.

Approved under Bylaw 105 (1985) (this bylaw applied from 1 December 1985):

- 25 July 1986 – Piles for a multi-storey building – Council reference 86/1765;
- 20 August 1986 – Erect a retail and car parking development – Council reference 86/3690; and
- 28 August 1986 – Car park building foundation – Council reference 86/2284.

At this point the design was changed to accommodate the cantilever over Tattersalls Lane:

- 21 January 1987 – Stage 8, extend car park over ROW - Council reference 86/3689; and
- 20 July 1987 – Stage 8 additions including structural cantilever – Council reference 87/1323.

The name of the engineering firm changed to Holmes Consulting Group from here on:

- 23 December 1987 – Erect office tower – Council reference 87/1727; and
- 24 August 1988 – Construction Stage II (Architect finish and structure details) – Council reference 88/3328.

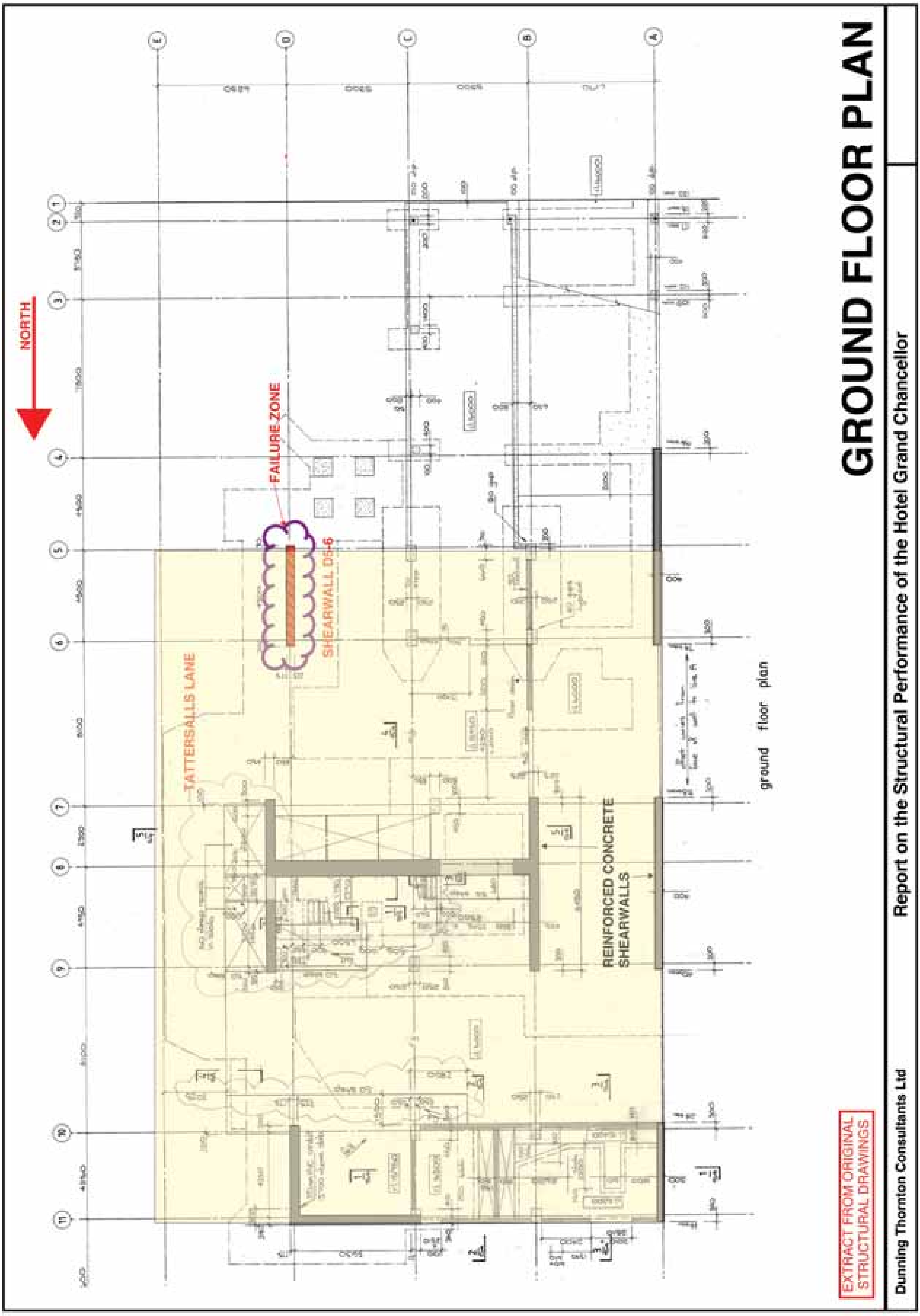


Figure 21: Ground floor plan (source: Dunning Thornton report)

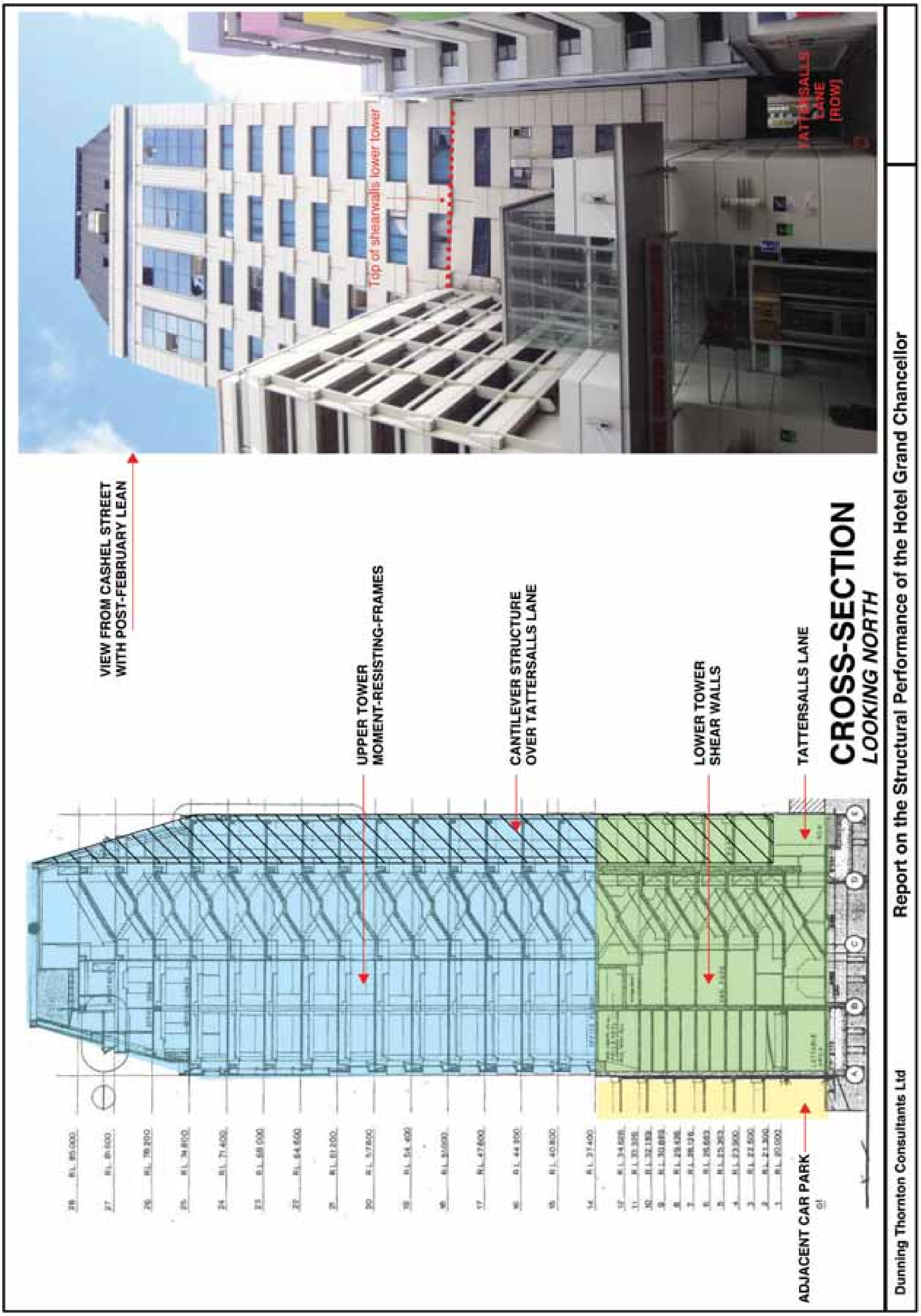


Figure 22: Cross-section looking north and view from Cashel Street (source: Dunning Thornton report)

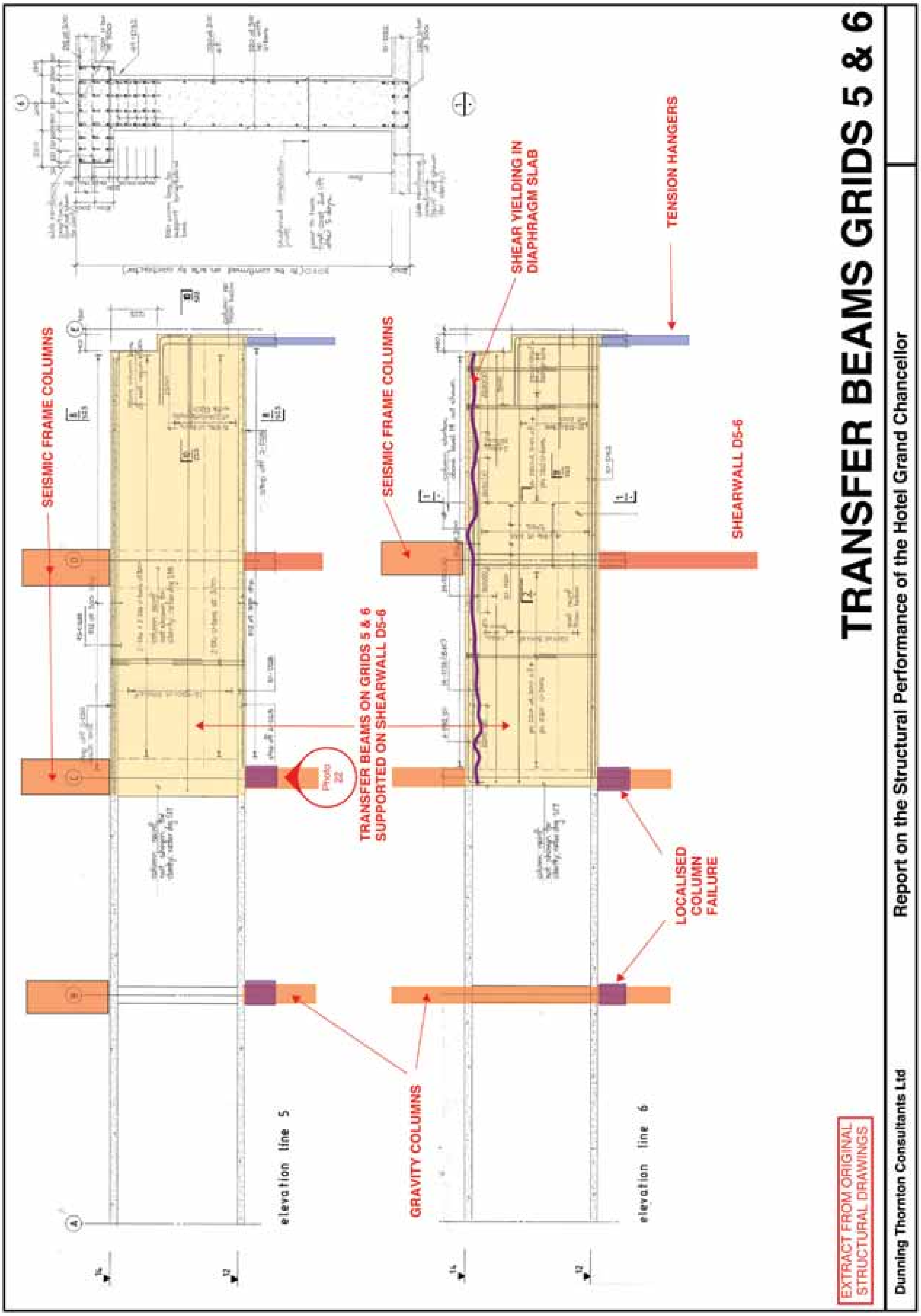


Figure 23: Transfer beams grid lines 5 and 6 (source: Dunning Thornton report)

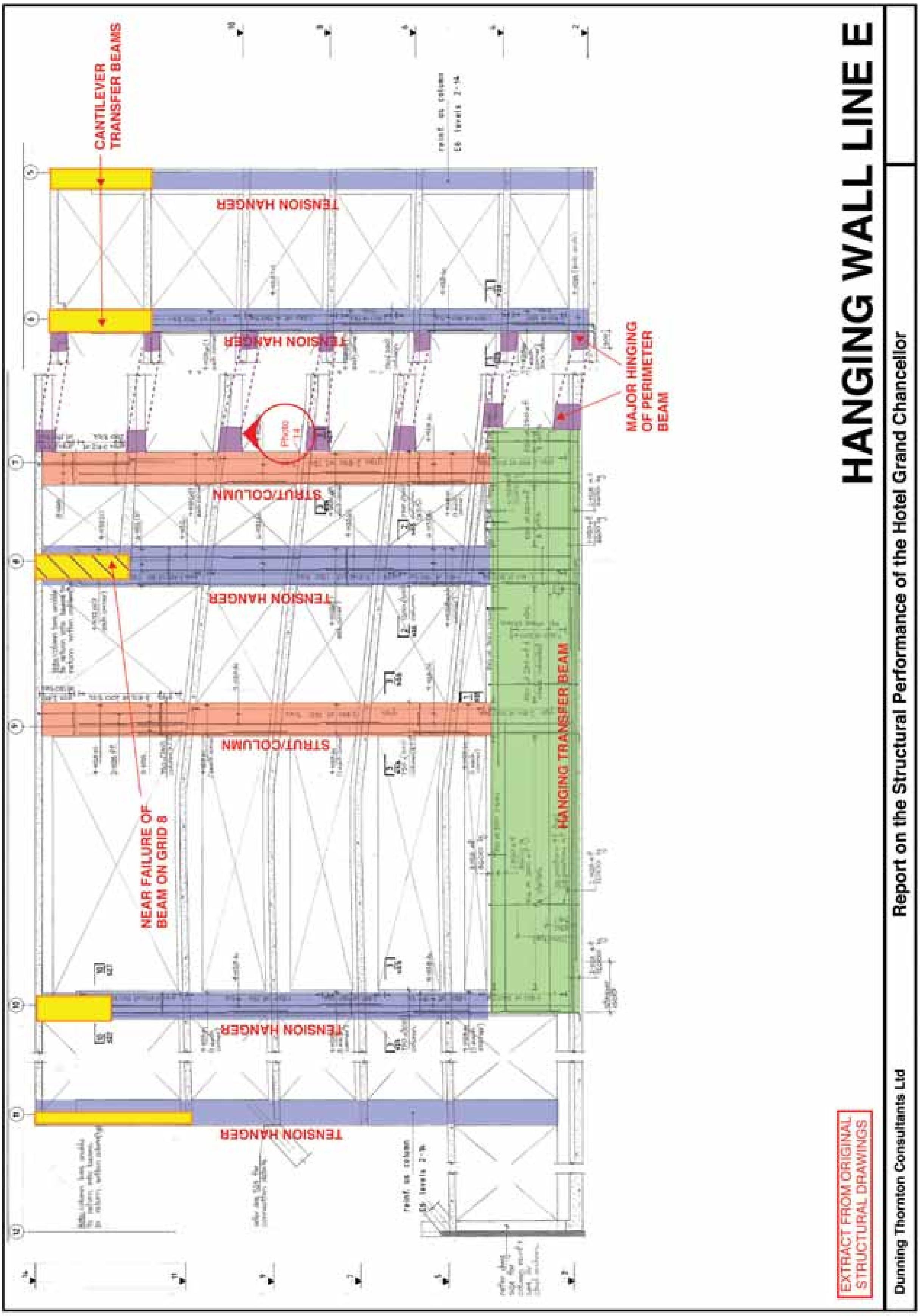


Figure 24: Hanging wall grid line E (source: Dunning Thornton report)

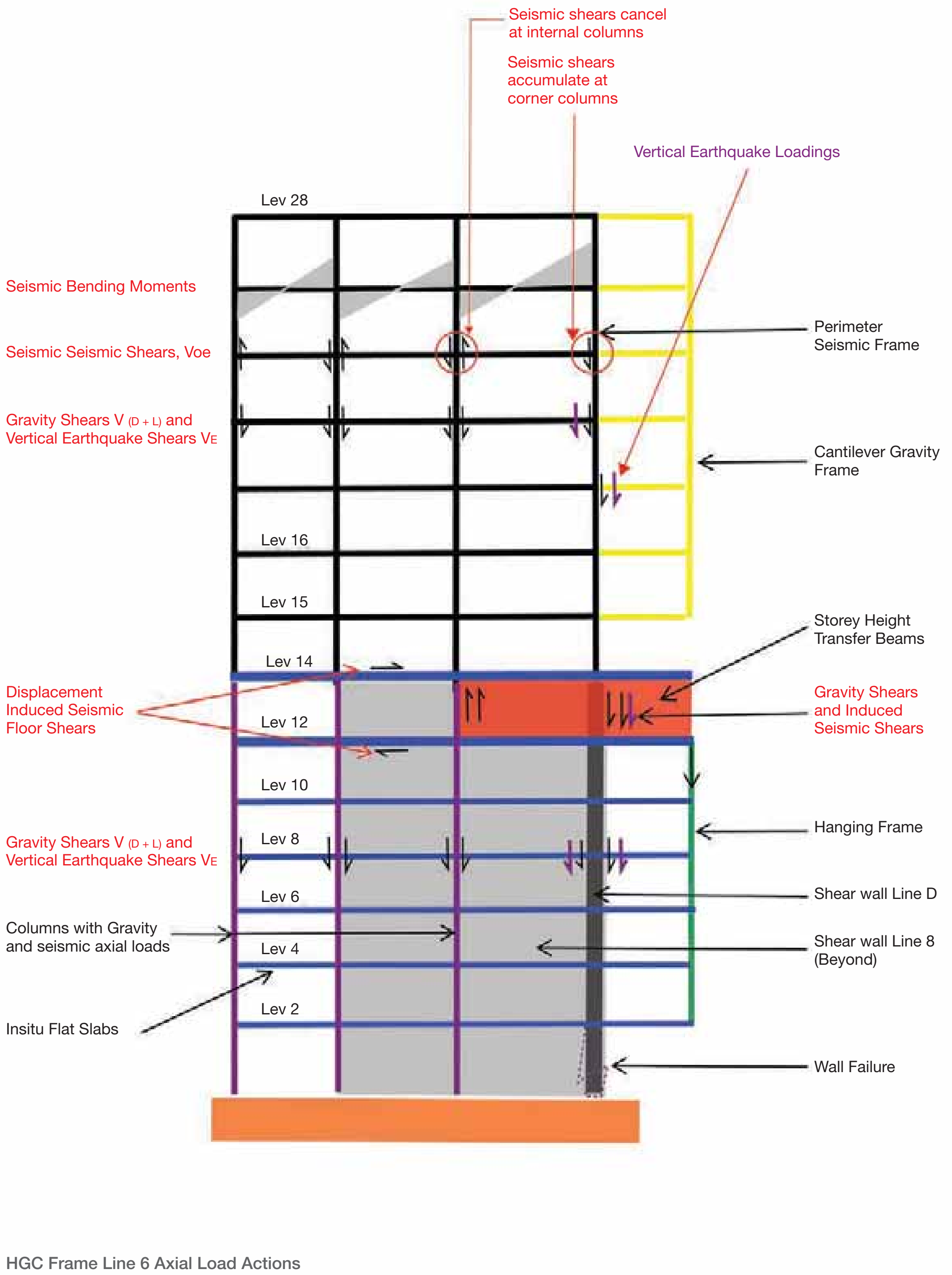


Figure 25: Dunning Thornton interpretation of axial load actions (source: Dunning Thornton report)
(Note that the Royal Commission does not necessarily agree with this interpretation)

3.2 Up until 4 September 2010

On 17 December 1992 a building consent application for the work necessary to change the use of the building from offices to a hotel and casino was submitted to the CCC (Council reference 9210270). This application was approved on 23 December 1992.

It is not clear from the CCC's records whether it turned its mind to the requirements of section 46(2) of the Building Act 1991, under which a change of use required the CCC to be satisfied that, among other things, in its new use the building would comply with the provisions of the Building Code for structural behaviour. However, design loading requirements in late 1992 have been assessed by the Royal Commission as being lower than at the time of the original design, and in the circumstances it would have been reasonable for the CCC not to require a further structural assessment to justify the change in use.

Over the period from 1993 to 1995, applications for building consents (in five stages) were submitted for the hotel fit out. A casino licence was never granted. The work approved, however, included some strengthening of the floor of the podium at level 14 in order to enable this floor to be used as a conference facility. These alterations would not have altered the overall seismic characteristics of the building. A final code compliance certificate for this work was issued on 22 October 1998 (Council reference 93012531). The building owner at this time was Grand Central (NZ) Ltd.

Other building and resource consents were subsequently issued by the CCC, but since they were for work that had no relevance to the structural performance of the building they are not discussed.

3.3 The earthquakes

3.3.1 The September earthquake

The nature and intensity of the September earthquake are described in section 2 of Volume 1 of this Report. The HGC building was located similar distances from all four primary seismic measuring stations for the Christchurch CBD, as can be seen in Figure 26. The Dunning Thornton report prepared for DBH used an average of the measurements from these four stations. The Royal Commission accepts that the measurements used are acceptable for the purpose of analysis.

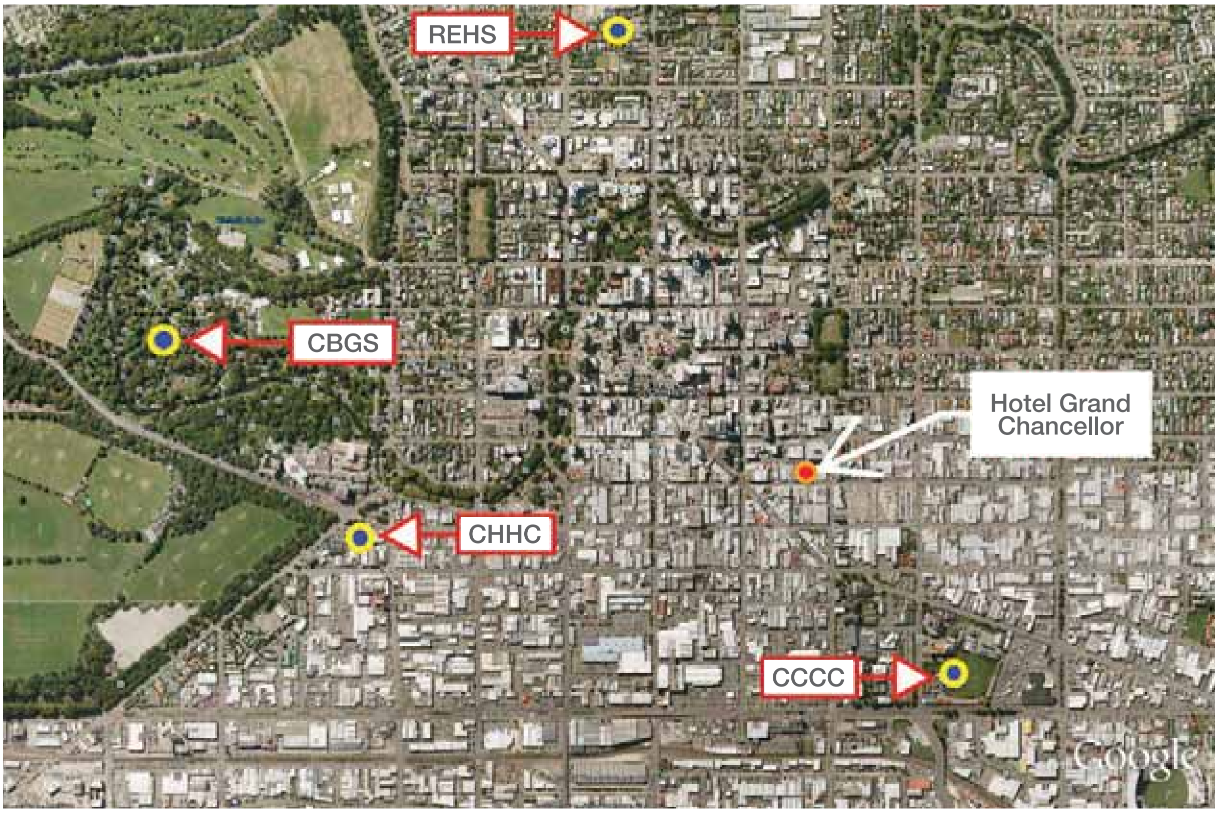


Figure 26: HGC building location in relation to the four seismic measuring stations

3.3.2 Between the September earthquakes and the Boxing Day aftershock

As discussed in a later volume of this Report, after the September earthquake a state of local emergency was declared and the CCC (under guidance from DBH) initiated a civil defence emergency management response. Starting on the day after the earthquake, teams were sent to all commercial areas of the CBD to undertake a Level 1 Rapid Assessment. These teams included at least one CCC officer, who was usually accompanied by a Chartered Professional Engineer (CPEng).

A Level 1 Rapid Assessment is an exterior inspection to look for obvious signs of damage that indicate immediate danger, or to identify that further investigations are required before use. A Level 2 Rapid Assessment is a more extensive visual inspection that includes the interior of the building.

On 5 September, both Level 1 and Level 2 assessments were carried out. Both resulted in a green "Inspected" placard, which placed no restriction on the occupancy or use, but encouraged owners to obtain a detailed structural assessment of the building.

The Level 2 assessment was carried out by Mr Gary Haverland, a senior structural engineer and director of Structex Ltd with over 24 years' experience. His inspection was at the request of Mr Stephen Martin, the hotel's general manager. Mr Haverland primarily identified cracked GIB plasterboard linings, tearing of floor coverings at the base of the stairs and flashing damage at the seismic joint between the HGC building and the neighbouring car park building. Some removal of linings was undertaken to inspect the stair support. All damage inspected was identified as superficial and not of structural concern.

Mr Martin employed Powell Fenwick Consultants Ltd to carry out a further structural inspection, which took place on 23 September 2010. This inspection was carried out by Mr Andrew Lind, a senior structural engineer with 18 years of experience. He was aware that buildings of this type (taller buildings with a longer initial period) have been subjected to higher than design loadings in the earthquake. Mr Lind was not able to see the key structural drawings and the critical nature of wall D5–6 was not obvious to him. As wall D5–6 was fully lined, the concrete could not be inspected for cracks. In evidence to the Royal Commission, he said he considered his inspection to have been more extensive than a Level 2 assessment, in that he observed beam column joints, and removed linings where they were damaged in order to investigate the underlying structure. He identified some hairline cracks, but otherwise the damage he observed was similar to that already noted by Mr Haverland. Mr Lind did not have any concern for the structural stability or strength of the building.

Mr Lind was, however, concerned about some of the stairs, having noted spalling at the landings from which some flights descended. He thought this was due to the stair units not sliding sufficiently at the base of the flight during the earthquake. At his direction the floor linings at every level were lifted to confirm the extent of the damage and a concrete patch repair was undertaken where necessary. The work was evidently carried out to his satisfaction.

The liftshaft was inspected at some point after this inspection but the actual date is not recorded.

A follow-up inspection of the beam-column joint in the conference room was undertaken by Mr Lind on 1 October 2010. There was no damage to the primary structure observed.

3.3.3 The Boxing Day aftershock

After the Boxing Day aftershock, Mr Lind carried out a further inspection at the request of Mr Martin, who was concerned about additional damage to the seismic joint between the hotel building and the adjoining car park building. There was also movement in the air conditioning and sprinkler pipes, with one of the sprinkler pipes bursting open. Fletcher Construction Ltd contractors were on site at the time carrying out repairs. They walked around with Mr Lind and removed linings where requested. In Mr Lind's opinion, there was no additional structural damage to the building.

On 1 February 2011, Goleman Exterior Building Care (a division of Goleman Co. Ltd) presented a report on the exterior damage to the building after an inspection carried out by industrial abseilers. This was addressed to Fletcher Construction Ltd. It was unclear from the evidence given to the Royal Commission whether this report was considered by an engineer before the February earthquake.

3.3.4 The February earthquake

The nature and intensity of the February earthquake are described in section 2 of this Volume.

The principal direction of the shaking was east–west, with a significant vertical component. The earthquake was of a short duration but had high accelerations and displacements because its epicentre was close to the CBD.

When the earthquake struck the HGC building was evidently subjected to strong east–west accelerations, resulting in failure of the ground floor D5–6 shear wall and the collapse of many of the stair flights. The reasons for the failure and the likely sequence of events are addressed later in this Report.

The Royal Commission has been assisted in its understanding of the failure of the building by:

- the Dunning Thornton report[1];
- the Expert Panel report[2]; and
- the review of both reports by Mr Holmes from Rutherford and Chekene prepared at the request of the Royal Commission.

3.4. Investigations

3.4.1 Investigation by Dunning Thornton Consultants Ltd

The findings of the Dunning Thornton investigation were presented to the Royal Commission at the hearing on 17 January 2011 by Mr Adam Thornton, structural engineer and author of the Dunning Thornton report. The executive summary stated:

> In the short but violent Lyttelton aftershock of 22 February 2011, the Christchurch Hotel Grand Chancellor building suffered major structural damage. The extent of damage suffered by the building was significantly increased by the collapse of a key supporting shear wall, which failed in a brittle manner.
>
> The building survived the 4 September 2010 earthquake and the 26 December 2010 aftershock events without apparent significant structural damage and was fully in use when the February event occurred. During the approximate 12 seconds of intense shaking that occurred at 12:51pm on 22 February, the building suffered a major structural failure with the brittle rupture of a shear wall in the south-east corner of the building. This shear wall had supported vertically approximately one eighth of building's mass and was also expected to carry a portion of lateral earthquake loads.
>
> As a result of the wall failure, the south-east corner of the building dropped by approximately 800mm and deflected horizontally approximately 1300mm at the top of the building.
>
> There was sufficient redundancy and resilience within the overall structure to redistribute the loads from the failing element and to halt the collapse.
>
> This major movement induced other damage including: column failure at the underside of the podium, beam yielding, stair collapse and precast panel dislodgement. The collapse of the stairs, in particular, was dependent on the wall failure. Other more minor structural damage was consistent with what may have been expected in a well performing reinforced concrete structure in a seismic event of this nature.
>
> The investigation found that, for the most part, the structural design appeared to be compliant with the codes of its day. However, for the failed wall, D5–6, it does appear that there were some items of non compliance that most likely contributed to the failure. The magnitude of possible axial loads was under-estimated and the wall lacked the confining reinforcing needed to provide the ductility required to withstand the extreme actions that resulted from the February 2011 aftershock. In addition the assessed response of the building to this shaking exceeded the actions stipulated by both the current and contemporary loadings codes for a building of this type, structural period (of vibration) and importance.

3.4.2 DBH Expert Panel review

The Expert Panel report concurred with the conclusions of the Dunning Thornton report.

The findings were presented to the Royal Commission at the hearing on 17 January 2012 by Associate Professor Dr Stefano Pampanin, one of the members of the Panel.

The conclusions and recommendations of the Panel were set out in paragraphs 6.10 and 6.11 of the Expert Panel report, as follows.

6.10. Conclusions

Examination and analysis suggests that the building structure was generally well designed. Indeed, the overall robustness of the structure forestalled a more catastrophic collapse. However the shear wall D5–6 contained some critical vulnerabilities that resulted in a major, but local, failure. Other shear wall failures of similar appearance have been observed in other buildings following the 22 February 2011 aftershock, and this suggests that a review of both code provisions and design practice is warranted.

6.11. Recommendations

This section contains some recommendations arising from observations made during the investigation of the Hotel Grand Chancellor building and the meetings of the Panel. Some are quite specific to structural features that are contained within the Hotel Grand Chancellor and some are more generic, relating to design codes and practice generally.

The matters set out below are ones that the Department should give consideration to:

- **Design rigour for irregularity**

  While current codes do penalise structures for irregularity, greater emphasis should be placed on detailed modelling, analysis and detailing. An increase in design rigour for irregularity is required.

- **Design rigour for flexural shear walls**

  The behaviour of walls subject to flexural yielding, particularly those with variable and/or high axial loads, has perhaps not been well understood by design practitioners. An increase in design rigour for wall design generally, and in particular for confinement of walls that are subject to high axial loads, is required.

- **Stair review**

  A review of existing stairs, particularly precast scissor stairs, should be promoted and retrofit undertaken where required.

- **Stair seating requirement**

  The introduction of larger empirical stair seating requirements (potentially 4%) for both shortening and lengthening should be considered. This should be included in earthquake-prone building policies.

- **Floor-depth walls**

  The consequences of connecting floor diaphragms with walls that are not intended to be shear walls requires particular consideration. A Design Advisory relating to walls/beams that are connected to more than one floor but which are not intended to act as shear walls, should be considered.

- **Design rigour for displacement induced actions**

  Designers generally have tended to separate seismically resisting elements from ‘gravity-only’ frames and other elements of so-called secondary structure. However, not enough attention has always been paid to ensure that the secondary elements can adequately withstand the induced displacements that may occur during seismic actions. Non-modelled elements should perhaps be detailed to withstand 4% displacement. Modelled elements should be detailed to withstand a minimum of 2.5% displacement. An increase in design awareness relating to displacement induced actions should be promoted.

- **Frames supported on cantilevers**

  Although this is not a common arrangement, caution needs to be taken when supporting a moment resisting frame on cantilever beams as effective ratcheting can lead to unexpected deflections. A Design Advisory relating to ratcheting action of cantilevered beams and frames should be considered.

During February 2012 DBH issued a final Expert Panel report. The only difference from the Stage 1 report was the renumbering of paragraphs 6.10 to 7.11, and 6.11 to 7.12.

3.4.3 William T. Holmes' review

The Royal Commission retained Mr Holmes to review both the Dunning Thornton report and the Expert Panel report. His findings were presented to the Royal Commission at the hearing on 18 January 2012, and may be summarised under the following headings:

Overall comments

- general agreement on failure caused by a heavily loaded and lightly reinforced wall; and
- the content of the investigative report results in questions with answers not available or not on the record:
  - the report relied on simplified analysis techniques apparently due to complexity of the building—particularly very strong vertical discontinuity (walls to frame);
  - the derivation of drifts estimated from displacement spectra are not clear—certainly not the vertical distribution of drifts (section 5.1 of the Dunning Thornton report);
  - the derivation of loading on the failed wall, D5-6, is not clear (Appendix F.1 of the Dunning Thornton report); and
  - very high vertical accelerations are noted in the February event, but their relative contribution to the failure is not estimated. In fact, it is stated that the wall probably would have failed anyway.

September 2010 versus February 2011 shaking

- although the shaking intensity in the period range of the structure was more intense in September 2010 than in February 2011, the explanation of the lack of damage in September is not satisfying;
- 'maximum possible displacements' are estimated at 700mm in September and 1050mm in February using an average of four elastic spectra from recordings. Two of the four recordings in September would have also yielded 1000 or more millimetres and one of the records from February only yielded a maximum of 850mm;
- reference to paper by Associate Professor Pampanin and others as an explanation is unclear; and
- comparison of inelastic displacement spectra with estimated displacement in September do not reconcile.

Possible explanations

- direction of strongest motion in September was north–south, which minimises interaction with global moment from cantilevers on east face (potential ratcheting). In February the strongest motion was east–west;
- damage in September in the frame superstructure was greater than reported; and
- inelastic spectra at base of upper moment frame were filtered by walled base structure in some way so that response was minimised. Brittle wall D5–6 did not go past its failure point (but did in February).

Lessons learned

- irregular structures, if allowed, must be carefully designed (peer review?);
- 'late' changes in design must be carefully considered (another example of bad things happening is the Kansas City walkway collapse);
- structures that incorporate major elements affected by shaking in two directions must be carefully considered (most structures designed for one direction at a time); and
- interaction of gravity framing with the lateral load system must be carefully considered (leaning columns, massive amounts of cantilevers, etc.) including the potential for ratcheting.

3.5 Discussion

3.5.1 Introduction

The Royal Commission agrees with many of the conclusions contained in the Dunning Thornton and the Expert Panel reports but has some reservations concerning aspects of the design of the building and its assessment in the reports, which we discuss below. In a number of cases the Royal Commission arrives at different conclusions from those given in the reports. These are briefly identified and are dealt with in greater depth later.

3.5.1.1 Design

The building was generally well-designed and detailed. However, there were two major errors made in the design analysis, which led to the partial failure of the building in the February 2011 earthquake. First, the axial load acting in wall D5–6 at ground level was underestimated, as clearly indicated in the Dunning Thornton report. This led to the wall being inadequately proportioned and detailed to sustain the structural actions imposed on it. Secondly, the modal response spectrum method of analysis was used in the design without allowance for the eccentric gravity loads acting on the structure. These actions gave the building a tendency to sway towards the east, which had a major influence on the seismic performance of the structure. This aspect was not considered in either the Dunning Thornton or the Expert Panel report.

3.5.1.2 Dunning Thornton report's analysis

The Royal Commission notes the following points related to the Dunning Thornton report's analysis of the building, which lead us to different conclusions:

1. The Dunning Thornton report indicates that the dependable base shear strength (in current design standards this is referred to as the design strength) for the frames at level 14 was $0.048W_t$ where W_t is the weight of the building above the structural walls (above level 14). This value appears to have come from information submitted to the CCC as part of the application for a building permit. The report further indicated that the actual strength was of the order of $0.08W_t$. The Royal Commission does not accept this value, as the effective strength varies with the direction of lateral seismic forces owing to the eccentricity of the gravity loads. This has a major effect on the seismic performance of the building.
2. The Dunning Thornton report comes to the conclusion that the stairs would not have collapsed without the failure of wall D5–6 at the base of the building. The Royal Commission does not accept this conclusion, as the analysis fails to allow for a number of actions that were recognised in current design standards and have been shown to have a significant effect on inter-storey drift, a key feature (action) initialising failure of stairs.
3. The Dunning Thornton report indicates that if the critical wall in the building, D5–6, had been designed to the current Standard, NZS 1170.5[3], it would have collapsed. This conclusion was based on the observation that the minimum design base shear strength in the current Standard[3] is lower than the corresponding value in the Loadings Code NZS 4203:1984[4] used for the design of the building. The Royal Commission has reservations about this conclusion.
4. The stability of any wall or column depends on three factors. The first is the strength for flexure and axial loads; the second is the detailing for confinement and shear strength; and the third is the magnitude of the displacement applied to the wall or column. The Dunning Thornton report discusses the first two points but does not give any indication of the displacement that the critical wall would have been subjected to from seismic actions.

3.5.2 Comments on aspects in the Dunning Thornton report

3.5.2.1 Fundamental period of vibration

In the September earthquake the predominant shaking for structures with fundamental periods of vibration in the range of 2.5–4 seconds was in the north–south direction, while the corresponding predominant shaking for the February earthquake was in the east–west direction[5].

The Dunning Thornton report states that the HGC building had a calculated initial fundamental period of vibration (prior to the yield of the tower frames) of around 2.8 seconds. As a structure yields it softens and, as a consequence, the effective period was calculated to be about four seconds. The Royal Commission notes that this increase in period is an assumption inherent in the equal displacement concept, and is implicit in designs based on elastic methods of analysis such as the modal response spectrum method. With this method of analysis the change in stiffness as the earthquake progresses is, therefore, already built into the basic assumptions on which the approach is based. It is valid to allow for softening that may have occurred because of yielding in the September earthquake, and the effect this had on the fundamental period of the building in its condition at the start of the February earthquake, but it is not valid to allow softening due to the increase in period during the earthquake. Using a longer period for the analysis of the February earthquake could be justified, therefore, only on the basis of damage sustained during the September earthquake.

An examination of the response spectra for the September earthquake shows that any predicted yielding in structural elements resisting seismic shaking in the east–west direction would have been minor, as the shaking in this direction was much less than in the north–south direction for the period range of interest. The assessment in the Dunning Thornton report used a response spectrum found by averaging the measured response spectra from the four sets of horizontal records recorded in the CBD for the north–south and the east–west directions. The corresponding spectra for these records are given on pages 40 and 41 of the Carr report[5] for both the elastic response and for ductile structures. From these spectra it can be seen that the response spectral lateral displacements in the east–west direction for the September earthquake are of the order of 210mm for a structure with a displacement ductility of 2 over the period range of 2.5–4.5 seconds. This level of ductility would induce little inelastic deformation in the structural members resisting east–west seismic actions and, consequently, any reduction in stiffness for this direction of loading would have been minor.

3.5.2.2 Response of buildings to the September earthquake

The Dunning Thornton report states on page 15 that the response of the structure did not match what was indicated by the response spectra. A number of possible explanations were advanced for this. However, one explanation that is likely to account for a major part of the apparent discrepancy is not mentioned. The two moment resisting frames orientated north–south, which resisted the seismic forces in that direction, comprised five bays in each case. Two of the bays had beams with clear spans of about 3.1m and the remaining three had clear span lengths of about 6.9m. Furthermore, precast pretensioned floor units spanned parallel to the beams and, in the longer spans, the critical sections of the potential plastic hinges were about 800mm from the support position of the precast floor units. When these frames were displaced in the north–south direction it is likely that:

- plastic hinges would have formed in the shorter spans at about 45 per cent of the lateral displacement required to initiate yielding in the longer spans; and
- the precast pretensioned units spanning past the plastic region would have contributed significantly to the flexural performance of the beam (see NZS 3101:2006[6], clause 9.4.1.6.2[6]).

The first effect would result in the lateral displacement response being bi-linear in form, with the longer beam acting as a spring. This would have reduced the residual displacement of the frames after each significant inelastic excursion. The second effect relates to elongation associated with plastic hinges in the longer span, which would have been partially restrained by the precast pretensioned units. These would have increased the strength and acted as a partial spring to reduce deformation. These two effects could be expected to significantly reduce the peak north–south displacement sustained in the September earthquake.

The Dunning Thornton report indicates that for the February earthquake the response spectra used for the analysis give a maximum displacement at a period of three seconds of 1050mm at the dynamic centre of mass for the fundamental period. In the Dunning Thornton report's analysis a value of 500mm was quoted for this displacement, but it is not clear how this reduction was justified. Carr, in his report[5], shows that the corresponding displacement for structures with displacement ductility in the range of 2–4 is of the order of 400mm–500mm for the period range of 2.5–4 seconds. However, this explanation is not given in the Dunning Thornton report. For a building with a relatively long fundamental period in a short intense earthquake, the equal displacement concept overestimates the displacement. Consequently, the assumed displacement of 500mm for the dynamic centre of mass given in the Dunning Thornton report is appropriate, but not for the reasons that were given.

3.5.2.3 Calculation of axial load on critical wall

A number of questions arise, which are not explained in the Dunning Thornton report, about the way in which the axial load on the critical wall, D5–6, was calculated:

1. The report indicates that the axial load was calculated following the capacity design steps given in the Concrete Structures Standard. However, the report does not state which edition of the Standard was used and there are marked differences between the 1982 and the current design Standards (2006)[6,7].

2. The capacity design steps in the Code for finding the maximum axial load levels in columns are based on the assumption that the lateral displacements of the building are well in excess of design levels for the ultimate limit state. The report predicts that the displacement ductility in the February earthquake was of the order of 3.3, which is well below the ultimate limit state design level of about 5 and far below the value of 7+ that should be sustained before collapse. Consequently, it is unlikely that the full capacity design axial loads would have been developed.
3. In the calculation of the peak axial loads in columns the flexural reinforcement in the beams was assumed to have its upper characteristic yield strength. This assumption gives a conservative basis for the purposes of design. However, in the assessment of a structure it is more rational to assume the reinforcement has average material properties. Using the upper characteristic strengths for the reinforcement would have led to a high estimate of the maximum axial load applied to the wall.

We conclude that the maximum axial load level given in the Dunning Thornton report is likely to have been overestimated by a few per cent. However, this discrepancy is small compared to the likely but unknown magnitude of the component of axial load induced by the vertical seismic ground motion.

3.5.2.4 Collapse of the stairs

The Royal Commission does not accept the Dunning Thornton report's conclusion that the stairs would not have collapsed without the failure of wall D5–6. In the report's analysis no allowance has been made for any of the following:

- the difference between the design and peak inter-storey drifts;
- P-delta actions;
- the difference in inter-storey deflections found using inelastic time history analyses and the elastic-based analyses modal response spectrum and equivalent static methods of analysis;
- the eccentricity of the gravity loads on the structure as illustrated in section 3.5.3 of this Volume, which arises from the cantilevering of the eastern-most bay of the building (see Figure 27). This causes the inter-storey drifts sustained during the earthquake to increase significantly in the eastward direction.

If allowance had been made for these actions the predicted inter-storey drifts would have been considerably greater than those quoted in the report. They would also have indicated that the stairs were likely to collapse even if the wall did not fail.

The Dunning Thornton report indicates that the drift the stairs must sustain is given by multiplying the inter-storey displacement between adjacent floors found from a response spectrum modal analysis by K/SM. In this term, K is 2.2 and S and M are both equal to 0.8, which results in the term having a value of 3.44. These values are given in the Loadings Code, NZS 4203:1984[4] for calculating the design inter-storey drift. However, this is a design displacement and not the peak value.

For stairs, it is essential to use the peak value and for this reason the Loadings Code, NZS 4203:1984[4] required the design drift to be doubled (see clause 3.8.4.2). It should be noted that the design lateral earthquake forces are calculated using a structural ductility factor of 4/SM, which is equal to 6.25. On the basis of the equal displacement concept, the peak displacement is 6.25 times the modal response spectrum drift, which is in close agreement with the value of 6.88 times the value specified in the Standard. We conclude that the calculated drift required for the design of stairs quoted in the Dunning Thornton report was incorrectly assessed. Comparative analyses of ductile structures based on inelastic time history analyses and modal response spectrum analyses have shown that P-delta actions can have a significant influence on inter-storey drift[3,8,9]. Allowance for P-delta actions in ductile moment resisting frames typically increases both the strength requirements and the inter-storey drifts by about 30–40 per cent.

Comparative analyses of ductile structures by inelastic time history and modal response spectrum analyses (the latter was used by Dunning Thornton) have shown that the latter method underestimates the maximum inter-storey drifts. This occurs as the deflected shape profile tends to change owing to the formation of higher modes of behaviour associated with the formation of plastic hinges in the structure. To allow for this effect, NZS 1170.5 requires the inter-storey drift derived from the drift envelope to be increased by the drift modification factor. For the HGC building the appropriate drift modification factor would be 1.5 (NZS 1170.5, clause 7.3.1.1)[3].

The Royal Commission acknowledges that the influence of P-delta actions on deflections and the drift modification were not incorporated in design standards in the 1980s but, in assessing the building's performance in 2011, it considers that allowance for all actions known to influence behaviour should be considered if the intent is to gain knowledge relevant to current design practice.

3.5.2.5 Would the building have collapsed in a NZS 1170.5 defined event?

The Dunning Thornton report (see page 30, section 10.4) contains the following statement:

> The design basis earthquake as defined by NZS 1170.5 is similar to, but a little smaller than, an event defined in NZS 4203:1984, for a building having a period equivalent to that of the HGC building. Therefore, there is a likelihood of possible collapse during NZS 1170.5 defined actions. A relevant issue is that the D5–6 wall did not have sufficient robustness to cope with an event larger than that defined by the Standard. This was exposed on 22nd February 2011.

We do not consider that this deals with the critical issue, which is whether the building would have performed adequately had it been designed to current New Zealand Standards (2011). This depends on the difference in the seismic design actions specified in the Loadings Standards in the 1980s[4], the current Earthquake Actions Standard[3] and the design requirements given in the corresponding Concrete Structures Standards, NZS 3101 of 1982 and 2006[6,7], compared with the details that were actually used.

The initial fundamental period of vibration of the building is given in the Dunning Thornton report as 2.6–2.8 seconds. Considering the earthquake actions as defined in the Standards, the lateral force coefficients for a fundamental period of 2.6 seconds in NZS 4203:1984[4] and NZS 1170.5:2004[3] for a ductile moment resisting frame are 0.048 and 0.031, respectively. However, allowing for the change in strength-reduction factors during this period (from 0.9 in 1980s to 0.85 in 1995)[7,10] the corresponding ratio of strengths would be 1:0.69.

In NZS 4203:1982[4] there was no requirement for strength to be increased to counter P-delta actions. However, in our current Standard allowance must be made for P-delta actions and two methods are given for this purpose in NZS 1170.5[3]. The simpler method increases the base shear strength by a factor of almost two, while the more detailed method changes the strength required by different proportions over the height of the building. A typical increase in strength with this approach would be of the order of 40 per cent. Using this value the ratio of equivalent strengths becomes 1:0.96 for the 1980s to the 2004 Standards respectively. On this basis, it can be seen that there was little difference in the strength requirements given by the 1984 and 2004 Standards[4,3].

As noted previously, wall D5–6 did not satisfy the requirements of the Concrete Structures Standard in 1982[7]. This was due to an error in assessing the axial load level. If the error had not been made, instead of a thickness of 400mm, the wall would have had a minimum thickness of 500mm, to satisfy the maximum slenderness ratio of 10:1 clear height for thickness. It would also have had more confinement reinforcement. If the wall had been designed to meet these requirements it is likely that it would have survived the February earthquake, although it is not possible to be certain of this unless the lateral displacement of the wall is known.

The stability requirements in the 2006 Concrete Structures Standard were based on the requirements given in NZS 3101:1995[10]. These had been developed as a result of structural testing and analytical work, where it had been found that walls buckled after being subjected to high flexural tension strains. Unfortunately, the case of buckling caused by high compression loads was not considered. A consequence of this is that buckling of walls under low axial load conditions is covered in the current Standard, but cases such as that which occurred in wall D5–6 for high axial loads are not covered by the Standard. Consequently, it would be theoretically possible to proportion wall D5–6 to be even more slender than was the case in the 1980s.

We consider that the answer to the critical question of whether the building would have performed adequately had it been designed to 2011 Standards is that it very likely would not have.

This is due to a weakness in our current Concrete Structures Standard on the allowable slenderness of walls with high axial load ratios of $(N/A_g f_c')$ (at or approaching $0.2A_g f_c'$), as identified above, rather than due to inadequacies of the design seismic actions defined in the Earthquake Actions Standard, NZS 1170.5[3]. In our view, urgent revision of the design limits for stability in structural walls subjected to moderate or high axial load ratios in NZS 3101: 2006[6] is required.

As far as the structural seismic actions are concerned, there is relatively little difference in the required lateral strengths and deformation capacities for walls. The current Concrete Structures Standard, however, would require Wall D5–6 to sustain a higher axial load level than was the case in the 1980s Standard, owing to changes made in the way that over-strength actions are calculated in beams. Consideration also needs to be given to the influence of vertical seismic ground motion on the design axial loads induced in walls and columns.

3.5.3 Method of analysis used by the Royal Commission

3.5.3.1 Analytical model

A schematic representation of the HGC building's structural system that resists lateral forces in the east–west direction is shown in Figure 27. Structural walls resist the lateral forces up to level 14 while, above this level, the forces are resisted by moment resisting frames. To maintain Tattersalls Lane at ground level the eastern-most bay of the building is cantilevered off the remainder of the structure.

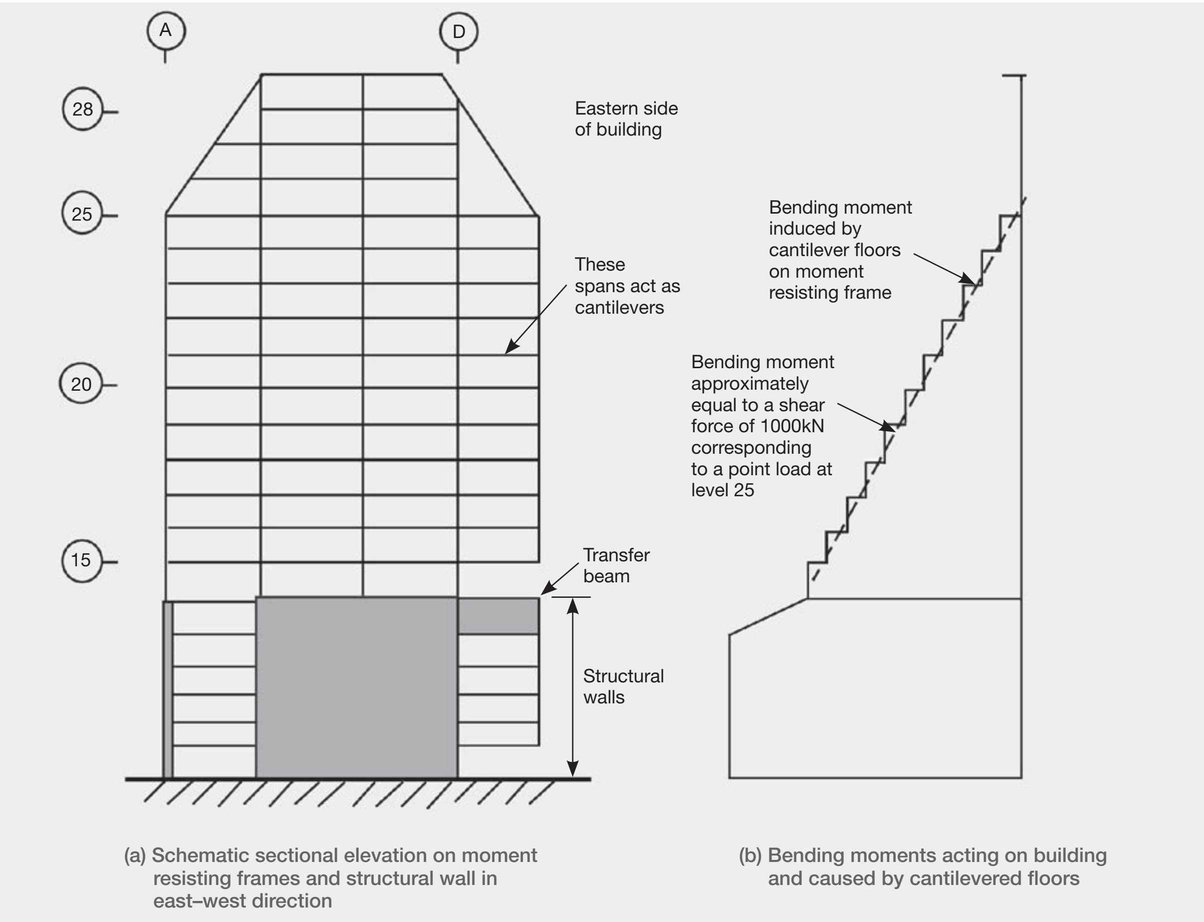


Figure 27: Action in HGC building from cantilever action of gravity load

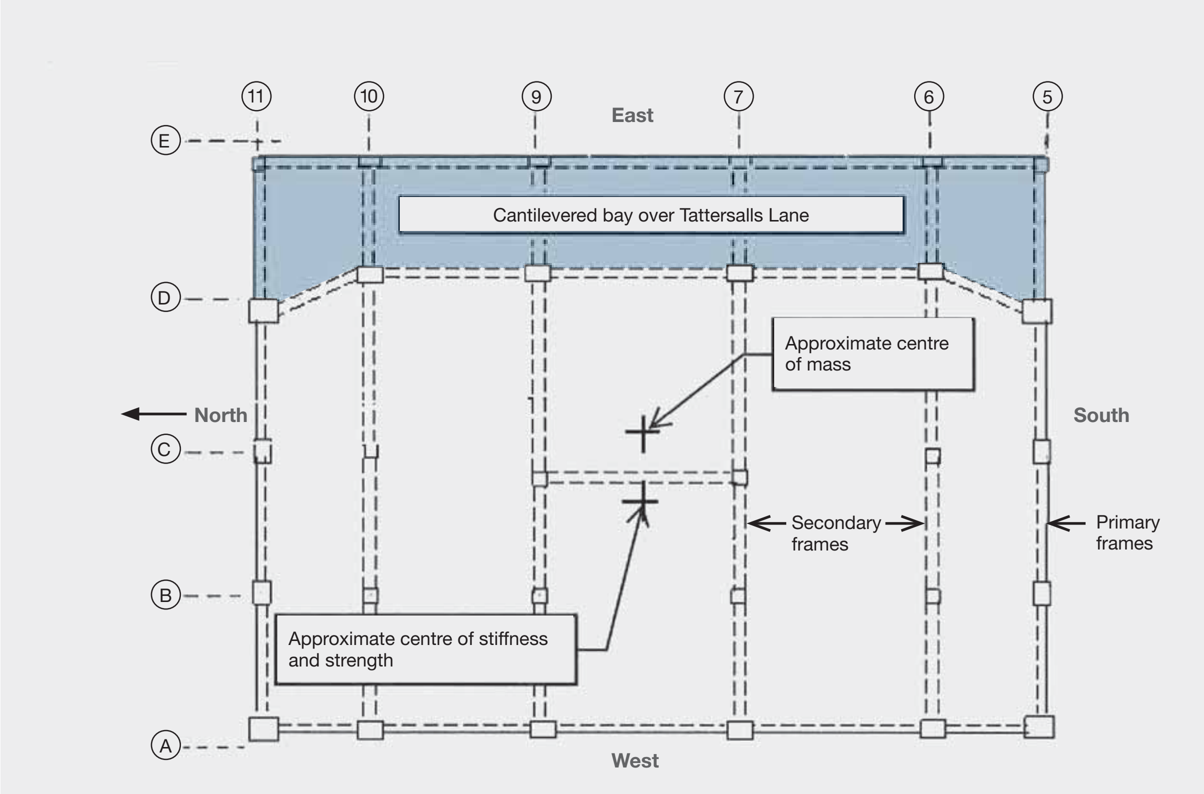


Figure 28: Plan of structural arrangement of tower above level 14

The dead and live loads acting on the cantilevered spans, as shown in Figures 27 and 28, induce bending moments as seen in Figure 27. These actions cause the building to deflect towards the east. The cantilever bending moments are approximately equivalent to the action of a single lateral force of 1000 kilonewtons (kN) acting at level 25.

Each floor in the levels above 14 contains frames that are intended to resist the lateral forces, and a set of lighter secondary frames that span in the east–west direction and resist the majority of the gravity loading. The lateral force resistant frames are located in grid lines A, D, 5 and 11, as shown in Figure 28. The gravity load frames are located between grid lines 6 and 10. The contribution of the gravity load frames to lateral resistance has not been included as they are relatively flexible compared to the lateral force resistant frames and they have been proportioned to reduce their lateral resistance by reducing their depths and/or longitudinal reinforcement at critical sections.

An analysis of the beams and columns in the building shows that the lateral force resistance provided by the frames in grid lines 5 and 11 in the east–west direction in the storeys above level 14 (the top of the structural walls) is about 7.8 per cent of the weight of the building above this level. This figure is based on nominal strength calculations ("ideal strengths" in terms of NZS 3101:1984). In terms of an equivalent design base shear, these values need to be reduced to allow for building torsion and strength-reduction factors. This gives an equivalent design base shear of the order of 6.4 per cent of the seismic weight at the top of the structural walls at level 14.

As noted above, the eccentricity of the gravity loading is equivalent to a lateral force of 1000kN acting on the seismic resistance frames. Based on nominal strengths, this effectively reduces the ideal storey shear strength at the top of the structural walls to 6.4 per cent and 9.2 per cent, respectively, of the weight of the structure above level 14 for lateral seismic forces acting towards the east and west. However, as the reinforcement content in the beams is reduced over the height of the structure, the relative strength ratio for eastward–westward forces also reduces with increasing height.

The use of the modal response spectral analysis for ductile structures is based on the assumption that the lateral resistance is equal for both the forward and backward deformation. Hence, using this method of analysis for the HGC building violates one of the basic assumptions of the method. As the lateral force corresponding to inelastic deformation is lower for displacement to the east than to the west, it is to be expected that the structure would tend to progressively deflect towards the east during the period of strong ground motion. This behaviour has been referred to as "ratcheting".

To investigate this effect we developed a simplified model of the HGC building. The model was restricted to assessing seismic displacements in the east–west direction. The structural walls up to level 14 were assumed to remain elastic and were represented by columns with a given shear stiffness.

The analytical model was developed in consultation with Professor Athol Carr, who subsequently made the analyses described in this Report. Further details of the model and the parameters assumed for the analyses and the details of the ground motion records are given in Appendix A on page 85.

The structure above level 14 was represented by three columns, 1, 2 and 3, as shown in Figure 29. The first column represents the storey shear strength based on the assumption that points of contra-flexure would develop at the mid-height of each storey, as seen in Figure 30. The shear resistance provided by this action is limited by the strength of the beams at the critical sections and this was found from an analysis of the beams on grid lines 5 and 11. These storey shear strengths were used to define the corresponding shear strengths in column 1 in the model. The corresponding shear stiffness was assessed from the strength and the associated inter-storey deflection at first yield of the beams. (Note that this represents a mathematical model of the HGC structure, not a physical model, and that the column numbers do not represent the grid lines in the building.)

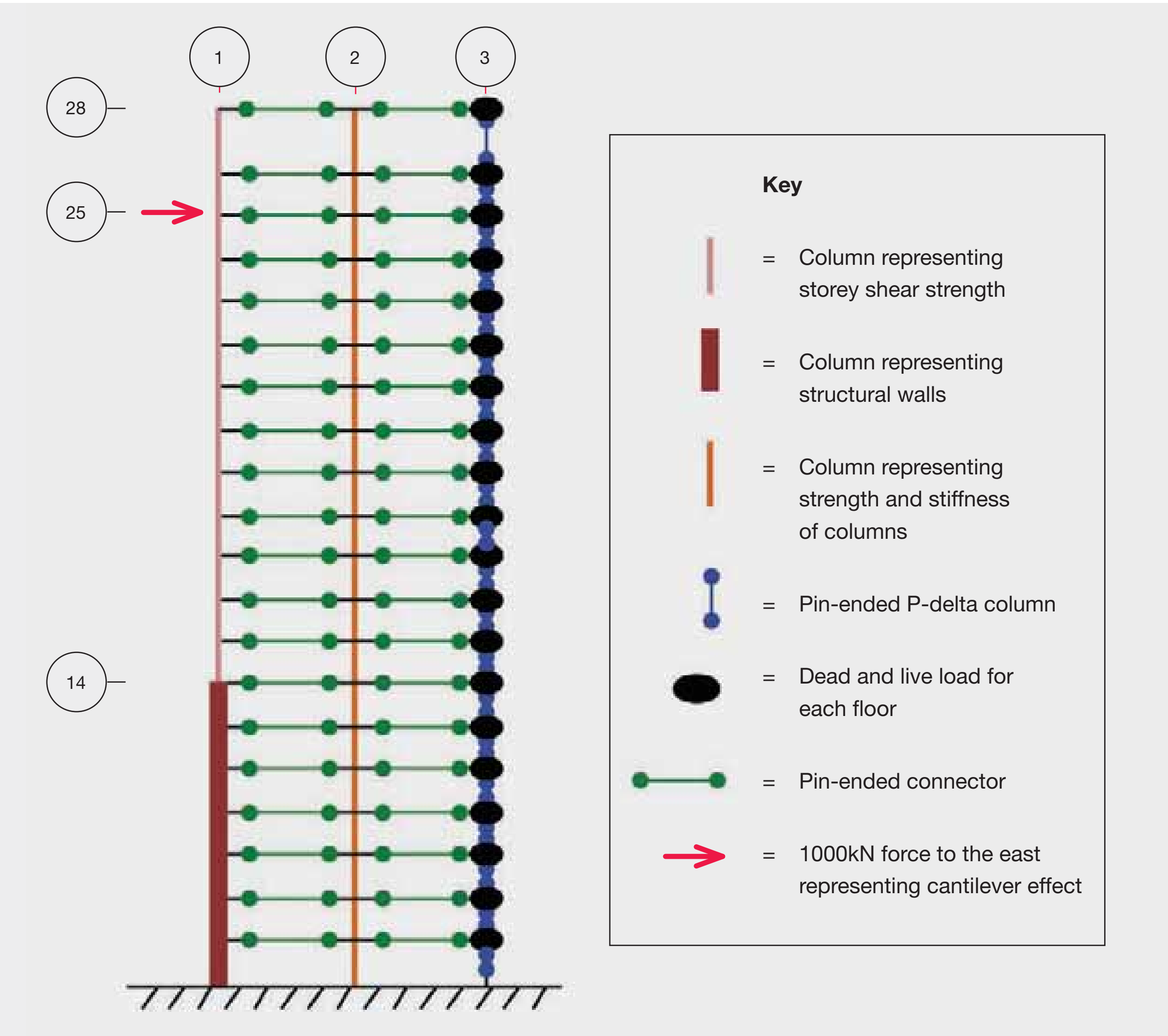


Figure 29: Schematic of the model for inelastic time history analysis

In any seismic loading situation the points of contra-flexure will move, provided the column strengths are adequate. Column 2 in the model, as shown in Figure 29, allows for this action. The flexural strengths of all the columns on grid lines 5 and 11, but excluding those on grid line E, were determined from the drawings, allowing for the gravity axial load that they resisted. The strengths at each storey of column 2 in the model are based on the sum of the calculated flexural strengths of these columns minus the corresponding column moments induced by the storey shear sway mechanism represented in column 1 in the model. The corresponding column stiffness values for the line 2 column in the model were calculated from the analysis of the columns at first yield but multiplied by 1.5 to allow for tension stiffening away from the potential plastic hinge region.

The third column, column 3 in the model, was added to allow actions arising from P-delta effects to be included, as indicated in NZS 1170.5 clause C6.5.1. This column was pinned at every level so it could resist axial loading alone but not contribute the lateral force resistance. The gravity loads at each floor were applied to column 3. As the first two columns in the model had their horizontal displacements slaved together and the P-delta column was tied to the other two columns by pin-ended struts, any P-delta actions induced by the gravity loads were transferred to the lateral force resisting elements of the building. The lateral stiffness of the shear elements was adjusted by changing the effective shear modulus until the fundamental period of the building was close to 2.8 seconds.

To allow for the cantilever portion of the frame, a single lateral force of 1000kN acting in the eastward direction was applied at level 25 (see Figures 27 and 29). This analytical model does not account for any reduction in strength as a result of torsional effects resulting from its irregular plan and the seismic actions in the north–south direction.

Using the model described above, inelastic time history analyses were made using the Ruaumoko analysis package[11] for the four recorded ground motions in the CBD for the east–west or near east–west direction. Two further analyses were made. The first of these was a composite earthquake involving one long analysis of the Resthaven (REHS) ground motions for the September, Boxing Day and February earthquakes, but with a period of no ground shaking between each earthquake record to allow the model to settle. The second was for an earthquake representing an Alpine Fault event. This was based on a record of the Japanese earthquake of March 2011, which was recorded on ground similar to Christchurch and at a similar distance from the fault.

Further information on the analytical model, the analyses and the earthquake representing the Alpine Fault event is given in Appendix A on page 85.

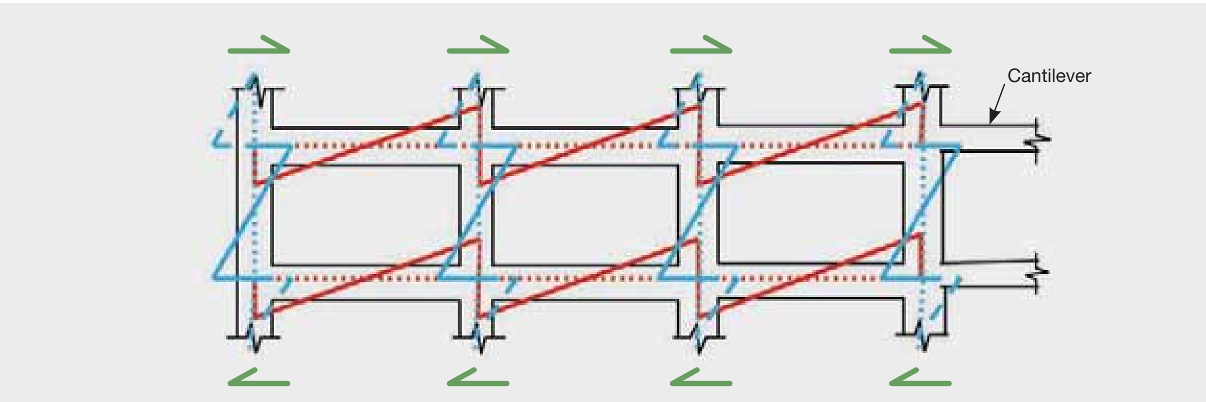


Figure 30: Storey shear strength (ignoring eccentric gravity loading)

3.5.3.2 Results of the analyses

After the inelastic time history analyses had been carried out, the displacement envelopes were calculated over the height of the building, together with peak inter-storey drifts and residual displacements. Results for the maximum storey displacements are given in Figure 31, below. The relatively stiff walls extended from ground level up to level 14, about 21m above the base. The predicted displacements of the top of the tower towards the east are three to five times greater than the corresponding maximum displacements to the west for the four CBD ground motion records.

The comparable Alpine Fault event, which had lower ground acceleration but a much longer duration, did not show the same degree of ratcheting in the eastern direction as the CBD ground motion records. The composite ground motion, involving one long earthquake in which the September event was followed by the Boxing Day and February earthquakes for the REHS station, showed a relatively small increase in ratcheting to the east when compared to the single REHS ground motion for the February earthquake.

To assess the significance of the eccentric gravity load acting on the structure, an analysis for the REHS ground motion was rerun assuming that there was no eccentric gravity loading. This was achieved by removing the lateral 1000kN force at level 25. The results of this analysis are shown in Figures 32 and 33 and can be compared with the corresponding values in Figures 31 and 34) where allowance was made for the eccentric gravity loading. It can be seen that without the eccentric gravity load the lateral displacement to the east is considerably reduced and the corresponding peak displacement to the west is of comparable magnitude.

Figures 34–37 show how the displacement at the top of the building varied with time for ground motions in the east–west direction measured in the CBD during the February earthquake (at the CBGS, CHHC, REHS and CCCC seismic measuring stations, as shown in the Dunning Thornton report, on page 13, and more completely defined by Carr[5]). Part (a) of each of these figures gives a plot of the recorded ground accelerations as a proportion of the acceleration due to gravity, while part (b) shows the corresponding displacement where the displacements to the east are given in negative values. In all cases, the analyses indicate that the building displaces progressively towards the east during the period of strong ground motion.

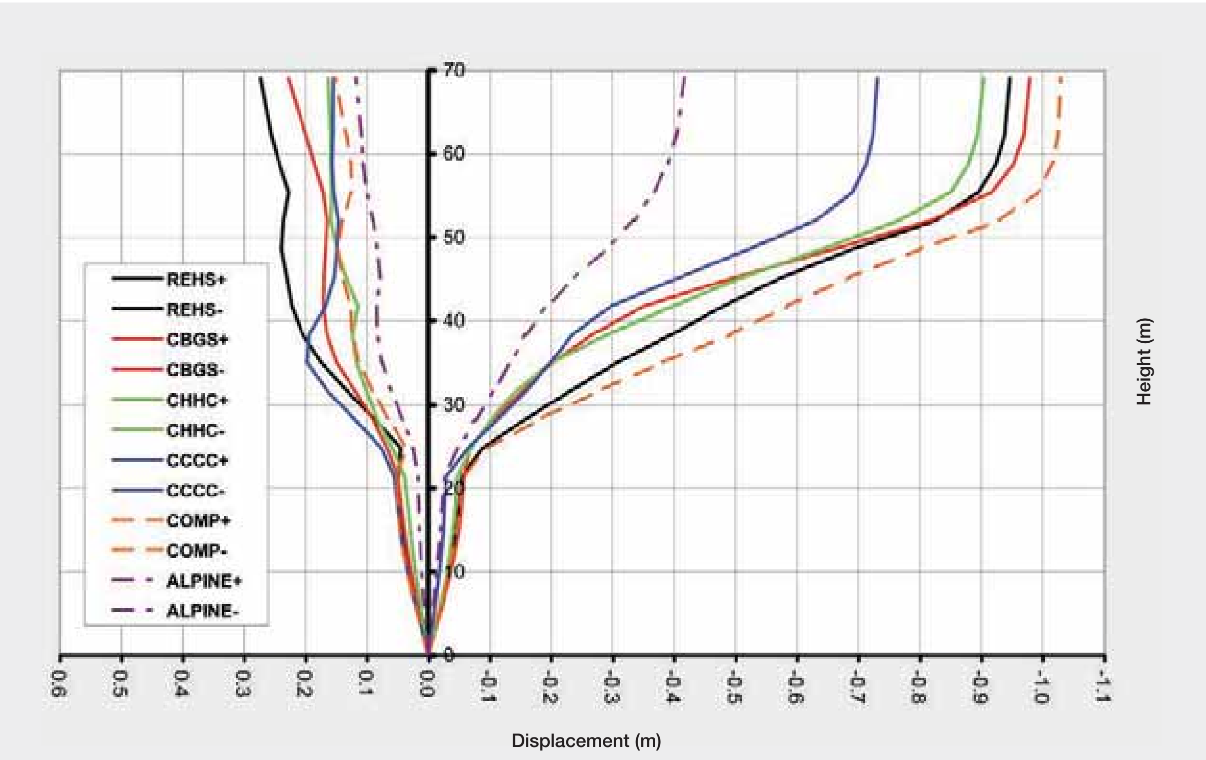


Figure 31: Maximum displacement envelopes for inelastic time history analyses

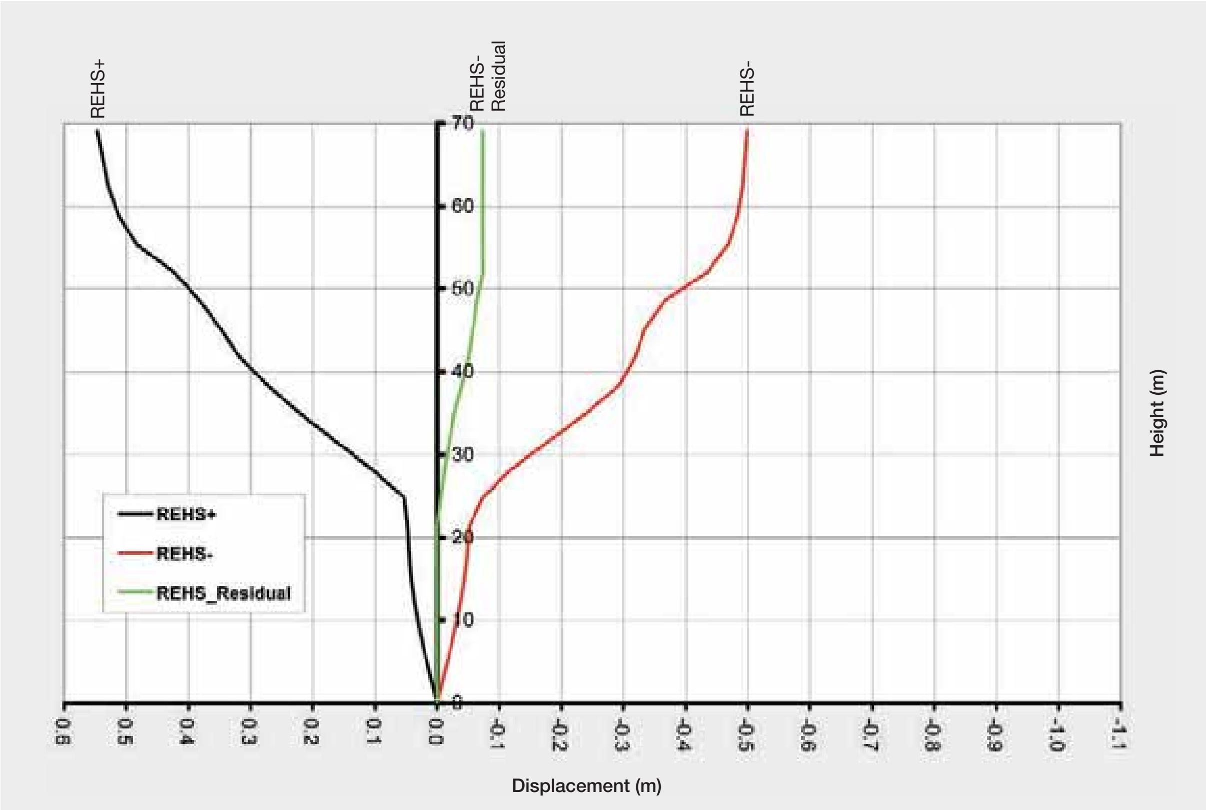


Figure 32: Maximum displacement envelope and residual displacement with no cantilever action

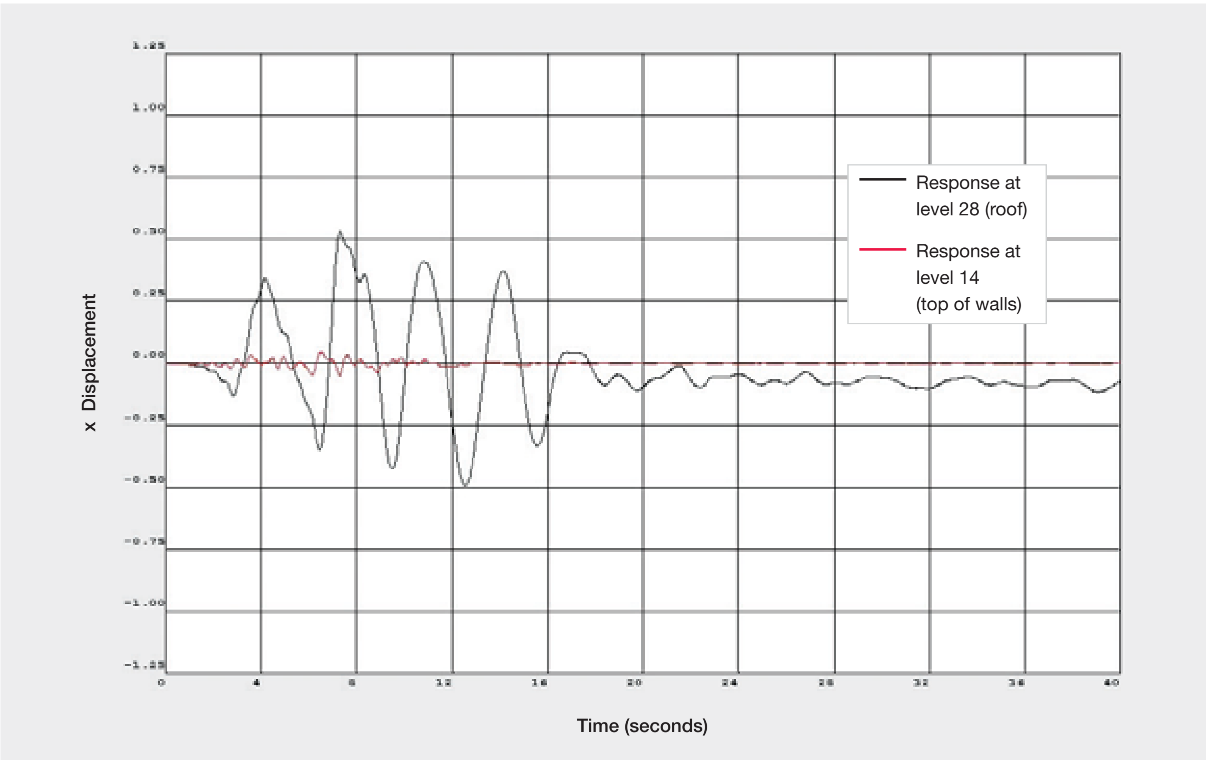
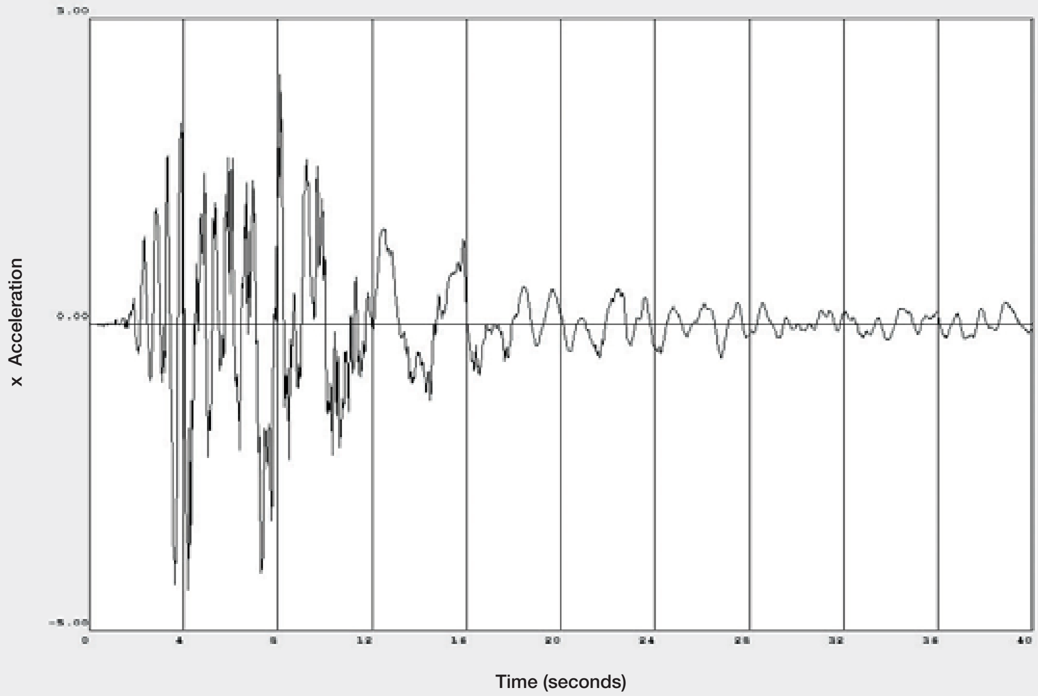
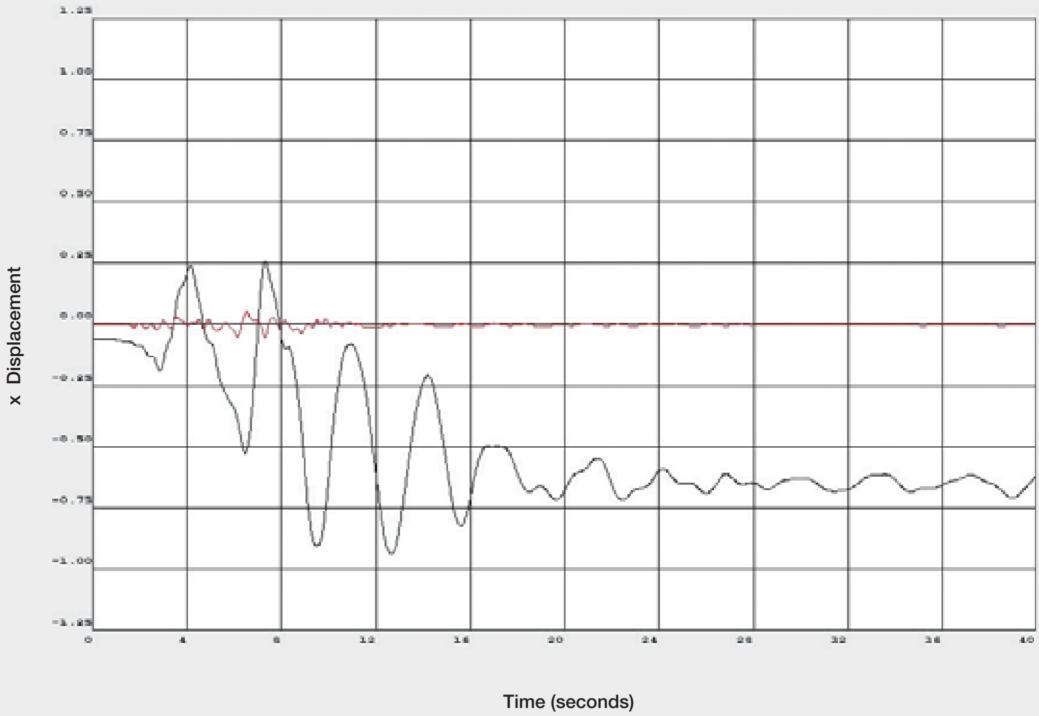


Figure 33: Displacement inelastic time history with no cantilever action (source: Athol Carr)

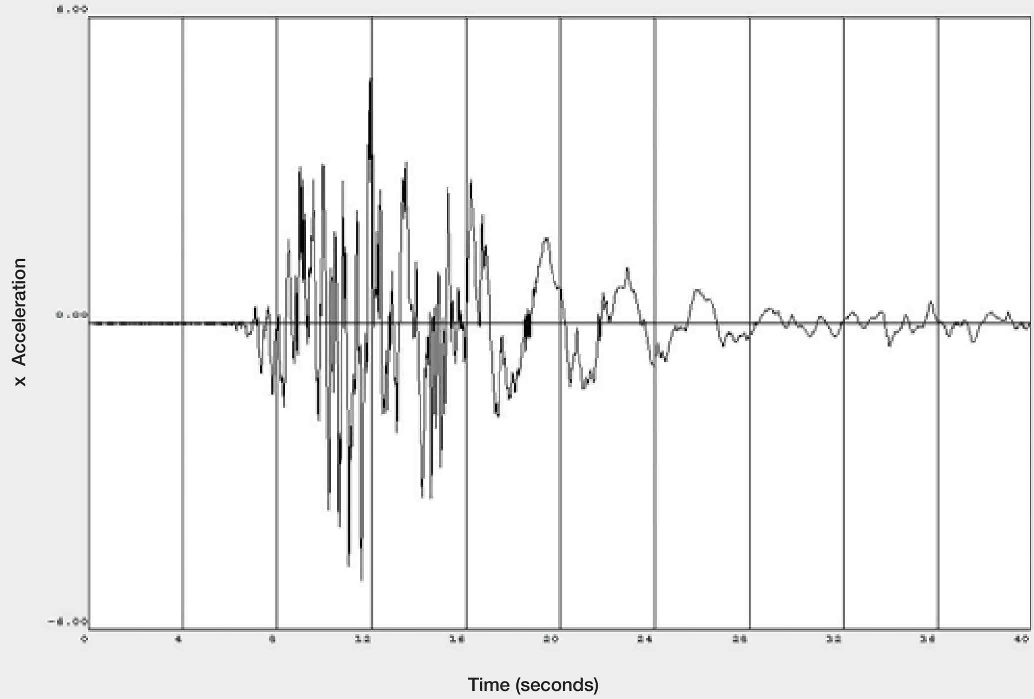


(a) Ground accelerations

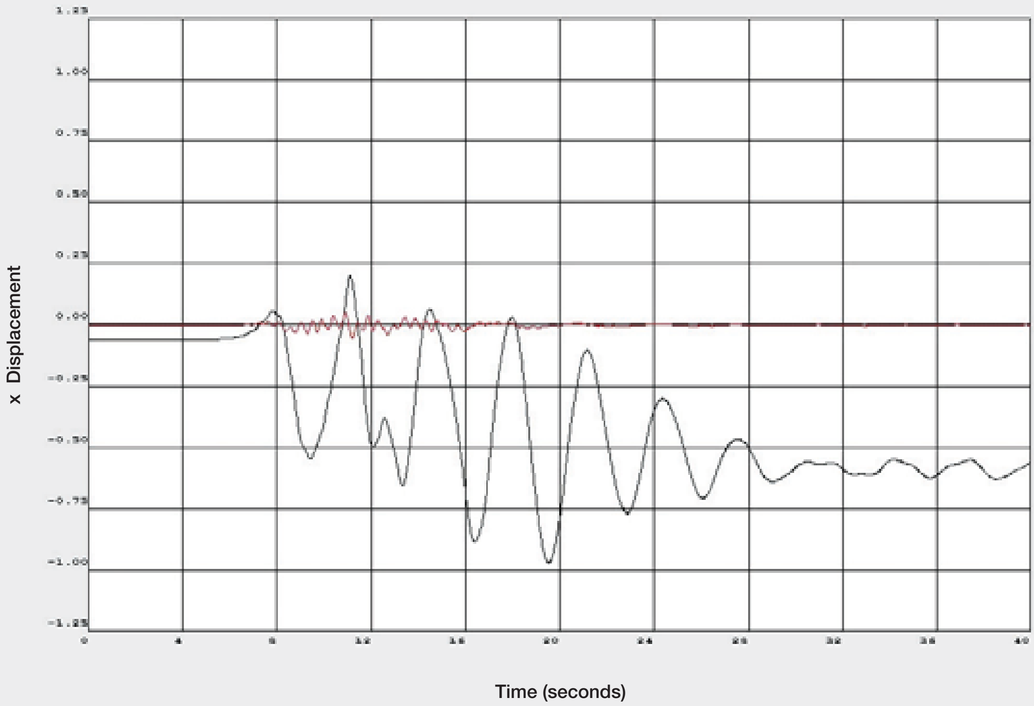


(b) The corresponding displacement at the top of the tower in the east–west direction

Figure 34: REHS earthquake record (source: Athol Carr)

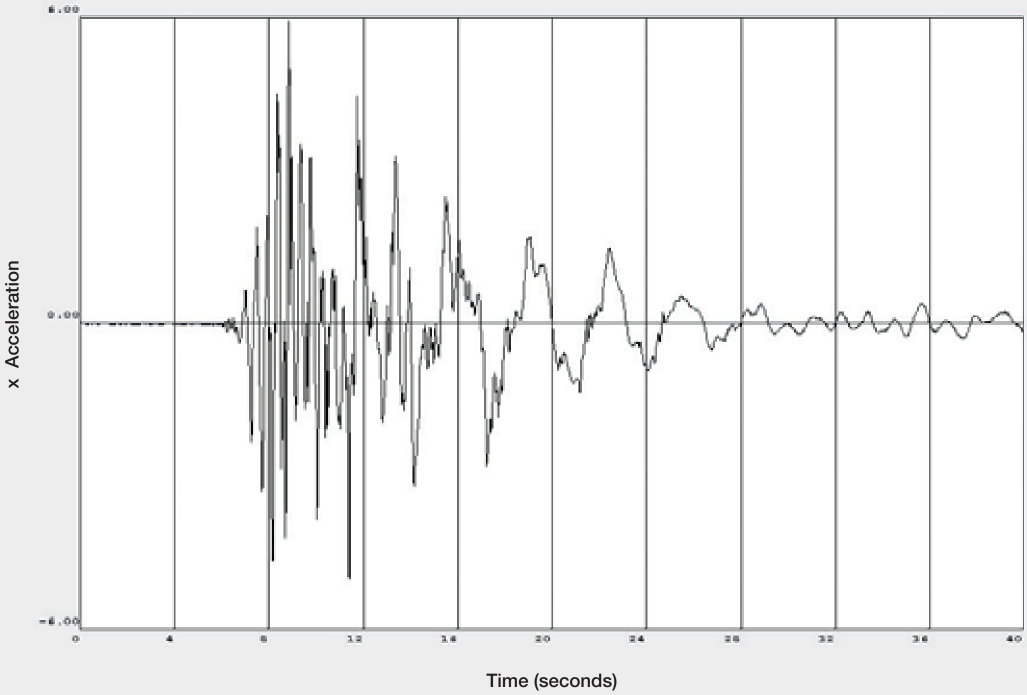


(a) Ground accelerations

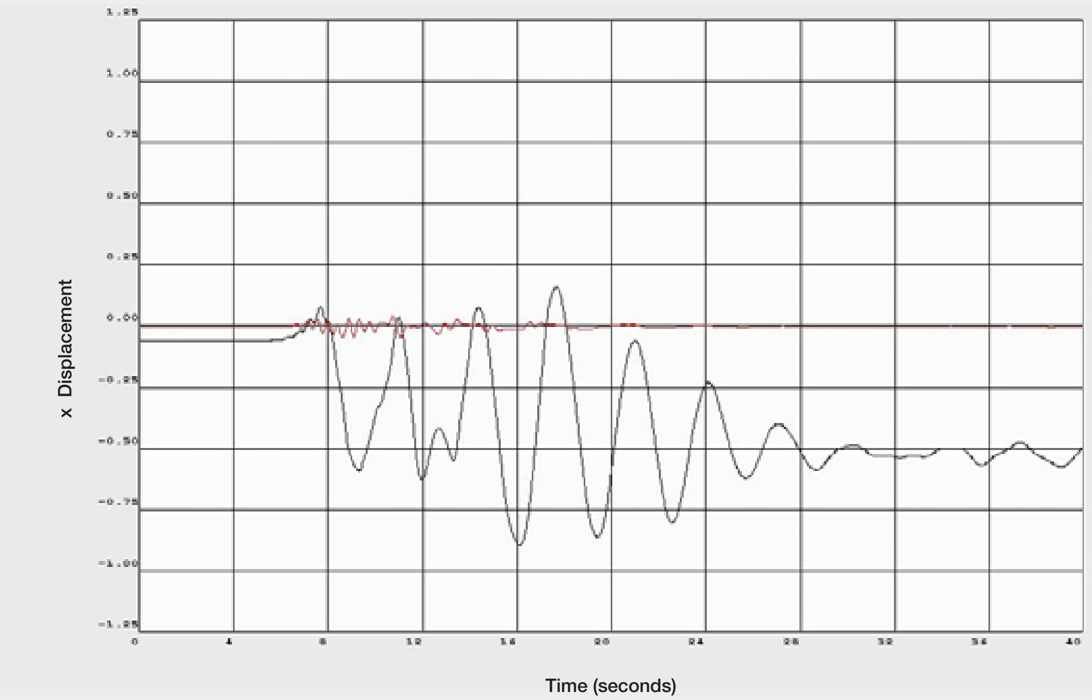


(b) The corresponding displacement at the top of the tower in the east–west direction

Figure 35: CBGS earthquake record (source: Athol Carr)

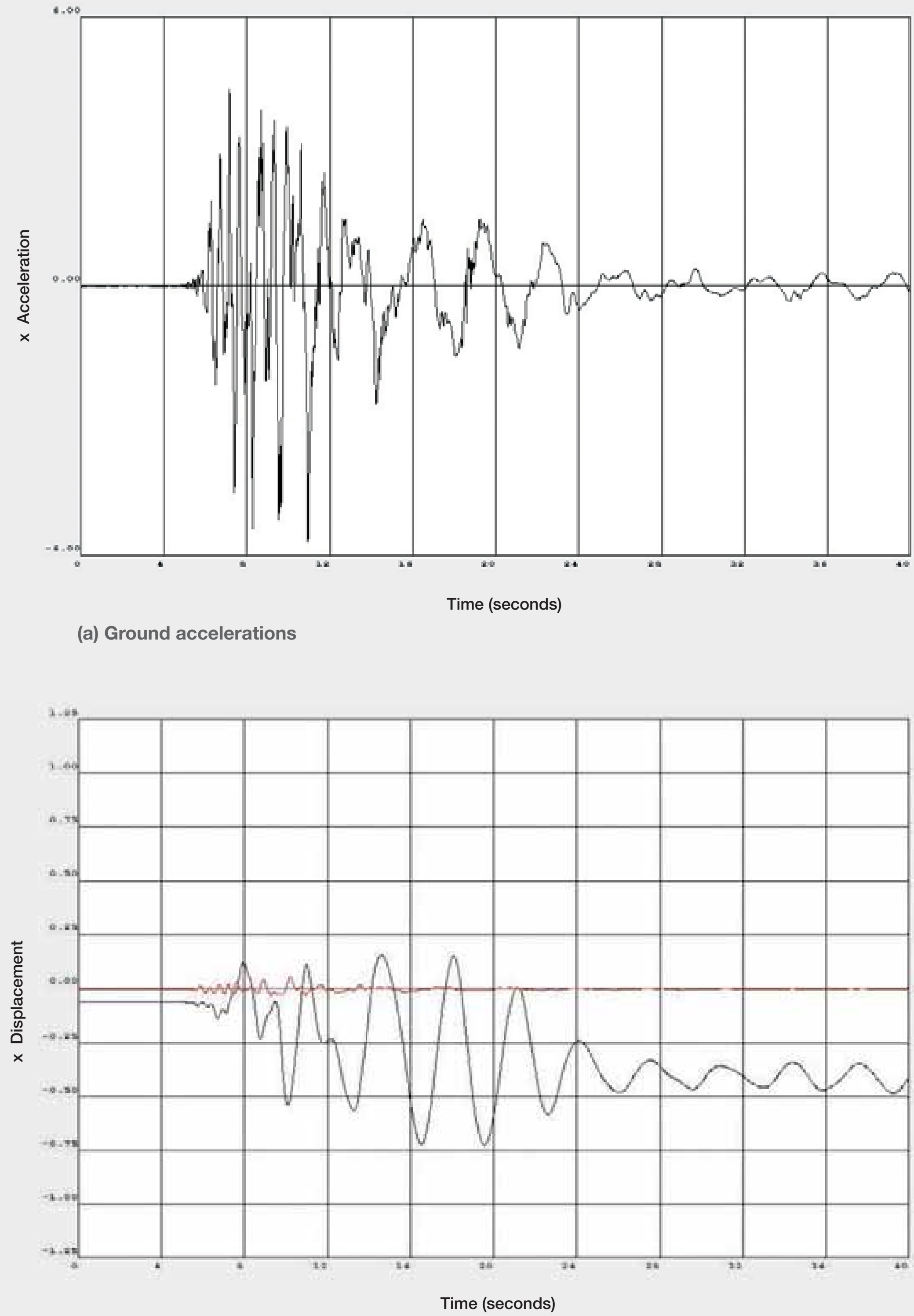


(a) Ground accelerations



(b) The corresponding displacement at the top of the tower in the east–west direction

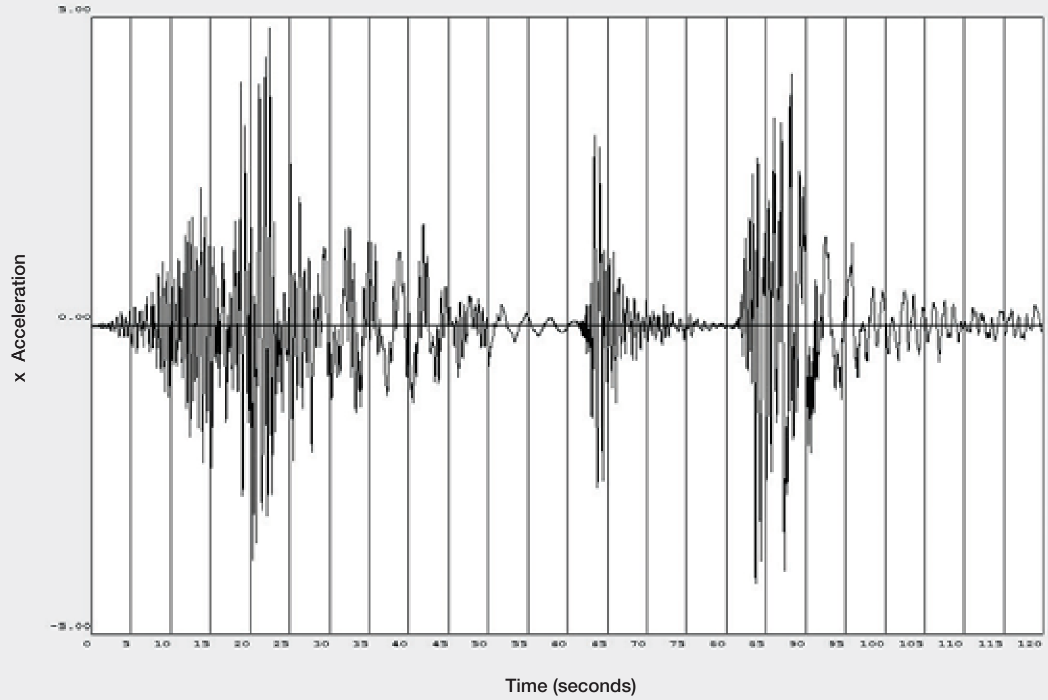
Figure 36: CHHC earthquake record (source: Athol Carr)



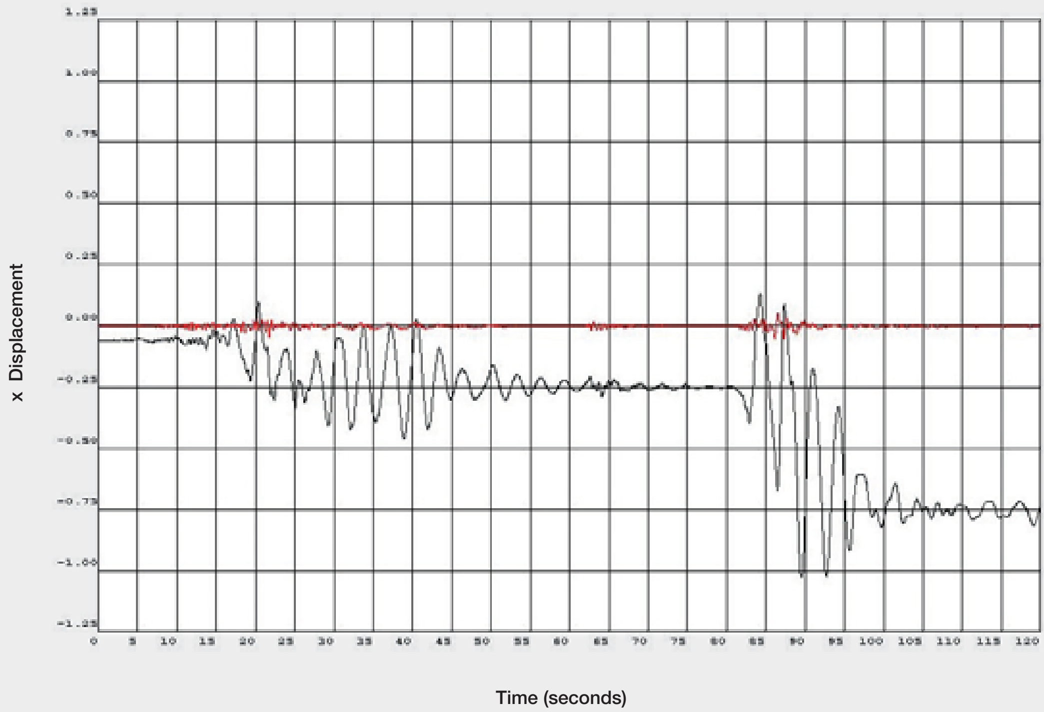
(a) Ground accelerations

(b) The corresponding displacement at the top of the tower in the east–west direction

Figure 37: CCCC earthquake record (source: Athol Carr)

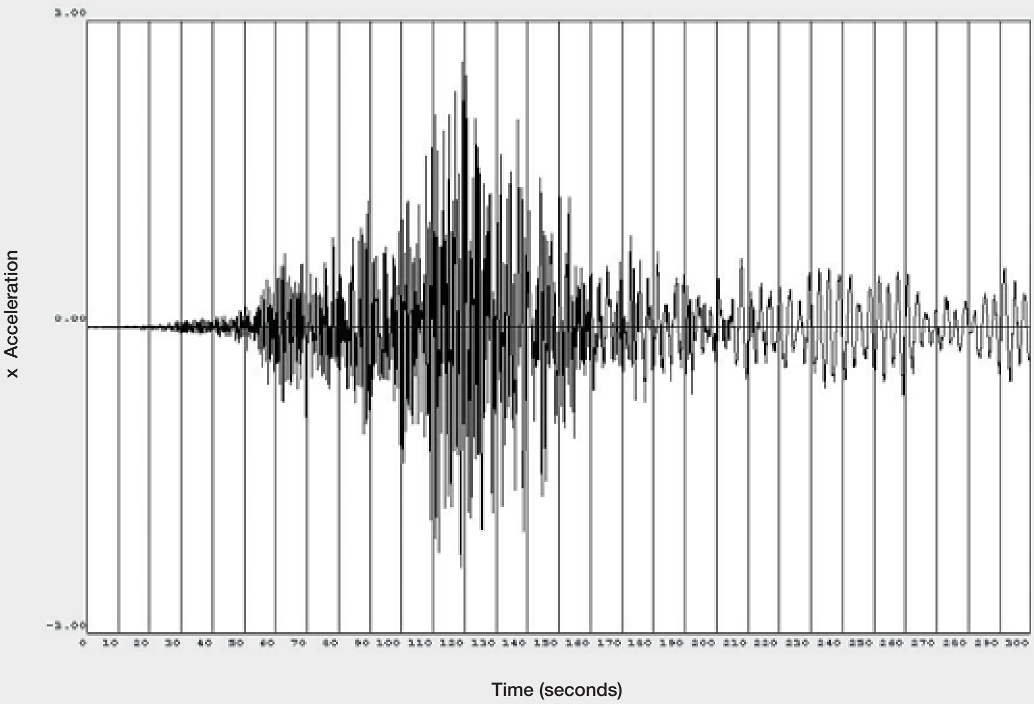


(a) Ground accelerations

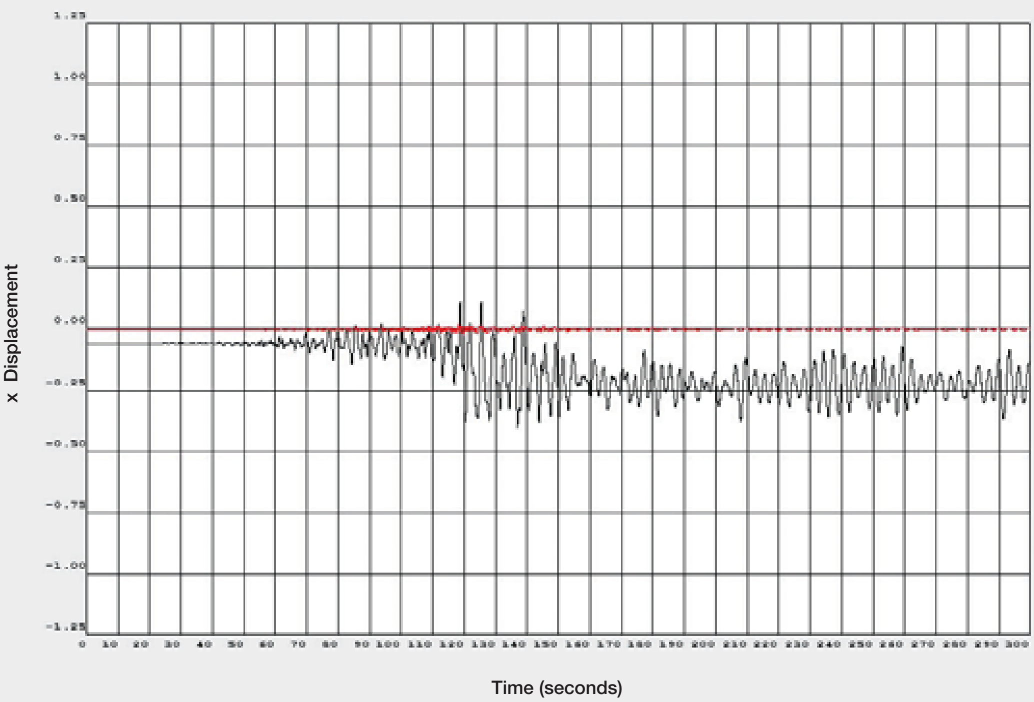


(b) The corresponding displacement at the top of the tower in the east–west direction

Figure 38: Composite REHS earthquake record (source: Athol Carr)



(a) Ground accelerations



(b) The corresponding displacement at the top of the tower in the east–west direction

Figure 39: Ground motion representing an Alpine Fault earthquake (source: Athol Carr)

Figure 38 shows the corresponding ground accelerations and displacement at the top of the tower for a composite earthquake comprising the September, Boxing Day and February earthquakes. It can be seen that the September earthquake had only a relatively small impact on the building in the east–west direction. This was due to the relatively small excitation in the east–west direction compared with that in the north–south direction[5]. The Boxing Day event had virtually no effect because the high frequency of ground motion did not excite a structure with a relatively long natural period of vibration. By far the greatest displacements were in the February earthquake, which as previously noted had a strong east–west component in the ground motion.

Figure 39 shows the corresponding ground motion acceleration record and displacement at the top of the building for an earthquake record that has been chosen to represent a potential Alpine Fault earthquake. It can be seen that the predicted behaviour is similar to that seen with the CBD earthquakes although the displacements are not as great. Owing to the distance of Christchurch from the Alpine Fault, the ground motion in an Alpine Fault earthquake is expected to be much less intense than that in the Christchurch earthquake sequence but of much longer duration. Clearly, a building with a lower lateral strength could have been much more seriously affected than in this analysis. This aspect needs to be considered given the very considerable reduction in design base shear strengths permitted in current and previous design Standards (NZS 1170.5:2004[3] and NZS 4203:1992[12]) compared with the corresponding values given in the 1976 and 1984 editions of NZS 4203[13,14].

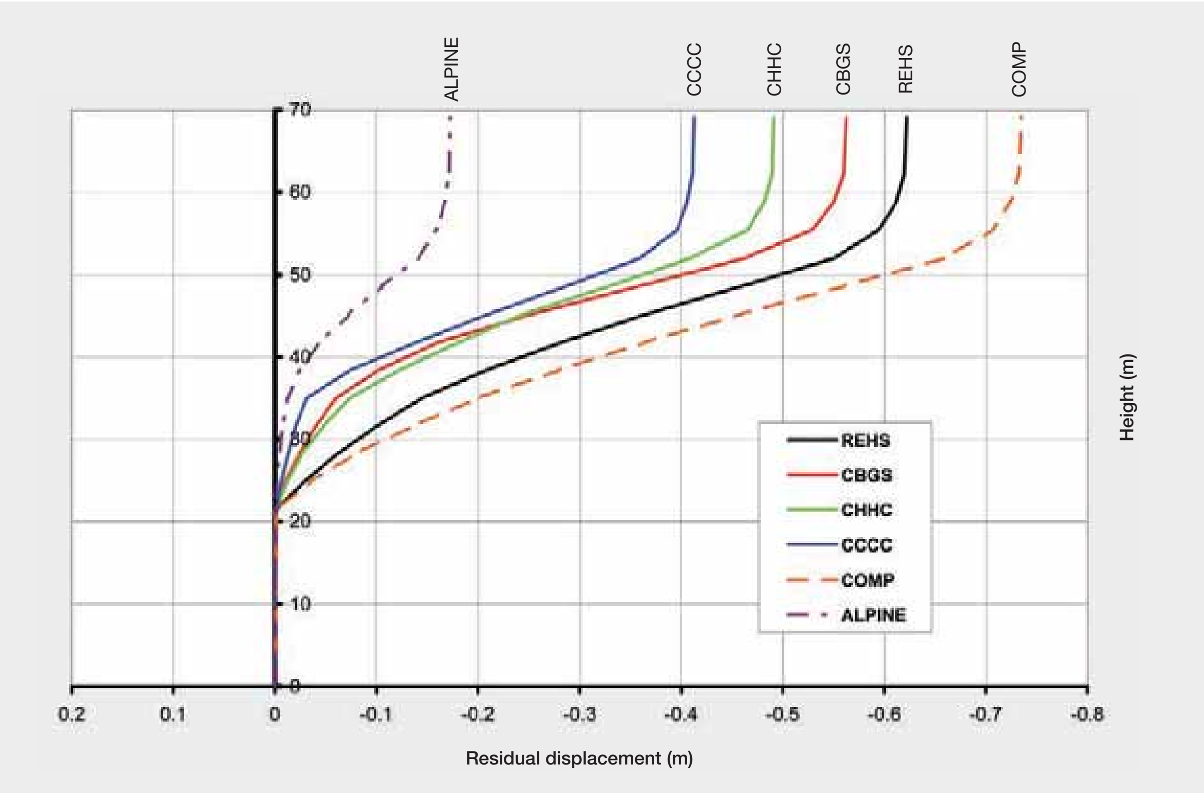


Figure 40: Residual displacement following earthquake ground motion

Figure 40 shows the residual predicted displacements over the height of the building after all motion has ceased for all the earthquake ground motion records. It can be seen that for the four CBD records the residual displacements were between 420 and 620mm.

Figure 41 shows the predicted peak inter-storey drifts for the different earthquake records. The four CBD earthquakes gave peak inter-storey drifts towards the east of the order of four per cent of the storey height for levels between 20 and 23. This would have been more than sufficient to cause the stairs in the HGC building to lose their support and consequently fail. The corresponding predicted peak drifts to the west were appreciably smaller and the stairs descending from the west to the east may have been damaged by being subjected to compression.

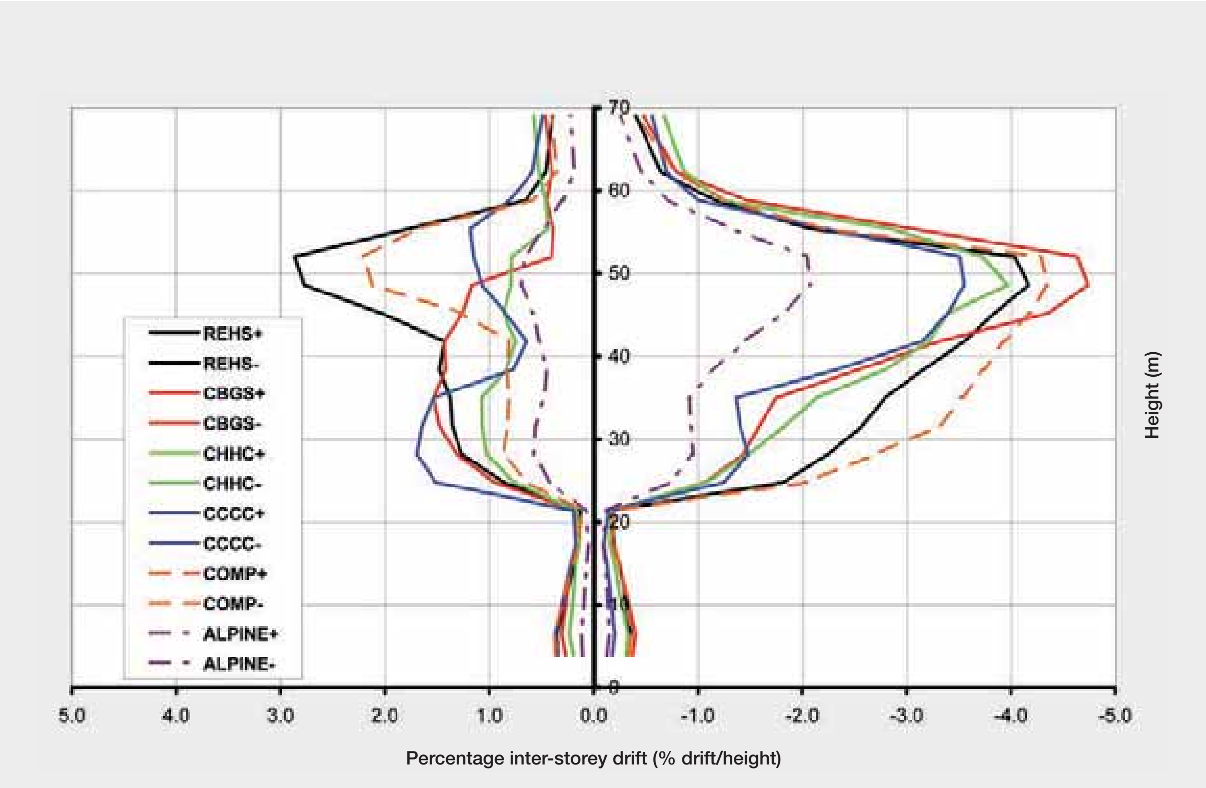


Figure 41: Residual displacement following earthquake ground motion

3.5.4 Failure of wall D5–6

The actions that determine the survival of a wall or column are the axial load level, the detailing for confinement and shear, and the displacements imposed on the wall. The Dunning Thornton report did not quantify the lateral displacements applied to the walls. The only lateral displacements recorded in their report are in the table on page 16 and in Figure 7. These values are not relevant to wall D5–6, which was located well away from the centre line of stiffness and lateral strength. Consequently, any torsional displacements induced on the wall were not reflected in the table or the figure.

Inspection of Figures B6 and B7 in the Dunning Thornton report (in Appendix B) shows that the centre of stiffness and strength near ground level is on the northern side of the major structural wall on grid line 8. It is likely to be close to the intersection of grid lines C and 9. The critical wall, D5–6, is about 14m from the centre of stiffness. Consequently, any torsional rotation of the building is likely to have imposed significant lateral displacements on the wall D5–6 (see Figure 42).

The building is likely to have a high torsional response near ground level for two reasons:

1. In the tower above the structural walls the centre of mass is appreciably eccentric to the centre of stiffness and strength and consequently any north–south motion will induce significant torsion in the building (see Figure 28).
2. The podium to the building effectively had a height of six floors, rising to level 14 (see Figure 17). The podium, measuring about 12m by 16m, is located on the southern side of the building between the approximate centre line in the north–south direction and the western edge of the tower (see Figure 42). The podium is supported by columns, which are laterally flexible compared to the walls. Consequently, a large part of the lateral resistance to earthquake forces from the podium would have been transmitted to the structural walls in the main part of the building, and that would have induced significant torsion in the structure. The resultant twist would have induced out-of-plane displacement in wall D5–6, which could be expected to play a significant part in the structural failure of the wall.

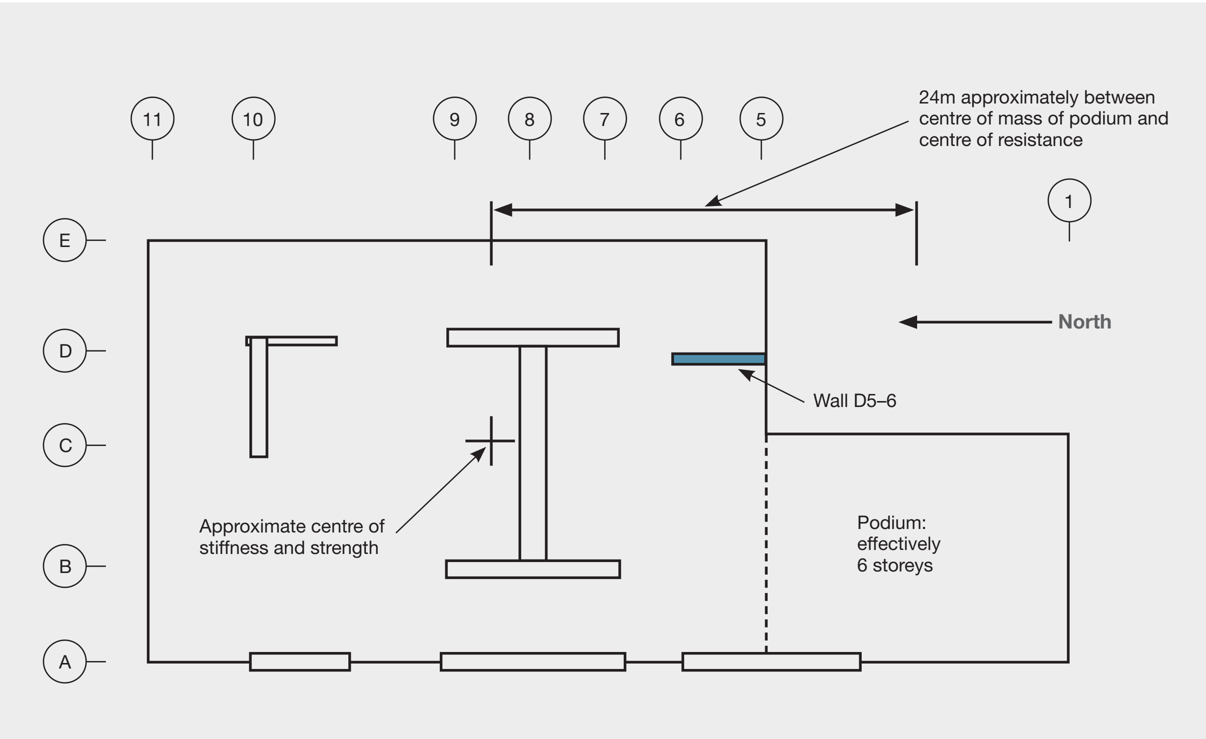


Figure 42: Location of podium to main structural walls at ground level

3.5.5 Transfer beams

Transfer beams located immediately below level 14 (see Figure 25) supported the gravity loads from the cantilevered portion of the building below level 14, which is the top of the structural walls that extended from the foundations to level 14. On grid line 8, the cover concrete in the transfer beam spalled close to where the gravity load was transferred to the transfer beam, exposing the stirrups at the mid-depth of the beam. In this location the stirrups consisted of 20mm U- shaped bars at a spacing of 200mm, which lapped at the mid-height of the beam (see Figure 27). When wall D5–6 collapsed the shock loading on the transfer beam would have been high and it must have caused the stirrups to be highly stressed. The splitting forces associated with bond in the lap length would have caused the cover concrete to spall (see the Figure in Appendix B, page 14, in the Dunning Thornton report). It is fortunate that complete collapse did not occur as this spalling would have greatly reduced the bond resistance of the stirrups and their capacity to resist shear in the transfer beam.

The beam was 600mm wide with a depth of close to 3600mm. The current Concrete Structures Standard, NZS 3101:2006[6] does not permit the use of lapped stirrups to resist shear in potential plastic hinge zones or where the shear stress exceeds 0.5 $\sqrt{f'_c}$ (clause 8.7.2.8). However, neither of these limits would have been critical in the design of the beam. The current Standard has limits in the spacing of stirrups across a beam section in the case where the beam is wide compared to its depth (clause 9.3.9.4.12). However, the criterion, as written, would not apply to the transfer beam situation.

To prevent the potential mode of failure observed in the transfer beam, it is necessary to prevent high bond stresses developing in closely spaced stirrup legs where U-shaped stirrups are lapped in cover concrete. It is recommended that where U-shaped stirrups are used only a proportion of them should be permitted to be lapped in the cover concrete, with the remainder lapped in the core concrete (see Figure 43). Alternatively, the stirrups could be lapped with hooked ends, where the hooks are bent into the concrete core in the beam. We recommend that the Standard be amended to require lapped U-shaped stirrups to comply with these proposals.

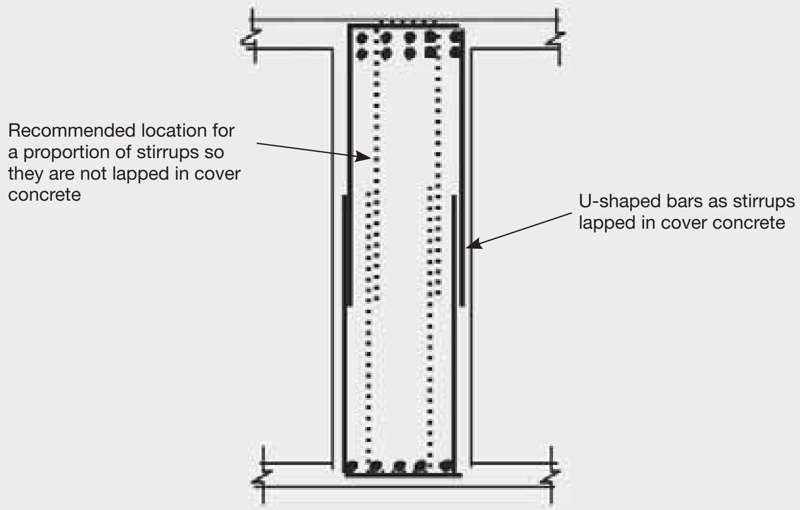


Figure 43: Location of U-shaped stirrups to avoid all being lapped in cover concrete

3.6 Discussion

The Royal Commission accepts the recommendations made by the Expert Panel. These are listed in section 6.11 of the Expert Panel report and in section 4.2, Volume 1 of this Report. However, we consider some further conclusions and recommendations are justified, as discussed below:

1. The HGC building was highly irregular in several ways. Two aspects of this irregularity were not identified in the Dunning Thornton report and at least one of these was not identified in the original design of the building. To identify these two aspects it is essential to have a clear understanding of the basic concepts of the dynamic behaviour of structures.

   (a) The first aspect arises from the eastern bay of the building being cantilevered off the structure to avoid closing Tattersalls Lane. As identified in section 3.5, this gives the building a tendency to sway to the east by reducing the lateral force resistance in this direction and increasing the lateral force resistance for displacement towards the west. The modal response spectrum and the equivalent static method of analysis are based on the assumption that the strength and stiffness of a structure are equal for both forward and backward displacement. The cantilevering action in the HGC building violates this fundamental assumption so the analytical results based on elastic methods of analysis are incorrect. The fact that this fundamental problem was not identified in the reports received by the Royal Commission highlights the need for structural engineers to have a clear understanding of the basic assumptions involved in seismic design. It is noted that the problem could have been avoided simply by changing the distribution of reinforcement in the beams to give the structure equal strength against lateral displacement in the eastern and western directions.

   (b) The critical design actions on walls consist of the axial loads, the bending moments and the lateral displacements imposed on the walls. It is necessary to have an assessment of all these actions to be able to design a wall or assess a wall's seismic performance. The torsional response of the HGC building is of particular concern in assessing the lateral displacements that were imposed on the critical wall. Much of this displacement is likely to have come from the seismic forces rising from the mass of the podium inducing torsion in the building.

   Where the modal response spectrum method of analysis is used in design, the practice is to sum the mass of each mode for the direction of shaking until the sum reaches 90 per cent of the mass of the building. The contributions from the remaining modes are discarded. It is possible in this case that the displacement contribution from the torsional mode associated with the podium was not included, leading to an under-estimate of the lateral displacement applied to the wall. A further complication, which has been drawn to the Royal Commission's attention by Professor Carr, is that where there are torsional modes, the sum of all the effective masses in all the modes may exceed the total mass of the building.

2. The axial load in a wall in a multi-storey structure cannot be accurately determined. The HGC building was highly indeterminate in all three dimensions. Bending moments in a wall, unless it is under a very high axial load, cause it to elongate, and this displacement can be restrained by surrounding structural elements, potentially significantly increasing the axial load on the wall. This behaviour has been observed in a large-scale test and found to have a dramatic influence on the seismic performance of the test building[14].

3. There is an urgent need to revise the provision in NZS 3101:2006 that deals with the stability of walls subjected to high axial loads. Until this revision is made it is recommended that rectangular walls subjected to calculated axial loads greater than 0.1 $A_g f'_c$ be proportioned so that the ratio of clear height between lateral supports divided by thickness does not exceed 10.

4. The provisions for shear reinforcement in beams in NZS 3101:2006 should be revised to limit the proportion of shear reinforcement that is in the form of lapped U-bars that can be lapped in cover concrete.

5. It is evident from the reports received by the Royal Commission on this building that there is significant misunderstanding by structural engineers of the relationship between design inter-storey drift and peak inter-storey drift. In addition, there is a lack of understanding as to why inter-storey drift values calculated from elastic-based methods of analysis need to be adjusted to allow for the change in form of the deflected shape profile caused by inelastic behaviour. This aspect should be more clearly identified in NZS 1170.5, where this effect is allowed for by the drift modification factor in clause 7.3 of the Standard.

Appendix A: Simplified model assumptions

Assumed properties

Property	Symbol	Value	Units
Concrete strength	f'_c	30	MPa
Concrete ultimate strain	ε_c	0.003	
H-section steel yield strength	f_{yh}	380	MPa
All other steel yield strength	f_{yd}	275	MPa
Concrete elastic modulus	E	25000	MPa

Loading	Symbol	Value	Units
Gravity floor load	DL + γLL	8	kPa
Columns line on grid lines 5 & 11 tributary area	TA	110	m^2
Equivalent external column width	W	7000	mm
Equivalent external column depth	D	1000	mm

Analysis assumptions for column strength and inter-storey shear

Refer to section 3.1, analytical model and Figure 29.

1. Nominal strengths have been calculated, for example no strength reduction factors have been applied for beams and columns.
2. As described in section 3.1, column 1 in the analytical model is given a shear strength in each storey, determined as described below.
3. The flexural strengths of the beams at each level were determined at the critical sections of the potential plastic hinges using standard flexural theory. These bending moments were extrapolated to the centre line of the columns. The sum of these column moments divided by the inter-storey height gives the storey shear strength based on the assumption that points of inflection occur at the mid-height of each storey.
4. As points of inflection may vary it is necessary to allow for any additional lateral strength that may arise at a level because of surplus strength in the column and the movement of the points of inflection.
5. The columns on grid lines 5 and 11, but excluding the columns on grid line E (see Figure 28, page 69) contribute to the lateral load resistance. In the analytical model these columns are represented by a single compound column, which is labelled as 2 in Figure 29, page 70.
6. The axial load assumed to act on the compound column, column 2, in the analytical model is based on the total tributary area supported by the columns represented by the compound column noted above at the level being considered and an assumed gravity load of $8kN/m^2$.
7. Column 2 in the analytical model is given a flexural strength equal to the value calculated for the compound column minus the corresponding bending moment associated with the storey shear actions in column 1. This is the surplus strength referred to above.
8. The critical moment in the compound column at each level is assumed to be at the mid-height between the face of the beam and the beam centre line.
9. The stiffness of the compound column was based on the section stiffness found from an elastic-based analysis at first yield of longitudinal reinforcement and in which the concrete was assumed to have no tensile strength. This stiffness was multiplied by 1.5 to allow for tension stiffening in the mid-storey region.

Analysis programme

The analyses were carried out using the two-dimensional version of Ruaumoko[11,15]. This programme was developed in the 1970s in the Department of Civil Engineering, University of Canterbury, for the analysis of inelastic buildings and bridges subject to earthquake and other dynamic excitation. Since 1990 it has been used by over 130 universities, building research institutes, highway authorities and consulting practices around the world for research, teaching and design. It has a very wide range of member models and inelastic and hysteretic representations to model numerous structural engineering systems. Further details may be found on the Ruaumoko website www.ruaumoko.co.nz

Damping model

The programme has a wide variety of damping models ranging from the simple but problematic Rayleigh damping found in most other programmes, to a variety of non-linear member damping models[16,17,18]. The model used for these analyses is that proposed by Wilson and Penzien[19], which allows the damping to be specified over all natural frequencies of free vibration in the structure. In the model used, the damping was specified at five per cent of critical damping at all frequencies of the structure.

Hysteretic models used for the inelastic structural members

The most commonly-used hysteretic model for reinforced concrete members is the Takeda hysteretic model[11,20]. This allows for degradation of the member stiffness as the member undergoes inelastic deformation. It was used for the shear members and column members in the structural model.

The first column in the model, representing most of the shear stiffness of the building, used a shear spring that was initially elastic for the lower levels and could yield using the Takeda hysteretic model for the post-yield behaviour.

The second, the flexural column members, used a Giberson beam member model[15] where plastic hinges using the Takeda hysteretic model were able to form at the ends of the column members.

The third, the P-delta column, used pin-ended struts that were tied to the other columns by stiff pin-ended links so that the P-delta actions on the structure would be readily available. If the P-delta column had been connected to the other columns by the computationally more efficient displacement slaving, then the P-delta actions could only be obtained by inference from the longitudinal forces in the P-delta columns and the inter-storey drifts of the P-delta columns.

Testing the HGC for an Alpine Fault event

Finding a ground acceleration record that could be used to represent what might be expected at a soft ground site in Christchurch for a magnitude 8 earthquake on the Alpine Fault is not easy. The best suggestion was to use a record from the magnitude 9 earthquake in Japan in March 2011, recorded on soft ground near Tokyo[21]. The magnitude of this earthquake is much greater than that for the Alpine Fault event, but the recording was made at a much greater distance from the epicentre than Christchurch in an Alpine Fault event. It was felt that these effects would, to an extent, cancel out.

The other point to note is that the 300-second duration of the record would also be longer than that expected from an Alpine Fault earthquake. In terms of the analyses, this final aspect did not matter as the analyses took only in the order of five minutes for the 300-second acceleration record. The magnitude of the accelerations is much less than recorded in the 22 February 2011 earthquake, but the longer duration of shaking could be important for structures whose strength and stiffness might degrade with increasing numbers of cycles of inelastic deformation. The 2011 Christchurch earthquake shaking was, fortunately, of very short duration.

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Section 4: Forsyth Barr building

The Forsyth Barr building at 764 Colombo Street is on the south-eastern corner of Colombo and Armagh Streets in central Christchurch, a short distance from Cathedral Square. In the 22 February 2011 earthquake, the main access stairs in the building collapsed. All stairs collapsed below level 14 in one stairwell and below level 15 in the other. People were trapped in the floors above.

Figure 44: Post-February view from Victoria Square

The discussion below covers:

- the history of the Forsyth Barr building prior to the 4 September 2010 earthquake;
- the September earthquake, the performance of the building in that earthquake and the actions taken as a result;
- the February earthquake and the failure of the stairs; and
- lessons the Royal Commission considers should be learned from this failure.

It reflects information gathered from a variety of sources including:

- the Christchurch City Council (CCC) as the regulatory authority administering building controls in Christchurch;
- Colliers International Property Management Ltd (Colliers), building managers at the relevant times;
- the Beca Carter Hollings & Ferner Ltd investigation into the failure for the Department of Building and Housing (DBH) (the Beca report[1]);
- DBH Expert Panel review of the Beca investigation (the Expert Panel report[2]);
- a review of the Beca and Expert Panel reports carried out on behalf of the Royal Commission by Mr William T. Holmes of Rutherford and Chekene; and
- evidence given and submissions made to the Royal Commission at a public hearing on 23 and 24 February 2012.

Figure 45: Fallen stair flights at the bottom of the stairwell

4.1 Original construction of the Forsyth Barr building

The Forsyth Barr building has 18 storeys and was designed in 1988 as a retail and office development. The developer was Paynter Developments Ltd and the building was sold on completion to Robt Jones (Canterbury) Ltd. It was originally called Robert Jones House.

The architectural design was carried out by Architecture Warren and Mahoney Ltd, with the final structural drawings prepared by Holmes Consulting Group Ltd (HCG). The contractor was Fletcher Construction Ltd.

A design certificate for the building was signed off on behalf of HCG by Mr R.A. Poole on 7 March 1998. The certification includes the following:

> I have exercised reasonable control over the design processes for the works defined above which have been designed in accordance with sound and widely accepted engineering principles to support the loads specified in NZS 4203:1984.
>
> I believe the stresses in the various materials of construction and force resisting elements of the structure including the foundation strata under the above loads are such as to ensure the safety and stability of the structure if the works are constructed in accordance with the above described drawings and specifications.

A building permit was issued by the CCC on 9 May 1988.

The building is founded on a shallow raft at a depth of around 2.5m below the ground floor level.

Both the Beca report and the Expert Panel report have concluded that there was no evidence that liquefaction or foundation failure played any role in the collapse of the stairs. The Royal Commission accepts those conclusions.

Lateral resilience of the building is provided by the frame action of the reinforced concrete beams and columns. For three storeys above the ground floor level, the floors extend beyond the footprint of the tower to form a podium on the south and east sides.

The building system for the tower is unusual in that it comprises two triangular portions that are reinforced concrete frames linked together by a precast concrete floor system. Figure 46 shows a typical floor plan and indicates some of the building features of original drawings. Figure 47 shows the lower floors and locates the tower in relation to the podium.

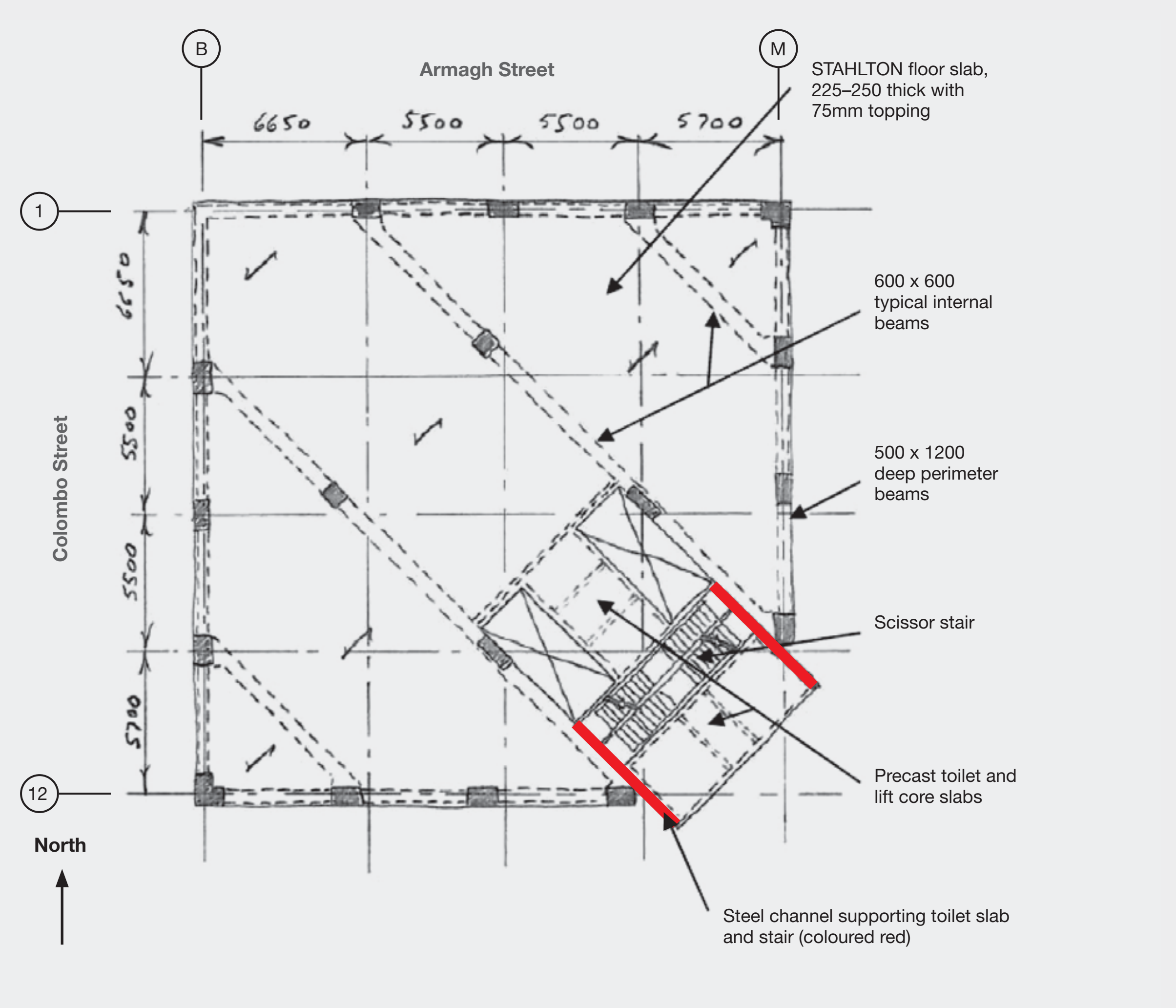

Figure 46: Typical plan above podium level (source: Beca report)

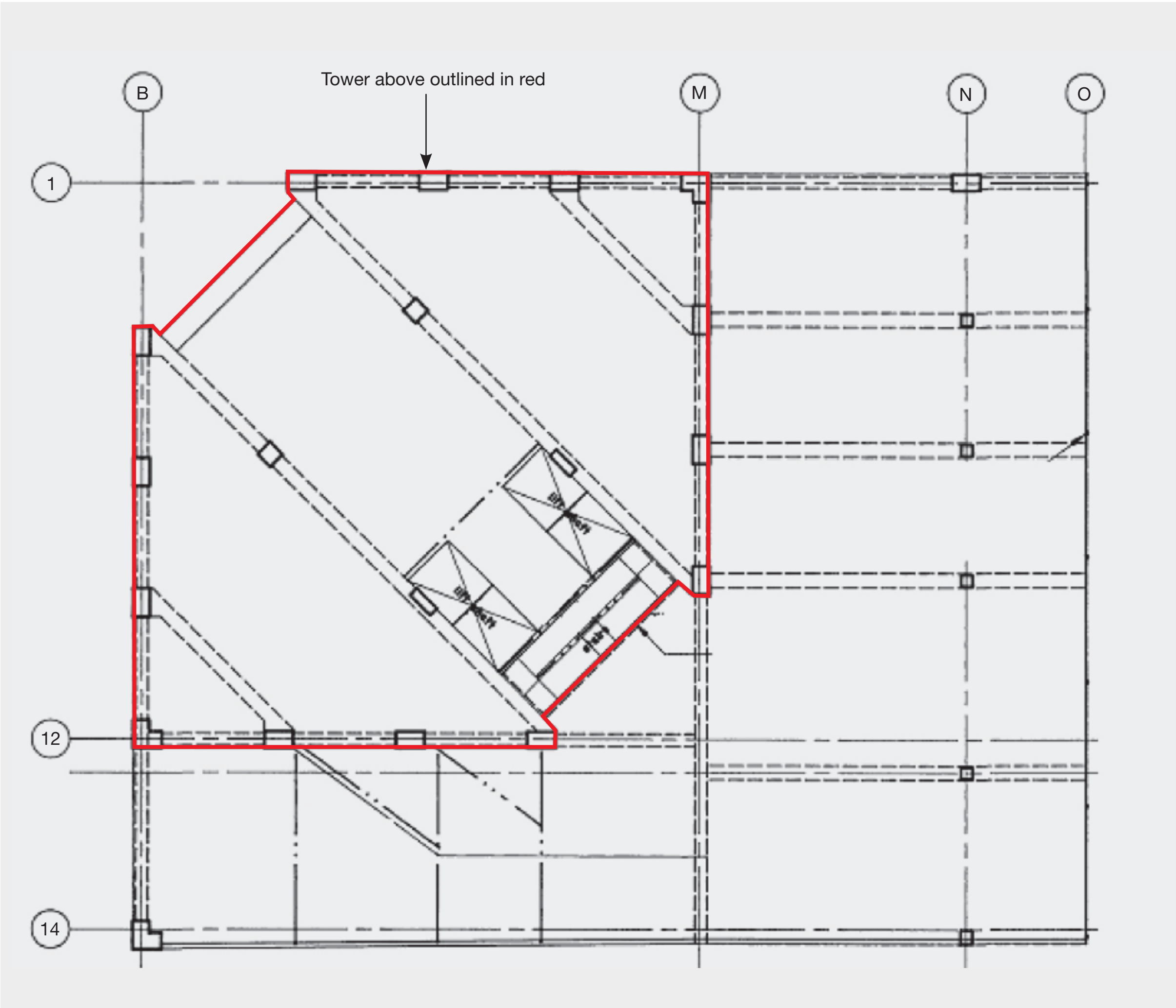

Figure 47: Lower levels showing podium (source: Modified from original construction drawings obtained from the CCC file)

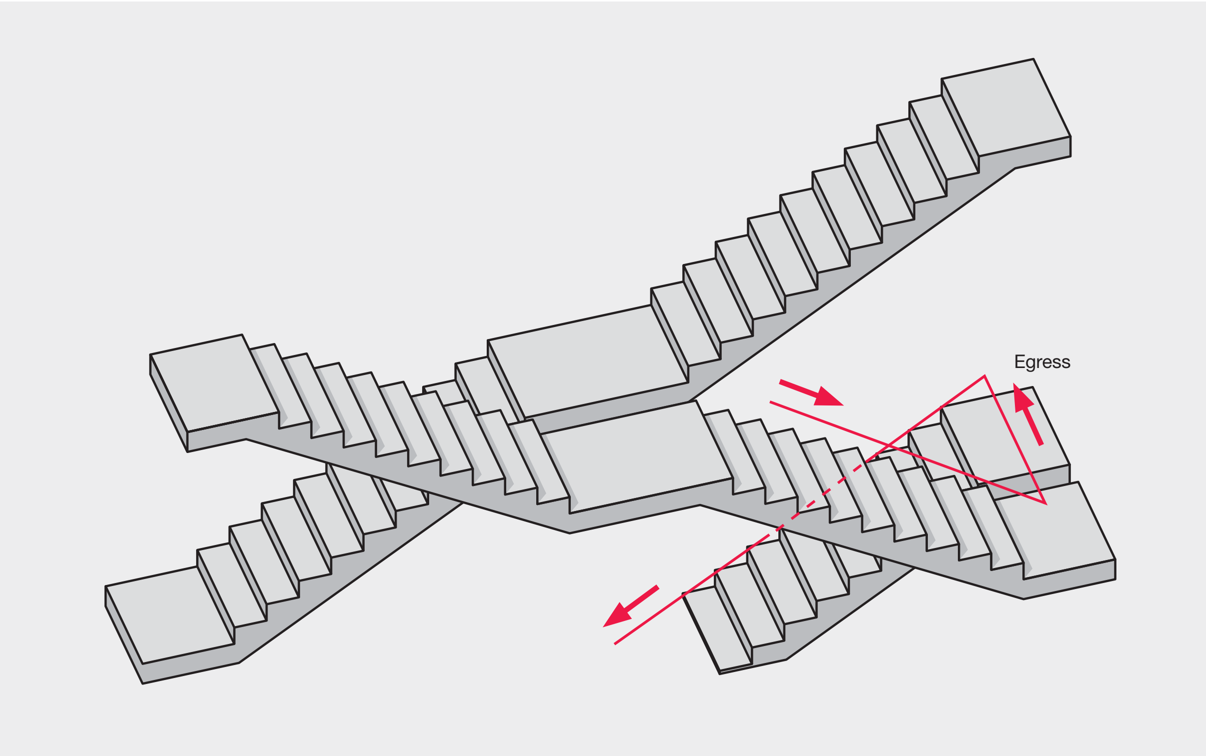

Figure 48: Example of scissor stairs (source: Beca report)

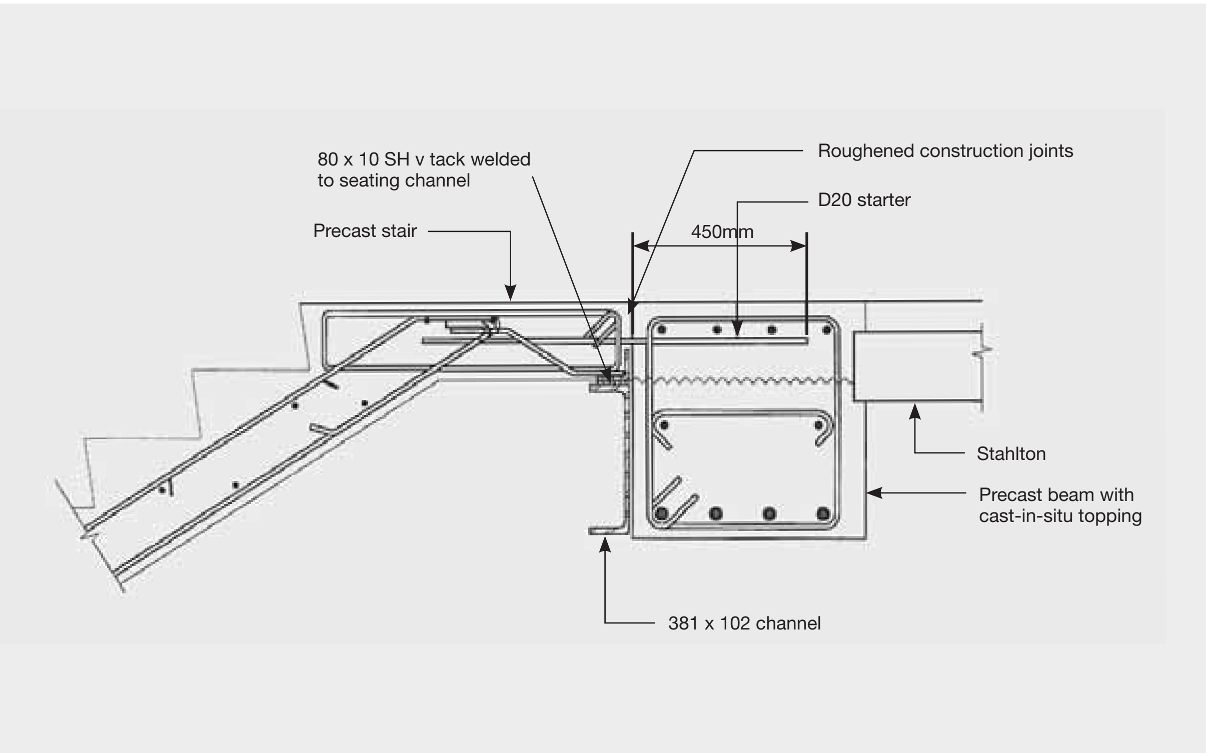

Figure 49: Detail at top of each stair flight (source: Beca report)

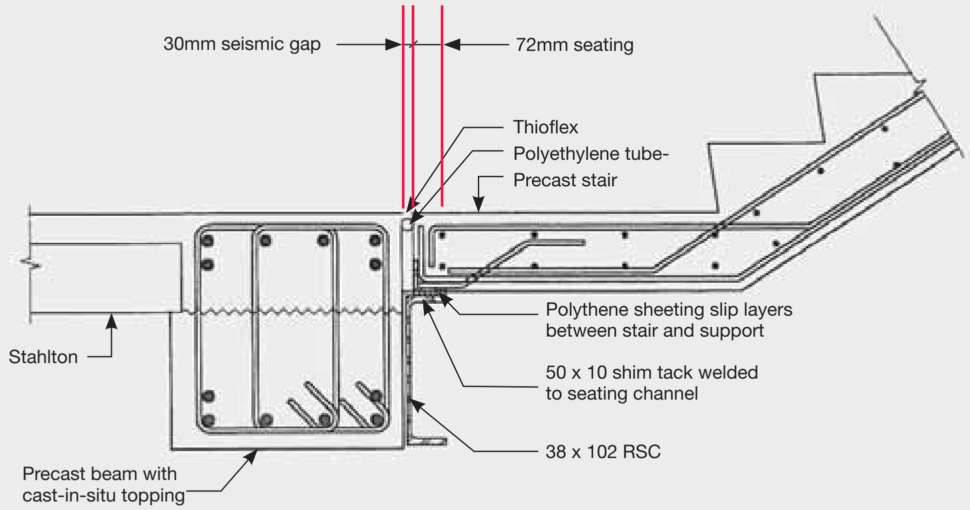

Figure 50: Detail at lower landing of stair flights (source: Beca report)

The stairs are orientated diagonally within the tower in a north-easterly to south-westerly direction. Figure 46 locates the position of the stairs, which are of the “scissor” type. In a scissor stair, two stairways are provided within a single shaft, separated by a light-weight partition in between and over the full height of the stairwell. Access is by winding up or down the stairwell, changing sides at each floor and passing under the flight of the other stair. Figure 48 shows the general arrangement. One of the risks associated with such a stair system is that if a flight collapses on one side of the shaft it may render both stairs impassable.

Most of the stair flights were precast concrete units, each cast as a single unit, not as separate flights, and connected by a cast-in-situ slab. Each flight was supported on a channel with a reinforcing bar cast into the supporting concrete beam at the upper landing, as seen in Figure 49.

At their lower landings, the stairs were seated on a steel channel with a horizontal gap specified as a 30mm seismic gap in Figure 50.

The steel channels also support the toilet slab, accessed off the landings (Figure 46, page 90).

There was evidence that during the course of construction, site instructions were given to the contractors about maintaining the seismic gap. However, Mr Paul Tonkin, site manager employed by Fletcher Construction Ltd during construction, gave evidence that he did not appreciate the critical importance of the seismic gap at that time.

4.2 Up until 4 September 2010

Between the time of the original construction of the Forsyth Barr building and 4 September 2010, numerous building permits, building consents, and exemptions from building consents were issued by the CCC. None of these approvals involved significant structural alterations of the building.

The building had a current building warrant of fitness at the time of both the 4 September 2010 and the 22 February 2011 earthquakes. One of the items on the compliance schedule and building warrant of fitness was an emergency lighting system, which was verified as being operative by Mr Russell Gracie (Independent Qualified Person #466) from Chubb New Zealand Ltd on 1 December 2010.

The current owner of the Forsyth Barr building is 764 Colombo Street Ltd.

4.3 The September earthquake

The nature and intensity of the September earthquake are described in Section 2 of Volume 1 of this Report.

The Forsyth Barr building is located at distances of 700–1350m from the four primary seismic measuring stations for the Christchurch CBD. The Resthaven Retirement Home site near Peacock Street (marked REHS in Figure 51) is the closest, and Beca used the ground motion records at this site.

In the September event, the predominant direction of the horizontal accelerations for buildings such as this, with an initial period of vibration of 2.0–2.2 seconds, was in the north–south direction. The Forsyth Barr building has similar strength and stiffness in both north–south and east–west directions.

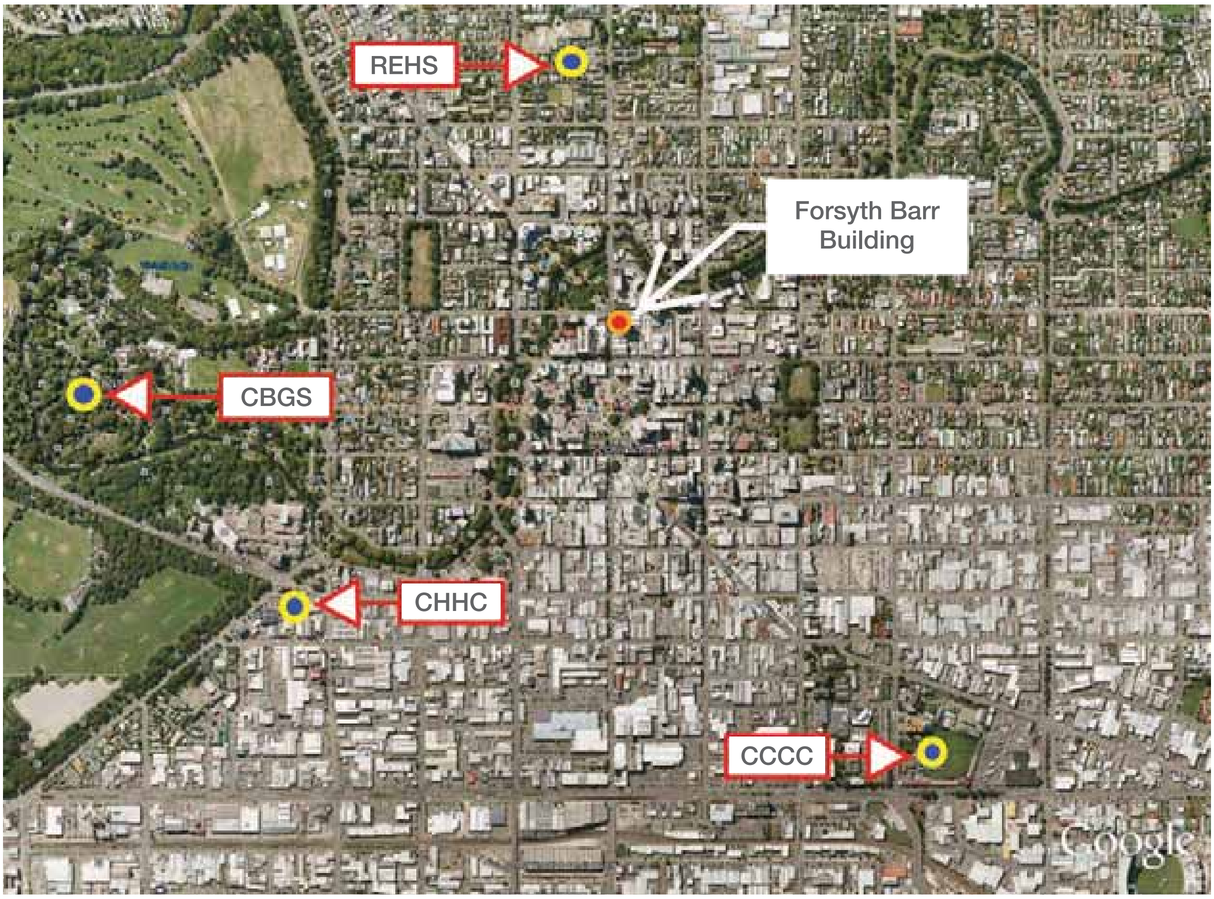

Figure 51: Location of Forsyth Barr building in relation to seismic measuring stations

4.4 Between the September and February earthquakes

As discussed elsewhere in this Report, after the September earthquake a state of local emergency was declared and the CCC (under guidance from DBH) initiated a civil defence emergency management response. As we have noted in our discussion of the failure of the PGC building, from 5 September teams were sent to all commercial parts of the Central Business District (CBD) to undertake Level 1 Rapid Assessments. These were exterior inspections to look for obvious signs of damage indicating immediate danger, or to determine whether further investigations were required before use of the buildings. A Level 2 Rapid Assessment is a more extensive visual inspection that includes the interior.

The Forsyth Barr building was the subject of a Level 1 Rapid Assessment on 5 September. As a result, a red placard was placed on the building, stating "UNSAFE – do not enter or occupy". Subsequent to this assessment, and on the same day, a Level 2 Rapid Assessment was carried out by Mr Peter Beazley and Mr Rob Jury of Beca, as instructed by the owner. This resulted in a yellow placard, marked "Restricted Use – no entry except on essential business". The classification was, "Y2– no entry to parts until repaired or demolished". The general concern was that the stairs had settled and might have become unstable.

In a report written after a further inspection on the following day, there was reference to "damage to the scissor stair around the landing area" and it was noted that a contractor had been brought in to assist with removal of the stair bulkhead on the level 7 landing. This was considered to be the most damaged stair, but the report said that the majority of the flights had similar damage. The report observed: "[Although] the deformations in the stairs are significant, we believe that the stairs still contain sufficient capacity for normal use." A steel beam under the car ramp was also identified as having a failure in a welded connection, and propping of this was recommended.

It is apparent that debris had been observed in the areas supposed to function as seismic gaps at the bottom landing of each flight of stairs. The report recommended that the debris be cleared from the gaps of "each stairflight to allow movement as originally intended".

The car ramp had been temporarily propped by the time the report was written, but access was restricted to pedestrians only (no vehicles). The placard was changed to green, "Inspected – no restrictions on occupancy or use" and, more specifically "G2 – Occupiable, repairs required". In evidence, Mr Jury expressed the opinion that it was not appropriate for a green placard to be withheld pending removal of the debris in the seismic gap. He said the stair flights did not appear to be significantly compromised as a result of the September earthquake and debris removal was something to be attended to by the building manager in due course. It did not make the building unsafe to occupy.

At about this time, although the exact date has not been provided to us, Pace Project Management Ltd (Pace) was engaged by Colliers to manage the earthquake repairs to the building. On 10 September Pace provided a quote to replace the vinyl on the stairs.

Two further inspections were carried out by Beca on 13 and 15 September. These are not relevant to the current investigation.

Beca expected to continue with inspections and assessment of the building. However, it had no further involvement from this time on. In mid-September the building managers engaged HCG to carry out further structural assessments.

On 8 October HCG provided a fee proposal that included as its first stage:

1. To complete a preliminary structural survey of the building to identify the general form and location of earthquake damage.
2. To complete a review of available documentation of the building to identify potential "hot spots" for more detailed investigation.
3. To coordinate with a contractor or maintenance staff to expose key structural members as required and/or commission testing if required for key elements.
4. To make an assessment of any strength reduction due to the damage and, if applicable, to estimate the remaining available strength of the building in terms of full code loading, in order to establish compliance with the CCC's 2006 Earthquake-Prone, Dangerous and Insanitary Buildings Policy[3], and to enable an informed decision to be made regarding future re-use.

The proposal also included some details of future stages should they be necessary.

On 12 October, Mr Andrew Christian of Pace wrote to Mr Michael Connelly at Colliers by email, reporting that the stopping on the stairwell was almost completed, and checking that this is what was expected.

On the same day, Mr Connolly sent an email to Mr John Hare at HCG saying:

> Please proceed with this report asap. Andy Christian of Pace has done a survey of the building so can advise on some areas of concern. I want to be sure the stairs are ok and fixed correctly. Please note some cracks were covered by the plasterer and these need to be double checked and probably fixed correctly.

On 1, 2 and 3 November HCG carried out the inspections that it relied on in the preparation of its report provided later that month.

On 3 November, Mr Hare wrote to Mr Connolly by email with an update. He expressed his opinion that the building had performed well and that it appeared to have suffered no significant structural damage. Mr Hare also said that HCG was happy with the repairs that had been completed and others that were ongoing.

On 4 November Mr Connolly responded to Mr Hare by email: “Thanks for this. I have concern about the apparent ‘drop’ in the stairs. I assume your report will cover this and the best way to repair”.

Mr Hare gave evidence that he did not read this email until some time after it was sent, although there were some telephone discussions about the stairs at about this time. Mr Hare stated that he did not consider the sag in the stairs was of immediate concern as “there was no sign of significant lateral drift of the primary structure that might have alerted HCG to significant concern”.

On 29 November, HCG presented a report titled “Forsyth Barr Tower Post-Earthquake Assessment and Repair Report.” The report was written by Mr Mark Sturgess, as Project Engineer, and reviewed by Mr Hare. This was a substantial report but it did not refer to any specific inspections of the stairs or details of damage to them. Mr Hare’s evidence was that an assumption was made that as Beca had already inspected the stairs and commented on them, the focus of the HCG report should be on the building’s primary structure. However, HCG had not seen the Beca assessments of the building as it had not been provided to them.

The only mention of stairs was in Appendix E of the report, which detailed the post-earthquake damage repair. In “1.7.1 Crack Damage”, the contractor is instructed to identify cracks to be repaired following the work detailed within that appendix, and to contact the engineer to arrange an inspection after preparation for, but prior to, epoxy injection or grouting. The stairs were one of the elements where cracks were to be repaired. This report was intended as a live document that would be updated and amended as work proceeded.

The Royal Commission has not been provided with any details or evidence of inspections after the 29 November report and there is no evidence of any inspection after the Boxing Day 2010 earthquake. In a written statement of evidence provided to the Royal Commission, Mr Christian stated that work necessary to replace the vinyl on the stairs was scheduled to commence in the week after the 22 February earthquake. Mr Christian said that his intention had been to advise HGC when the vinyl was lifted that the landings should be inspected further, and that this should take place before the vinyl was re-laid. Unfortunately, the 22 February earthquake intervened. It is unclear from the evidence that a closer inspection of the stairs would have exposed the inadequacy of the seismic gaps.

4.5 The February earthquake

The nature and intensity of the February earthquake are described in section 2 of Volume 1 of this Report.

The shaking was mainly in the east–west direction, with a significant vertical component. It was of a short duration, but had high accelerations and displacements because its epicentre was close to the CBD.

When the earthquake struck, the Forsyth Barr building was evidently subjected to strong accelerations in the east–west direction, resulting in rapid failure of the scissor stairs. The reasons for failure and the likely sequence of events are addressed later in this Report.

As the failure occurred in the middle of a working day, many people were trapped in the upper floors. Fortunately, no one fell down the remains of the stairwell (a significant risk, as both the main and emergency lighting systems to the stairwell failed so the only light available was when a door to the landing was opened). Evacuation of the occupants required lowering a number of people on ropes to the podium roof to the east, and the rest by crane some hours later. The Royal Commission was impressed by the account of the evacuation given in evidence by Mr Grant Cameron, a solicitor who practised on the sixth floor of the building, and we consider it worth setting this out in full:

At the time of the earthquake on the 22nd of February 2011, I was sitting in my office talking with Shaun Cottrell. He's one of our associates. It was immediately a lot more violent than anything we had previously experienced and I crouched forward in my chair pondering whether to jump under my desk. As I leaned forward I noticed a very large book case beginning to fall from the wall behind Shaun and although I thought it was going to hit him, I didn't have a chance to yell a warning because we were consumed by a tremendous noise and all the violence of the earthquake. Fortunately the bookcase missed Shaun but all of my other furniture and belongings crashed to the ground and we could hear furniture falling all around the office, women screaming and there was general chaos.

Although my office was positioned on the Armagh Street frontage immediately adjacent to our boardroom which in turn is situated right on the corner of Colombo and Armagh Streets, the interior wall of my office was glass and so I could see clearly into the interior of the firm. My wife Ilze is the office manager and from the outset I could see her standing by her desk with her eyes and mouth wide open in obvious astonishment but strangely with thick clouds of dust swirling around her. Later we discovered that these clouds were formed by concrete dust from the collapsing stairs being blasted back into our suite through the air conditioning ducts.

Shaun and I clambered over furniture, files and other debris and rushed out into the main body of the office. I began calling for the staff to all come down to the Board Room where we could start a head count and get ourselves sorted out. Naturally there was a lot of concern and people were quite upset. I should add there that I also sent a staff member next door, the adjacent tenancy was the Ombudsman and we got all of their staff into our Board Room as well.

As we returned to our Board Room David Maclaurin, one of our solicitors, came into the reception area from the direction of the lift wells. I noticed half of his shirt was hanging out of his trousers and he exclaimed that he had been in the toilets when the quake struck. He looked utterly shocked and then blurted out, "You won't believe it but the bloody stairs have collapsed." It took me a moment to register what he was saying but he was quite insistent that the stairs had completely disappeared. He also commented that "half the bloody landing has gone as well" and then described having to carefully clutch to the walls around the sixth floor landing in the dark in order to get from the toilet back into our offices.

Suddenly we realised that we could be trapped. Two or three of us then ran round to the corridor on the south-west side of the lift wells to see if the stairs leading away from the landing beside the ladies' toilets were in place. However, the internal door between the corridor and the stairwell landing was jammed shut with a lot of rubble behind it. It took quite a few shoulder shoves to slowly push it open and even then we probably only got it open 12 to 15 inches. There was about 18 inches of concrete rubble jammed up behind it. As I put my head through the now partly opened door I could see that all the stairs had disappeared, as had the dividing wall between the stairwells. There was just a gaping hole stretching down through the middle of the building with blackness both above and below.

There were other people standing on other levels both above and below who had also opened the same doors on their respective landings and so there was a little bit of light shining in from behind these various doors and just enough for us to all take in the damage. It was now plain that everybody was trapped on their respective floors.

This reinforced my view that the big risk factor was fire. With all the stairs gone there had to be a real risk that electrical fittings would have been damaged or destroyed and at the same time there was a good chance that the fire hydrants might not operate because the plumbing to those may also have been damaged.

We returned to the Board Room and had a very quick talk about the options. I suggested to everyone that we probably had enough electrical extension cords in the office to provide ourselves with a form of rope whereby perhaps we could lower people to the carpark on the eastern side of the building. On that eastern elevation the car park extended up for three floors from ground level and jutted out from the main tower block. Our office overlooked that carpark and as the distance from our floor to that carpark was about 30 feet I was reasonably sure that we would have enough extension cords to come up with a solution. If we could lower staff to that level they could either then run down the car park ramps to the street or if they were damaged they could escape over rooftops on the eastern side of the building.

The staff quickly began retrieving extension cords from around the office and I set about tying reef knots to link them up. The first cord formed from two such extension leads would probably have been of the right length but other such ropes would likely require at least a couple of joins. As I was busy with this exercise one of the secretaries from the Ombudsman's Office grabbed my sleeve and told me there was a Civil Defence cabinet situated at the back of their office. I asked her what was in it but she didn't know and so a few of us rushed around to find a large steel cabinet with double doors situated in the back corner of their office. Upon opening it we found there were several coils of rope, quite a few sets of gloves and, to my great surprise, a sledgehammer. We grabbed these materials and shot back around to our boardroom.

I then explained to all our staff that we had a simple choice. We could stay where we were and wait for some form of rescue or we could attempt to escape down the side of the building. To await rescue necessarily meant some sort of crane being found and we had no way of knowing if and when such a crane might be available. After all it was plain to all that this earthquake had been very serious and emergency services would have many other priorities right at that time. I should add there that from our offices we could see the PGC building flat on the ground and we could also see the smoke coming from what later proved to be CTV. Also as we were experiencing some nasty aftershocks and given that the stairs had collapsed we couldn't be sure how secure the building might be. Although there didn't appear to be any column damage we had no way of knowing if the building had been seriously weakened.

I explained if there was a fire we may have very limited time to react and described how we intended to use the ropes we'd just found and the unanimous view was that we should attempt to leave the building. We then jammed a desk into an office doorway near the window through which we intended leaving. Once that had been positioned and all furniture was cleared away from our departure point the relevant window was quickly removed with a sledge hammer and we organised two or three males on each rope and having been a mountaineer John Haines from the Ombudsman's Office took responsibility for tying the two ropes, or tying two ropes around each person. I called for volunteers, Jai Moss one of our associates stepped forward. I asked him to remain in the car park level so that he could help others following untie the ropes. He was happy with that. He was safely lowered to the car park. I called for further volunteers but when nobody moved my wife stepped forward. She too was lowered without incident and at this point the others began to realise that this was quite a safe exit methodology. So this [Figure 52] is a photograph showing the exercise.

The photograph depicts David Maclaurin on the right-hand side, my wife about to go out the window, my head's just behind, about to push her out and you can see the – but you can see the vehicle situated on the car park below.

MR MILLS:

Q. Who's in the lower window?

A. That was another office. I can't remember exactly who was in that particular one.

Q. That's a different operation from yours?

A. Different operation and when the cranes arrived were able to assist them and there was the Japanese ambassador may have been, I think, on the third one. You can see another window missing there as well.

WITNESS CONTINUES READING BRIEF OF EVIDENCE

Over the next 90 minutes or so we were able to lower about 15 people. Ultimately, only three of us remained when two cranes arrived on the scene. One was positioned at the eastern end of the building, that's in Armagh Street, and the operator advised he could soon uplift us in a cage. So we used the short interval while he was setting up to lower as many computers and other equipment that we could and shortly after 4:00pm the remaining three of us were lowered to Armagh Street in that crane.

The Royal Commission has been assisted in its understanding of the failure of the stairs by:

- the Beca report;
- the Expert Panel report; and
- a review of both of the above by Mr Holmes prepared at the request of the Royal Commission. Mr Holmes indicated his agreement with the conclusions in both the Beca and Expert Panel reports, and we do not need to discuss his review further.

Figure 52: Post-February evacuation. Mr and Mrs Cameron are in the top right broken window

4.6 The Beca investigation

The findings of the Beca investigation were presented to the Royal Commission at the hearing on 23 February 2012 by Mr Robert Jury (the author of the Beca report) and Dr Richard Sharpe, both from Beca. The collapse scenario that they inferred is illustrated in Figure 53.

The Beca report contained the following as the executive summary:

In our opinion:

Original Design

- The stairs as designed met the 1988 design requirements for the prescribed earthquake loads.
- The precast stair units in the tower were cast into the floor at their upper levels, and free to slide horizontally, within limits, at their lower ends.
- Testing of concrete and reinforcing steel from some elements after the collapse did not indicate that they were less strong than required by the design.

Modifications

- We have viewed the stairs removed from the building after the 22nd February 2011 earthquake.
- We have inspected the seismic gaps at the lower landings of stair units still in place at Levels 14, 15 and 16.
- It would appear that the stair units were precast as one unit rather than two flights interconnected with a cast in situ concrete mid-height landing. It is considered unlikely that this change had any effect on the collapse.
- There is evidence of modification to the lower end of at least four stair units that may indicate the prescribed seismic gap at that end was not achieved in all cases during construction.
- Repair of the floor coverings in the seismic gap areas of the landings was underway at the time of the 22nd February 2011 earthquake. No evidence that these repairs had an impact on the stair collapse has been identified during this investigation.

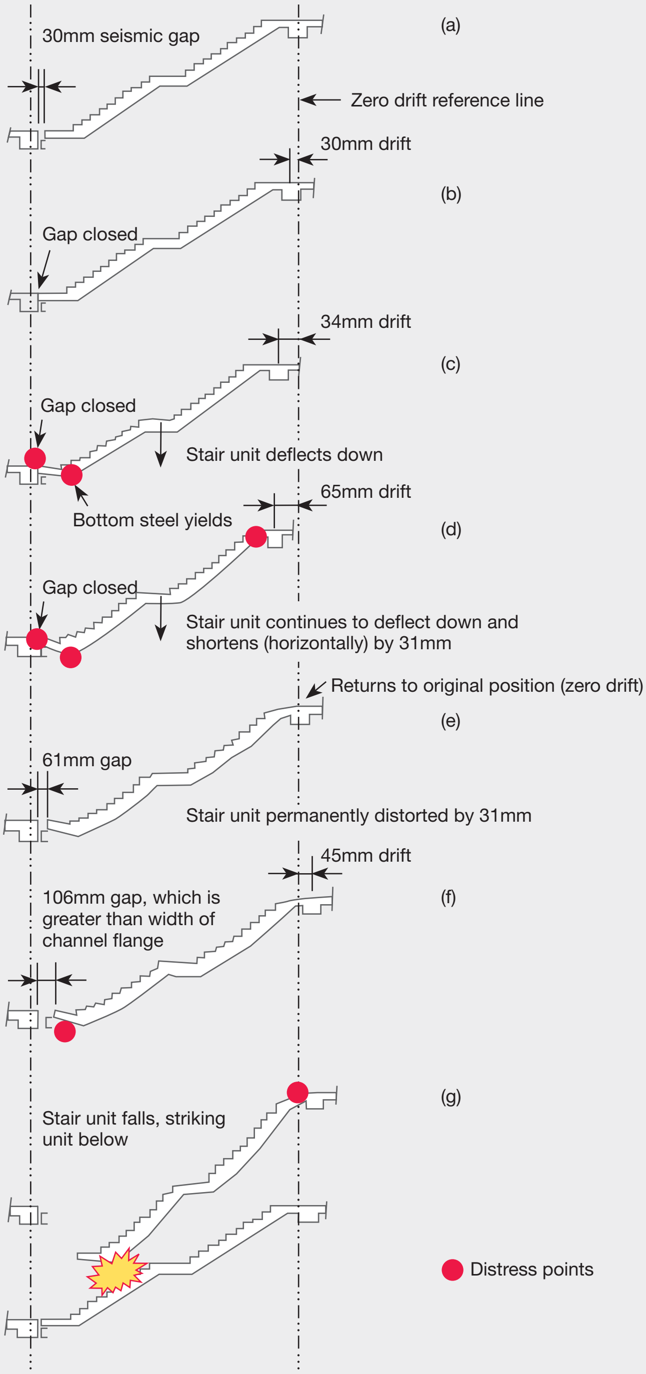

Figure 53: Inferred collapse sequence (source: Beca report)

Comparison with Current Code

- The clearance requirements for stairs from structure are essentially the same as was the case in 1988.
- The seismic gap provided in the original design would not meet current requirements by a factor of approximately 1.2.

Damage prior to 22nd February 2011

- Damage to the stairs and the structure was observed and/or reported after the 4th September 2010 earthquake as follows:
  - Cracking and vertical displacement in some of the stair units and to the floor coverings at the landings.
  - Cracking in the main structural frame members.
  - Failure of a weld in the region of a carpark ramp.
- Inspections of the most-damaged stair units carried out immediately after the September earthquake did not indicate that there had been any significant movement at the lower support.

Damage after the 22nd February 2011 Earthquake

- The main stairs from the ground to Level 15 (on one side) and ground to Level 14 (on the other side) collapsed, bringing with them the light-weight wall between them in the stairwell.
- The upper part of a column supporting the south-east corner of the podium roof was significantly damaged.
- We have not inspected the interior of the building other than at Levels 14, 15 and 16, but we have sighted two reports dated 31 March 2011 and 13 April 2011 that have been prepared by the owner's engineer that describe the extent of damage to the structure.
- Our interpretation of these reports is that the damage to the structure is relatively minor.
- Laser scanning of the north and west facades of the building does not indicate any significant permanent distortion of the structure.
- The removal of the collapsed stair units necessitated cutting them in half at their middle landings, and no records are available of which units were already broken/damaged at their mid-height landings or from which levels the various pieces originated.

Mode of Collapse

- The sequence of the stairs collapsing has not been determined. It seems likely that the uppermost stair units collapsed first, possibly progressively spearing the units below.
- Interviews with occupants suggest that all the stair collapses occurred during the main shock over a short period of time.
- It is likely that support at the bottom landing of one or more units was lost first, allowing the unit to pivot downwards about its upper end which was cast into the upper landing. In most cases, the cast-in reinforcing steel at the upper landing has yielded and then snapped, presumably allowing the stair unit to fall down the building in a near vertical attitude. We have been advised that at least some of the units did not detach from their upper connections and were left hanging in the stairwell until removed by USAR.
- On any one unit, the lower seating support could have been lost for one of (or combination of) three reasons:
  - A stair flight has been compressed, resulting in bending downwards and yielding of the reinforcement, because the seismic gap was smaller than needed in the earthquake of 22nd February 2011. The resultant permanent shortening of the flight was sufficient for the lower landing to fall off the steel seat on the reversal of the relative motion. Analyses completed by Beca indicate that inter-storey displacements (drifts) were likely to be highest between Levels 10 and 14.
  - The lower stair landing failed in shear when the unit was subjected to compression after the seismic gap was closed.
  - The effective horizontal length of the flight was shortened when struck by the flight above after the flight above lost its seating and rotated downwards about its upper landing. The consequent V-shaped lower flight would drag its lower landing off its seat.
  - A free-falling stair unit simply "pole-axed" the still-intact flight, causing it to fail catastrophically and fall.

Reasons for Collapse

- The damage observed and/or reported after the 4th September 2010 and 26th December 2010 earthquakes is not considered to have significantly weakened the stairs to make them more vulnerable in the 22nd February 2011 earthquake.
- The actual seismic gaps at the bottom landings were too small for the earthquake shaking experienced on 22nd February 2011.
- The stair units were not designed to resist compression that would arise from the closing up of the seismic gap.
- The characteristics of the lower seat did not allow any latitude if the building inter-storey displacements in an extreme event were such that they exceeded the gap provided.
- Construction tolerances and the possibility that the seismic gap at the lower stair support had been filled (construction debris or mortar), would have reduced the level of building horizontal displacement required to fail the stair.
- Our analyses predict that the stairs would have collapsed even if the gaps were clear of obstructions.

Commentary

- The seismic gap specified on the drawings met the prevailing design standards at the time the building was designed.
- We have been unable to definitively establish whether the specified gap was provided everywhere, and whether there was construction rubble/dirt/mortar in the gaps that would have reduced their effectiveness.
- The specified gap would have not been sufficient to avoid compression if the current Code derived displacements had been applied.
- There is evidence that the available seismic gap was not large enough to prevent some stair flights being compressed and slightly damaged during the 4th September 2010 earthquake.
- The specified gap was sufficient for the shaking experienced in the 26th December earthquake.
- The owner’s structural engineers inspected the building after the 4th September and 26th December earthquakes, and advised the owner that it was acceptable to occupy.
- General instructions had been given after the 4th September earthquake for any cracks over a certain size to be repaired by injection of an epoxy mortar. No evidence could be found to suggest that vertical accelerations (or response of the stair over its length) experienced in the 22nd February earthquake caused the stair failure.

Recommendation

- Known alternatives to the seismic gap detail used in this building should be used on all new buildings, and for replacing the stairs in this building. These alternatives minimise significantly any likelihood of the stair collapsing because of insufficient displacement allowance.
- DBH should issue an advisory note, warning of the potential issues and lack of resilience with the gap and ledge stair detail for new and existing buildings.
- Consideration should be given to including a provision in the Building Code requiring clearances and seatings for stairs to be capable of sustaining a nominal drift of twice that estimated for the Ultimate Limit State (ULS), after allowances for construction tolerances.
- The concept that a specified seismic gap must not be compromised under any circumstances should be promoted.

4.7 DBH Expert Panel review

The Expert Panel report concurred with the conclusions of the Beca report.

The findings were presented to the Royal Commission at the hearing on 23 February 2012 by Professor Nigel Priestley, one of the members of the Expert Panel.

The conclusions and recommendations of the Expert Panel were set out in paragraphs 8.12 and 8.13 of its final report, as follows:

8.12 Conclusions

Although the seismic gap at the lower stair support met the code of the day, it was too small for the aftershock event of 22 February 2011. There is also evidence that the available seismic gap was not large enough to prevent some stair flights being compressed and slightly damaged during the 4 September 2010 earthquake. The specified gap was sufficient for the shaking experienced in the 26 December 2010 aftershock.

The seismic gap specified on the drawings met the design standards prevailing at the time the building was designed. The specified gap would not have been sufficient to avoid compression if the current (2010) code-derived displacements had been applied.

When comparing the stairs as constructed in the Forsyth Barr Building with the current code, it was found that the original design would not meet current requirements (introduced in 1992) as the 1988 design requirements for clearance between stairs and structure would only be 80% of current requirements.

It could not be definitively established whether the specified seismic gap was provided everywhere, or whether there was debris, mortar or polystyrene in the gaps everywhere, which would have reduced the effectiveness of the gap. Despite the presence of extraneous material in the spaces intended for seismic movement, indications are that the stairs would have collapsed even if this material had not been present and the stairs had been fully free to move.

There was no evidence found in the investigation that indicated that repairs that were underway to the stair coverings prior to 22 February 2011 had an impact on the stair collapse.

The fact that the stairs had been precast as one unit, rather than as two separate units to be connected at mid-height landing, was not considered to have been likely to have had any effect on the collapse.

No evidence (physical or analytical) could be found to suggest that vertical earthquake motion (or response of the stair over its length) experienced in the 22 February 2011 aftershock caused or significantly contributed to the stair failure.

8.13 Recommendations

Following the investigation of the Forsyth Barr Building stairs and subsequent discussions with the Panel, a number of issues have arisen that the Department should give consideration to:

- **Alternatives to seismic gap detail**

  Known alternatives to the seismic gap detail used in this building should be used on all new buildings, and for replacing the stairs in this building. These alternatives minimise significantly any likelihood of the stair collapsing because of insufficient displacement allowance.

- **Advisory note for gap-and-ledge stair detail**

  An advisory note that warns of the potential issues and lack of resilience with the gap-and-ledge stair detail for new and existing buildings should be issued.

- **Building Code provision for clearances and seatings for stairs**

  A provision should be included in the Building Code requiring clearances and seatings for stairs to be capable of sustaining at least twice the Ultimate Limit State (ULS) inter-store displacements, after allowances for construction tolerances.

- **No compromise on seismic gaps**

  The concept that a specified seismic gap must not be compromised under any circumstances should be promoted.

4.8 Discussion

In the February earthquake there was extensive damage to stairs in a wide range of buildings and in a number of cases the stairs collapsed. The failure of the means of egress from several multi-storey buildings caused the public considerable concern. The Forsyth Barr building was a relatively modern building in which an extensive collapse of stairs trapped people in the building for a number of hours. The public concern raised by this issue was one of the reasons why the performance of the Forsyth Barr building was one of the buildings specifically named in the Royal Commission's Terms of Reference as part of the representative sample, and why the seismic performance of the Forsyth Barr building was assessed in detail by DBH. We record that, apart from the failure of the stairs, the building's structure performed well in the earthquakes and sustained little damage.

4.8.1 Cause of collapse

The Royal Commission accepts the conclusion given by Beca as to the cause of the collapse of the stairs in the Forsyth Barr building in the February earthquake and supports the recommendations related to the performance of stairs in multi-storey buildings made by the Expert Panel. These have been reproduced above. However, we note that in our Interim Report we made more conservative recommendations on the inter-storey drift that stairs should be designed to sustain. In particular, we note that the Expert Panel recommendations make no allowance for loss of seating caused by elongation of beams, an effect that can be significant. We also note that there are two references dealing with the design and assessment of stairs for new and existing buildings, "Report to the Royal Commission – Stairs and Access Ramps between Floors in Multi-storey Buildings"[4], and a report that is being prepared for DBH by the Engineering Advisory Group and will be available later in 2012[5]. The second reference, which we have seen in draft, contains a comprehensive treatment of design and assessment of stairs in multi-storey buildings.

We highlight four other matters:

1. Critical importance of the seismic gap

In the light of the evidence of Mr Tonkin, as discussed above, we record that it is very important that contractors be aware of the critical importance of the seismic gap specified on construction drawings, and that this be kept clear of extraneous materials at all times. In the present case it was unclear whether the full gap was, in fact, allowed for each flight in the construction process, and it may be that the weight and overall dimensions of the precast units (which were cast as a single unit before being manoeuvred into place) made their precise positioning difficult.

2. Maintenance of seismic gap

We consider that it would have been desirable to remove the debris observed when the building was inspected after the September earthquake, as a matter of urgency. We accept, however, that strict adherence to the original design, and timely maintenance of the specified gap to be free of debris would have been unlikely to prevent the collapse.

3. Emergency lighting

It is of concern that, as noted by Mr Cameron, the emergency lighting system failed in the stairwells as a result of the shaking in the February earthquake. We consider that multi-storey buildings should be equipped with emergency lights that are activated when power is cut to the normal lighting system, without the need for communication or power delivery by wires that might be vulnerable to local explosions, fires or material falling during earthquakes.

4. Analytical method

There were a number of points made in the hearing about the robustness of the analysis of the structure of the building. While these do not affect the conclusions reached in the Beca report, it is important that potential weaknesses in the approach should be identified so that other structural engineers, who may be following the same approach to the assessment of other multi-storey buildings, are made aware of the potential shortcomings.

In the assessment Beca applied the earthquake record obtained at the REHS site to its analytical model of the building, using an elastic time history method of analysis. The authors of the Beca report were asked a number of questions about the choice of this earthquake ground motion record in their analyses. Answers to these questions were given by Mr Jury, and Professor Priestley also responded on this issue.

The questions from the Royal Commission concerned the choice of the REHS earthquake record and approximations inherent in the method of analysis. In particular, it is noted that of the four earthquake records obtained in the CBD, the displacement spectra for the REHS site stand out as inducing appreciably greater displacements than the others in the period of interest, which is 2.0–2.5 seconds. The second point related to the underestimation of inter-storey drifts that occur when elastic methods of analysis are applied, compared to inelastic time history analyses. In the design of new buildings an allowance is required for this effect by application of the drift modification factor (NZS 1170.5 clause 7.3) when the design is based on elastic-based methods of analysis such as the modal response spectrum and elastic time history methods.

Mr Jury answered that the REHS record was chosen as it was the closest to the site of the Forsyth Barr building and there was no specific information available to indicate that the foundation soils were significantly different from those at the REHS site. Subsequently, Professor Priestley indicated that the analysis would have been more robust if the analysis had been repeated for other earthquake records available in the CBD. He made the additional point that in assessing inter-storey drifts allowance should be made for inter-storey displacements associated with both first and second modes of response. The elastic time history analysis does allow for this effect.

Professor Priestley also agreed that elastic time history analyses are likely to underestimate inter-storey drifts because of a change in the deflected shape profile that occurs as a result of inelastic deformation. He noted that his estimates gave displacement ductility values of the order of 2–3. He pointed out that the inelastic deformation associated with this level of ductility could be expected to increase inter-storey drifts compared with those obtained from an elastic time history analysis. We agree.

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Section 5: General observations of damage

This section of the Report illustrates common types of damage to primary structure that were observed in the buildings constructed between the 1930s and 1970s. It follows with some common types of damage that were not to the primary structure, but could have affected the safety of the building.

5.1 Damage to the primary structure

5.1.1 Introduction

Unreinforced masonry (URM) and stone buildings are well known to be vulnerable in earthquakes. Construction of these building types came to an end during the 1930s as councils adopted by-laws based on NZSS 95: 1936 – New Zealand Standard Model Building By-Law[1]. This Standard was established after the 1931 Napier earthquake. Around this time the importance of inertial seismic forces on buildings was recognised and incorporated in the design of buildings. However, it was not until the 1960s that the significance of ductility on seismic performance was appreciated by practising structural engineers.

A key change in design philosophy occurred during the period from the late 1960s to the late 1970s, when the concept of capacity design was developed and introduced into design standards. There was no single date on which it can be said that the concepts of capacity design and ductility were adopted by designers. A few fundamental concepts were practised by the Ministry of Works, with details in its 1968 Code of Practice[2]. Some ductile detailing requirements from the American Concrete Institute design code ACI 318:1971[3] were used by some engineers in New Zealand and in the Provisional New Zealand Standard of the 1970s, which was based on the ACI code. In 1975 the book "Design of Reinforced Concrete Structures" by Park and Paulay[4] set out a number of the basic concepts of capacity design.

The date of 1976 is often quoted as a milestone as this was when the Loadings Code, NZS 4203: 1976[5] set out the requirements for capacity design.

There was information readily available in 1977 and 1978 on capacity design in the New Zealand National Society for Earthquake Engineering Bulletins[6], but it was not until 1982 that the Concrete Structures Standard, NZS 3101:1982[7] defined the detailing necessary to achieve the required ductility. From the late 1960s to the early 1980s there was a wide variation in design practice, with some designers applying the new concepts while others maintained their previous practice.

Much of the damage that occurred in buildings constructed between the 1930s and 1970s is well known by designers and the issues around these failures have already been addressed in the current standards. Patterns of observed earthquake damage and the lessons that should have been learnt have been described by Paulay and Priestley[8] in 1992.

Kam et al[9] have stated that the structural deficiencies in these buildings include:

- no capacity design principles;
- lack of confining stirrups;
- inadequate reinforcing and anchorage details;
- poor material properties and use of plain reinforcing bar; and
- irregular configurations.

Deformed reinforcement was not widely available in New Zealand until the mid-1960s.

The majority of multi-storey buildings in the Christchurch Central Business District (CBD) are reinforced concrete and reinforced masonry structures, with few older steel-framed and timber structures. A draft report dated 5 December 2011 on "The Seismic Performance of Reinforced Concrete Buildings in the Christchurch CBD"[10] was prepared for the Christchurch City Council (CCC) by Pampanin et al from the University of Canterbury. This report describes the damage observed after the 22 February 2011 earthquake.

Many of the observations of building damage were assessed non-invasively, so further damage may have remained hidden by wall linings, ceilings or floor coverings. Nevertheless, typical patterns of damage observed and their severity can be related to the time when the building was designed. Some of the more common forms of damage identified in the Pampanin report are shown in Figures 54–59, with an emphasis on buildings constructed from the 1930s to early 1980s.

5.1.2 Common observed damage patterns

5.1.2.1 Column failures

(a) 198 Gloucester Street (1929)

(b) 82 Chester Street East (1957)

(c) 141 Hereford Street (1979)

(d) 221 Gloucester Street (1974)

(e) 79–83 Hereford Street (1968)

Figure 54: Typical column failures

Figure 54 illustrates some of the common column failures that occur in earthquakes, affecting buildings that were not detailed for ductility or designed using capacity design principles. In these cases brittle shear failures in columns were relatively common (Figures 54(a) and (b)) and column sway mechanisms formed as shown in (Figure 54(c)). In all cases shown in Figure 54 the columns had inadequate shear and/or confinement to prevent premature failure.

Figures 54(d) and (e) illustrate the “short-column” effect, which leads to premature shear failure. In many cases this was due to the stiffening effect of so-called non-structural masonry infill or spandrels built hard up against the columns. When the infill is partial height, the column is stiffened in comparison with other columns at the same level, which may not have adjacent infills (that is, interior columns). These short, stiff columns attract high shear forces, sometimes with disastrous effects.

Figure 55: Beam-column joint damage, 277 Manchester Street (1945, with 1953 alterations)

The columns in Figure 54 illustrate lack of ductility associated with inadequate confinement reinforcement in a plastic hinge region. Paulay and Priestley[8] show that high compression strains can be induced in the concrete from the combined effects of axial force and bending moment. Unless adequate closely spaced well-detailed transverse reinforcement is placed in the potential plastic hinge region, spalling of the concrete can be followed by instability of the compression reinforcement which buckles as shown in Figure 54(c).

5.1.2.2 Beam–column joints

The beam–column joint is the region where the beams at each level connect into the columns. In current design practice these joint zones are designed to be stronger than the beams that frame into them. The beam–column joint zones are subjected to high shear forces during severe earthquakes and, if the joint zones are inadequately reinforced, excessive loss of strength and stiffness can occur. In extreme conditions collapse can occur. Figure 55 illustrates shear cracking developing in a corner beam–column joint.

5.1.2.3 Structural wall failure

Unless adequately designed for the levels of flexural ductility and shear force expected under strong ground shaking, flexural or shear failures may develop in structural walls, as shown in Figure 56.

Figure 56: Structural wall failure, 29–35 Latimer Square (1967)

5.1.2.4 Conventional coupling beam failure

Beams coupling structural walls are often subjected to high ductility demands and high shear forces as a consequence of their short length. It is difficult to avoid excessive strength degradation in such elements (as shown in Figure 57) unless they have been diagonally reinforced. The concept of using diagonal reinforcement in coupling beams was not introduced until the mid-1970s.

Figure 57: Conventional coupling beam failure, 180 Manchester Street (1964)

5.1.2.5 Structural masonry

Reinforced masonry buildings are made of solid grouted or cavity walls, which are known as structural masonry. Masonry is also commonly used as an infill in frames and it is considered to be non-structural. As discussed earlier, if this infill is not adequately isolated from the frame it can lead to stiffening effects and/or short-column failures.

In-plane diagonal shear failures are common in solid-grouted and cavity-type masonry walls as shown in Figure 58. Failures of masonry elements can also be attributed to construction deficiencies[11]. In some cases the masonry block voids were only partially grouted, and under-reinforced.

Masonry shear cores around liftshafts and stairs are also common in buildings. These can become damaged with diagonal and/or sliding bed-joint shear cracking.

Figure 58: Structural masonry failure, 35 Cambridge Terrace (1964)

5.1.2.6 Punching shear

Buildings constructed using flat slabs supported directly on columns without beams are susceptible to punching shear cracking and failure (Figure 59). Demolition of these systems has shown them to be fragile with the potential of floor "pancaking". Failure of slabs in punching shear can arise when inter-storey drift induces bending moments in the column, and these are then introduced into the floors. The shear stresses associated with the transfer of bending moments can greatly increase the stresses at the critical section and lead to a brittle failure.

Figure 59: Punching shear cracking, 33 Lichfield Street

5.2 Building damage that is not part of the primary structure

5.2.1 Introduction

Analysis of individual buildings, and reports from other agencies, have highlighted that the safety of a building can be severely compromised by individual items and systems not complying with the structural requirements of the Building Code. We now discuss some examples of parts of buildings that failed in the earthquakes.

5.2.2 Compromised egress routes

5.2.2.1 Doors

There was evidence of doors jamming as a result of the permanent deformation of the Hotel Grand Chancellor, discussed in section 3 of this Volume of the Report. Maintenance workers needed to force doors open in order to evacuate the building.

5.2.2.2 Emergency lighting

In the Forsyth Barr building the emergency lighting failed, as discussed in section 4 of this Volume. The failure was likely to have been caused by the complete collapse of the stair. It is fortunate however that no one fell down the stairwell because of the lack of lighting. Emergency lighting systems are also often attached to suspended ceiling systems, which commonly failed.

5.2.2.3 Fire separations

The safety of buildings subsequent to the earthquakes was compromised by the failure of lightweight fire separations not detailed and installed to accommodate the flexibility of the main structure. This was demonstrated in the stairwells of the CCC Hereford Street offices (see section 6.12 of this Volume) and repairs were required to plasterboard firewalls after both the September and February earthquakes. The plasterboard is now reinforced with steel strapping to the outside, which will not prevent damage to the lining but will prevent it from falling on people and from blocking the escape route.

5.2.2.4 Shelving

The majority of buildings had shelving systems that did not remain standing during the major earthquakes. Even if the shelving did remain standing, the contents of the shelves often fell. This can be of particular significance where the primary escape route from the building is compromised.

5.2.3 Falling hazards

5.2.3.1 Ceilings

Failures of ceilings, in particular suspended tile ceilings, have been widely reported as a result of the earthquakes. This was discussed as having occurred in the Pyne Gould Corporation (PGC) building after the September earthquake (see section 2 of this Volume). The solution in that case was to replace the heavy ceiling tile system with a lighter system. Unfortunately, because the building collapsed on 22 February there is no way to determine how well the new tile system performed. However, there have also been reports of the failure, during the 23 December 2011 aftershock, of suspended tile ceilings that had only just been replaced as a result of the February earthquake. This occurred in the Christchurch Hospital Riverside Block.

The performance of ceilings in the February earthquake is considered in a report by Dhakal, MacRae and Hogg, published in the Bulletin of the New Zealand Society for Earthquake Engineering (NZSEE), Volume 44, Number 4, December 2011[12]. That paper also includes conclusions and recommendations that are summarised below:

1. Most damage occurred at the perimeter of ceilings and increased with the size of the ceiling.
2. The observed damage primarily occurred in ceilings with heavier ceiling tiles. Wherever possible, heavier tiles should be avoided.
3. Earthquake design for ceilings should be for life-safety rather than serviceability.
4. Several ceiling failures were the result of the failure of services above the space, bulkheads and partition walls. Service installation requirements should be strictly complied with. Similarly, improved design guidelines for ceiling systems that take into account the interactions with partition walls are needed.
5. Poor installation practices in the case of ceilings, services and partitions appear to have caused more failure than weakness in design. Quality control measures should be implemented to ensure compliance.
6. Replacing ceilings quickly after an earthquake has been a priority in some cases. However, if not carried out correctly, further damage can occur in aftershocks and future earthquakes.

We agree with these observations.

5.2.3.2 Heating, ventilation, and air conditioning (HVAC) systems

The failure of the support of HVAC systems, in particular the way the failure affected ceilings, is considered in the Dhakal et al report. HVAC systems are commonly of metal construction and could be a significant danger if they fell on a person.

5.2.3.3 Lighting

Lighting systems, in particular those associated with suspended tile ceilings, but also longer channel lighting systems, have regularly failed and fallen into the room below. An example of the failure of a long channel system in the CCC Hereford Street offices in the September earthquake, and the solution to prevent a reoccurrence, is discussed in section 6.12 of this Volume.

5.2.3.4 Fire safety systems (excluding sprinklers)

No particular examples of the failure of fire safety systems have been bought to the attention of the Royal Commission. However, with many buildings having detector and alarm systems attached to the suspended tile systems, it follows that damage must have occurred. The major risk from falling objects in this case remains with the tile system itself rather than the attached fire safety systems.

5.2.3.5 Racking systems

The performance of racking systems in the Canterbury earthquakes has been considered in a report by Uma and Beattie[13]. The Royal Commission has not considered this matter further.

5.2.3.6 Non-structural partition walls

Although there have been many reports of damage to non-structural partition walls, the Royal Commission has seen no evidence that lightweight walls have failed in a manner that has created an immediate danger to people. The exception to this is lightweight walls that have impeded an exit route.

5.2.4 Risk of fire in buildings

We have seen little evidence of outbreaks of fires in buildings that remained standing after the earthquakes. One fire is known to have occurred as a result of the September earthquake but the specifics have not been investigated by us. There does, however, remain a significant risk if there is an outbreak of fire. The passive and active systems to protect the people occupying buildings and allow them to exit in safety should remain operative during and after an ultimate limit state event.

5.2.5 Discussion

Egress from a building during an emergency and the protection of the egress route should be considered as a life-safety issue, and consequently the means of egress should perform adequately in an ultimate limit state event. This also applies to building elements that could fall and injure people underneath.

Structural engineers focus on the primary structure, with the ancillary structures generally being managed by the designer responsible for the architectural elements. There is often no overall supervision of the structures within the building by a person with knowledge of how the building is expected to behave in an earthquake.

There is a significant amount of work that can now be carried out without a building consent. Although there is an obligation for all work to comply with the Building Code, the restrictions in the Building Act on those who may carry out work only apply to residential buildings. Evidence discussed in the reports in the NZSEE Bulletin Volume 44, Number 4, December 2011[9, 11, 12, 13] (referred to above) suggests that systems that may be of proprietary design and have adequate provision for seismic movement are not necessarily being installed in accordance with those designs, or with proper regard to the limitations of those designs.

5.2.6 Conclusions

We conclude that:

1. The principles of protecting life beyond ultimate limit state design should be applied to all elements of a building that may be a risk to life if they fail in an earthquake. This is already applied to stairs and the same factors of safety should apply to other critical non-structural building elements.
2. In the design of a building the overall structure, including the ancillary structures, should be considered by a person with an understanding of how that building is likely to behave in an earthquake.
3. Any element of a building that is considered to be a life-safety issue if it fails should only be installed by a suitably qualified and experienced person, or under the supervision of such a person. The regulatory framework necessary for this is discussed in Volume 7 of this Report.

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Section 6: Individual buildings not causing death

This section of the Report includes details of the 14 individual buildings that have been assessed by the Royal Commission.

6.1 Buildings designed prior to the introduction of Loadings Code NZS 4203:1976[1]

6.1.1 48 Hereford Street: Christchurch Central Police Station

Current status
In use; repairs may have been undertaken but are presumably of a minor nature as no building consent has been obtained.

Figure 60: The Christchurch Central Police Station seen from the west bank of the Avon River (source: Ross Becker)

6.1.1.1 Introduction

The Christchurch Central Police Station was designed in 1968 by the Ministry of Works. As a Government building it was built to more rigorous design requirements than the minimum New Zealand Standards of the day. No building permit from the Christchurch Council City (CCC) was required for Crown-owned buildings at the time.

The building is a 15-storey reinforced concrete structure, three levels of which are a podium that is about twice the plan area of the tower above. The tower is approximately central to the major portion of the podium, with a seismically separated portion of the podium to the west, as is shown in Figure 61. It is located about 60m from the western bank of the Avon River.

There is relatively little information on the foundation soils. However, based on an existing soil profile along Hereford Street by Elder and McCahon[2] we think it likely that the building is founded on sandy gravel for a depth of about seven metres, and below that a layer of about six metres of loose sand of medium density. After the February earthquake it was noted that minor liquefaction had occurred at the north-eastern corner of the building and there was some differential settlement between the seismically isolated portion of the podium and the main structure. In a survey it was found that there was up to a 100mm differential settlement between the eastern and western ends of the main podium (grid lines 1 and 7 in Figure 61).

The foundation system is a deep reinforced concrete cellular raft system. The total depth of this raft is about 2.5m.

6.1.1.2 Building structure

The gravity loads and lateral forces are resisted by ductile reinforced concrete moment resisting frames. The arrangement of structural members on typical floors in the building is shown in Figures 61–63. The floors consist of 152mm thick reinforced concrete. The beams, columns and floor slabs were all cast-in-situ. Precast concrete panels were used as non-structural elements for cladding and also for walls in the vicinity of the lift/stair core. These were installed with seismic gaps to prevent them from interfering with the seismic performance of the building. The stairs were fixed to the floors at their upper level but designed to slide at their lower level on two sheets of polythene.

In the tower there are 20 columns arranged in a grid to give four bays of 6400mm in the east–west direction and three bays of 6400mm in the north–south direction. The columns in the lower levels of the tower are 762 by 762mm and in the upper levels they are 686 by 686mm. Beams are made continuous with the columns. The beams are 762mm deep with a web width of 686mm in the lower levels of the tower, and 686mm deep with a web width of 610mm in the upper levels of the tower. The 152mm reinforced concrete floor slabs are tied into the beams as illustrated in Figure 64.

Detailing of the structure is of a high standard, having regard to the fact that it was designed in 1968. While it is not up to current standards, in many aspects it is close. It is apparent from the structural details that the columns were designed to be considerably stronger than the beams, which ensured that in the event of a major earthquake a beam sway mode would develop, provided that the beam-column joints did not fail. The detailing of the beams and columns ensures that plastic hinges, should they form, are located in the beams against the column faces. The columns are confined by ties and most of the longitudinal bars are adequately restrained against buckling. In some cases the spacing between the bars was greater than required by current standards. In the beams the stirrups have been placed to enclose all the flexural reinforcement. This detail does not conform to current standards, in that the top and bottom reinforcing bars located in the middle of the beam are not constrained against buckling (see Figure 64). In the beam-column joint zone it is apparent that the joint zone shear reinforcement is less than what would be expected for a building designed to current design standards.

It is clear from the drawings that the design incorporated many of the concepts of capacity design, which was at a very early stage of development in 1968.

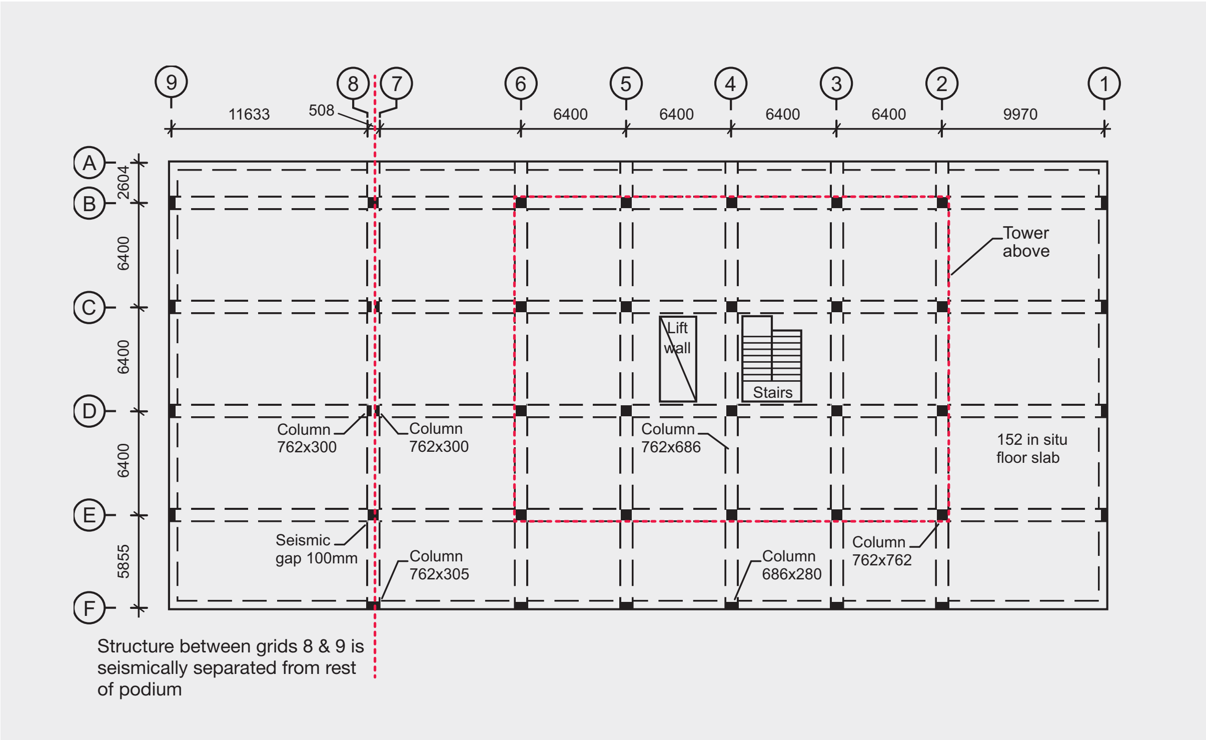


Figure 61: Structural members at podium level

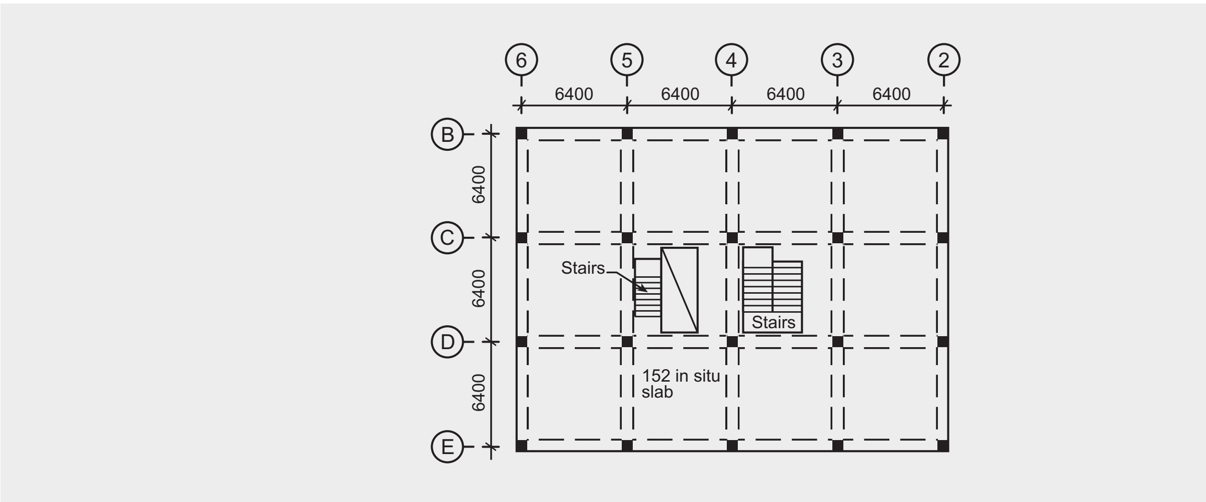


Figure 62: Structural members, levels 5 to 13

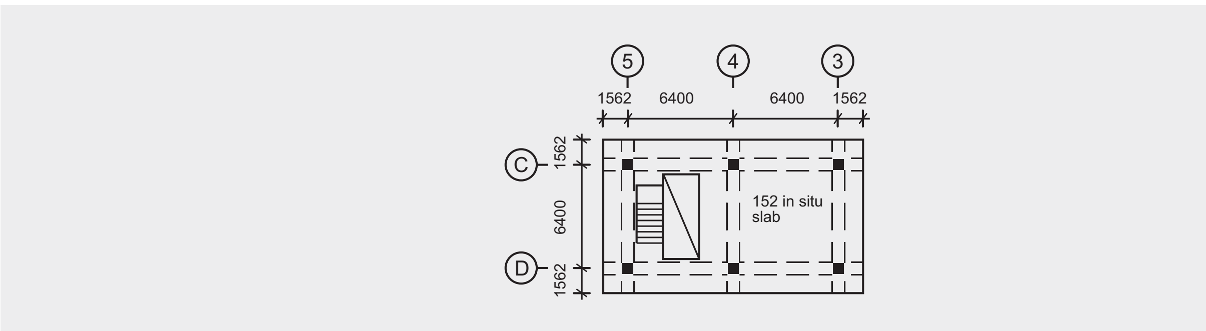


Figure 63: Structural members, level 15

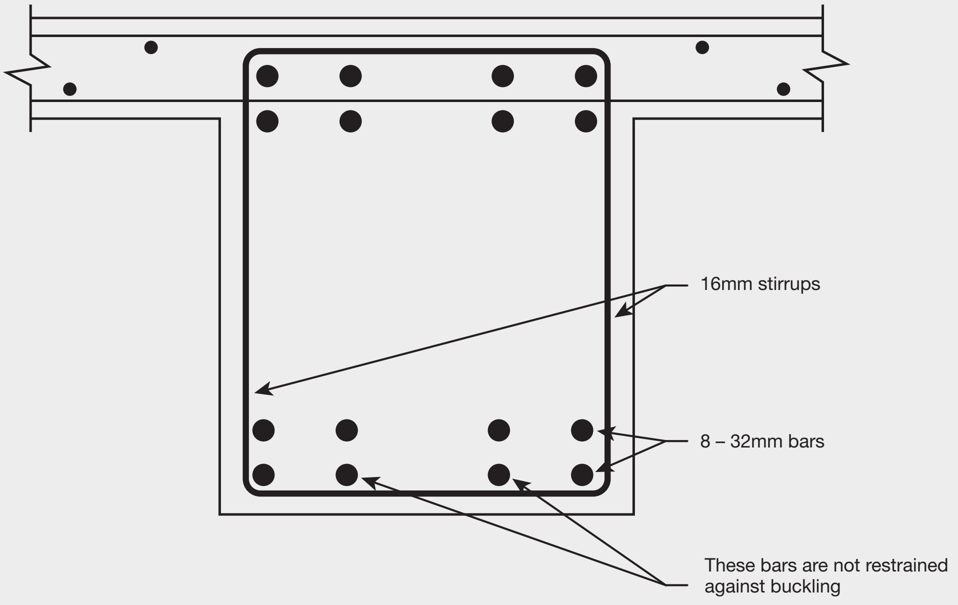


Figure 64: Typical beam details

The structural arrangement is robust in that there is a high level of redundancy, with five moment resisting frames that resist seismic forces in the east–west direction and four frames that resist the forces in the north–south direction. All the columns are effectively tied together by the reinforced concrete in situ floor slabs.

6.1.1.3 Structural damage

No significant structural damage was recorded in structural elements in the September earthquake, though there was some non-structural damage.

In the February earthquake there was appreciable non-structural damage, but relatively minor damage to the main structural elements. As noted previously, a limited amount of differential settlement occurred, possibly because of liquefaction, but this did not have significant adverse effects on the building. The good performance of the building almost certainly owes much to the sturdy cellular raft foundation that was used.
Some cracking was observed in the beams in the floor levels that were inspected, with cracks up to 2mm wide in the beams at the column faces.

6.1.1.4 Assessment of seismic performance

The building was designed to comply with the Ministry of Works code[3], and with the then current codes of practice for design loads and concrete structures[4,5]. The lateral force coefficient for a public building in 1968 was 0.06 for a building with a fundamental period of 1.2 seconds or more. However, to interpret this coefficient in terms of current design standards it is necessary to make allowance for changes in practice since the building was designed. In 1968 elastic design was widely used, while today ultimate strength theory is used. To allow for this change the 0.06 is multiplied by 1.25 (MacRae et al[6]). The design strength is taken as the product of the appropriate strength-reduction factor (0.85 for reinforced concrete) and the nominal flexural strength. For the purpose of assessing probable strength, a strength-reduction factor of 1 should be used. The nominal flexural strength is calculated from the lower characteristic material strengths, which means that in 95 per cent of cases the flexural strength is greater than the nominal value. The ratio of probable material strengths to their corresponding lower characteristic values is about 1.1:1.

In practice, reinforcement contents are greater than the minimum areas required to provide the design strengths found in an analysis, owing to the need to maintain similar reinforcement arrangements along members, and in the 1960s no allowance was made for the contribution of reinforcement in the slabs to the strength of the beams. These two factors would typically increase the strength by a factor of 1.2. Using the ratios given above, the base shear coefficient of 0.06 corresponds to the probable base shear strength, in terms of current practice, of 0.12.

The building was assessed for the Royal Commission by Compusoft Engineering Ltd. As part of its assessment Compusoft examined the acceleration response spectra calculated from the ground motion records obtained at the CCCC, CHHC, and CBGS sites (see section 1.6 of this Volume). The records from the REHS were not included as Compusoft considered the soils in that location were not representative of those on the site of the Police Station. The acceleration response spectra for these three records plotted in terms of acceleration due to gravity (g) are reproduced in Figure 65 for the September earthquake in the north–south direction and Figure 66 for the February earthquakes in the east–west direction. These directions were chosen as they were dominant for the period range of interest. From an analytical model which Compusoft developed, the fundamental periods of vibration were found to be 2.0 seconds in the north–south direction and 2.15 seconds in the east–west direction. Based on these values the figures show that the lateral force coefficients for elastic response are close to 0.25 and 0.32 for the September and February earthquakes respectively. This implies that displacement ductilities were of the order of 2 and 2.6 respectively. The structural damage observed in the February earthquake appears to be consistent with displacement ductilities of this order.

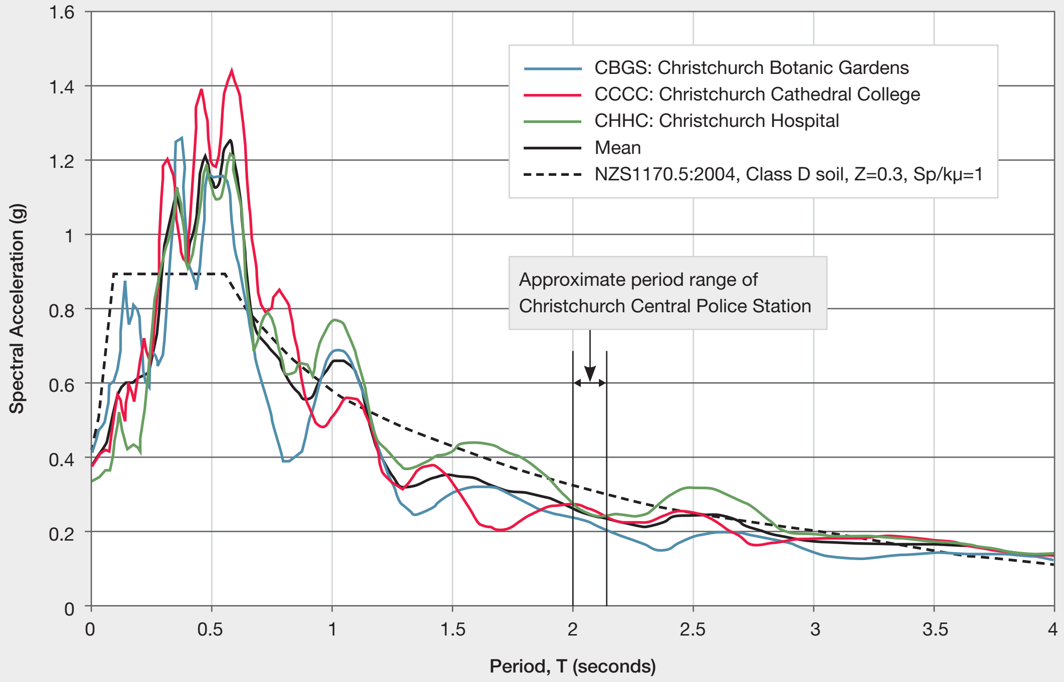


Figure 65: Spectral acceleration in the September earthquake for ground motion in the north–south direction for the stations CBGS, CCCC and CHHC (source: Compusoft)

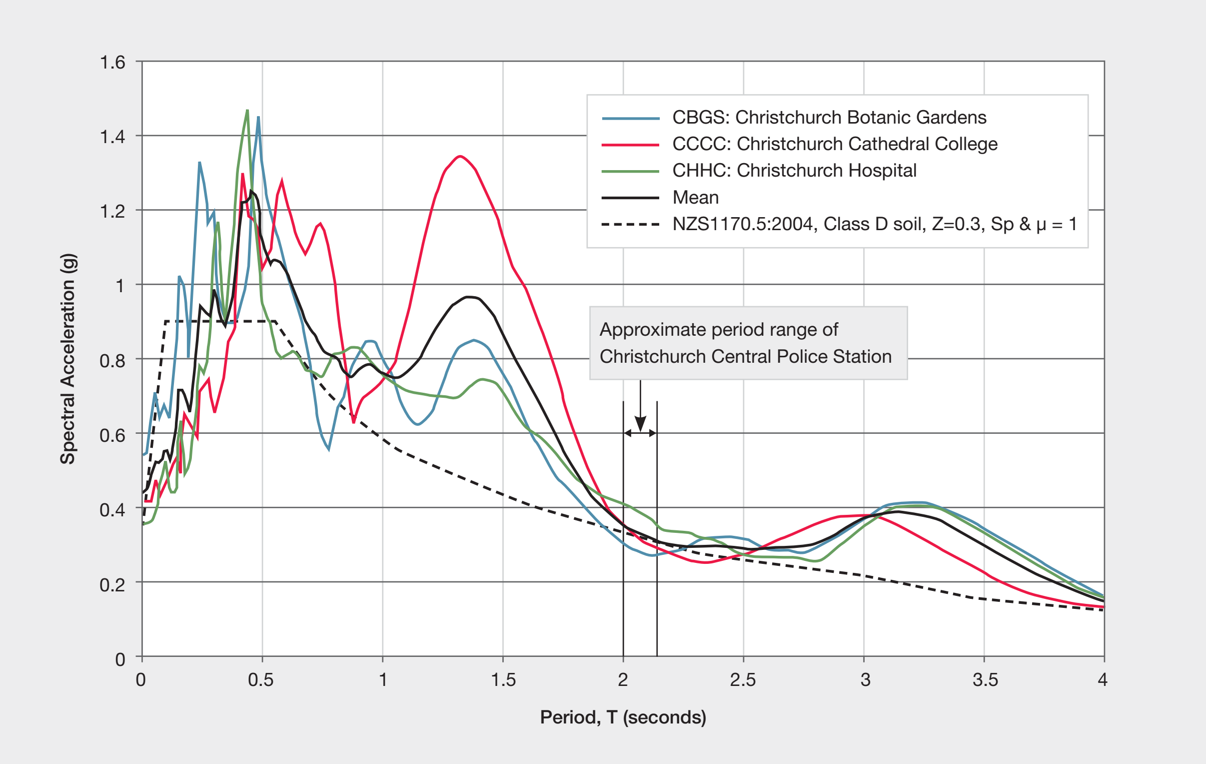


Figure 66: Spectral acceleration in the February earthquake for ground motion in the east–west direction for the stations CBGS, CCCC and CHHC (source: Compusoft)

In a previous assessment of the building its proportion of compliance with New Building Standard (NBS) was assessed as 20 per cent for an importance level 2 building (in terms of importance in AS/NZS 1170.0:2002[7]), with predicted performance being limited by the detailing of the beams and columns, which do not fully comply with current design standards. However, the performance of this building has been shown to be well beyond the implications of the assessed level. This indicates that there is a need to quantify the performance of structural members that do not fully meet the current design provisions. This should be possible if the results of the numerous tests that have been made in New Zealand and elsewhere were compiled in a readily available document.

6.1.1.5 Non-structural damage

There was significant non-structural damage in the building in linings and to the precast panels. These were detailed with a 25mm clearance gap, which proved inadequate to prevent them from being damaged. This underestimate of the required gap was very likely due to the 1960s practice of assessing deflections on the basis of gross section properties, whereas today practice deflection calculations are based on section properties that allow for the reduction in stiffness associated with flexural cracking (MacRae et al, 2011)[6].

As noted in the report on Clarendon Tower (see Figure 113, page 74), a reinforced concrete building that sustains inelastic displacement loses some stiffness for subsequent earthquake events. In the present case that may have contributed to the reported observations of occupants that the building felt more lively after the February earthquake, but it would not have been weaker in structural terms. Nevertheless, the acceptable extent of loss in stiffness should be considered in the design of new buildings.

6.1.1.6 Conclusions

1. The performance of the building in the earthquakes was very satisfactory in terms of the structural damage that occurred. The very robust nature of the building, which was due to its high level of redundancy and its symmetrical, regular form, contributed to its good performance.
2. The detailing of the building was excellent for the time it was designed.
3. The building would have lost some stiffness as a consequence of its inelastic deformation (see Figure 113 on page 174). When considering serviceability of a building, it is important to consider this reduction of stiffness as a part of the design of a ductile structure.

4. There was appreciable non-structural damage to lining and precast panels in the building, which is an issue that needs to be considered for new construction. The precast panels were detailed with 25mm separation to prevent damage in the event of an earthquake.
5. In a previous assessment of the building its proportion of compliance with NBS was assessed as 20 per cent for an importance level 2 building, with predicted performance being limited by the detailing of the beams and columns. Some guidance is required for engineers involved in assessing percentage NBS for the deformation capacity of structural elements that do not fully meet all the requirement of current design standards.

6.1.2 53 Hereford Street: Christchurch City Council Civic Offices

Current status
Repaired and in full use.

Figure 67: View from Worcester Street

6.1.2.1 Introduction

The building currently used as the CCC civic offices was originally designed as the Post Office mail sorting centre in 1972. As it was owned by the Crown, no building permit was required or obtained. Design and supervision was undertaken by the Ministry of Works, which signed it off as complete in 1974. The original structure was designed using the Ministry of Works Code of Practice for Public Buildings[3]. Structural details indicate that the fundamental concepts of capacity design, which were being developed at the time, were applied in the structural design.

The building underwent substantial alterations and extensions between 2008 and 2010 to convert it into the civic offices. These works were approved under a series of building consents, with a final code compliance certificate being issued on 18 August 2010. The building is now six storeys plus a basement below the extension area and a sub-basement below the original building, with mezzanines on five levels.
The overall plan size is about 78m by 37.6m. The inter-storey heights are close to 5.82m in the upper storeys and 6.9m in the first storey.

Figure 68: View from the north-east (source: Compusoft)

Foundations for the original building consist of a reinforced concrete cellular raft system with a total depth of about 2.5m. The base of the raft is 1270mm deep and the top slab is 305mm deep. Support between the two slabs is provided by a grillage of 1200mm-wide reinforced concrete beams with numerous openings in them. This formed the sub-basement with limited access that was partially used for water storage.

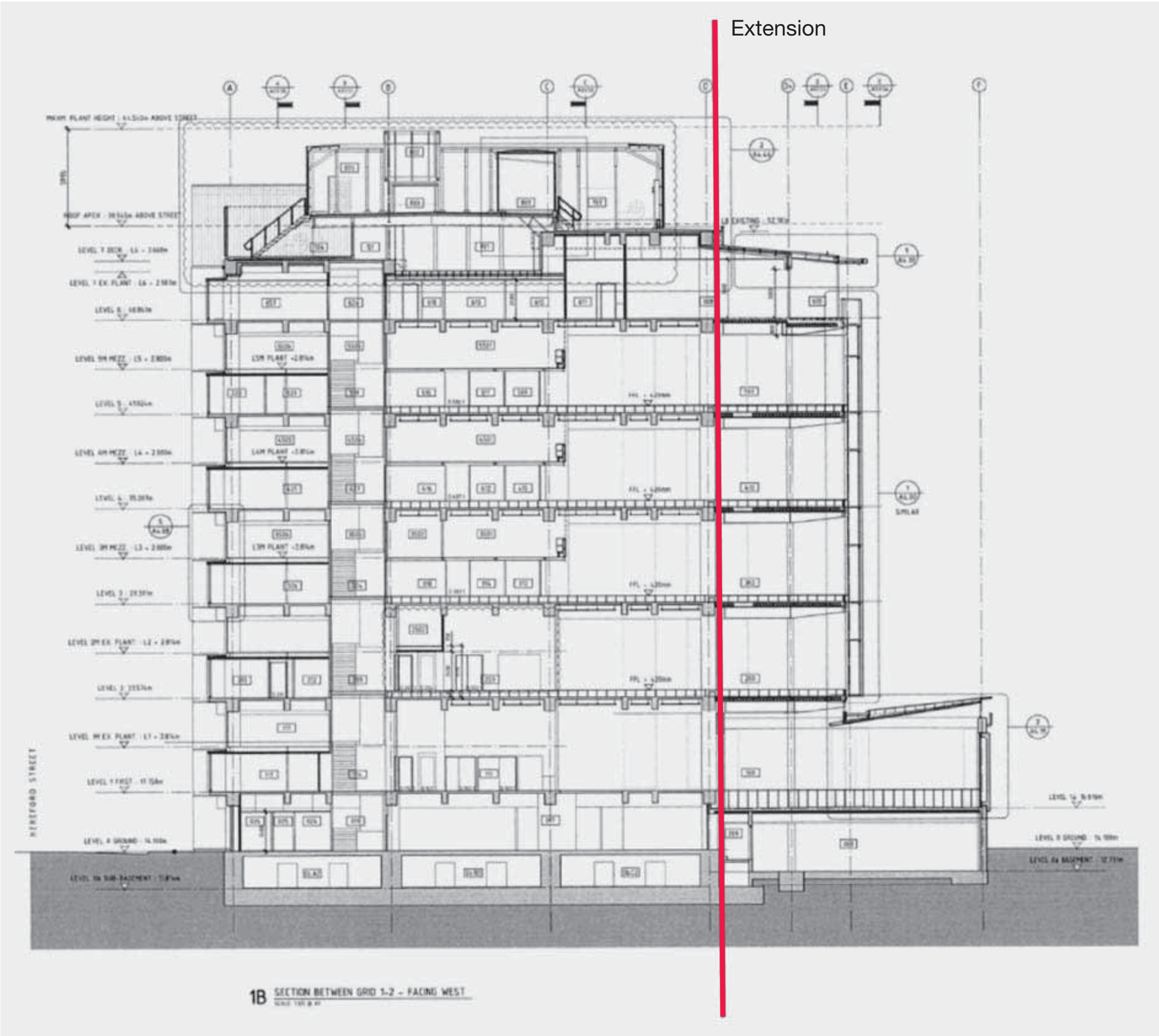


Figure 69: Cross-section (source: modified from original drawings)

The extension of the foundation consists of three foundation beams 1200mm deep and 1500mm wide, two with 2500mm thickenings at their ends to accommodate a high-voltage cable duct. The pad foundation is a 3600mm square with a depth of 1200mm.

The structural system in the original building resisted both gravity loads and lateral forces with moment resisting frames. Reinforced concrete columns were constructed on a grid pattern to give bays of 9754mm in each direction. There are eight bays in the east–west direction and three bays in the north–south direction. Primary beams are supported by the columns to give four moment resisting frames in the east–west direction and nine in the north–south direction. Two secondary beams were added in the bays between moment resisting frames in the east–west direction to provide support for the 127mm reinforced concrete floor slab.

The structural arrangement was very similar to that used in the Police Station but in this case there was no podium. With this structural arrangement there was minimal eccentricity between the centre of mass and the centre of lateral stiffness and strength.

Extensions to the building in 2008 involved the addition of a further bay of 8776mm on the northern side for all the elevated floors above the second storey. The support for these floors was provided by 400mm concrete-filled tubular steel columns, which were at 9754mm centres in the east–west direction and at a distance of 5.0m from the northern-most moment resisting frame. The floors are supported by steel beams that span from the moment resisting frames over the columns and for a distance of 3776mm past the columns to provide support to the double-skin façade system on the northern face of the building.

The floors were built up of 200mm hollow-core reinforced concrete units that spanned in the east–west direction between the steel beams. This was topped with in situ concrete to a depth of 80mm that was reinforced with mesh, with some additional reinforcement added to the hollow-core units. Below the second floor the bay length was increased as shown in Figure 70.

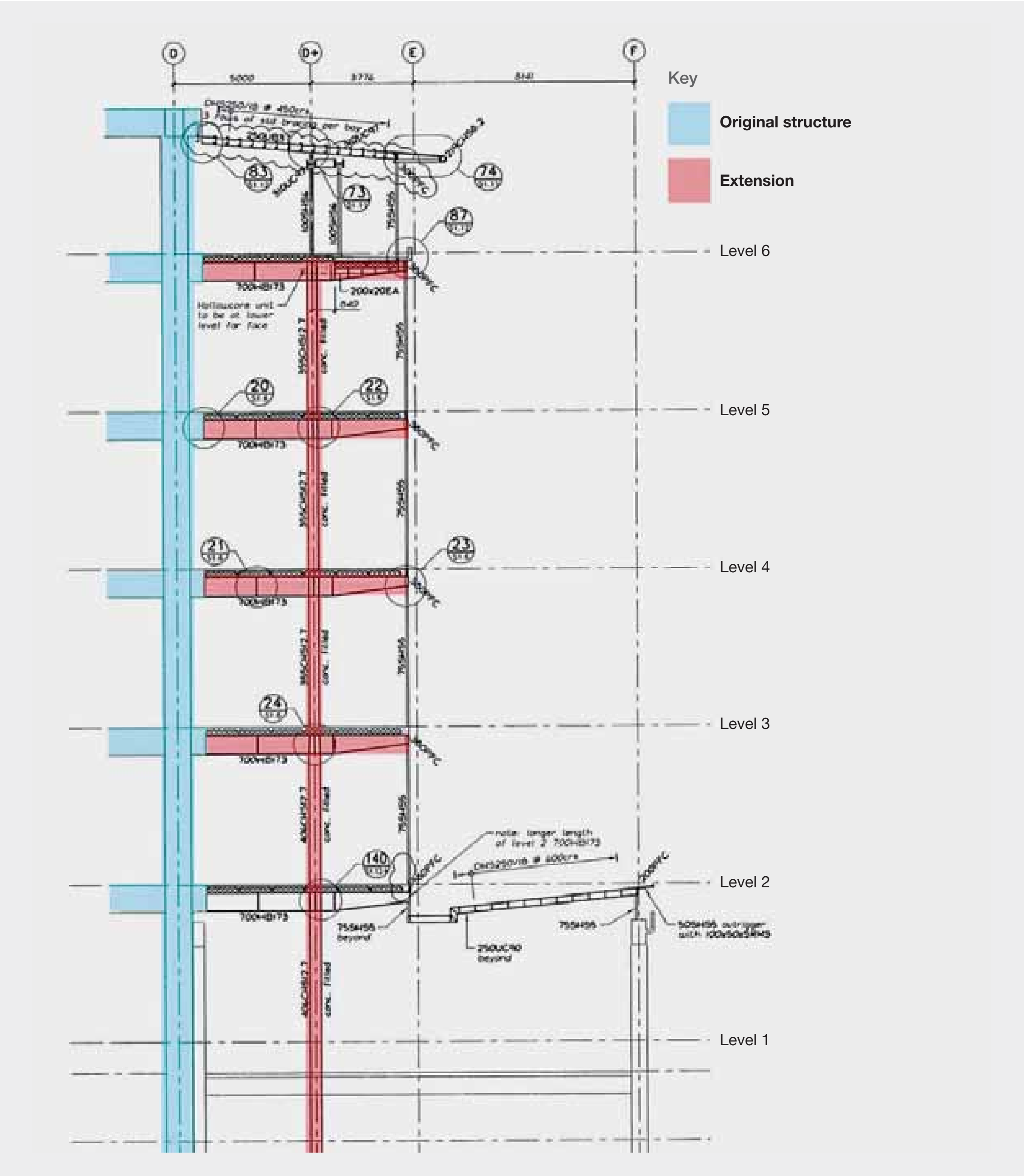


Figure 70: Typical section through extension structure (source: Compusoft)

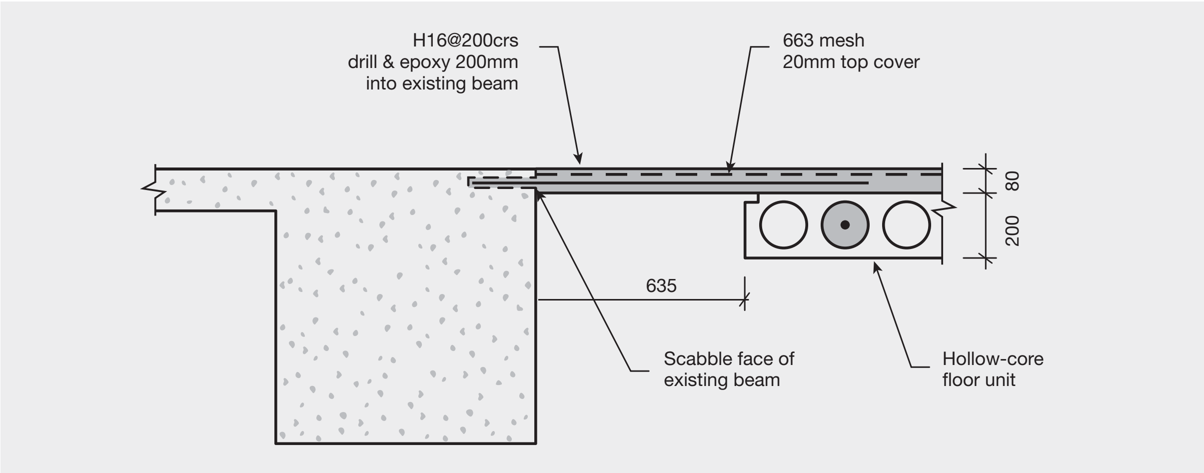


Figure 71: Section at interface of original and extension structure (source: Compusoft)

The lateral seismic forces arising from the extension were carried back into the original part of the building. For this purpose 16mm bars at 200mm centres were placed in the topping concrete and anchored into the main part of the structure (see Figure 71). The addition introduced a limited amount of eccentricity for seismic forces in the east–west direction.

6.1.2.2 Building structural performance

The Royal Commission was assisted in its assessment of the building by a report prepared by Compusoft. The building suffered relatively minor damage as a result of the September 2010, February and June 2011 earthquakes. The primary structural damage has been summarised in the table below:

Structural aspect	Earthquake		
	September 2010	February 2011	June 2011
Original frames	-	Spalling of concrete in columns adjacent joints Shear cracking in beams	Some cracking
Extension structure	Yielding where steel beams connect to existing structure Crushing and spalling of concrete in the infill slab	No apparent movement at steel beam connections Cracking of concrete at edge of infill slab	Movement where steel beams connect to existing structure Cracking and spalling at double-tee seating
Stairs: general	Cracks in topping concrete at stair landings	Spalling to edges, cracking through stairs in places Stairs safe to use	Cracks to landings at level 3 and 4, alongside previous repair
Stairs: level 1 to 2	Cracking and spalling of top connection, cracking of "sliding" base connection	Stairs jammed and considered unsafe to use	-
Foundation	-	Moderate liquefaction at eastern end of structure	-

Summary of primary structural damage over three earthquakes, CCC Civic Offices (source: Compusoft)

6.1.2.3 Damage to elements that are not part of the primary structure

The building suffered from damage to elements that are not part of the primary structure during the earthquake sequence. Fortunately this did not result in injury to people in the building, but failures of this nature are likely to be typical of many buildings in Christchurch. Some of the damage in this building is considered below.

As a result of the September earthquake, long lighting channels that were suspended by wires failed and fell onto the spaces below (see Figure 72). These weighed up to 34kg each so it was fortunate that the building was unoccupied at the time. Analysis by Powell Fenwick Consultants Ltd after the September earthquake indicated that the lighting channels were subjected to loadings well above the design Standards of the time (NZS 4219:1983[8] and AS/NZS 60598.1:2003[9]). This is with the assumption that the fixtures are only required to perform adequately in a serviceability limit state earthquake, as their failure was considered to be a financial loss issue rather than one related to life-safety. After the September earthquake these lighting channels were re-suspended with steel rods and braced to resist seismic loads in the most critical direction (see Figure 73). They performed adequately in the February earthquake.

Figure 72: The lighting channels in 2010, prior to occupation of the building by the CCC

Figure 73: Lighting channels suspended by steel rods, braced for seismic loads in the critical direction

Bookcases also toppled in the September earthquake, and were re-fixed at the base with more consideration of seismic loadings before the February earthquake. We do not have a record of the performance of these in the February earthquake but they are an example of where egress routes can be obstructed.

As was the case with the Police Station there was extensive damage to the linings of the building. Of particular concern was the failure of some of the linings, which obstructed egress routes. The fixing of fire-resistant linings in the stairwells between moment resisting frames did not allow for the movement of those frames in the earthquakes.

6.1.2.4 Conclusions

Performance was comparable to that of the Police Station, with the structure only sustaining relatively minor damage. Its good performance reflects the high quality of its design at a time when the concepts of capacity design were being established. It also reflects the advantages of having:

- a regular structure with multiple lateral force resisting elements;
- minimal eccentricity between the centre of mass and the centre of stiffness and strength of the lateral force resisting system; and
- robust cast-in-situ concrete floors to act as diaphragms.

There was limited damage to the structure at the junction between the extension and the main building. The extension was tied to the main building by 16mm reinforcing bars at 200mm centres that were anchored into the existing structure and the in situ concrete topping above the hollow-core units. While there was sufficient reinforcement to satisfy the requirements of NZS 1170.5:2004[10], the method of calculation was incorrect. The horizontal force was assessed by bending theory applied to the floor loaded by a horizontal force with the floor acting as the beam. The span of this beam was 8m, its depth 87m and the calculations surprisingly assumed that plane sections remained plane. In addition the lateral force coefficient for each level was 0.05, which might have been adequate for the base shear for the building as a whole but it is close to 1/30th of the corresponding value found from a parts and portions analysis. However, the designer had the good sense to ignore his calculations and specify that a much greater quantity of reinforcement be used in the junction than was indicated by the calculations.

The analysis of the failure of the luminaires highlighted the assumption that fixtures and fittings only need to comply with the design load requirements of a serviceability limit state earthquake. The Royal Commission considers that where the failure of a fixture or fitting is likely to risk the life of any person, the ultimate limit state loadings should be applied. Also, the design of linings is principally the responsibility of architects and it is important that the need to ensure that egress routes remain clear in the event of an earthquake is emphasised to them.

6.1.3 100 Kilmore Street: Christchurch Town Hall

Current status
Proposed to be repaired.

Figure 74: View of entrance to the Town Hall lobby looking south from Kilmore Street (source: CCC)

6.1.3.1 Introduction

The Christchurch Town Hall was designed and built between 1968 and 1972 (Figures 74 and 75). It is a T-shaped building comprising an auditorium, a theatre and three large conference rooms. At the southern end of the building is a restaurant with adjacent kitchen areas. For the general layout see Figure 76. The structure extends over three storeys, with a plan area of about 6500m^2. The Avon River is immediately adjacent to the southern side of the building.

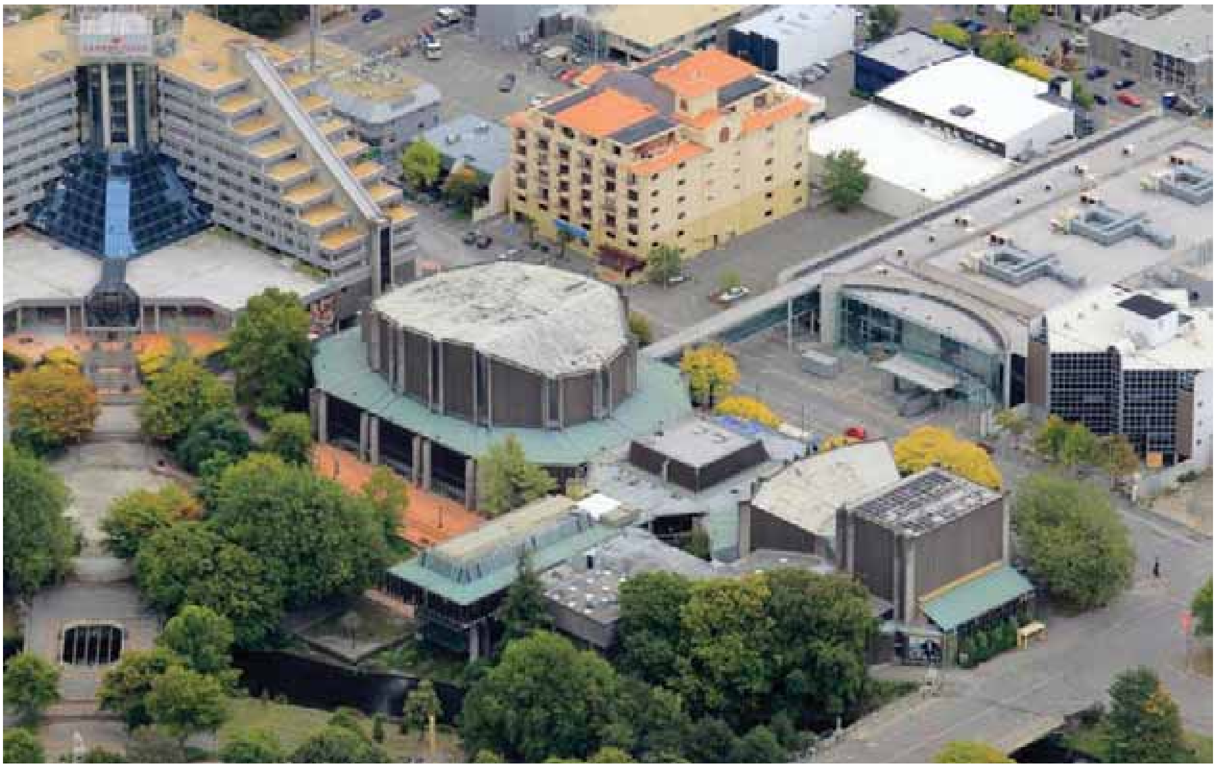


Figure 75: Aerial view looking north-west with Colombo Street in the foreground (source: Becker Fraser Photos)

6.1.3.2 Foundations

Generally, the Town Hall's foundation system consists of shallow foundations such as strip or rectangular footings, although a relatively small extension added in 1976 was supported on deep foundations. In the auditorium, slender piers are supported by pad footings at both the interior oval and the exterior. Foundations for the slender piers at the interior oval are further connected to each other by strip footings under the reinforced concrete walls. At the exterior they are tied together by small beams cast on grade.

The theatre employs similar foundation elements, with slender piers supported by pad footings, reinforced concrete walls by strip footings and several small beams cast on grade interconnecting various elements. Slender piers to the lobby are also supported on pad footings with small grade beams. In addition several significant concrete ducts run under the slab on grade. One of these ducts continues to the restaurant, where slender piers are again supported on pad footings. More significant tie beams exist between pad footings in the north–south direction of the restaurant, with smaller grade beams running east–west.

The kitchen block rests on a reinforced concrete mat foundation that is thickened near its centres, where it supports an interior reinforced concrete wall. As noted above, in contrast to the original structure, the 1976 addition uses reinforced concrete piles rather than shallow foundations.

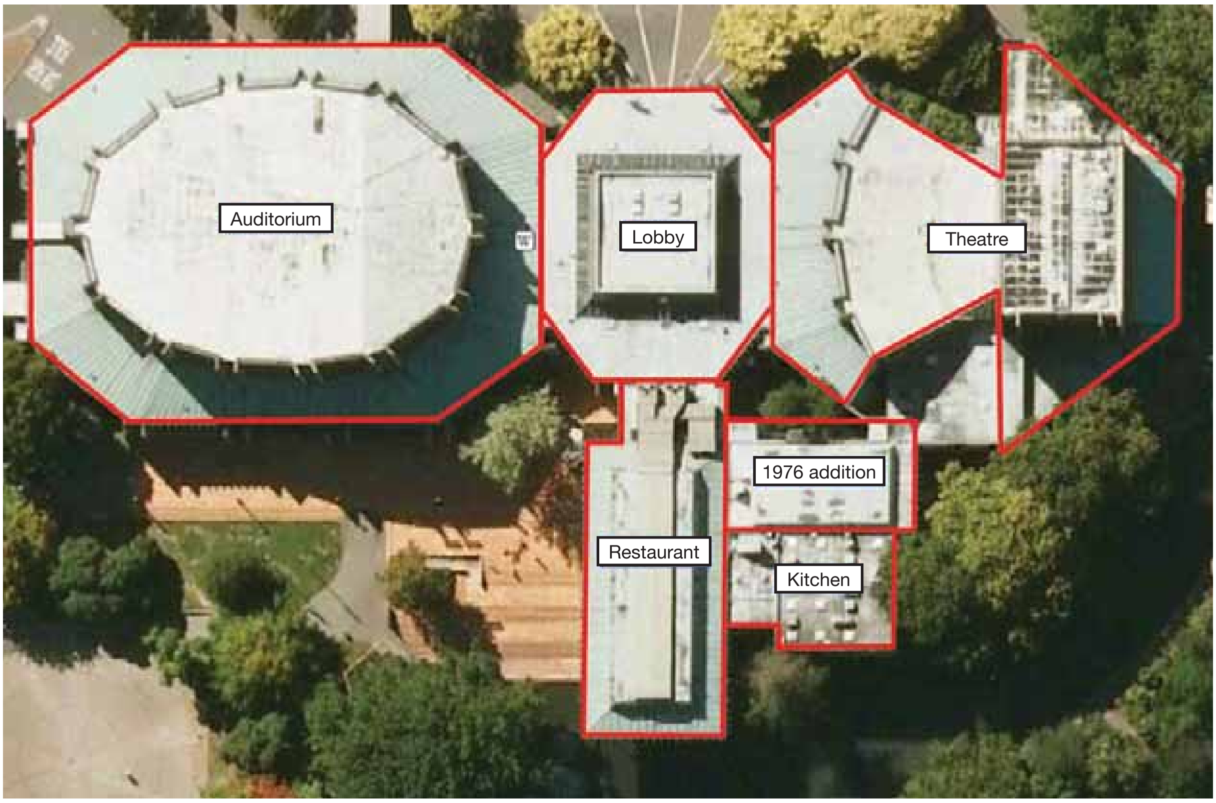


Figure 76: Aerial view showing the various uses

6.1.3.3 Superstructure

Reflecting the building occupancy and use, the lateral force resisting system varies between portions of the structure. Because the damage observed was primarily identified as being due to liquefaction and lateral spread, the superstructure is not discussed fully in this Report.

6.1.3.4 Building structural performance

The Royal Commission was helped in the assessment of this building by a report prepared by Rutherford and Chekene, consulting engineers, from California.

The Town Hall suffered significant damage during the Canterbury earthquake sequence, with the February event producing by far the greatest effects. Most of the superstructure damage appears to have been caused by widespread liquefaction and lateral spreading that resulted in differential settlement and building separation. Localised eruption of sand and the presence of sand and silt in the Town Hall's basement are the most obvious evidence of liquefaction, while ground cracking near the Avon River suggests extensive lateral spreading. Foundation settlement varied from 70 to 460mm, but more typically between 200 and 350mm over most of the building.

Lateral spread varied from no displacement at the northern side of the building (Kilmore Street) to as much as 350mm close to the Avon River bank. Available reconnaissance reports indicate that no foundation bearing capacity failures were observed. Several portions of the superstructure tilted either towards or away from the Avon River to accommodate the severe ground movement. Structural response due to ground shaking may also explain some superstructure cracking, although it is difficult to identify in the presence of such dramatic settlement damage.

6.1.3.5 Conclusions

The damage to the Town Hall is primarily due to liquefaction settlement and lateral spread of the ground. Given the large displacements caused by the ground damage, the building has performed well. It is a complex network of structures, and as the damage from shaking could not be clearly isolated from the ground failures, and the building was not built to current standards, there is little value in the Royal Commission commenting further on its superstructure.

6.2 1976 to 1984: Buildings designed to Loadings Code NZS 4203:1976[1]

6.2.1 166 Cashel Street: Canterbury Centre/ Westpac Tower building

Current status
To be demolished.

Figure 77: Westpac Tower, looking northwest (source: Ross Becker Photos)

6.2.1.1 Introduction

The Canterbury Centre/Westpac Tower (Figure 77) was designed in 1981 and a building permit was issued that year by the CCC. It was constructed over the next two years as the Canterbury Savings Bank building. The building is a 13-storey reinforced concrete building with a basement, and is interconnected through a seismic gap with a three-storey podium that also has a basement. The tower is of hexagonal form, with the tower orientation offset from the essentially rectangular form of the podium (see Figure 78). Only the performance of the tower is considered in this Report.

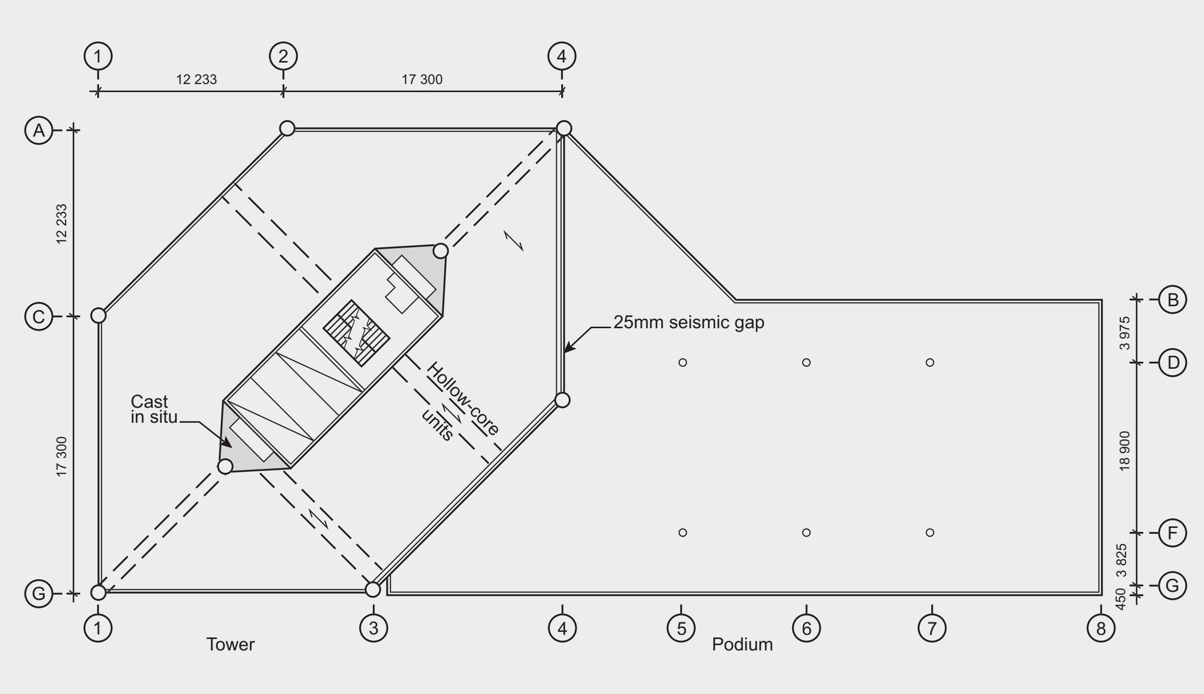


Figure 78: Plan on level 3 showing relationship of podium to tower

6.2.1.2 Foundations

There is no specific information on the geotechnical profile below the building, but the ground liquefied, with sand ejected through the basement floor. This is reported in a damage assessment carried out after the February earthquake but is not recorded in information available from the September earthquake.

The building is built on a raft-type foundation consisting of outrigger beams from the core to pick up the weight of the external columns to help resist overturning loads. The raft has possibly rotated up to 58mm, but from the information available the variations in level are too small to be conclusive. The indications are that the tower raft has settled by up to 70mm.

6.2.1.3 Gravity load system

The floors comprise precast hollow-core units with 50 to 65mm of in situ concrete topping, reinforced with hard drawn wire mesh. The hollow-core units were supported on the shear core walls, the two internal beams which span between the external columns at A4 and G1 and the internal columns (see Figure 78), and on the perimeter truss-shaped precast beams (see Figure 79). The support at the external beams is provided by a steel angle section that was anchored into the web of the beams. At each end of the shear core there is a region of in situ concrete floor that provided support to a few of the hollow-core units.

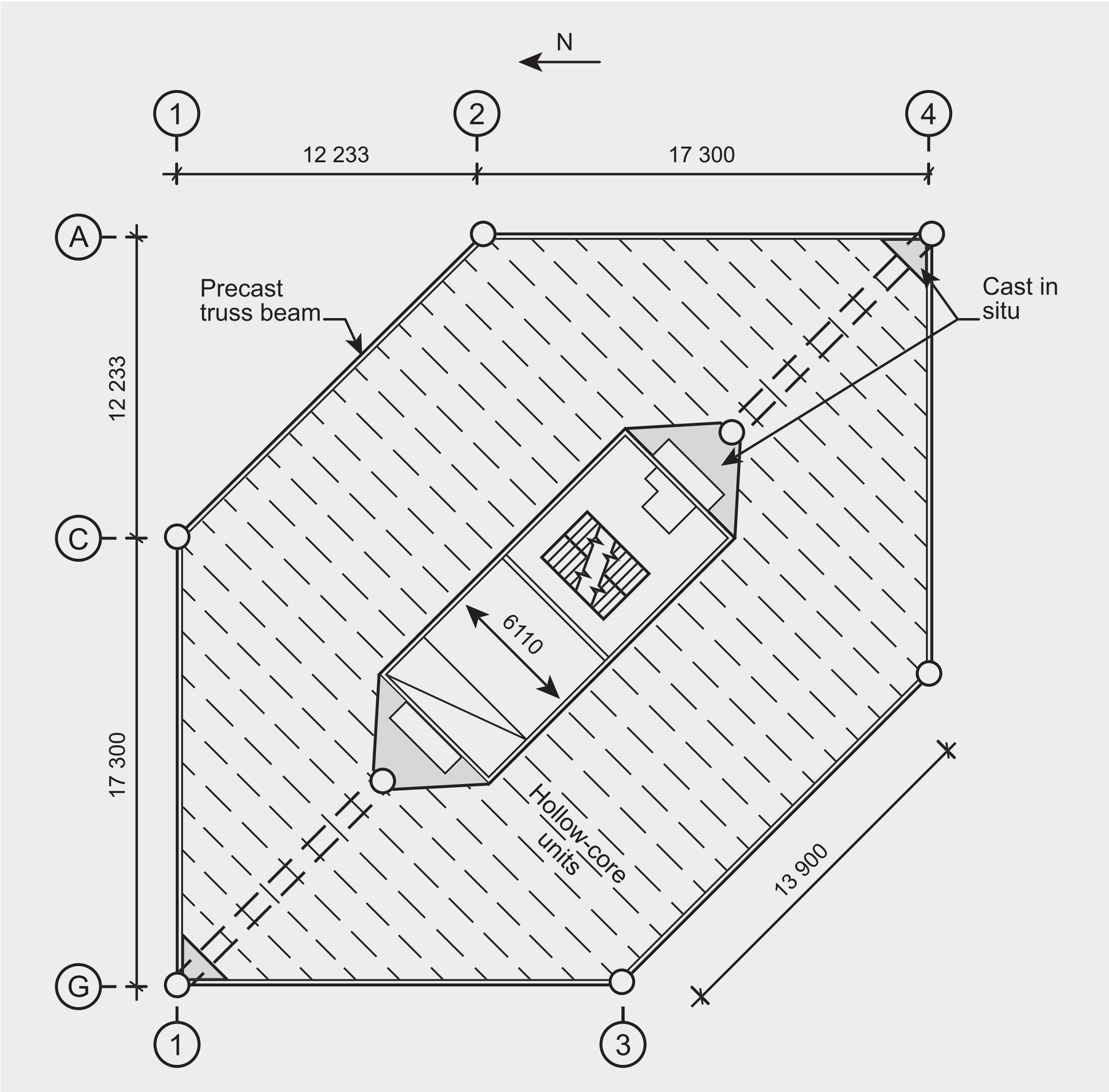


Figure 79: Plan on typical level in the tower

6.2.1.4 Seismic load system

The primary lateral force resisting system to the tower is provided by reinforced concrete ductile shear walls that form a shear core, which is located centrally within the hexagonal floor plate of the building. A perimeter frame consisting of six columns and the truss beams provides support for gravity loads. Their contribution to resisting seismic forces would have been minimal, owing to the flexibility of the columns and beams relative to the shear core.

A seismic gap of 25mm was provided between the podium and the tower.

The precast stairs are generally supported by structural steel legs cast into them and mortared into pockets at both ends. This detail provided little allowance for inter-storey drift but because the stairs were located within the shear core any shortening or lengthening of the stairs would have been small.

The Loadings Standard current when the building was designed was NZS 4203:1976[1]. The design forces that were used would have been less than 75 per cent of the current requirements in 2010 (based on a Z value of 0.22). The design was carried out before publication of the first Concrete Structures Standard, NZS 3101:1982[11], which contained detailed information on design for ductility. However, the detailing on the drawings shows that many of the ductile detailing concepts that were being proposed at the time were employed. It should be noted that detailing requirements have been considerably improved over the last 30 years, but the structural design was clearly advanced for its time. There was some confinement in the walls, and the beams above doorways in the shear core were designed as diagonally reinforced coupling beams.

6.2.1.5 Performance of the building

The Royal Commission was assisted in the assessment of the performance of this building by a report prepared by Spencer Holmes Ltd.

As a result of the September earthquake the building suffered some damage, in particular:

- flexural and shear cracking of the lower shear walls and coupling beams;
- tearing of the floor slabs adjacent to the shear core walls;
- cracking of the floor slabs adjacent to the exterior beams;
- damage and spalling at seismic gaps;
- spalling of external columns (and minor rusting of reinforcing exposed); and
- destruction of level 13 non-structural cladding (glazing).

Damage assessed after the Boxing Day aftershock was reported as an overall summary to that date:

1. For the tower, minor flexural and shear cracking of the core walls was observed throughout. Local buckling of the southern side of the core wall occurred at level 1, with significant cracking at this level.
2. Extensive spalling of cover concrete occurred on the exterior columns and significant damage was observed to the columns at the beam connections. Cracking and crushing extended to the core of the column section in these locations. Minor cracking of the precast truss beams occurred.
3. Failure of hollow-core flooring was observed at sliding seismic joint locations. Relative movement of the tower and podium caused failure of the sliding corbel seating for the level 2 bridge. Tearing of the floor topping was observed at some levels adjacent to the core as well as extensive pullout of cast-in inserts connecting the floor to the exterior beams.
4. Sliding connections for the podium roof were extensively damaged, with a residual displacement between the tower and podium. Non-structural cladding (glazing) at level 13 was badly damaged.
5. Liquefaction occurred at the site, with sand ejected throughout the basement slab. Possible differential settlement of the podium's shallow foundations occurred, with minor rotation of the tower raft foundation implied from the verticality survey.

At the time of the February earthquake, the repairs of damage from the previous earthquakes were under way and overall damage from all events to that point was reported:

1. Moderate cracking of the shear core walls was observed throughout, particularly at the first floor level where local wall buckling occurred. Steel material testing indicated that the core wall reinforcing had lost up to 90 per cent of its strain capacity in this area. Extensive spalling and cracking of the external columns occurred, resulting in significant damage to the precast truss beam seating.
2. Failure of hollow-core flooring units and the level 2 bridge occurred, resulting in local collapse hazards mitigated by temporary propping or cordoning. Tearing of the floor slab topping was observed in some isolated locations, as well as extensive pullout of the cast-in inserts connecting the floor to the exterior beams. The sliding connection for the podium roof failed and the top floor (level 13) glazing was completely destroyed.
3. Liquefaction occurred at the site, with sand ejected throughout the basement slab.
4. There were differential settlements of the podium's shallow foundations of up to 70mm, with minor rotation of the tower raft foundation implied from the verticality survey. The verticality survey indicated relatively minor residual displacements.
5. In general the structural damage sustained was considered relatively extensive and substantial repairs or replacement would be required. As a result of the earthquakes the building's capacity was reduced, although it was not considered to pose an immediate collapse hazard in a moderate earthquake.

6.2.1.6 Conclusions

Only a few of the more significant aspects of damage are discussed here.

6.2.1.6.1 Spalling and buckling of reinforcement in shear walls

The walls were detailed for confinement and buckling restraint of bars below level 1. However, from the observed damage it appears that reinforcement yielded over a greater length than anticipated in the design. As there was damage to the seismic joint between the podium and tower it is likely that interaction between the two structures could have caused plastic hinging to occur at higher levels in the tower than would otherwise be anticipated (see Figure 80).

Current design requirements (NZS 3101:2006[12]) require ductile detailing to extend for a distance equal to the height of the shear core divided by six. Damage to the expansion gap of 25mm was not unexpected, for two reasons. First, the design standard of the time recommended the use of stiffer section properties for assessing design actions than is now the case, which would have led to predictions of lower lateral displacements than would be expected from current practice. Secondly, the design displacement was taken as 50 per cent of the peak displacement calculated on the basis of the equal displacement concept.

The buckling of the longitudinal reinforcement observed in the wall below level 3 highlights the importance of providing lateral restraint to longitudinal bars in the potential plastic hinge zones (ductile detailing lengths) of walls (refer to Figures 81–83). The extensive spalling of the cover concrete results in the loss of anchorage of the transverse reinforcement, which creates a potential problem that has not been addressed in the current design standards.

Figure 80: Damage to seismic joint (source: Spencer Holmes)

Figure 81: Spalling on shear wall (source: Spencer Holmes)

Figure 82: Buckling of vertical reinforcement level 1 (source: Spencer Holmes)

Figure 83: Buckling of reinforcement level 2 (source: Spencer Holmes)

6.2.1.6.2 Concrete spalling at junction of truss beams and columns

Extensive spalling of the cover concrete in the columns occurred adjacent to the junctions with the truss beams, which were embedded in the columns (refer to Figures 84 and 85). The detail was designed for gravity load transfer but the connection detail was not designed to allow for the relative rotation that could result from inter-storey drift. The relative rotation between the truss beams and the columns generates a prising action of the beam on the column, with the cover concrete outside the spiral spalling over an appreciable length. The extent of this spalling may have been increased as each truss beam was anchored onto a longitudinal bar in the column, which might have been pulled sideways by the prising action (see Figures 86 and 87).

Figure 84: Spalling of circular external column (source: Spencer Holmes)

Figure 85: Spalling of circular external column at junction with precast truss (source: Spencer Holmes)

As mentioned above, the precast truss beams were anchored into the perimeter columns. However, during an inspection of the building it was noted that there was an outward movement of the columns relative to the floor, of the order of 25 to 30mm (see Figure 86).

Figure 86: Tearing between precast truss and floor (source: Spencer Holmes)

The origin of this movement can be traced to elongation associated with the structure, as shown in Figure 87. The depth of the beam at the junction with the column is 900mm. A storey drift of two per cent would cause, on average, an elongation of 9mm at each column when the seismic motion ceased. Where the enclosed angle between the truss beams meeting at a column is 135°, the application of 9mm elongation from each bay corresponds to an outward displacement of 24mm, which is similar to the observed movement. Where the enclosed angle is 90° the corresponding movement is 13mm. The forces required to restrain the outward movement of the columns would be very high and it would not be practical to provide this restraint.

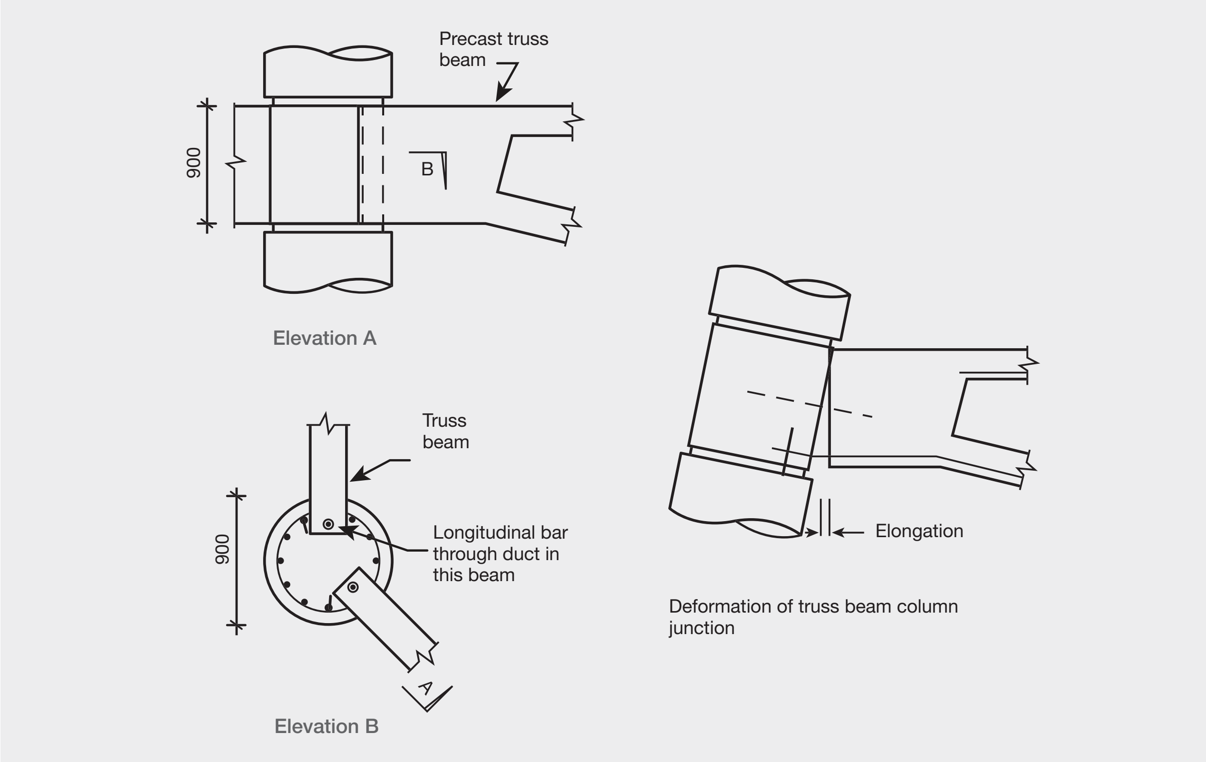


Figure 87: Junction of truss beam and a perimeter column

6.2.1.6.3 Incompatible deformation damage

The walls making up the shear core behaved as flexural members, causing the floors adjacent to the walls to rotate. The gravity load system consisting of the columns and truss beams tended to be displaced laterally without rotation. These two different patterns of deformation imposed rotation and twisting on the precast and in situ floor components. We consider that this source of incompatible deformation largely accounts for the observed damage to the floors.

6.3 1984 to 1992: Buildings designed to Loadings Code NZS 4203:1984[13]

6.3.1 90 Armagh Street: Craigs Investment House building

Current status
The building is still standing, but its future is not known to the Royal Commission.

Figure 88: View from the north-west (source: Becker Fraser Photos)

6.3.1.1 Introduction

Craigs Investment House at 90 Armagh Street was designed during 1985 and 1986. Building permits were issued by the CCC in 1986 (the foundations and the main structure were the subject of separate permits); the completion date is not certain from the information available. The building is 10 storeys high plus a basement, with a total height of 35.1m (Figure 88). It is located approximately 25m from the banks of the Avon River.

6.3.1.2 Structural system

6.3.1.2.1 Foundations

The foundations comprise a concrete raft 300–900mm thick with the retaining walls to the basement 300mm cast in situ concrete.

6.3.1.2.2 Structural floors

The ground floor is a 250mm thick cast in situ slab that acts as a diaphragm to transfer seismic loads to the perimeter basement wall on the northern side. The elevated floors are formed by double-tee precast reinforced concrete units, with a 65mm in situ concrete topping reinforced with standard 665 mesh reinforcement. These floors act as diaphragms. The location of the stairs and lifts in the tower creates a significant cut-out in the floors, which is offset to the south-west of the building's centre lines (see Figure 89). The double-tee units in the floors span in the north–south direction. They are supported by moment resisting frames on the southern and northern sides of the building (on grid lines 1 and 6 respectively) and by an intermediate moment resisting frame on grid line 3.

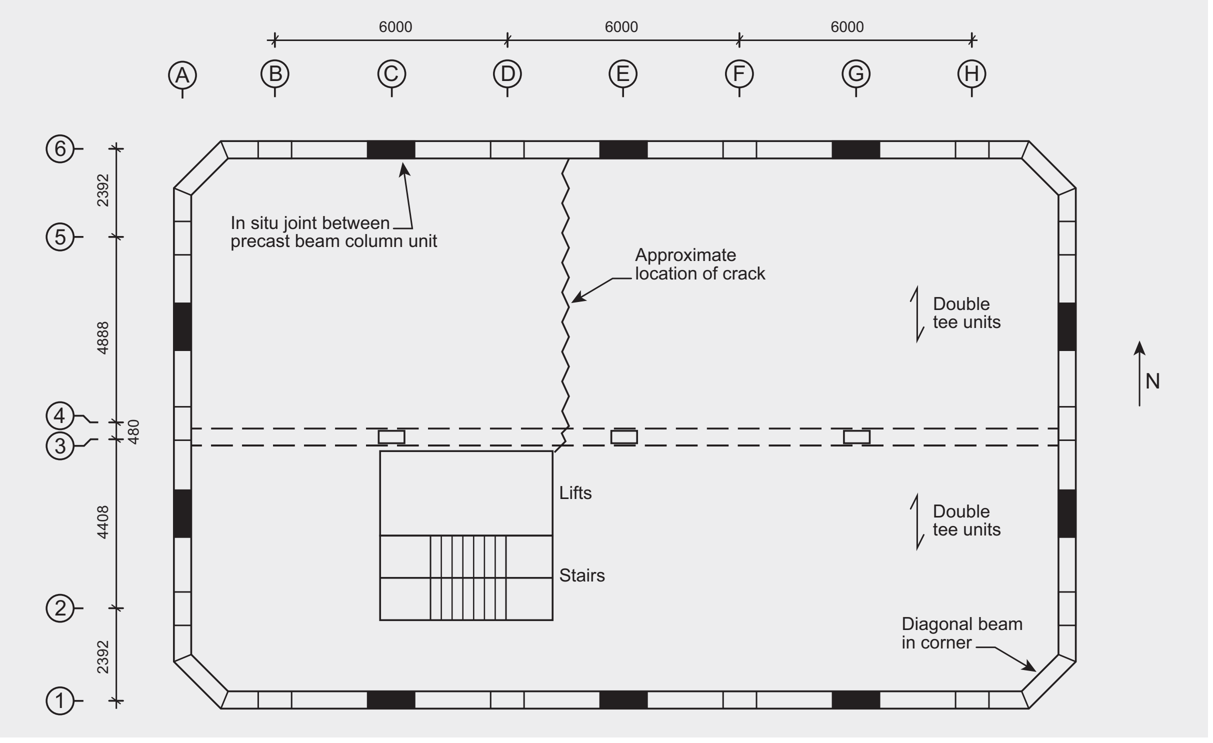


Figure 89: Typical upper level plan

6.3.1.2.3 Lateral force system

The east–west lateral force resistance in the tower is provided primarily by the reinforced concrete moment resisting frames on the southern and northern sides of the building, with a minor contribution (due to its low relative lateral stiffness) of the moment resisting frame on line 3. The lateral force resistance in the north–south direction is provided by the moment resisting frames on the western and eastern walls of the building, on grid lines A and J (see Figure 89). The frames were built from precast units. These were tee-shaped units consisting of a column with a beam on top (see Figure 90). The longitudinal reinforcement in the beams projected by 1100mm, which allowed it to be lapped within in situ concrete joints between the precast units (see Figure 90). The column sections measure 900mm by 450mm and the beams are 900mm deep by 320mm wide. The longitudinal reinforcement in the columns is Grade 380 and in the beams it is Grade 275. To establish continuity between the beams there is a cast in situ joint in the mid-span region of the bays. Continuity of the columns is provided by joining the longitudinal column reinforcement with mechanical splices (NBM type U).

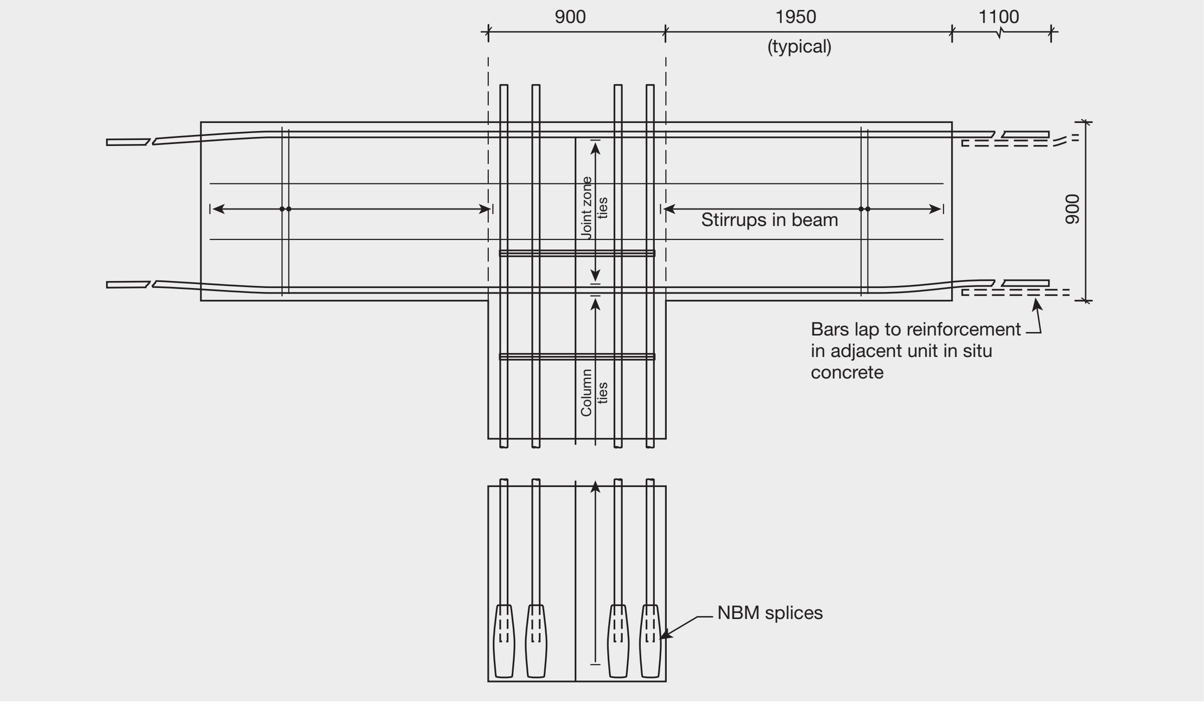


Figure 90: Typical details of precast beam column units

There are no corner columns in the tower. As shown in Figure 89, the beams at the elevated levels of the building near the corners cantilever out from the column to support a diagonal beam across each corner. The supports to the diagonal beam act as pin connections. This allows differential movements to occur between the east–west and north–south frames.

Both the moment resisting frames on the northern and southern sides of the building have three bays bounded by columns, with an additional bay at each end that contains the diagonal corner beams. The moment resisting frame close to grid line 3 has four bays. The beam and column dimensions in this frame are smaller than the moment resisting frames on the building perimeter, reducing its lateral stiffness. The primary function of the frame on grid line 3 is to provide support for gravity loads. The moment resisting frames on the eastern and western sides of the building, on grid lines A and J, each have two bays bounded by columns with an additional bay at each end containing the diagonal corner beam.

6.3.1.2.4 Stairs

There is a single stairway in the building and the detail shown on the drawings indicates that the flights from each level are seismically separated at mid-storey height. There was no indication that egress via the stairs was compromised by any of the earthquakes.

Precast panels

Precast concrete wall panels 100mm thick were fitted to the southern and eastern walls. Each panel was supported by two mechanical splices cast into the beam, with two fixing brackets that had slotted holes to fix the lower level of the panel and allow for lateral movement.

6.3.1.3 Performance of the building

The Royal Commission was assisted in the assessment of the performance of this building by a report prepared by Spencer Holmes.

6.3.1.3.1 Foundations and precast panels

There has been no information provided to the Royal Commission indicating that the building was damaged by earthquakes before 22 February 2011. As a result of the February earthquake, the building tilted by about 0.5 degrees towards the south-east, which equates to a tilt of about 300mm at the top of the building. We infer that this tilt was due to liquefaction and lateral spreading of the land causing the foundations to settle and rotate. The heavy precast concrete wall panels on the southern and eastern sides, and the additional forces on the foundation soils from the Victoria Square apartments building at 100 Armagh Street (on the eastern side), would have contributed to the lateral deflection to the south-east. We note that the Victoria Square building is also assessed, in section 6.4.1 of this Volume.

There has been damage to glazing, which was due to a clash with a decorative feature on the Victoria Apartments building as well as crushed drainage pipes in the seismic gap between the buildings. The damage indicates that there had been minor pounding against the adjacent structure.

Little damage was found in the structure of the foundations other than the rotation and settlement described above, which was likely to be due to liquefaction of sand and silt layers in the foundations. Appreciable liquefaction was observed in the vicinity of the building.

The precast panel walls of the eastern and southern sides were not damaged, indicating that the seismic separation provided for these was effective.

6.3.1.3.2 Moment resisting frames

The moment resisting frames in general appear to have performed as expected. Below level 7 some single cracks were seen in the beams at the column faces. However, in some cases a fan of diagonal cracks had formed in a similar pattern, as has been observed in many laboratory tests.

Above level 7, the cracking in the beam-column joint transitioned into a crack to the column at the underside of the joint that extended to above the bottom horizontal beam reinforcement, where it passed through the beam-column joint zone. The high-frequency ground motion provided a possible explanation for the horizontal cracking observed. The vertical spectral accelerations for the sites where ground motion was measured for a period of close to 0.1 seconds ranged from 1.5–2.4g (refer to the Carr report[14]). If the frequency of vibration of the beams in the vertical direction was close to 0.1 seconds, the high accelerations could account for the observed cracking.

In a number of cases horizontal cracks had formed in the beams in the mid-span region where the longitudinal bars from adjacent precast units in the same frame were lapped into the in situ concrete. In some cases the concrete below the lapped reinforcement had spalled. The laps as detailed would have satisfied the requirements in NZS 3101:2006[12] for lapped splices. It was not clear why cracking and spalling was observed at these locations, as the stress in the bars there should not have been high. It is noted that the clear gap between the lapped longitudinal bars is 42mm, which was equal to 1.3 bar diameters.

There was some damage to the diagonal corner beams. These beams were supported by pin joints located in the beams, which cantilever towards the corners. The movement allowed to accommodate the relative movement between the two cantilevered beams appears to have been inadequate for the relative movements induced in the February earthquake. Consequently some damage was induced in the diagonal and cantilever beams.

6.3.1.3.3 Floor diaphragms

The diaphragms in the first to fifth floors had cracked, with crack widths of 4mm. It appeared that the 665 mesh in these floors had fractured at these cracks. The higher floors may have been damaged in a similar manner but they were not examined for this cracking. On each floor, the cracks ran in a north–south direction across the floor between the eastern side of the opening in the floor for the stairs and lifts, to near grid line 6 on the northern side of the building (see Figure 89). We consider that the formation of these cracks is due to elongation of the beams associated with the formation of plastic hinges in the east–west moment resisting frames on grid lines 1, 3 and 6.

6.3.1.4 Conclusions

We conclude that:

1. The building was designed with ductile moment resisting frames providing the lateral force resistance. Its performance in the February 2011 earthquake is consistent with the design philosophy inherent in the structural design standards current at the time of design and at present. The beams in the moment resisting frames developed cracks that remained open because of yielding of the reinforcement. In some cases fan-shaped cracks formed; these remained open, indicating that the yield zone had extended for some distance along the beam. Single column face cracks are typical in beam-column joint tests conducted at curvature ductility levels of one third the maximum permitted in NZS 3101:2006[12]. Other cracks may have formed in the beams but unless the reinforcement had yielded at these cracks they would close and be difficult to notice in an inspection.
2. The tear in the floor diaphragms is consistent with beam plastic hinge formation and the associated elongation in these plastic hinges, which applied tension to the floors.
3. There was some spalling in the lap zones of the longitudinal reinforcement located in the mid-span region of the beams within in situ concrete. There is no clear explanation for this damage. The laps conformed to the requirements for lapped splices in NZS 3101:2006[12]. The limited horizontal spacing between the bars may have contributed to this cracking, but this spacing is within the limits in NZS 3101:2006[12].
4. The principal problem with the building was the differential settlement of the foundations, which was likely due to liquefaction of the foundation soils below the spread foundations.

6.3.2 20 Bedford Row: Bedford Row Public Car park building

Current status
Demolished.

Figure 91: View of Bedford Row Public Car park building from Bedford Row, looking south-east

6.3.2.1 Introduction

The Bedford Row Public Car park was a multi-level car park building with frontages to both Lichfield Street and Bedford Row. It was six storeys high, with each storey on the eastern and western sides offset by half a storey, to give a total of 12 levels. These levels were linked by ramps. The total plan size was about 35m by 40m.

The design and approval for the building both occurred in 1987. We are not sure when construction was completed but expect that it was some time in 1988. The design certificate for the building verified that it was designed to the NZS 4203:1984[13] Loadings Code and the concrete construction was to be in accordance with NZS 3101. This is presumed to be a reference to NZS 3101:1982[11], as that Standard applied at the time.

6.3.2.2 Foundations and ground floor

The foundations were shallow ground beams to the perimeter and through the middle (running north–south). These beams were supported on the east and west sides of the building with piles of unknown depth. The ground floor was an unreinforced concrete slab on grade.

6.3.2.3 Building structure

The above-ground floors were 500mm deep double-tee prestressed precast concrete floor units spanning about 17m, with a 65mm concrete topping reinforced with 664 mesh. The floor units were supported on a corbel to the east and west sides of the building (see cross-section B–B, Figure 94) and by precast beams at the centre line of the building. The beams were in turn supported by rebates on a central row of precast columns (see cross-section A–A, Figure 95). An outline of the structural arrangement is shown in Figure 92. The ramps were built using 220mm deep double-tee units spanning about 10m between two beams. The eastern and western walls were 200mm thick precast tilt-up concrete panels to a height of 6.27m, with a further 6.9m of 200mm thick blockwork. The walls on lines 1, 2 and 7 were made from precast panels.

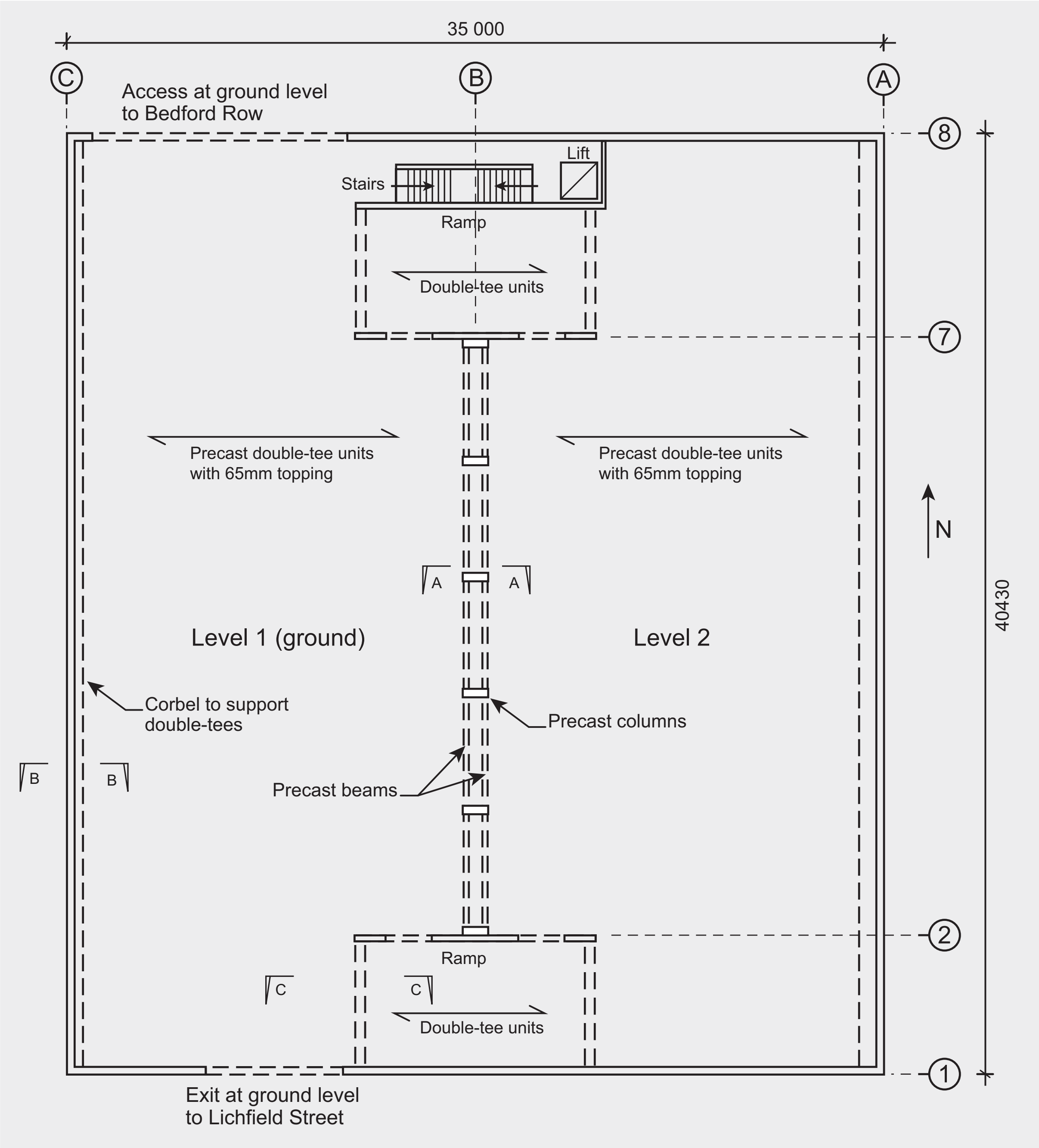


Figure 92: Floor plan

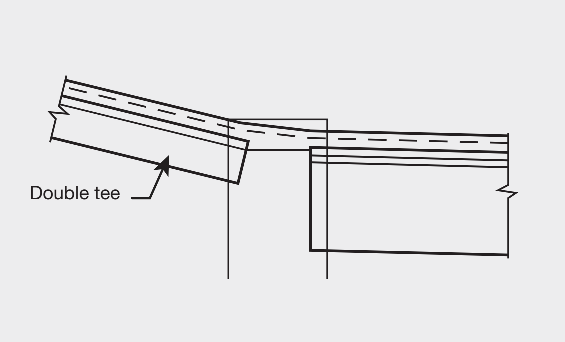


Figure 93: Cross-section C–C

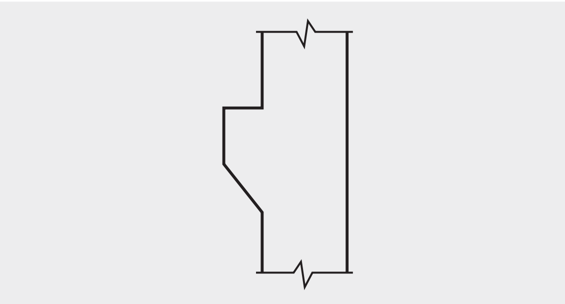


Figure 94: Cross-section B–B

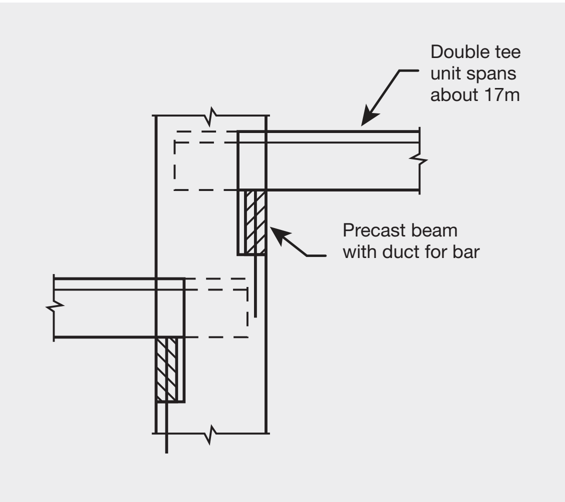


Figure 95: Cross-section A–A

The primary lateral force resistance in the north–south direction was provided by the in-plane actions of the eastern and western walls, which were constructed with precast tilt-up panels 4035mm in width. The walls were extended above the panels by reinforced concrete blockwork. In the east–west direction the shear walls along the Lichfield Street and Bedford Row frontages were 240mm thick cast in situ reinforced concrete. These were not continuous along the frontages, and on the Bedford Row side comprised a double wall on either side of the stairwell/liftshaft.

Precast concrete spandrel panels were fixed on the northern and southern faces of the building to act as balustrades to each level.

Most of the stairs were precast flights that were tied into cast in situ landings at the top of each flight. The lower support may have been sliding in most cases but the drawings are not entirely clear in this matter.

The floors, which were required to act as diaphragms to distribute the seismic forces to the structural walls, provided part of a complex load path. The floor diaphragms lacked an adequate continuous tension chord along the eastern and western walls. In addition, some of the induced membrane forces within the diaphragms were required to be transferred to the resisting walls by inclined ramps.

6.3.2.4 Performance of the building

The Royal Commission was assisted in the assessment of the performance of this building by a report prepared by Spencer Holmes Ltd.

The building was placarded green, “Inspected – no restriction on use or occupation” after the September earthquake. There has been no information provided to the Royal Commission that identifies damage to the building as the result of earthquakes up until the Boxing Day aftershock. After this event, the building was placarded red “Do not approach or enter this building” and a notice issued under section 124 of the Building Act 2004, on the premise that the building was dangerous. This was because cover concrete had spalled and it was considered that loose sections could dislodge in a significant aftershock, endangering the public. Concrete also spalled on a central column on level 10. Loose areas of concrete were removed and the building was opened up to level 9 later in January 2012; the opening of upper levels was delayed until the level 10 column was repaired.

The February earthquake resulted in significant damage to the building and as a result it has now been demolished. The most obvious damage was the failure of the support of one of the precast central beams that supported the double-tee floor system at level 3. The photograph in Figure 96 was taken at level 1 (ground level) and shows that the beam supporting level 3 lost support, resulting in the double-tee units collapsing onto a number of vehicles in the car park at level 1.

Figure 96: Collapse of the double-tee units from level 3

6.3.2.5 Discussion

The lateral forces induced by the earthquake in the east–west direction were transmitted to the northern and southern walls by the diaphragm action of the floor slab and the inclined ramps that connected each level of the building. To transmit the seismic forces to the northern and southern walls the ramps had to sustain tension and compression forces. Under tension in the ramps there was an upward component of force that acted in consequence of the change in grade above the beams (see cross-section C–C, Figure 93). We consider that this component would have been sufficient to separate the in situ concrete from the supporting beam, resulting in some damage to the concrete at the junction between the ramps and the floor diaphragms.

Seismic-induced forces in the north–south direction were resisted by the eastern and western walls, which, as noted previously, were constructed from precast tilt-up panels, with reinforced block work in the higher levels. Some diagonal cracking and sliding shear was observed in these walls. The seismic forces were transmitted to these walls by the continuity between the topping concrete on the double-tees. This was established by reinforcement bent out from the wall panels and into the in situ concrete on top of the precast double-tee units where it lapped the 664 mesh reinforcement. The individual wall panels acted as cantilevers to resist the in-plane lateral forces. This action led to small vertical displacements being induced at the junction between the panels. The floor was continuous across the junctions between the precast panels. The relative vertical movement across at the junctions, caused by the flexural action in the panels, damaged the floor and in some cases also damaged the double-tee units in the vicinity of the walls.

The level 3 floor appears to have partially collapsed when the concrete failed on the underside of a rebate in the central columns. The detail is shown in Figure 97. The precast beams that supported the 500mm deep double-tee units were held in place in the rebates of the central columns by 24mm bars that extended into the rebate from the column into ducts in the beams. Once assembled, the ducts were grouted (see Figures 97 and 98).

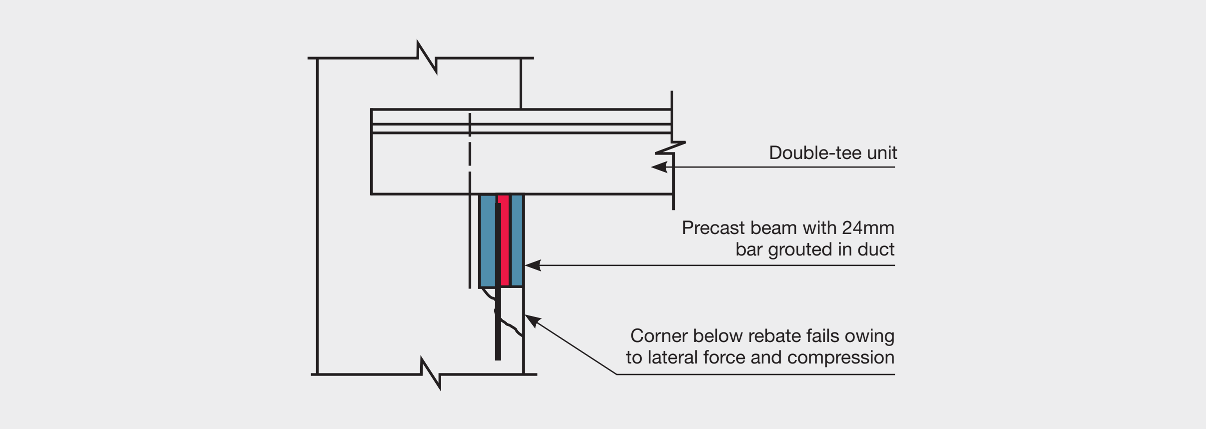


Figure 97: Support detail for beam and double-tee units on the central columns

Figure 98: Undamaged connection at top; the lower connection has failed (largely unseen)

There was no connection between the in situ concrete on the precast double-tee units, the rectangular support beam and the column. The only way that lateral force could be transmitted between the double-tee units and the support beam was through friction. We consider that, under the high lateral forces generated in the February earthquake, the lateral friction force applied to the precast beam was sufficient to cause the beam to rock over the outside edge of the rebate in the column. The gap left between the double-tee unit and the top surface of the rebate would have been insufficient to prevent a wedging action. The resultant pressure on the lower surface of the rebate was sufficient to cause the concrete below the rebate to spall, resulting in collapse of the beam.

There was appreciable damage to the support zones of the double-tee units, where friction forces had caused spalling on the underside of the units, and in one or two cases there was some diagonal cracking in the webs.

6.3.2.5 Conclusions

We conclude that:

1. The load path for seismic forces acting on the floors included transferring the forces into the ramps and the structural walls. The load path required:
   (a) the floors to act as horizontal beams. However, there was no continuous tension chord in the floors to allow this action.
   (b) the ramp to resist both tension and compression forces. The out-of-plane forces resulted in significant spalling damage at this location. The edges of the diaphragm to both the floor and the ramp at these points did not include an adequate continuous tension chord.
   (c) a tension chord to resist flexural tension in the diaphragms at the eastern and western sides of the building. No specific chord was provided. The wall panels were not connected for horizontal tension.

2. The friction between the double-tee units and the precast beams created a horizontal force at the top of the beam that it had not been designed to sustain. This caused the major failure that is visible in Figure 96. It is important to ensure that elements not intended to contribute to the seismic resistance of a building are designed to sustain forces and displacements that may be imposed on them. The use of low-friction bearing strips at the supports of the double-tee units would have avoided this problem.
3. At the connection of the eastern and western walls to the diaphragm floors there was an incompatibility between the displacements in the sliding vertical movement of the wall panels under seismic actions and the relative rigidity of the floor diaphragms. The implications of incompatible displacements on adjacent structural elements should be considered in design and appropriate steps taken to avoid loss of strength from the development of these displacements.

6.3.3 79 Cambridge Terrace: Bradley Nuttall House building

Current status
Unoccupied with temporary repairs in place – future yet to be determined.

Figure 99: Bradley Nuttall House viewed from Cashel Street

6.3.3.1 Introduction

The reinforced concrete building located at 79 Cambridge Terrace, known as Bradley Nuttall House, is a seven-storey office building situated approximately 40m from the banks of the Avon River (Figure 99). The building is square in plan, measuring about 24m in each direction. A building permit was issued by the CCC in October 1985, indicating that design would have been carried out earlier in that year. The Royal Commission does not have any information as to the construction commencement or completion dates.

6.3.3.2 Building structure

A plan view of the layout of the building with the main structural elements labelled is given in Figure 100. The lateral force resisting structural system consists of four ductile reinforced concrete walls that form a shear core around the stairs, lifts and toilet facilities. To allow for access into the shear core at each level, one wall (wall E) has voids and diagonally reinforced precast coupling beams at each wall end. The coupling beams are positioned above the doorways and act to tie wall E into the perpendicularly-oriented core walls (walls 3 and 4). The shear core is eccentrically positioned on the south-western side of the building.

The concrete floors are connected to the shear walls and act as a primary load path for inertial forces to track back into the walls. The elevated floors consist of 100mm deep prestressed concrete beams spaced at 900mm centres with timber infill that acts as permanent formwork for the 75mm thick cast-in situ concrete floor reinforced with 665 steel mesh. The precast beams are supported on precast shell beams that span 7.2m in the south-western to north-eastern direction between column supports.

The columns extend from the ground floor up to the sixth level. The 400mm square columns are essentially gravity columns that support the weight of the floors and roof. The shear core extends up a further two levels to where the plant and service rooms are, at the seventh level, with the lift motor room on the top level, shown in Figure 101.

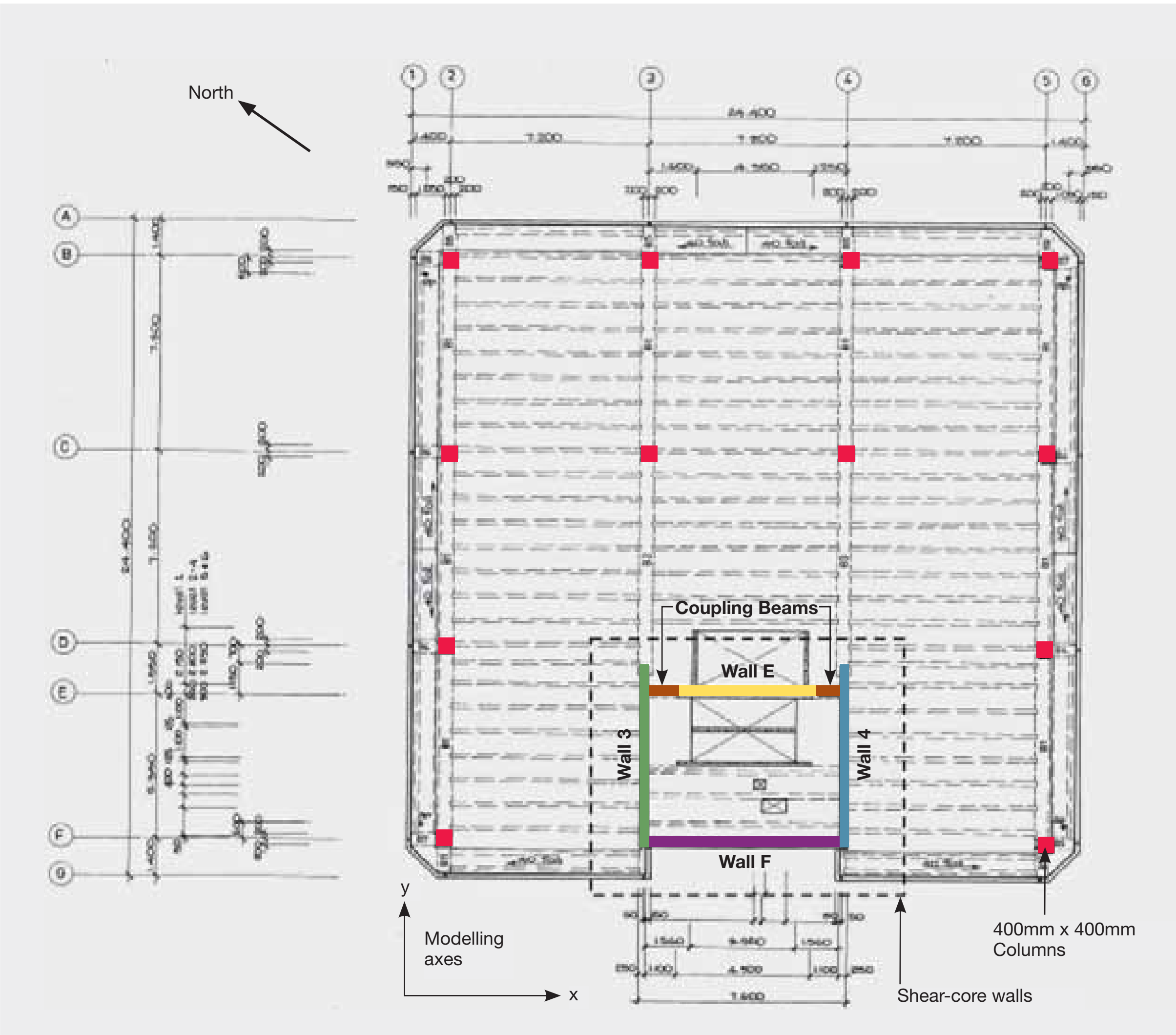


Figure 100: Typical floor plan, levels 1–6 (source: Modified from original approved plans)

The foundations transfer loads to the ground through a combination of shallow pads and bulb piles. The pads that are directly connected to the shear core structure consist of reinforced concrete beams on bulb piles located roughly in a perimeter surrounding the shear core. The reinforced bulb piles extend 9.5m below ground and are assumed to act as tension piles when the shear core is overturning from seismic loadings. The column foundations on the north-western side of the building are separate from the foundation beam and pile system and are simply founded on 2.8m square bearing pads. The ground floor is a 100mm thick unreinforced slab cast on compacted hard fill.

Other features of the building include precast concrete scissor stairs located within the shear core. These stairs have steel extensions top and bottom that are grouted into cast in situ concrete landings. Reinforced block walls that cantilever up from the floor slab are located on the ground floor. Around the liftshaft these block walls are also tied into the concrete floor of level 1. The roof is a light steel structure arrangement of steel roofing, supported on steel purlins, beams and posts. The building exterior is clad with precast concrete panels.

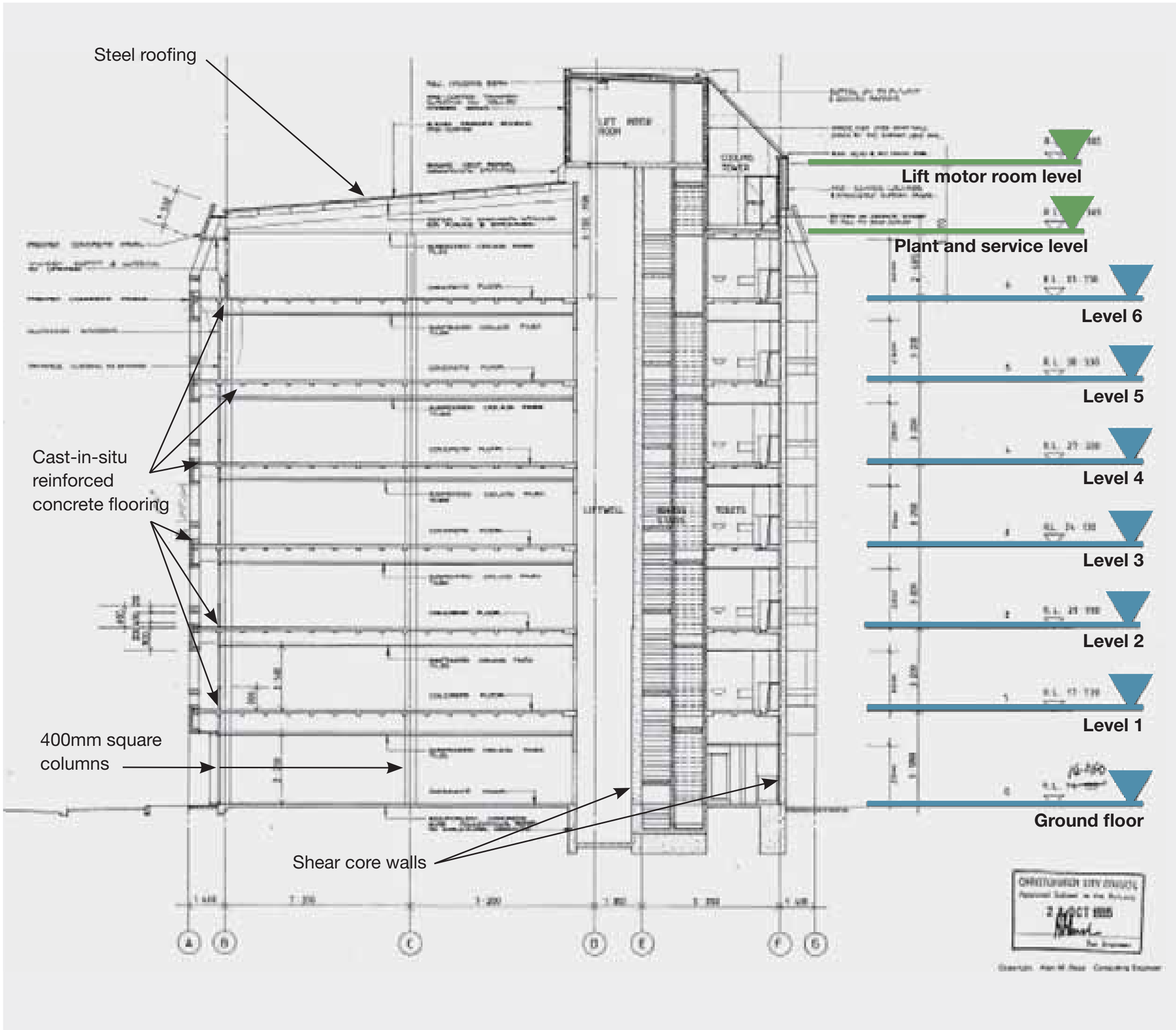


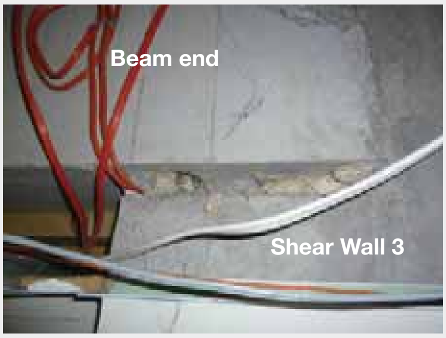
Figure 101: Cross-section through building (source: Modified from original approved plans)

6.3.3.3 Post-earthquake structural inspection

A consulting structural engineer, Mr Michael Fletcher of Buchanan & Fletcher Ltd, was engaged by the owner to carry out structural engineering inspections and assessments after the February earthquake. The account that follows is based on Mr Fletcher's report.

The main visual inspection was conducted on 14 March 2011. Mr Fletcher's observations on damage caused by the earthquakes are summarised as follows:

Area of interest	Observed damage
Concrete columns	– Some cracking in ground floor columns – In the level 3 columns there is some cracking in the cover concrete and in the top of the corner columns
Shear walls	Walls 3 and 4 – These walls show an extensive pattern of diagonal cracking at the lower levels with cracks at about 300mm centres. Cracking in the upper levels is more widely spaced. Where checked, the cracks run right through the walls. The cracks are at about 45 degrees in both directions and extend the full height and length of the walls – Crack widths are in the order of 0.5mm at the lower levels reducing to 0.2mm near the top of the building – Generally, the damage to wall 3 is more significant than the damage to wall 4 Wall F – Multiple horizontal cracks are visible for at least three levels above ground, regularly spaced at about 300mm centres (Figure 102(a)) Wall E – This is the wall located at the back of the liftshaft and connected to walls 3 and 4 by precast concrete coupling beams – No cracks were observed in wall E itself but there is vertical cracking visible at the joint between the coupling beams and the main wall at most levels. Crack widths are estimated to be 1.0mm at level 1 down to 0.1mm at level 6. Diagonal cracking in the coupling beams in the lower levels is also noted Junction between wall 3 and end of wall F – There is a significant horizontal crack (see Figure 102(b)) up to 0.7mm in width. Adjacent to wall 3 (on the other side) is a door opening in wall F

Area of interest	Observed damage
Shear walls *(continued)*	– Cover concrete has spalled off at the base of wall F at the north-western end  (a) Exterior view of wall F  (b) Junction of wall 3 and wall F **Figure 102: Horizontal cracking in shear walls at ground floor**
Connections between shear walls and floor beams (on lines 3 and 4)	The concrete floor beams transfer some of the inertial forces into the end of walls 3 and 4 – Inspections at levels 1, 2, 4 and 6 showed vertical hairline cracking in the beams – Some moderate damage to the shear-wall-to-beam connection was observed, with some spalling at the end of the shear wall, as well as vertical cracking in the beam and wall (see Figure 103)  **Figure 103: Connection of line 3 beam into the end of wall 3 at level 2**
Connections between shear walls and floor slabs	These were checked from the underside at a number of locations with no sign of damage or movement noted
Site and surrounds	The building is located about 40m from the Avon River In the adjacent building there is slumping in the ramp leading down to the car park basement There is no evidence of liquefaction or settlement on the other three sides of the building or between the building and the river

Further information on the consultant's findings and recommendations, results from a detailed elastic seismic analysis and future proposed works have been excluded from this Report.

6.3.3.4 Geotechnical site investigation

Following the post-earthquake structural inspection, a geotechnical investigation was carried out by Geotech Consulting Ltd who prepared a site investigation report dated 29 November 2011. According to the Geotech report, the ground conditions at the site consist of a surface layer of sandy gravel underlain by loose to very loose saturated sands (susceptible to liquefaction between 5–13m), with dense sands located at around 19m depth.

The post-earthquake structural inspection suggested that there were no significant surface manifestations of ground movements. The building may have settled differentially to a small degree (up to 50mm) and some slumping in nearby buildings indicated that some liquefaction occurred in the area.

6.3.3.5 Method of analysis used for this report

The Royal Commission carried out a non-linear time history analysis with the assistance of Professor Athol Carr, in order to study the seismic performance of the building.

The analysis was carried out using the programme, Ruaumoko 3D[15], which is capable of modelling the post-elastic properties of structural elements as they undergo yielding. The inelastic time history method of analysis was used. Some of the key modelling assumptions included:

6.3.3.5.1 Earthquake loading

The earthquake loading was scaled to the current new building design level loading. The three scaled earthquake records chosen[16] were the Tabas (Tabas 16/09/78), Smart (Taiwan 14/11/86) and F52360 (Taft Lincoln School (tunnel 21/07/52)) records. These records were scaled in accordance with NZS 1170.5:2004[10], with a hazard factor (Z) of 0.3 and type D soil for a normal-use office building.

An analysis was made using the unscaled Christchurch Hospital (CHHC site) record from the February earthquake.

6.3.3.5.2 Assumptions

1. The weight of the building includes the self-weight of structural elements, dead loads from fixtures and fittings (for example, partitions and cladding) as well as the long-term live load.
2. All analyses were modelled with a rigid foundation. To check the sensitivity of foundation stiffness a trial case with foundation beams and soil springs was modelled.
3. The floor plate is assumed to be rigid for in-plane forces but modelled to allow flexural deformation. The floors were connected to the walls by link elements to allow the transfer forces to be recorded.
4. Beams and columns were assumed to have a cracked stiffness of 0.4 I_{gross} with the shear walls having a cracked stiffness of 0.25 I_{gross} over the full length of the members.
5. Inelastic shear, tension yield and moment-axial interaction capacities were modelled over the first storey of all four walls. The second storey allowed for flexural yielding with the walls elastic above this level. The shear capacity of the walls was based on a concrete contribution, V_c, of $0.17\sqrt{f'_c}$, plus an allowance for axial load, plus the horizontal reinforcement in the wall, V_s. Inelastic shear was based on the SINA hysteresis model.
6. The moment-axial interaction capacities of the columns were modelled.
7. Five per cent viscous damping was assumed for all 205 modes of free vibration.

6.3.3.6 Findings

A time step of 1/20,000th a second was required to keep the analysis stable. This is due to the structure having relatively short members with little mass, resulting in a high frequency. The fundamental period of the rigid base structure is 1.0s in the translational north-east to south-west direction (y-direction, see Figure 100) and 1.25s in the fundamental torsional mode. With the flexible foundation modelling these natural periods became 1.32 and 1.26 seconds respectively.

The primary modes of vibration for the structure indicated that an earthquake would induce a highly torsional building response.

The analyses show that the walls are subjected to axial tension forces that have the potential to yield the vertical wall reinforcement. Therefore, to model the flexure and axial behaviour at the base of the walls a multi-spring element was used to represent the concrete and steel properties. A gapping spring element represented the concrete stiffness, with no tension and a bi-linear spring element represented the vertical steel properties.

6.3.3.6.1 Walls

Shear deformations had a significant effect on the building's response. By modelling the inelastic shear, a different building behaviour was observed compared to modelling the walls with elastic shear stiffness and flexural yield at the base of the walls. Once shear deformation occurs, this type of deformation occurs more easily than deformation caused by flexure in the walls.

Tension yielding is most critical in wall F owing to the combination of its lower axial compression load and the overturning in the y-direction. All walls indicate cracking during the design-level earthquake excitations, with some yielding of the steel once or twice during the event. For the February earthquake all four walls yielded in axial tension. Wall E yielded numerous times, while wall F underwent the most significant axial forces.

6.3.3.6.2 Connection of floors to walls

The design-level connection force was calculated to be greatest at the upper levels, with the coupled forces from torsional deformation of the building giving about 2200kN acting in opposite directions on walls 3 and 4 at level 6. The connection peak force is achieved only once or twice over the time history record. In the February earthquake analysis the magnitude of these shears reached about 5200kN.

6.3.3.6.3 Columns

The static gravity axial loads on the ground floor in the corner columns were found to be about 1100kN but for the interior columns on grid line C the static axial force was about 2400kN. The columns did not pick up any significant additional axial force caused by seismic motion.

The corner columns underwent the most severe deformations/drifts as they were at the greatest radius from the centre of the shear core. Design level inter-storey drifts of around 0.8 per cent were calculated from the analysis. In the February earthquake the analysis indicated that the inter-storey drifts were about 2.4 per cent. The torsional response of the building was illustrated by the movement of the corner columns, predominately in a direction pivoting about the centre of the shear core walls.

6.3.3.6.4 Foundations

Modelling the foundation stiffness has the effect of lengthening the building period. The walls sitting directly on the foundation beams show increased vertical and rocking movements. Since the bulb piles are located in a liquefiable layer there is a potential for these piles to uplift under axial tension forces, but this effect was not modelled.

6.3.3.7 Conclusions

Our conclusions are as follows:

1. **Walls**

   The analysis was consistent with the observed damage, in that the walls were highly stressed in the February 2011 earthquake.

Analyses indicate that under a design-level earthquake, walls 3 and 4 underwent shear cracking and some shear yield at the base between the ground floor and level 1. Diagonal shear cracking was observed in both directions along the full height and length of these walls. Wall F was shown to be subjected to high axial tension loads and extension, and the horizontal cracking pattern observed indicated that this occurred. The analyses predicted that the vertical bars in wall F yielded and possibly fractured, or that the wall foundations uplifted to stop bar yield and/or fracture from occurring. Uplift of the foundation for wall F would require tension failure or rocking of the piles under the wall footing. Owing to the weight of the building the structure would have self-centred and closed the cracks that had opened up in wall F, making the damage possibly appear less extensive than it really was.

The building concept results in an eccentricity of the centre of mass from the centre of lateral stiffness. Had this eccentricity been reduced, the seismic performance of the building would have greatly improved.

2. **Connection of floors to walls**

   Damage was observed close to the junction of the precast concrete beams and walls 3 and 4. Vertical cracking indicated that this connection was subjected to axial tension forces. There are two main load paths for the inertial forces to be transferred into the shear walls. The most direct is through the end of the precast beams. The other load path is the connection of the floors to the sides of walls 3 and 4 through steel reinforcing bars placed perpendicular to the length of the walls (see Figure 104).

These bars are cast in situ with the concrete floor slab and walls and extend a distance on either side of the wall faces. This load path requires movement and diagonal cracking in the floor to develop the full connection strength. Adding the strength of the two connections together is not a conservative approach, as development of the full strength for each connection will occur at different deformations.

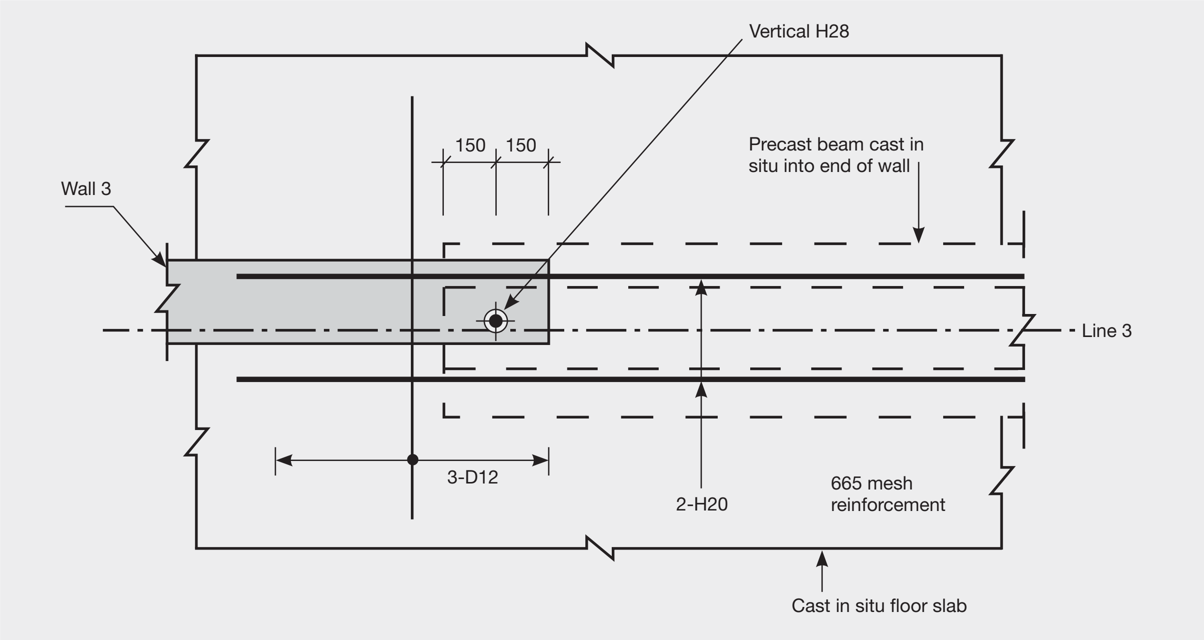


Figure 104: Plan view of floor and precast beam connection to wall 3 (or 4)

The indirect load paths of this connection make it a detail we would not recommend.

A strut and tie calculation of the nominal connection capacity between wall 3 or 4 and the adjacent concrete floor topping is in the order of 2200kN per wall. This is assuming the minimum strut angle of 25 degrees as stated in NZS 3101:2006[12]. The Christchurch earthquake record theoretically shows demands that exceeded this nominal capacity. There are a number of possible reasons why this may not have occurred:

1. In practice, the angle between the axes of the strut and the tie could be less than 25 degrees. Since the strut was forming against the concrete there was less chance of slipping and a higher connection force might be achieved.
2. Since the shear core was encompassed by the floors, some slab reinforcement acted in tension to stop the floors from separating perpendicular to the length of the walls, acting like a vice clamp around the walls.
3. The actual ground motions at the site ground surface may have been different from those measured at the Christchurch Hospital site, particularly if liquefaction occurred at depth.
4. The stiffness and ductility in the connection may have reduced the force demands.

3. Columns

The columns have beams framing between them in one direction (along grid lines 2, 3, 4 and 5) with only the concrete slab spanning in the perpendicular direction. A typical column cross-section in the upper levels is illustrated in Figure 105.

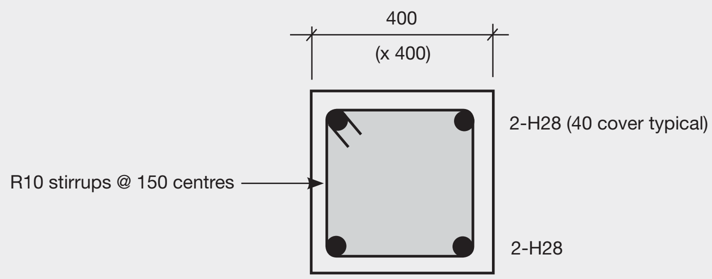


Figure 105: Section of columns from levels 2–6

Failure of a column would lead to a loss of gravity support that would highly stress other elements, potentially leading to the collapse of the building. The columns were inadequately confined with R10 stirrups placed at 150mm centres at the top and bottom of the column with minimal longitudinal bars. An approximate calculation of the nominal deformation capacity of the unconfined upper-level columns was in the order of 0.5 per cent inter-storey drift, which was below the design level demand of 0.8 per cent calculated in the inelastic analysis. The theoretical demand during the February earthquake was in the order of 2.4 per cent inter-storey drift.

In the Royal Commission's analysis, the deformation capacity was calculated conservatively by assuming full fixity at the top and bottom of the columns. In practice the slab would have rotated, reducing the rotations in the columns and accommodating a larger inter-storey drift. The damage observed after the February earthquake indicated that deformations in the columns caused the cover concrete to crack and spall.

It is important that gravity-only structural elements are also designed and detailed to accommodate the seismic-induced deformations of the building, as is required by current standards.

4. Vertical accelerations

In general the vertical accelerations from the analysis do not seem to have a significant effect on the structural behaviour. There is some variation in the axial forces in the columns but the short duration of axial force is not expected to greatly affect the lateral load resistance of the columns. The analysis assumed rigid foundations. The actual characteristics of the soft soil beneath the foundations may have reduced the vertical accelerations that were transmitted into the structure.

6.3.4 151 Worcester Street Building

Current status
Demolished.

Figure 106: View from Worcester Street

6.3.4.1 Introduction

The building at 151 Worcester Street was designed in early 1987 and received a building permit in November 1987 after a late change in the location of the north-eastern ground floor column to accommodate the turning circle of a loading truck. This change in layout introduced some irregularity to the structure of the ground floor. It is assumed that the building was designed to satisfy the Loadings Standard current at the time, NZS 4203:1984[13] and the Concrete Structures Standard, NZS 3101:1982[11].

The building had seven levels of offices, with parking on part of the ground floor. The plan area was about 18.8m x 17.4m.

6.3.4.2 Building structure

The building was built on reinforced concrete foundation beams supported by 500mm in situ concrete compression bulb piles about 10m long.

The plan shape of the building is rectangular. Above the first level the columns were arranged in a grid pattern of three bays of 5920mm in the north–south direction and three bays of 5415mm in the east–west direction (see Figure 108). The elevated floors were constructed from 100mm deep Stahlton prestressed ribs at 900mm centres with a 25mm timber infill and a 75mm concrete topping reinforced with D10 bars at 375mm centres both ways. The precast units spanned in a north–south direction between beams that were at

5920mm centres. The ground floor was a concrete slab that was cast above a grid of foundation beams.

From the drawings it appeared that the lateral forces in the east–west direction were designed to be resisted primarily by the two internal moment resisting frames located on grid lines 2 and 3, with the external frames on grid lines 1 and 4 primarily providing support for gravity loads. In the north–south direction it appeared that the intent of the design was that the lateral forces would be resisted primarily by the external moment resisting frames on lines A and D.

As noted previously, there was some irregularity in the structure of the ground level owing to the need to set back the column in the north–east corner to accommodate the turning circle for truck access. As shown in Figure 107, the two columns in the second and higher levels on grid line 1 were replaced by a single central column in the first level (ground floor). In addition, the north-eastern corner column in grid line D was set back 1000mm from line 1.

All the columns in the perimeter frames on grid lines 1 and 4 had a diameter of 350mm. The central column on grid line 1 in the ground floor was reinforced with nine Grade 380MPa 24mm bars and an R12mm Grade 275MPa spiral at 75mm centres. All other columns on grid lines 1 and 4 in all storeys were reinforced with six Grade 380MPa 20mm bars and an R10mm Grade 275MPa spiral at 300mm centres.

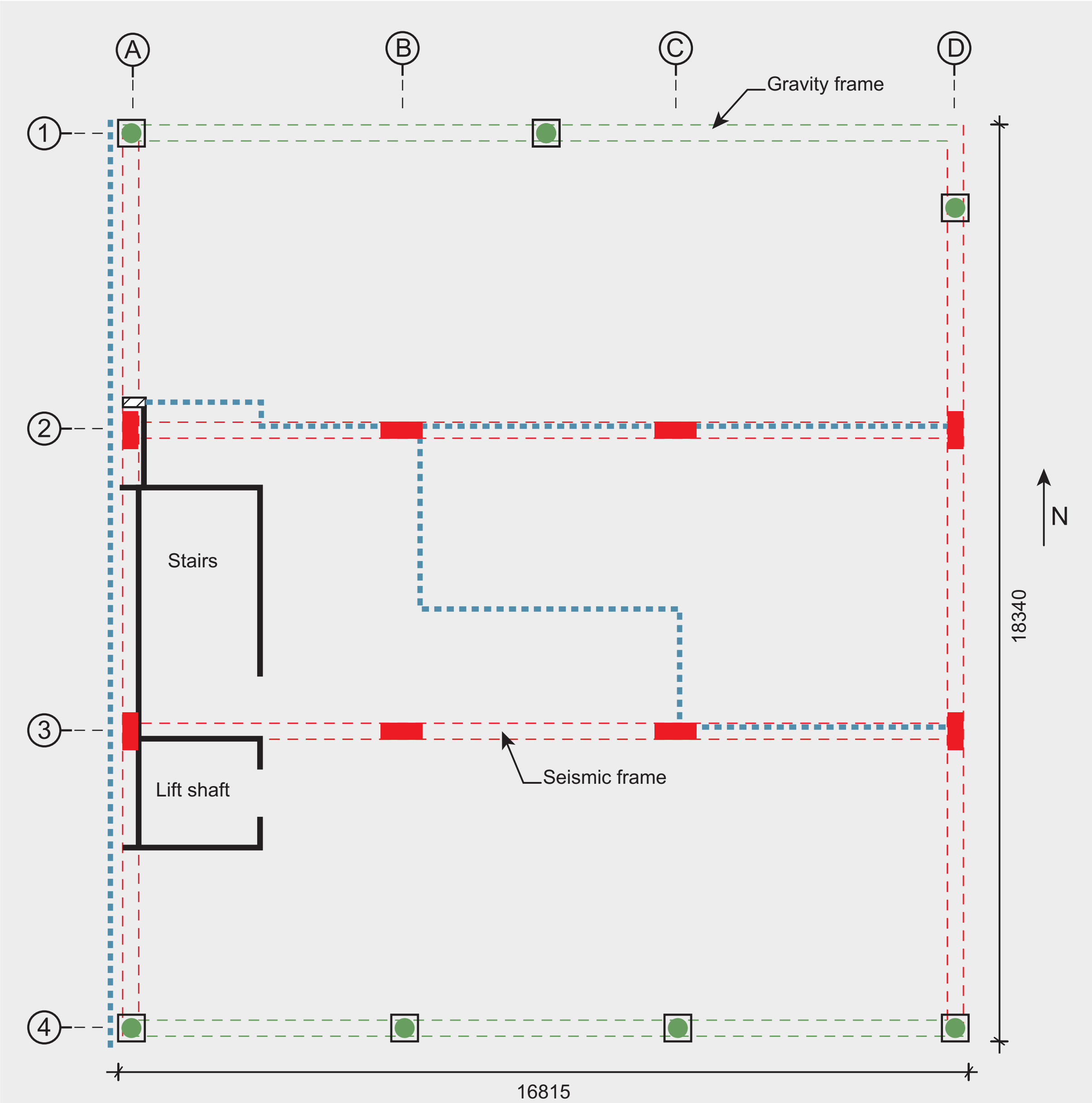


Figure 107: Ground floor: red indicates the seismic frame, green indicates the gravity frames and the western wall cladding

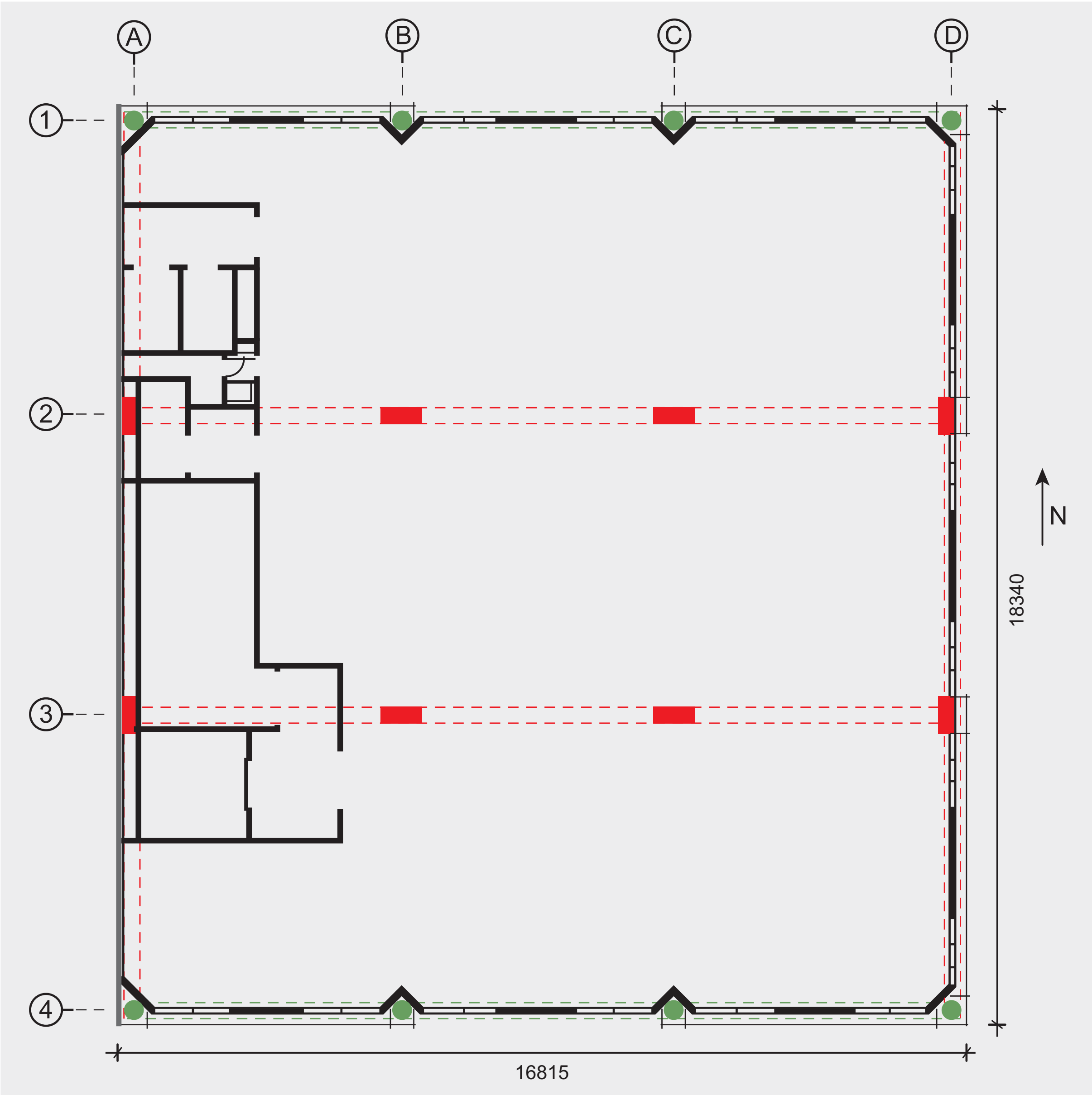


Figure 108: Typical upper floor: red indicates the seismic frame, green indicates the gravity frames and the western wall cladding

The western wall of the building was clad with precast concrete panels, which were detailed with 15mm gaps at all edges. The top floor had a light steel frame and light steel roofing.

The stairs were constructed in reinforced precast concrete with 15mm gaps at both ends of each flight, which were specified as being filled with a flexible sealant. However, photographs show this had been filled with mortar. These gaps were not specified as being seismic gaps, and the purpose may have been principally for construction tolerance.

6.3.4.3 Building performance

The Royal Commission was assisted in the assessment of this building by a report prepared by Spencer Holmes Ltd.

After the 4 September earthquake a Rapid Level 1 Assessment on 5 September found no damage to the building. This was followed by internal and external inspections by Evans Douglas Consulting Engineers Ltd between 6 September and 19 October, when it was noted that there was some non-structural damage to plasterboard walls and some spalling of concrete to the underside of the stairs at the floor/stair junctions. The spalling of the concrete was explained as indicating some hinging of the joints at these points, but it was not considered to have affected the structural integrity of the building.

No information has been provided to the Royal Commission about the building's performance in the Boxing Day aftershock. However, the February earthquake caused major damage to the building. Below is a summary of the notes recorded by Beca in a report, "Earthquake Damage Assessment – 151 Worcester St" (dated 27 May 2011) about an inspection they carried out in May 2011:

1. A column sway mechanism had formed between levels 1 and 2, resulting in a permanent offset of 20-30mm at the front of the building.
2. There was significant damage to perimeter columns and beam-column joints at the lower three to four levels, indicating column hinging had occurred in the moment resisting frames. Significant damage to internal columns and beam-column joints at the lower three levels was noted, including extensive concrete spalling and exposing of reinforcement that indicated hinging in the interior frames.
3. There were cracks and local failures in the infill concrete block walls in the truck dock area. There was extensive damage to glazing and cladding.
4. It was anticipated that there would be extensive cracking in the floors, but these were not able to be inspected. However, there was some indication of damage to the support zones of the precast units.
5. There was considerable damage to the stairs, which were not considered safe to use.
6. There was extensive damage to glazing and cladding and to the precast panels located on the eastern side of the building. This damage tended to be concentrated around beam-column frame areas on the lower floors.
7. Damage to cladding was observed at the adjacent building site where pounding had occurred.

At the time Beca carried out its inspection any damage to the western wall panels would not have been visible owing to the position of an adjacent building (which was subsequently demolished).

An external inspection was carried out for the Royal Commission when the demolition had just started. The following details were noted:

1. There was damage to the precast concrete panels on the west wall. The panels were detailed with mid-panel connections top and bottom and at mid-height between the floors with a 15mm clearance all round. It appeared that mortar and weathering sealants had reduced the clearance, so that the rotation induced by inter-storey drifts in the building resulted in the compression crushing and spalling of the panels at their corners, and excess tension in the lower connection causing severe cracking and spalling at the base of the panels. The top connections of the panels (with a stronger detail) were relatively unaffected.
2. Inelastic deformation and some hinging had occurred at the lower level of the main frame rectangular columns and at the top and bottom of some circular columns.
3. There was some vertical cracking of the column capital at level 1 in the north-western corner, which was assumed to be as a result of axial load/compression dilation of the column capital.
4. There was some evidence of elongation in the beams in the northern wall. However, the floors could not be accessed to examine the significance of this deformation on their performance.

6.3.4.4 Interpretation of damage

A hand analysis of the building carried out by the Royal Commission indicated that the lateral strength would have satisfied the requirements in NZS 4203:1984.[13] Even though some of the detailing of the columns would not satisfy the current standards the building would not, in the Royal Commission's opinion, have been classified as earthquake-prone in February 2011. Our interpretation of the observed damage is that the building responded in a torsional mode in the February earthquake as a result of the high ground accelerations and the eccentricity caused by the combination of:

- the additional stiffness in the west wall due to the lack of effective separation between the precast panels on this wall;
- the additional stiffness from the infill block walls on the ground floor as the walls were not effectively separated from the structure; and
- the lack of adequate seismic gaps between the stairs and the floors.

The locking-up of the panels caused by inadequate separation gaps would have moved the centre of rigidity towards the western wall, which would have significantly increased the torsional response of the building. The two seismic frames, which were designed to resist lateral forces in the east–west direction, would not have been effective in restraining the torsional motion because they were near the centre line of the building.

The block wall in the ground floor may have been partially effective in limiting inter-storey drift as a column sway mechanism appears to have developed in the second storey.

6.3.4.5 Conclusions

A major contribution to the damage sustained by the building arose from its irregularity. While there was some irregularity in the design, this was greatly increased by:

- the failure to effectively isolate the infill block walls from the structure;
- inadequate seismic gaps for the stairs; and
- inadequate seismic gaps for the precast panels and the failure to ensure these gaps did not become filled with weathered sealant and mortar.

The circular columns in the perimeter frames were, with one exception, inadequately confined to resist the imposed displacement. The one exception was the column on the northern side of the building between grid lines B and C. This column was subjected to a relatively high axial load in comparison to the other columns on the perimeter and it was confined by a 12mm bar spiral with a 75mm pitch. All other circular columns contained a 10mm bar spiral with a pitch of 300mm.

6.3.5 78 Worcester Street: Clarendon Tower building

Current status
Demolition had commenced at the time of writing.

Figure 109: View from the north-west

6.3.5.1 Introduction

The Clarendon Tower building (Figure 109) was designed in 1987 and was approved for construction by the CCC in two building permits issued that year. The building had a total of 20 levels, the lowest level being a part basement and the uppermost two levels being smaller service levels for the plant and lift rooms.

The ground floor and the level above this was a podium that covered the site. It was about 40.4m by 50.5m. The tower (Figure 110) above this was offset to the north-west and measured about 24m by 37m.

On part of the northern and western faces of the building the original three-storey historic façade was retained.

The majority of the levels were used as offices, with some retail activities on the ground floor and the level above this. Parking was provided from the basement up to the roof of the podium.

The building was about 43m from the banks of the Avon River at its closest point.

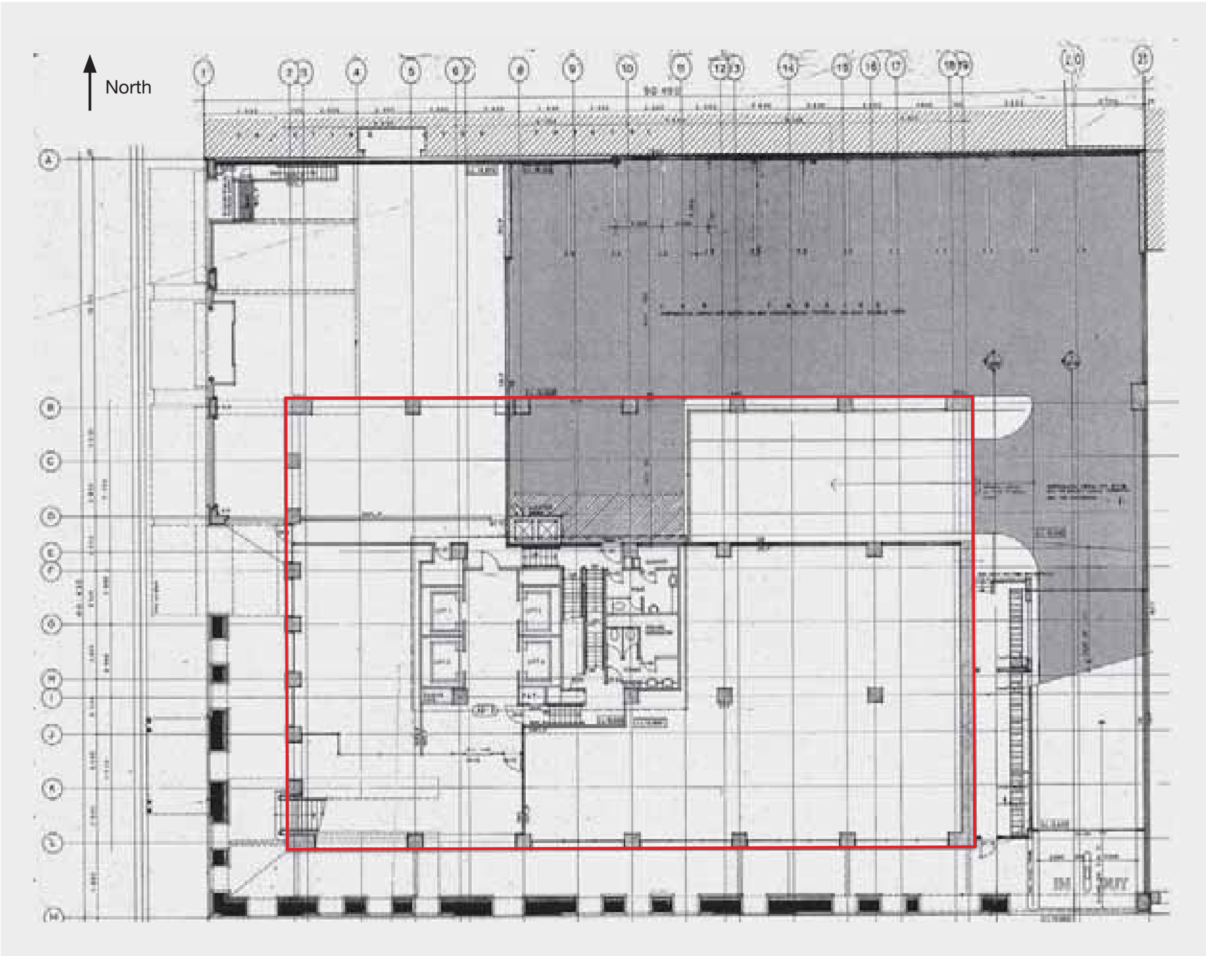


Figure 110: Ground floor. Tower structure above is outlined in red; dimensions of the Tower are shown in Figure 111 on page 170

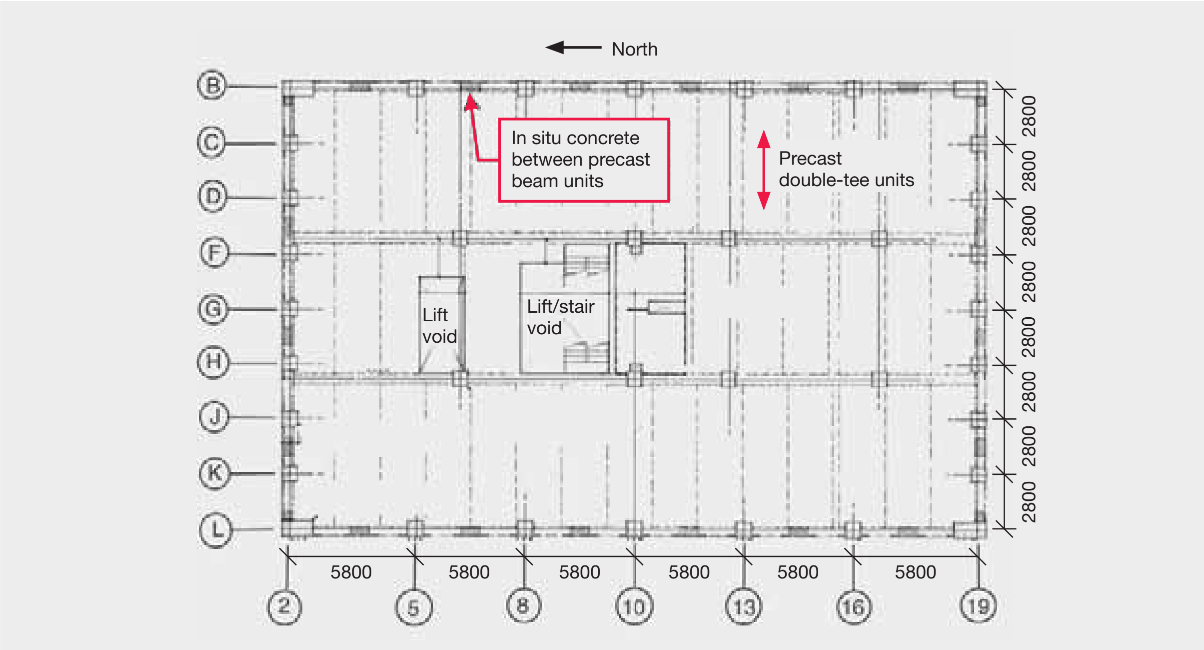


Figure 111: Typical tower plan

6.3.5.2 Building structure

The building was founded on a reinforced concrete spread footing. There was no evidence of liquefaction or ground spreading in the immediate vicinity.

The floors were flange-hung 250mm deep precast double-tee units with 65mm minimum of seating and overlaid with a 60mm cast-in-situ topping reinforced with cold-drawn wire mesh. The topping was required to act as a diaphragm. The tees spanned about 7.7m from east to west, being supported on grid lines B, E, I and L with precast beams, which in turn were supported by cast-in-situ columns.

Lateral force resistance was provided predominately by perimeter moment resisting frames. In the perimeter frames on the eastern and western faces of the tower there were six bays in the frames: four that spanned 5800mm and two that spanned 6500mm. In the corresponding moment resisting frames on the northern and southern faces there were eight bays in the frames, each having a span of 2900mm. A consequence of this was that the building was about twice as stiff for lateral deflection in the east–west direction as in the north–south direction.

The columns in the perimeter frame were cast-in-situ and were 800mm by 800mm, except in the corners, where they were 1500mm by 800mm. The beams were precast so that they fitted over the columns with the column reinforcement passing through ducts in the precast beams, the ducts being grouted after the beams were fitted in place. In situ concrete was cast to join the beams together. In the in situ zone between the beams, the reinforcement was bent into the diagonal, as shown in Figure 112(a). The ends of the reinforcing bars, labelled B in the figure, were welded to steel plates. The plates were bolted together to establish the continuity of the reinforcement. Closely spaced ties were provided over the length of the diagonal bars to prevent buckling of this reinforcement (for clarity these ties are not shown in Figure 112). In the in situ concrete there were some relatively small bars in the top and bottom of the beam, with some stirrups to form a light cage of reinforcement (not shown in Figure 112). Additional reinforcement was placed along the length of the precast beam to increase the flexural strength and prevent yielding caused by flexure.

In the north–south direction continuity was established by hooked laps for the bottom bars and straight laps for the top bars, with stirrups passing around the top and bottom reinforcement.

6.3.5.3 Building performance in the earthquakes

The Royal Commission's understanding of the performance of the building has been assisted by a report dated February 2012 by Rutherford and Chekene, consulting engineers, of San Francisco, prepared at the request of the Royal Commission. Based on that report, the Royal Commission finds that in the 4 September earthquake there was extensive cracking in the floor and limited cracking in some of the beams. The floor cracks in some cases were several millimetres wide and some of the mesh in the topping concrete had fractured. These cracks were injected with epoxy and where mesh had fractured (or it was suspected to have fractured), additional reinforcement was set into the topping concrete. The repair work on the building was essentially completed before the 22 February 2011 earthquake.

In the February earthquake, very extensive cracking occurred in the floor diaphragm with wide cracks developing between the northern and southern frame perimeter beams and the double-tee precast floor units. The crack widths were of the order of 20–30mm and 10–20mm wide at the northern and southern ends of the building respectively, with the mesh fracturing at these cracks.

Wide cracks of the order of 10mm also developed between the double-tee units and the intermediate internal beams.

There were pronounced flexural and diagonal cracks in the beams, some up to 15mm wide in levels 7 and 8. This cracking was located close to the junctions between the precast beams and in situ concrete joining the beams together. In addition, some cracking occurred in the beams at the column faces. Cracking in the floor caused the mesh in the topping concrete to fail in a number of regions and in some cases the elongation greatly reduced the support length for the double-tee floor units, leading to concern that support for some of the double-tee units could be lost.

Frame elongation was found to be greater in the north perimeter frame and was 20–50mm in some of the floors, resulting in failure of the mesh that connected the topping concrete in the floors to the perimeter beams. With this connection broken, the tie between the floor and some of the columns was lost. Compression forces in the beams caused by elongation resulted in the columns bowing outwards from the floors. The frame on the northern side of the building suffered more damage than the frame on the southern side.

In the February earthquake the stairs collapsed over several levels. This was explained by an inadequate separation that resulted in the stairs being subjected to compression, and led to compression/flexural hinging at the centre of the stairs, in a similar manner to that observed in the Forsyth Barr building.

6.3.5.4 Analysis of the building

Rutherford and Chekene assessed the fundamental periods of vibration to be in the order of 2.5 and 1.9 seconds in the longitudinal (north–south) and transverse (east–west) directions respectively. In addition, they noted that their respective second mode periods were about 0.8 and 0.7 seconds, and that there was a torsional mode with a period of about 1.25 seconds. This torsional mode was a result of the podium being offset from the centre of the tower and the original heavy façade of the previous building on the site being retained with the new structure. The section properties they used in their analysis were comparable to those recommended in the commentary to the Concrete Structures Standard, NZS 3101:2006[12].

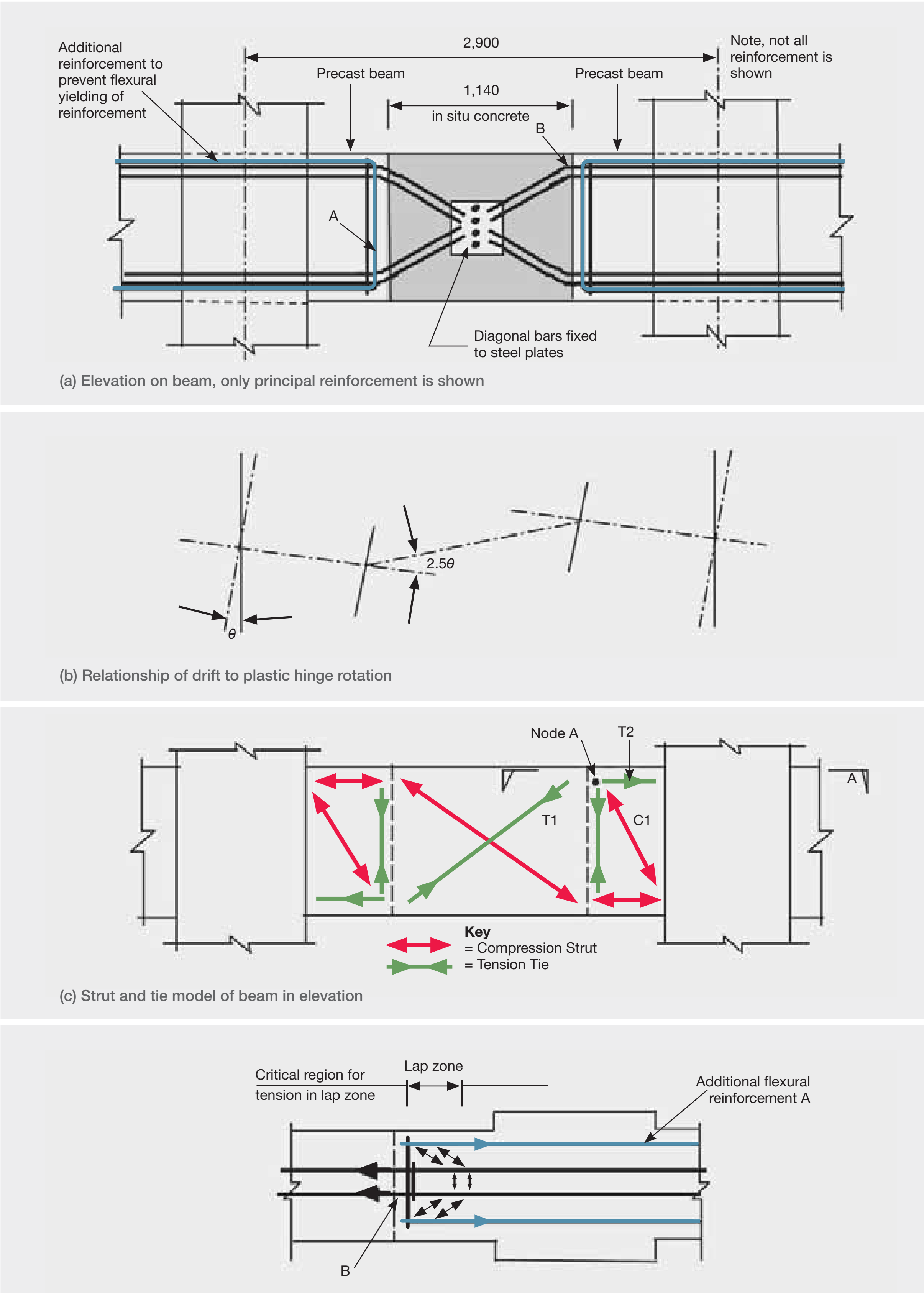
With reference to Figures 2–5 shown on pages 9 and 10 in this Volume, it can be seen that in the September earthquake a building with a fundamental period close to two seconds is likely to have been subjected to earthquake actions in the east–west direction that were comparable to design values given in NZS 1170.5[10]. In February, the corresponding values were of the order of twice the design level. In the north–south direction, the earthquake actions in the September earthquake were close to twice the design level actions while, in February and June respectively, the corresponding values were about 40 per cent and 20 per cent in excess of design values.

Rutherford and Chekene analysed the building using the Model Response Spectrum method with response spectra for the four earthquake records recorded in the Christchurch CBD and a design response spectrum, including a S_p factor of 0.7, as defined in NZS 1170.5[10] for the type D soil conditions of Christchurch. The assessment of inter-storey drifts in levels 5 to 10 ranged from 1.3–2.8 per cent for deflections in the east–west direction. The drifts in the north–south direction were appreciably less. Rutherford and Chekene also indicated that owing to the torsional mode the predicted inter-storey drifts were a little greater in the northern perimeter frame than in the southern perimeter frame.

6.3.5.5 Discussion: beams and floor slab

The beams were designed to deform in a shear mode similar to the approach used in coupling beams between structural walls. With these kinds of beams the diagonal reinforcement is intended to yield in tension and compression. The approach has been well tested in laboratories and used extensively around the world[17]. The intention with the Clarendon Tower beams was that any inelastic deformation would be sustained by yielding of the diagonal bars. This arrangement, as noted above, has been observed to give good ductile performance in coupled walls.

To achieve the objective of limiting any inelastic deformation to the diagonal reinforcement when this detail is used in the mid-region of a beam, additional flexural strength must be added to the beam outside the zone containing the diagonal bars. In an attempt to achieve this objective, additional flexural reinforcement was placed in the precast beam unit. Looking at the beam in elevation, we consider this additional reinforcement formed a continuous loop around the perimeter of the beam with the bars bent at 90 degrees in the corners (see bars marked A in Figure 112). As shown in Figure 112, this additional reinforcement was placed on either side of the bars that were bent diagonally down in the in situ concrete. Assessment of the strengths in accordance with standard structural theory would indicate that the flexural strength was adequate to achieve this objective. However, tests of the strengths, which were carried out a few years after the building was constructed, showed that the detail did not work as intended[18].



(a) Elevation on beam, only principal reinforcement is shown

(b) Relationship of drift to plastic hinge rotation

(c) Strut and tie model of beam in elevation

(d) Section A

Figure 112: Strut and tie model of a coupling beam in the Clarendon Tower

A strut and tie analysis of the beam indicated that the critical section was located at the interface between the in situ concrete and the precast beam (see Figure 112(a)) and not in the diagonal reinforcement in the in situ concrete, as had been intended. The strut and tie model is shown in Figure 112(c). The magnitude of forces forming the strut and tie truss can be calculated from equilibrium. At node A in Figure 112(c), a tension force in the reinforcement, shown as T2, is equal to the horizontal component of the diagonal strut force, C1, plus the horizontal component of the diagonal bars' tie force, T1. The diagonal compression force, C1, provides the shear resistance and acts against the bend in the diagonal bars at node A. Consequently, tension force, T2, is resisted in the horizontal extension of the diagonal bars immediately on the column side of the in situ concrete. The magnitude of tension force, T2, in the bars reduces over a short distance as bond action transfers some of this force to the additional bars, marked A in Figure 112(a) and (d). As a result, only a short length of this reinforcement sustains the inelastic deformation. Strain levels in the reinforcement were, therefore, high. The situation was not improved by the location of the bend in the reinforcement that would have strain-hardened the bars and caused strain-ageing to occur.

With the details as described above, it would be anticipated that yielding in the beam would be confined to the central bars in the precast beams where they were bent into the in situ concrete zone. This was shown to be the case in laboratory tests[18].

However, only the beam was modelled in the tests and not the floor slab close to the beam. When the floor slab, which contained precast prestressed units, was included in an assessment of the strengths it was found that the slab could add appreciable flexural strength to some regions of the beam but not to others. In some cases it was found that the inclusion of the likely contribution of the slab could explain why yielding had occurred at some column faces in addition to the critical sections at the end of the diagonal bars.

In this structure, elongation due to the formation of plastic hinges in the beams was a major cause of damage and reduced performance of the building in the earthquakes. Tests have shown that the magnitude of elongation in beams is primarily a function of the rotation imposed on the plastic hinges and the number and magnitude of the load cycles[18,19,20,21].

Figure 112 shows the deflected shape of the beam associated with inelastic deformation of the bars at the ends of the precast beam segments. It can be seen that the rotation of the plastic hinges is about 2.5 times the drift angle, which is much greater than would have occurred if the plastic hinges had formed at the column faces. It is clear that this detail greatly increased the elongation induced in the east–west direction of the building.

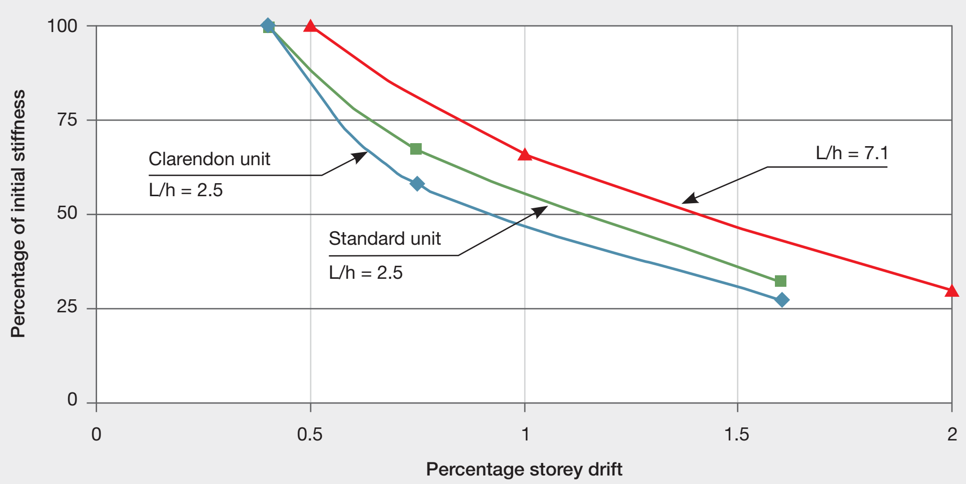


Figure 113: Stiffness shown as a percentage of initial pre-yield stiffness when reloading after the first half-cycle has been applied

Figure 113 shows some results of stiffness degradation of frames that have been subjected to inelastic deformation with plastic hinges forming in the beams. The initial stiffness was based on the load deflection characteristics measured in tests in which the test units were loaded to close to 75 per cent of their calculated flexural strength. The stiffness after inelastic loading to the given drift limit has been calculated from the observed load deflection characteristics when the load direction was reversed after the previous peak displacement. The stiffness was based on displacements in the range of a zero load to a value corresponding to 75 per cent of the theoretical strength. Three cases are shown in the figure. The first two come from reference[18], the first being for the test unit representing the beam used in the Clarendon Tower. The second unit was from the same reference where the horizontal reinforcement between the two precast units was joined by using hooked bar laps. In both cases the ratio of clear span to depth of the beam was close to 2:5. The third test comes from a two-storey frame test[21]. In this case the clear span to depth ratio was close to 7:1. In all three cases no floor slab was included in the test units. It can be seen from Figure 113 that a very significant stiffness decrease occurs when only moderate levels of inelastic deformation have been imposed on the frames. For the beams with a low span to depth ratio the stiffness was reduced to close to 50 per cent of its initial value, while for the unit with a span to depth ratio close to 7:1, the stiffness was reduced to about 67 per cent of its initial value. If the floor slabs had been represented in the test units it is likely the stiffness degradation would have been smaller.

It was noted in the Rutherford and Chekene report that the northern end of the building sustained greater damage than the southern end. From the analysis, it was found that some torsion was induced in the building owing to the heavy façade and the podium in the first three levels; this was assessed as inducing greater lateral displacements of the northern end of the building than the southern end. Once inelastic deformation had been sustained in the northern end frame, the reduction in stiffness would have been significant. Even if this decrease had been half that indicated in Figure 113 it would have significantly increased the eccentricity of the seismic forces and further amplified torsional response; then the damage sustained at the northern end of the structure would have been even more pronounced.

6.3.5.6 Conclusions

We conclude that:

1. Faulty detailing of reinforcing in the perimeter frames on the northern and southern ends of the building led to inelastic deformation in regions where it was not anticipated and this reduced the ductility of the structure. A strut and tie analysis of the detail would have revealed the problem but the building was designed before this method of analysis was widely accepted by the engineering profession.
2. The detail used to connect the floor slabs (which acted as diaphragms) to the perimeter frames was not adequate for the forces that were induced. This was partly due to the high forces associated with elongation and partly to the use of non-ductile mesh. It should be noted that mesh was widely used for reinforcement in the in situ concrete placed above precast floor units until recently (2005).
3. Although the building had only minor irregularities in the structural system, the sustained damage indicates that it was subjected to a high torsional response. We conclude that this was largely due to the loss of stiffness in the northern frame, which greatly increased the eccentricity of the seismic forces to the effective centre of stiffness of the building. Examination of a number of results of tests on moment resisting reinforced concrete frames shows that for the type of beam used in the building, relatively small inelastic displacements can significantly reduce the lateral stiffness. This would have accentuated the torsional response of the building.

6.4 1992 to 2008: Buildings designed to Loadings Standard NZS 4203:1992[22]

6.4.1 100 Armagh Street: Victoria Square apartment building

Current status
Future yet to be determined, possibly to be relevelled.

Figure 114: View from north (source: Compusoft)

6.4.1.1 Introduction

Victoria Square is a 14-storey reinforced concrete apartment building designed in 2004 using NZS 3101: 1995[23] and NZS 4203:1992[22]. A building consent was also approved (in three stages) in 2004, with a final code compliance certificate issued in 2007.

The building consists of a podium from the ground floor to the fourth floor of about 650m^2 per floor. The floor area of the fifth to the seventh floors is about 475m^2 per floor, and above that the floor areas are about 300m^2 per floor (see Figure 114).

6.4.1.2 Ground and foundations

The building foundations consist of a combination of piles and shallow concrete pads.

The piles are 1200mm diameter reinforced in situ concrete tension piles founded within a sandy gravel layer below the building. They vary from 6–18m in depth. They are on the east, west and south on the perimeter of the building, as shown in Figure 115. The piles are linked by foundation beams that are intended to resist compression loads.

The concrete pads are within the building and along its northern perimeter under isolated columns, and also under the liftshaft and other internal walls.

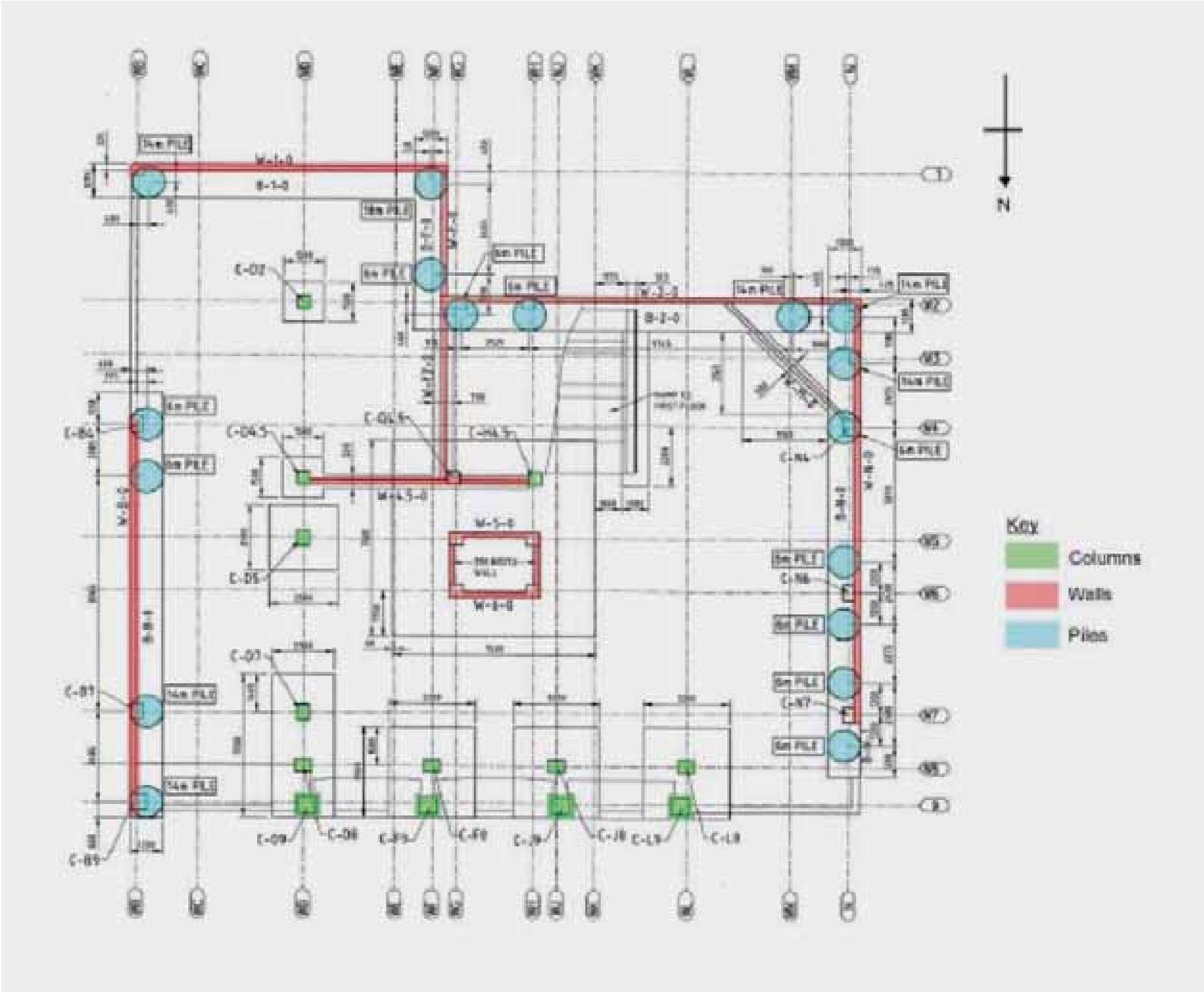


Figure 115: Pile layout (source: Compusoft)

6.4.1.3 Structure

From the information available the Royal Commission concluded that the failure of the foundations was the most significant damage to the building. For this reason the above-ground structure is not discussed further in this Report.

6.4.1.4 Performance of the building

The Royal Commission was assisted in its assessment of the building by a report prepared by Compusoft.

Assessment of the building after the September earthquake indicated that there was no significant structural damage to the Victoria Square building as a result of that earthquake. Only minor non-structural cracking was seen.

After the February earthquake, damage reports included minor cracking of concrete shear walls, spalling of concrete at stair landings and damage to non-structural cladding panels.

Although the structural damage resulting from the February earthquake was minor, the earthquake caused a significant permanent overall displacement of the Victoria Square building. The magnitude of this displacement was reported as about 450mm at the top of the building, or 0.8 per cent drift. This displacement is clearly visible in photographs. Survey results indicated that the total displacement/drift can be attributed to settlement of the foundations. The structure was found to have settled by 220mm at the north-western corner and 160mm at the north-eastern corner (both levels relative to the south-eastern corner).

6.4.1.5 Discussion

The reasons for the foundation settlement appear to be readily explainable. Two different factors contributed to the differential settlement:

- the layout of the different types of foundation element under the structure; and
- the distribution of soil liquefaction in the vicinity of the structure during the earthquake.

From preliminary maps presented in the Royal Commission Interim Report, Compusoft interpreted that the boundary of severe liquefaction extended from the north to the front face of the building. Based on available geotechnical information they considered it was most likely that liquefaction at the northern side of the Victoria Square building would have occurred at depths of 8–14m, with some possibility of liquefaction from 16–20m.

The failure of the ground, together with shallow foundations not designed for liquefaction at the northern side of the building, would have resulted in the significant rotation towards the north. The structure also experienced a less significant rotation to the west. The principal cause of this rotation is likely to have been structure-soil-structure interactions, specifically, the existence of a second large structure immediately to the west of the Victoria Square building (Craigs Investment House, 90 Armagh Street, which is also assessed in this Report) (Figure 88, page 142). This would have resulted in both structures influencing the stress state of the soil in the vicinity of the boundary line. A secondary contribution may have been the provision of piles of only 6m in length at the north-western corner of the structure. Such short piles are unlikely to have penetrated below the liquefiable material under this region of the structure (Figure 116).

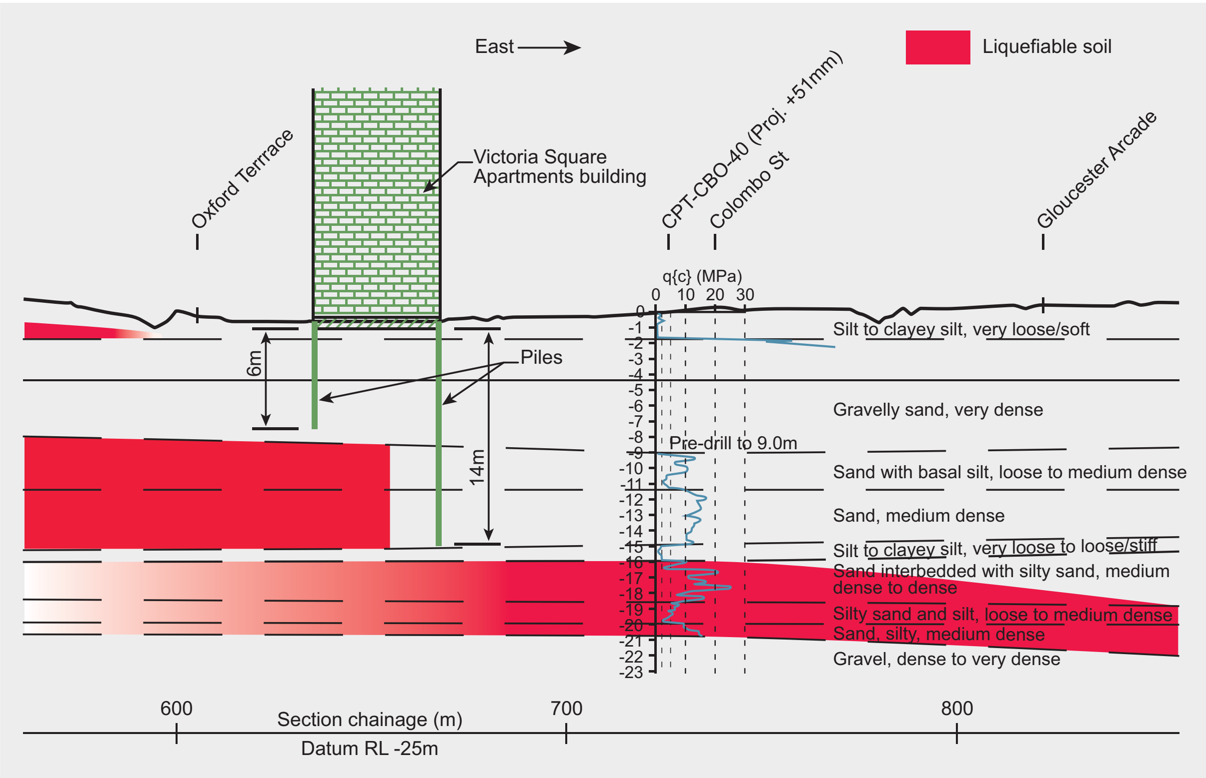


Figure 116: Cross-section from west to east showing the zones of liquefiable soil (source: Compusoft)

6.4.1.6 Conclusions

The February earthquake resulted in a significant overall rotation of the structure. This rotation was caused by severe liquefaction immediately to the north of the structure combined with the shallow foundation elements on that side of the structure. There are two key points from the performance of this building:

1. The geotechnical conditions in the vicinity of a structure, as well as directly under the structure, should be considered as a part of design.
2. Particular consideration should be given to the underlying geotechnical conditions when specifying hybrid foundations for a structure.

6.5 2004 to 2011: Buildings designed to Earthquake Actions Standard NZS 1170.5:2004[10]

6.5.1 62 Gloucester Street: Gallery Apartments building

Current status
Demolished.

Figure 117: View of the northern tower from Gloucester Street (source: Becker Fraser Photos)

6.5.1.1 Introduction

The Gallery Apartments building was located at 62 Gloucester Street, adjacent to the Christchurch Art Gallery. It was designed during 2005 and 2006. Three building consents were issued by the CCC in 2006 and code compliance certificates were issued in 2007. It was a 14-storey building constructed in two seismically separated towers. The southern tower had seven levels of parking and the services core for both towers.

The northern tower comprised a two-level art gallery, with about half of this being of double height. There were 12 levels of residential apartments above this with a single apartment on each floor. The plan dimensions of this tower were about 9.3m by 11m.

For the purposes of this Report only the northern tower has been assessed.

6.5.1.2 Foundations

A geotechnical investigation undertaken for the design of the building indicated a potential liquefaction hazard at depth but it was considered this would not have a significant impact on the building.

The foundations of this tower comprised 900mm diameter reinforced concrete piles that extended to depths of 4m on the eastern side and up to 7m on the western side. These supported 1.5m deep foundation beams at the perimeter and on grid lines 1 to 5 (shown in Figure 118).

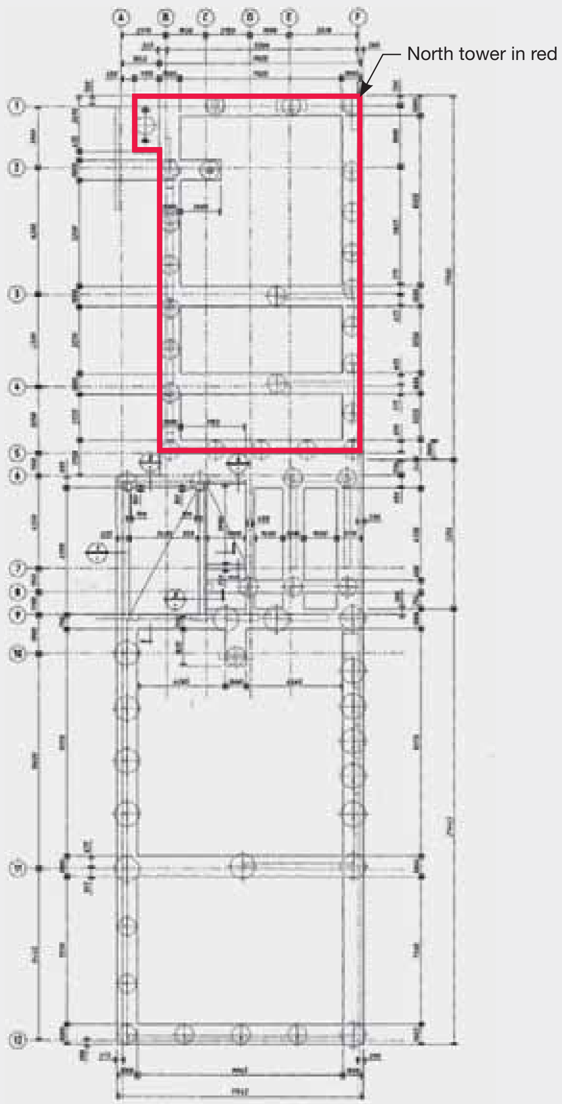


Figure 118: Foundation plan; dimensions shown in Figure 119 on page 182

6.5.1.3 Floors and gravity load system

The ground floor was a 100mm thick on grade reinforced concrete slab. The floors above the ground floor in this tower comprised Interspan precast prestressed concrete ribs with timber infill and a 135mm thick in situ concrete topping reinforced with cold drawn mesh. The ribs spanned between the eastern and western walls. They were supported on a cast-in-situ edge beam (Figure 119).

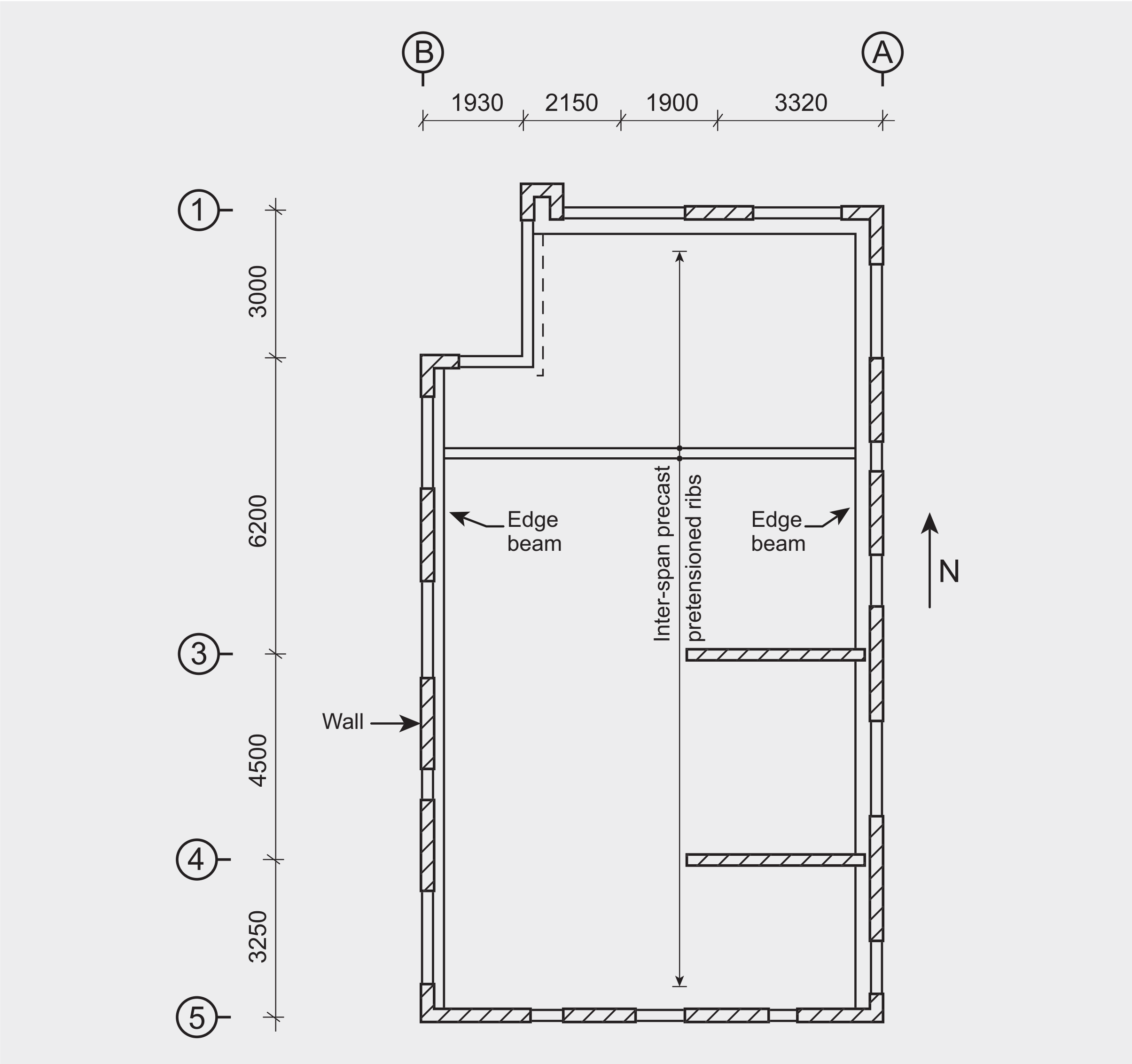


Figure 119: Typical floor plan of northern tower

6.5.1.4 Seismic system

The seismic system consisted of structural walls built up from precast panels 175–325mm thick. Each panel was typically two storeys high and continuity of reinforcement was provided by grouted couplers at the horizontal junctions between the panels.

6.5.1.5 Structural damage

The damage to the northern tower as a result of the February earthquake is recorded in a report dated 20 May 2011 and prepared by the Holmes Consulting Group Ltd (HGC):

> Our inspection did not include invasive opening up of linings or floors and ceilings therefore only a selection of exposed structure was reviewed. The most obvious structural damage observed is as follows:
>
> - The column in the north-west corner of the northern tower appears to have settled. This can be seen from the street and is observable in the suspended floors when walking around the apartments.
> - Failure of the precast panel connections on the western elevation of the northern tower. USAR engineers have advised that reinforcing bars have fractured over significant lengths of these walls. External steel straps have been provided by contractors as a temporary securing measure.
> - Failure of the internal shear wall at ground floor level in the northern tower. Significant shear cracking is apparent. USAR engineers have advised the reinforcing had buckled in the end of the wall. Prior to our visit contractors have provided a steel jacket to the end of the wall to provide some confinement. Steel straps are welded to the jacket and fixed through the wall to provide some shear capacity and hold the jacket in place. There is a vertical crack in the end of the wall above the steel jacket which may indicate the reinforcing steel is beginning to buckle in this area also.
> - Uneven floor surface at the two internal shear wall locations in the northern tower. Floors are raised in these areas suggesting either settlement of the side walls or elongation of the shear walls due to yielding of reinforcing steel.
> - Damage to the floor outside the passenger lifts and at the seismic joint between the two towers is evident. The exact extent of this can't be determined without stripping back the finishes.
> - Shear cracking of front wall (northern tower) at level 6 has occurred. "Gib" plasterboard has popped at this level and allowed inspection, other levels were not inspected.
> - Flaking of paint to SHS balcony column, possibly due to flexure (level 3).
> - There appears to be a lack of grout in the Reid couplers at front of car lift which should be investigated further to confirm. If so, the tension capacity of these reinforcing bars cannot be relied upon.
> - Shear cracking has occurred in the in situ concrete floor stitch beams at panel openings. Some concrete spalling of the Interspan rib adjacent to the support connection was also apparent in one location.
> - Movement is visible between the precast stair units at the in situ concrete stitches and at the precast unit to landing connection.
>
> In conclusion, the building has sustained severe damage. Although it is not considered an immediate overall collapse hazard, the building is not safe to spend any significant amount of time in due to the extent of structural damage and the high possibility for unknown damage still not identified.

It can been seen from the floor plan (Figure 119), that the lateral force resisting system in the east–west direction is highly eccentric to the centre of mass, while there is little eccentricity for seismic forces acting in the north–south direction.

6.5.1.6 Structural assessment

The Royal Commission was assisted in the assessment of the performance of this building by a report prepared by Spencer Holmes Ltd.

According to a design features report that was submitted to the CCC with a building consent application, the building was designed using the Earthquake Actions Standard, NZS 1170.5:2004[10] and the Concrete Structures Standard, NZS 3101:1995[23]. A modal response spectrum analysis was used to determine the seismic design actions. A geotechnical report recommended that the foundation soils be assumed to be type D. The analysis was made based on a seismic hazard factor of 0.22 and a structural ductility factor of 3. From the modal analysis, the designers determined the fundamental periods of vibration of the northern tower in the east–west direction to be close to 4 and 3.5 seconds respectively (depending on whether the accidental eccentricity was placed to the north or south of the calculated centre of mass). The corresponding periods for the north–south direction were close to 3.7 seconds and in this case the offset for accidental torsion to the east or west of the centre of mass had little effect.

In the building assessment carried out by Spencer Holmes, the fundamental periods of vibration in both the east–west and north–south directions were calculated to be close to 3 seconds. This assessment was based on an equivalent static analysis and the assumption that the effective section properties of the walls were equal to 0.34 times the gross section properties. Given the limited spread of cracking observed in the walls, the Royal Commission considers that the effective fundamental period was in the range of 2.0–2.5 seconds in both the east–west and north–south directions. Because of the lower periods of vibration the seismic design forces would have been higher but the displacements would have been smaller than those used in the design.

The relatively long fundamental periods of vibration quoted in the design features report indicated that a relatively low stiffness must have been assumed for the design of the walls. From this it was evident that the predicted inter-storey drifts would have been high.

The commentary to NZS 3101:1995[23] recommends that the section properties be taken as 0.25 times the gross section properties to allow for flexural cracking. We consider such a low value as 0.25 is not appropriate for structural walls where only limited flexural cracking is possible owing to the low reinforcement content. This problem should be addressed in the commentary to the current edition of NZS 3101:1995[23].

The centre of lateral resistance in terms of both strength and stiffness was highly eccentric to the centre of mass for seismic actions in the east–west direction (see Figure 119). As a result, earthquake shaking in the east–west direction induced high torsional actions into the building. Spencer Holmes found that this direction of seismic forces induced higher critical actions in the eastern and western perimeter walls than when the corresponding seismic forces were applied in the north–south direction. The eccentricity for the north–south seismic actions was small. In the September event, for the period range of 2.0-3.0 seconds the seismic actions in the north–south direction were dominant and the corresponding values in the east–west direction appreciably smaller (see Figures 2 and 3 in section 1 of this Volume of the Report). In the February earthquake, the east–west motion dominated more than the north–south motion. This could explain why the building was not significantly damaged in the September earthquake but suffered major damage in February.

6.5.1.7 Discussion: Structural detailing

6.5.1.7.1 General

The choice of a structural ductility factor of 3.0 by the designers, combined with the use of the Concrete Structures Standard, NZS 3101:1995[23], required the use of limited ductile detailing in the structure. However, inspection of the drawings indicated that nominally ductile detailing was used, but referred to as “elastically responding” in NZS 3101:1995[23]. The structural walls did not contain the confinement reinforcement required in their compression zones, as specified in the Concrete Structures Standard, NZS 3101:1995[23]. Hence, there was a mismatch between the assumed design ductility and the detailing. With nominally ductile detailing the structural ductility factor was 1.25; the corresponding base shear should have been at least twice as high as that corresponding to a structural ductility factor of 3.0.

6.5.1.7.2 Structural walls

Appreciable spalling of concrete occurred in the wall on grid line 3 at ground level, to the extent that temporary securing was judged necessary by Urban Search & Rescue (USAR) engineers immediately after the February earthquake. The confinement required for limited ductile structures would have limited this structural damage and the extent of the spalling that occurred at this location, as well as securing structural safety.

Spalling of concrete in the walls was observed to expose some of the couplers used to join the longitudinal reinforcement at the junctions between the precast panels (see Figure 120). The size of these couplers located close to the face of the wall increased the likelihood of spalling. The stability of the couplers would have been assured if they had been restrained by reinforcement anchoring them into the body of the wall.

During an inspection of the building after the February earthquake it was noticed that a number of the couplers joining the wall panels had not been grouted.

Figure 120: Spalling of concrete exposing the couplers at the bottom of structural walls (source: Spencer Holmes Ltd)

Examination of one of the walls showed that the expected crack pattern of primary and secondary cracks that had been observed in structural tests did not develop in this building. Instead, in many cases only one relatively fine crack was found in the anticipated plastic hinge zone of each wall. When the concrete had broken out at one of these cracks it was found that the bars had failed in tension. We assumed that the gravity load acting on the walls would have been sufficient to close the crack after the earthquake, causing any reinforcement in the mid-region of the wall that had not failed in tension to yield back in compression. Hence, only narrow cracks were evident after the earthquake.

There appeared to be two reasons for this unanticipated behaviour. First, there was insufficient reinforcement to induce the formation of secondary cracks in the concrete and, consequently, yielding of the reinforcement was confined to the vicinity of the primary crack. Secondly, the development and yield penetration into the concrete on each side of the crack was much less than had been anticipated.

To assist in establishing the reason for the formation of single cracks in the walls in the potential plastic hinge zones, the Royal Commission requested Holmes Solutions Ltd to investigate the properties of the concrete. They cut four cores from structural walls near the front of the building. Two were tested in compression and two were subjected to split cylinder tests to determine the tensile strength of the concrete. In addition, Schmidt Hammer tests were carried out on the structural walls on the northern and southern sides of the building. The core tests indicated cylinder compression strengths of 56 and 46.5MPa with associated tensile strengths of 2.4 and 3.4MPa. It should be noted that the coefficient of variation for tensile strengths was very much higher than the corresponding value for the compression strengths. Hence, little reliance should be placed on the tensile strengths. The Schmidt Hammer tests indicated the concrete compression strengths ranged from 54 to 70MPa.

Inspection of the drawings indicated that Grade 500 reinforcement was used on the walls and the proportion of longitudinal reinforcement in the potential plastic hinge zones was generally 0.0017–0.0022. The minimum reinforcement permitted in NZS 3101:1995[23] was 0.0014. The corresponding minimum given in NZS 3101:2006[12] is dependent on the grade of concrete used. For 30MPa concrete, the minimum proportion of longitudinal reinforcement is 0.0028.

To form a secondary crack, the strain-hardened strength of the reinforcement crossing a primary crack must be sufficient to stress the concrete surrounding the bars to a level that exceeds its tensile strength. Several walls in the first two storeys were 325 mm thick and were reinforced in the longitudinal direction (vertical) with 12mm Grade 500 reinforcement placed on each side of the wall at a spacing of 420mm. The critical location for the formation of a secondary crack is at a level where there is horizontal reinforcement immediately above the existing crack. Hence, for the purposes of assessing secondary crack formation the effective width of the wall was the width minus the area taken up by the horizontal bars, which in this case was 325 – (2x12) = 301mm. As the longitudinal bars are at a spacing of 420mm, the effective area of concrete related to two bars (one on each side of the wall) was 301 x 420 = 126 420mm^2.

The tension force carried by the two bars was equal to the cross-sectional area of the bars multiplied by the strain-hardened stress that may be resisted by the reinforcement. For Grade 500 bars, the strain-hardened stress should be equal to, or greater than, 1.15 times the design yield stress, giving a value of 575MPa and a total tension force of 133kN in the two bars. The corresponding stress in the concrete was 1.02MPa. The minimum measured tensile strength of the concrete was 2.4MPa. For comparison, the corresponding average tensile strength predicted from the CEB-FIP Model Code[24] for 30MPa concrete was 2.9MPa, and for 46MPa concrete it was 3.9MPa. Clearly, secondary cracks could not be expected to form.

In the current Concrete Structures Standard NZS 3101:2006[12] the minimum reinforcement proportion has been increased to

$p = f'_c / 4f_y$ from $0.7/f_y \geq 0.0014$ in NZS 3101:1995[23].

For 30MPa concrete, the minimum proportion of reinforcement was close to twice the previous minimum value. However, even the more recent provision for minimum reinforcement only corresponds to a tensile strength in the concrete of close to 1.4MPa.

6.5.1.7.3 Incompatible deformation of walls

The walls on grid lines 3 and 4 are positioned close to separate walls on grid line F. These four walls formed two pairs, each in a T-shaped configuration (see Figure 121). In each pair the two walls were connected to the floors but not directly to one another. When the walls were subjected to seismic actions in the east–west direction incompatible displacements were induced between them, as shown in Figure 121. This deformation broke up the floor in the area close to the junction of the walls.

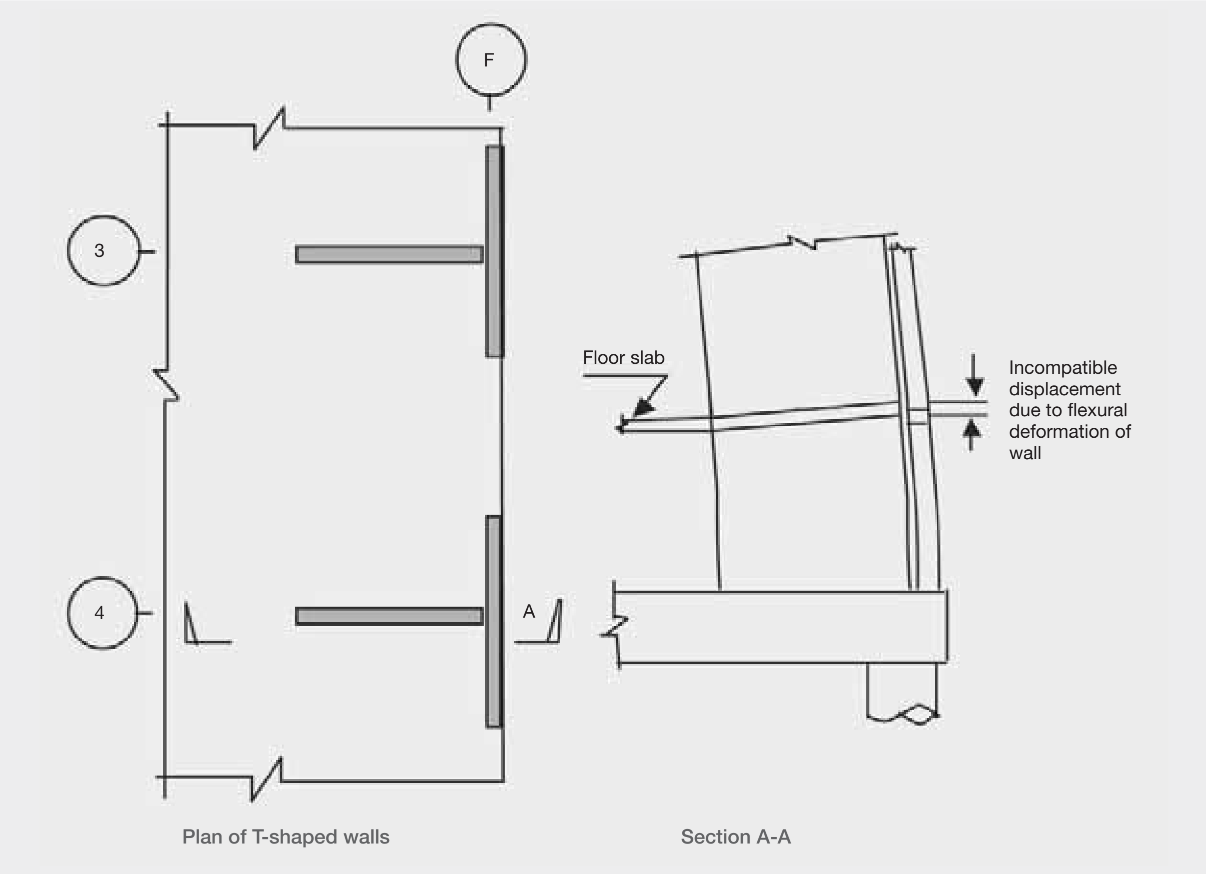


Figure 121: Incompatible displacement between structural walls

6.5.1.7.4 Support of precast ribs on walls

The Interspan ribs were supported on in situ concrete edge beams cast against concrete panels that formed the structural wall. A typical detail is shown in Figure 122. With this arrangement, the web of the pretensioned rib was supported by shear friction. This detail did not perform well and spalling occurred in the ribs and in the edge beam. Figure 123 shows the spalling that exposed the pretensioned strands. At the ends of the ribs in the transfer length of the strands, there were high bond stresses that increased the tendency for spalling to occur.

Shear friction strength decreases with increasing crack width. Reliance on shear friction for load transfer in such situations was uncertain. Out-of-plane displacement of the wall would apply a prising action to the rib and the pretension strands anchoring it into the edge beam. Consequently the clamping force acting across the crack, which supports the rib, can be lost. In this event, the rib relied on support through the in situ concrete topping. A diagonal tension force must be sustained in the web of the rib, as shown in Figure 122, to transmit this force to the topping concrete. This is not a reliable load path, so positive bearing support on the lower surface of precast floor units should be used.

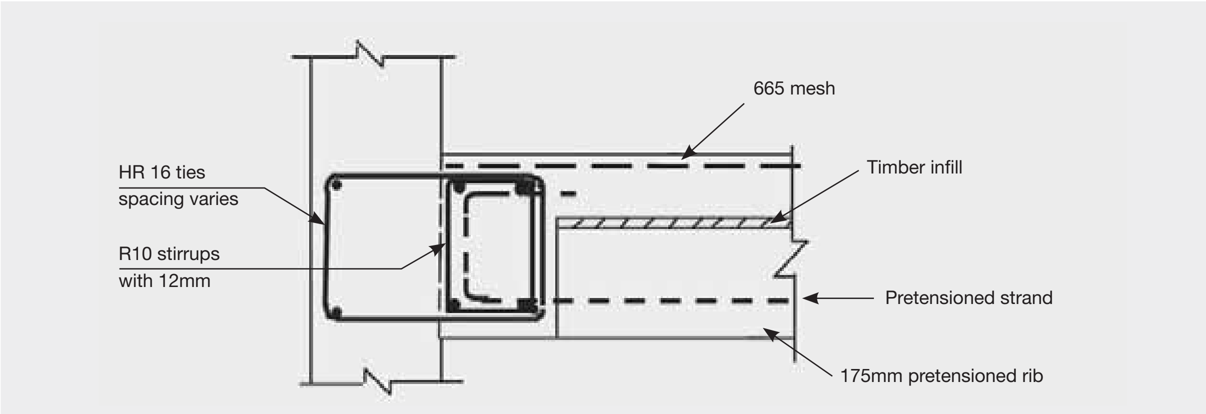


Figure 122: Support of the pretensioned rib on the edge beam

Figure 123: Exposed prestressing strands from the Interspan rib into the edge beam (source: Spencer Holmes Ltd)

Figure 124: Spalled rib and edge beam (source: Spencer Holmes Ltd)

The longitudinal edge beams were continuous along the eastern and western walls. With in-plane deformation of the walls, high flexural and shear forces were induced in these beams by the imposed deformation. The edge beams were separated from the structural walls for a distance of 800mm at the gap between adjacent walls. These edge beams were not detailed for ductility. Figure 124 shows the damage in one of these beams.

6.5.1.8 Conclusions

We conclude that:

1. The building was analysed as having limited ductility but detailed as if it was nominally ductile (elastically responding in terms of NZS 3101:1995[23]). For a nominally ductile structure the Earthquake Actions Standard, NZS 1170.5[10] would require the building to have a minimum strength of close to twice the corresponding value associated with the assumption of limited ductility (structural ductility factor of 3).
2. The longitudinal reinforcement in the walls complied with the minimum requirements in the Concrete Structures Standard, NZS 3101:1995[28]. However, the crack patterns showed there was insufficient reinforcement to cause secondary cracks to form. The yielding was confined to the primary crack and the high strains imposed caused some of the bars to fail in tension.
3. The current Concrete Structures Standard, NZS 3101:2006[12], has increased the minimum area of longitudinal reinforcement that must be used in walls. However, even with this increased area it was doubtful whether there would have been adequate reinforcement to generate secondary cracks, which would have allowed the yielding to spread, so reducing peak strains in the reinforcement.
4. The walls were designed and detailed to act as rectangular members. However, in two locations they were mounted at right angles in a T-shape. Both walls were joined to the floor so they were effectively coupled at these locations. Under seismic actions incompatible displacements were imposed on the floor slab and these zones were damaged.
5. In situ concrete edge beams were tied into the precast walls to provide support for the precast floor ribs. At the gap between the walls, the edge beam was separated from the walls by a distance of 800mm. In-plane deformation of the walls resulted in incompatible deformations being imposed on some of these beams and they were extensively damaged. The damage would have been reduced, but not prevented, if the nominally ductile detailing used had been replaced by ductile detailing in the edge beam. We note that the current Standard, NZS 3101:2006[12] would require ductile detailing of this beam, while the 1995 edition only required limited ductile detailing.
6. The precast prestressed floor ribs were supported by shear friction against the side of the in situ edge beams. Appreciable spalling of concrete occurred below the pretensioned strands and into the face of the edge beam. The ability of this form of support to provide safe support to a floor is questionable, because out-of-plane movement of the wall applies a prising action to the units and the pretensioned strands (which apply the clamping force necessary to sustain shear friction transfer). The prising action destroys the clamping force required for shear friction to act.

6.5.2 2 Riccarton Avenue: Christchurch Women's Hospital building

Current status
Substantially undamaged and has remained in use.

Figure 125: View looking north-east (source: Holmes Consulting Group)

6.5.2.1 Introduction

The Christchurch Women's Hospital building was designed in 2001 and 2002 and construction was completed in 2004 (see Figure 125). The building is also discussed briefly in section 3 of Volume 3 of this Report.

6.5.2.2 Building structure

The nine-storey building is the only base-isolated structure in the South Island and is positioned adjacent to the western end of the Parkside building complex, with a 550mm seismic gap between the two structures. The two buildings are connected via drop-in plates at each of the floors from lower ground floor to level 4.

The primary structure consists of precast prestressed floor ribs (spanning north–south) with a 100mm thick topping slab on timber infill planks. The floor is supported on precast beams (east–west) that span onto cast in situ interior and exterior columns. The lateral force resisting system in the north–south direction from the lower ground floor to the underside of level 3 is a dual system that uses reinforced concrete moment resisting frames at the ends of the building and eccentric K-braced frames forming the sides of the stair/service shafts. From level 3 to the roof, the reinforced concrete moment resisting frame forms the lateral force resisting system. The east–west direction lateral system consists of full height moment resisting frames on the northern and southern faces of the building. The entire building is supported (both for vertical gravity loads and lateral seismic shears) at the underside of the lower ground floor on lead rubber isolator bearings (see Figure 126). These are connected with a grid of stiff transfer beams.

The stair, lift and service shafts are framed with structural steel beams and posts and a composite steel deck and concrete topping forms the floors in these areas. The staircases are precast concrete seated on steel beams and tied into the floor topping slabs with reinforcement.

Above level 6 there are two mechanical/service floors covered by a structural steel portal frame and a lightweight roof system.

Figure 126: One of 40 lead rubber isolator bearings (source: University of Canterbury)

6.5.2.3 Building performance

HCG completed a full structural review of the Christchurch City Campus for the Canterbury District Health Board. The findings in this report are outlined as follows.

HCG observed little structural damage and there were few indications that ductile action had taken place in the concrete moment resisting frames or the steel braced frames above the isolated level. This is in keeping with the philosophy of a base isolation system, which concentrates the earthquake-induced deformations to the isolated level of the building.

Ground motions recorded at the Christchurch Hospital GeoNet site (CHHC spectra) had stronger horizontal ground motions in the September earthquake than in the February event, owing to the amplification of accelerations in the longer period range. Once the isolators yield they have a period of 2.5–3.0 seconds and a damping of 30 per cent and 22 per cent in a design-based earthquake (DBE – also referred to as ULS or ultimate limit state) and a maximum considered earthquake (MCE), respectively. The recorded acceleration response spectrum exceeded the original site-specific MCE design spectrum for the September record in the north–south direction. Of note is that the CHHC site is one of the four primary seismic measuring stations for Christchurch. It is located near this building on another part of the hospital site and the measured ground movements are, therefore, likely to be similar to those that affected this building.

HGC generally found that structural damage above the isolator level was limited to cracking of the floor slab and some stair landings. In places the cracking of the slabs was consistent with pre-existing shrinkage crack patterns but their extent and width may have increased as a result of earthquake movements. Some diagonal cracking was observed in the transfer beams supporting the elevator pit and spanning back to adjacent isolator bearings. Given that cracks were not extensively observed in other transfer beams, HGC thought it was possible that this cracking had occurred as a result of vertical acceleration of the liftshaft during the February earthquake. We note that bearings are stiff vertically to support gravity loads and, therefore, do not isolate seismic vertical accelerations. Mechanical equipment “excited” by vertical accelerations may also have caused flexing and damage to slabs and cantilevers.

From its evaluation of the structural drawings and observations at the site HGC did not consider that there were any critical structural weaknesses in the lateral force resisting system. However, it considered that the cracks in the precast ribs forming the lower ground floor were a significant weakness and required immediate attention. Another weakness identified was the detailing of the stairs at mid-landing. Based on the structural drawings, it appears that the preferred allowance for relative movement between the floor levels cannot be accommodated by the detailing used and will need to be remediated to ensure that no further damage occurs under large earthquake demands.

6.5.2.4: Conclusions

We conclude that:

1. The base isolation system for this building generally performed as expected and, to a large extent, limited the damage caused by horizontal accelerations.
2. Vertical accelerations are not usually damped by base isolation systems. In addition, high-frequency displacements can be transmitted through the lead core of the isolators. This might cause damage to sensitive equipment, or induce large vertical actions by accelerating heavy components such as liftshafts.

6.5.3 224 Cashel Street: IRD building

Current status
Still standing but proposed to be demolished.

Figure 127: View of the IRD building from the corner of Cashel and Lichfield Streets (source: Becker Fraser Photos)

6.5.3.1 Introduction

The IRD building at 224 Cashel Street (Figure 127) was designed between 2004 and 2006 and issued with a building consent in four stages, receiving a code compliance certificate on 16 October 2007. It is a seven-storey office building with the Inland Revenue Department as the primary tenant and retail businesses on the ground floor. It is rectangular in shape, with overall dimensions of about 40m by 60m. It is relatively regular, with minimal eccentricities to the seismic structure.

6.5.3.2 Foundations and ground floor

The foundations are on 900m and 1200mm bored concrete piles founded in dense sands at depths of up to 12m below street level. The shear core is supported on two interconnected 2.5m deep reinforced concrete rafts supported by the piles, with the rafts being at the eastern and western ends of the shear core. Extensive ground investigations were carried out before the design was undertaken when the risk of liquefaction was considered.

The ground floor is a 100mm thick conventionally reinforced concrete slab.

6.5.3.3 Structure

A typical elevated floor plan is shown in Figure 128. The elevated floors are built from 300mm thick hollow core precast floor units with a 90mm thick in situ concrete topping. These floors are supported on precast concrete beams on the northern and southern sides of the building perimeter, the shear core walls and internal moment resisting frames on grid lines E and H between the eastern and western walls and the shear core. Throughout the structure extensive use is made of precast concrete in both the moment resisting frames and the shear core.

The roof is lightweight steel with structural steel roof framing.

Precast fin panels made of reinforced concrete are connected to the exterior of the eastern and western perimeter walls.

Exterior façade frames are attached to the moment resisting frames along lines B and L on the northern and southern walls of the building.

The primary lateral force resisting element is the central shear core, which is assisted in the east–west direction by the moment resisting frames in the northern and southern walls. The shear core comprises precast panels that are interconnected with in situ concrete. In the north–south direction, concrete panels are connected with diagonally reinforced coupling beams to form coupled shear walls. In the transverse direction the walls are built up from hit and miss precast and in situ concrete.

The primary purpose of the internal moment resisting frames on lines E and H is to support the gravity loads.

The main stairs consist of precast flights and landings with an in situ topping to the landings. They are located within the shear core. No provision appeared to have been made for inter-storey drift but the stairs suffered no apparent damage in the earthquakes.

At the time of design, NZS 4203:1992[22] was the relevant verification method for the New Zealand Building Code, and NZS 1170.5:2004[10] was not cited (therefore needed to be considered as an alternative solution to the Building Code). Both Standards were considered as a part of the design, which indicates that the building should have complied with NZS 1170.5[10] at the time of the February earthquake.

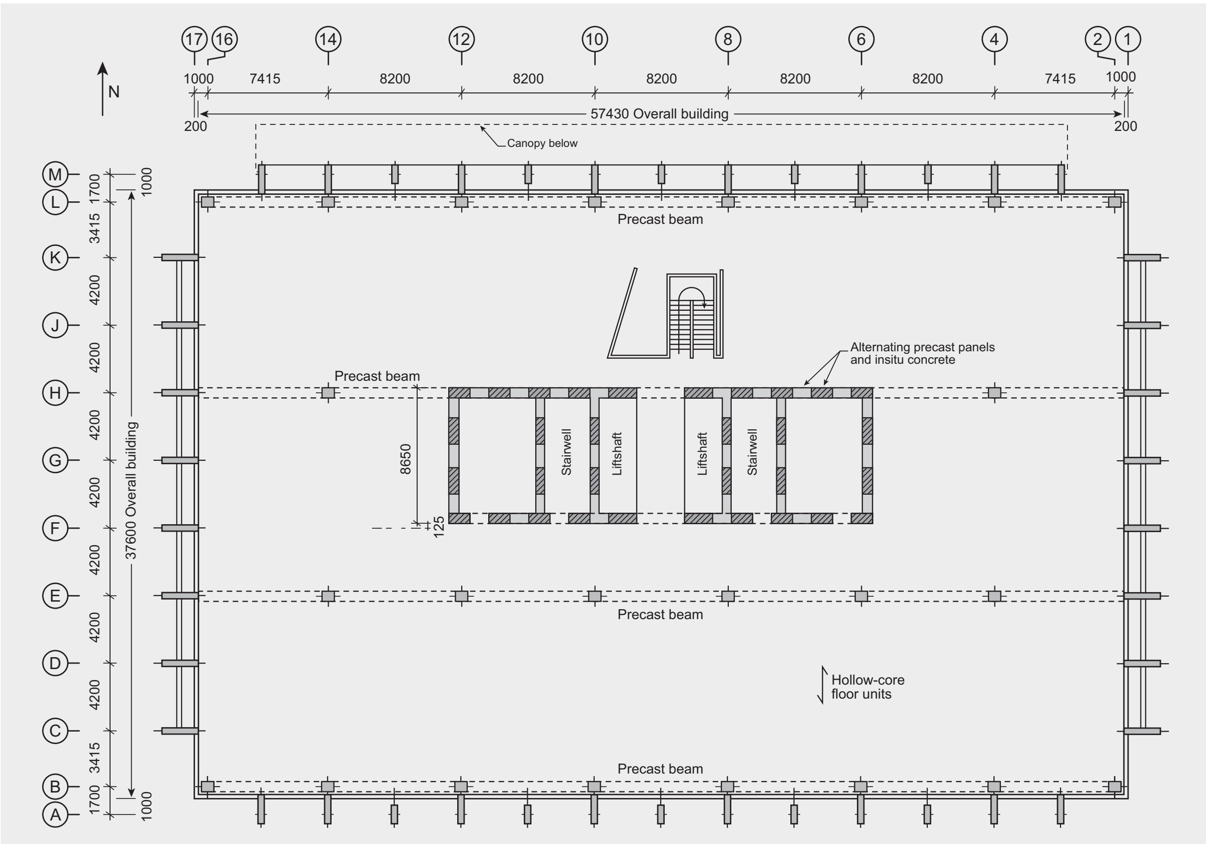


Figure 128: Typical upper floor

6.5.3.4 Building performance

The Royal Commission was assisted in the assessment of the performance of this building by a report prepared by Spencer Holmes Ltd.

6.5.3.4.1 General observations

We have no evidence to show there was any significant damage to the building as a result of the September and Boxing Day 2010 earthquakes or in the aftershocks associated with those events.

The effective initial fundamental period of the IRD building was assessed as between 0.8–0.9 seconds depending on direction of excitation. The February earthquake was particularly damaging to buildings with a period range of 0.5–1.5 seconds. This building was likely to have been subjected to shaking exceeding 1.5 times the intensity of design seismic actions for the ultimate limit state. From the structural drawings it is apparent that it was a robust structure and detailed to comply with the Concrete Structures Standard, NZS 3101:1995[23].

Design Standards for reinforced concrete structures (NZS 3101:1995[23] and NZS 3101:2006[12]) recommend that section properties should be based on gross section properties multiplied by a factor that allows for the reduction in stiffness due to flexural cracking. We assumed that the recommended section properties would have been used in the design. However, from the extent of flexural cracking observed in the concrete after the February earthquake, it was clear that this cracking was considerably less than that consistent with the recommended allowance for cracking. Consequently, the fundamental period was likely to be around 0.6 seconds rather than the value of about 0.8 seconds assumed in the design. The response spectra calculated from the recorded ground motions in the CBD indicate that a reduction in the initial fundamental period from 0.8 to 0.6 seconds would increase the seismic forces and reduce seismic displacements.

6.5.3.4.2 Foundations

In the geotechnical report on the foundation soils, it was recommended that the strength of foundation soils could be based on 0.8 times the average measured value for load combinations that included seismic forces.

The piles were founded in sands at a depth of about 12m. In the February earthquake differential settlement of 20–90mm occurred between the shear core and the perimeter frames. The greatest differential settlement was between the corner columns and the shear core. It was likely that this differential settlement was associated with:

- liquefaction reducing friction on the sides of the piles and the end bearing strength of the piles; and
- the different levels of gravity load and seismic forces acting on the different structural elements.

The seismic forces sustained by the shear core may have been underestimated and this may have contributed to the increase in the settlement of the shear core. This underestimation of design actions arises because current design practice does not allow adequately for the interaction of floor diaphragms with some structural wall systems. This issue is discussed later.

The foundations were designed to allow for the level of liquefaction expected in an Alpine Fault earthquake. However, the extent of liquefaction in the February earthquake exceeded that anticipated level.

6.5.3.4.3 Moment resisting frames

Seismic resisting moment resisting frames are located on the northern and southern sides of the building. They appear to have performed well in the February earthquake. Flexural cracks, which ranged from one to five millimetres, were observed in the beams at the faces of the columns. In one or more of the corners of the building it was noted that the crack width was greater at the top of the beam than at the bottom. This was consistent with the measured differential settlement of the shear core relative to the perimeter frames and the redistribution of gravity load bending moments caused by yielding of the reinforcement. A crack 5mm wide indicated that material strains of about one third of the maximum permitted in NZS 3101:2006[12] were sustained in the earthquake. It is likely that other secondary cracks opened up during the earthquake but largely closed again when the ground motion ceased, making it difficult to see them in subsequent inspections. The top flexural reinforcement in the beam consisted of three 25mm and two 20mm bars with a 600mm wide beam. This level and concentration of reinforcement should have been adequate to initiate secondary flexural cracks. However, unless the strains in the reinforcement at the column face were sufficient to induce appreciable strain hardening in the reinforcement, the increase in strength would not have caused yielding to occur at one or more of the secondary cracks. Without this yielding, it would be difficult to see the cracks, which were likely to be less than 0.05mm wide.

6.5.3.4.4 Floor diaphragms

The opening up of the cracks in the beams at the column faces was a clear indication that elongation had occurred in the beams. This movement was reflected in the floor slabs, which sustained some cracking at right angles to the beams and between the precast hollow-core floor units.

The floor diaphragms appeared to have performed adequately. A positive moment flexural crack was seen in the soffit of the hollow-core units close to a support in one floor. Such cracking has been shown to seriously reduce the strength of hollow-core floors. However, in this case there would be no loss of strength as the hollow-core unit was tied into the supporting beam by reinforcement with in situ concrete being placed in broken-out cells at the supports[26].

On the northern and southern walls in the shear core (lines F and H) many of the hollow-core units were supported by TAC20 connectors anchored into the walls. The TAC20s were placed into broken-out cells in the hollow-core units that were later filled with in situ concrete (Figure 129). The connection performed adequately during the earthquake, but with this arrangement the support of the precast unit depends to a large extent on shear friction at the interface and to a lesser extent on potential dowel action of the reinforcement. The dowel action of the bars is limited (refer to NZS 3101:2006[12]) and shear friction decreases with increasing crack width. As it was not possible to accurately predict the width of cracks that might be induced in severe earthquakes, we do not recommend that shear friction be relied on to support precast floors.

6.5.3.4.5 Shear core

The north and south shear core walls consisted of individual wall panels linked by diagonally reinforced coupling beams. Coupling beams have been extensively used in construction throughout the world. However, one aspect of their behaviour has received little attention. A diagonally reinforced coupling beam will elongate in a very similar manner to a reinforced concrete beam. Any such elongation was partially restrained by floor slabs which, in this case, extended round the shear core. This restraint applies axial forces to the coupling beams, increasing their strength. As a consequence, higher lateral forces may be resisted by the walls than was anticipated in the design. An assessment indicates a likely increase in the strength of the coupling beams of the order of 50 per cent compared with strengths assessed by current design practice. An increase in resistance of this order would have significantly increased the seismic forces applied to the foundations and might have contributed to the settlement of the foundations of the structural walls seen after the earthquake.

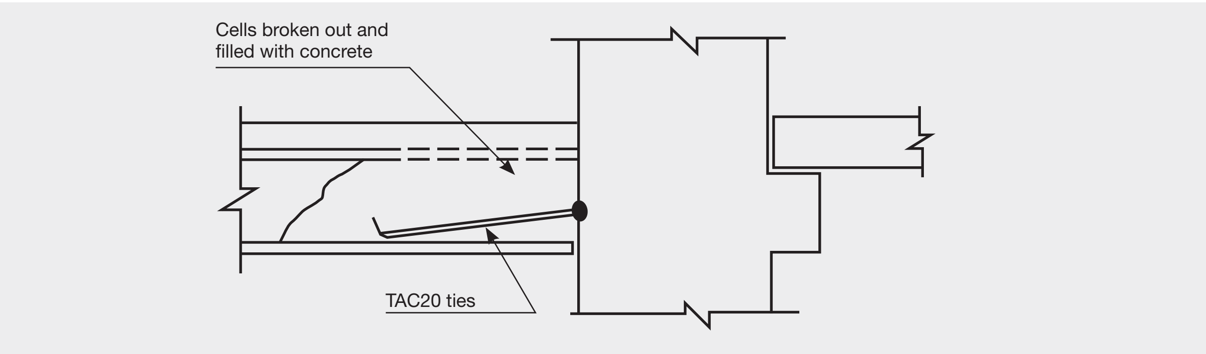


Figure 129: Support of hollow-core unit on shear core wall using TAC20 connector

6.5.3.4.6 Canopy

The canopy that spanned between the IRD building and the adjacent car park pavilion collapsed in the February earthquake. Given the intensity of the earthquake this was not surprising as the peak displacement between the two buildings would have been close to twice the value given by NZS 1170.5[10]. The factor of 2 arises from the design displacement being less than the peak value owing to the use of the structural performance factor (S_p) and the calculated spectral displacements in the CBD being about 50 per cent greater than the values calculated from the design spectral values.

6.5.3.5 Conclusions

We conclude that:

1. Liquefaction on bored piles is likely to have reduced the strength of the soils, which may have contributed to the observed settlement of the building.
2. The recommended use of a strength-reduction value of 0.8 for the design of soil strengths for over-strength actions was likely too high for granular soils.
3. The floors restrained the elongation of coupling beams in coupled structural walls. This restraint could increase the strength of coupling beams and impose higher seismic actions on the walls and foundations than would be calculated by standard design practice.
4. Cracking in beams in the moment resisting frames appeared to consist of single cracks at the column faces. However, it was likely that other cracks formed by the reinforcement did not yield and, consequently, closed. They would not have been apparent without the use of a crack microscope.
5. Reliance on the support of floor units by shear friction cannot be recommended.
6. Peak displacements induced in the February earthquake were significantly greater than indicated by standard design calculations.

6.5.4 166 Gloucester Street: Pacific Tower building

Current status
Under repair.

Figure 130: Pacific Tower building, looking south-east from Gloucester Street (source: Clifton et al.[27])

6.5.4.1 Introduction

The Pacific Tower building (formally the C1 Tower) was designed in 2006 and 2007, with four building consents issued by the CCC in those years for different stages of the development. Amendments to the consents were issued up until 2009, with a code compliance certificate issued in 2010. It is a 22-storey steel-framed building with precast concrete cladding panels (Figure 130).

6.5.4.2 Structure

Lateral load resistance is provided by eccentrically braced frames in both K and D configurations, as well as moment resisting frames. There are vertical irregularities in the configuration of the eccentrically braced frames and moment resisting frames that require the floors at levels 2, 6 and 11 to act as transfer diaphragms.

The floors and roof are typically built with a 150mm thick composite steel deck supported on composite steel beams. The topping is reinforced with 10mm diameter Grade 500 reinforcement spaced at 300mm centres in each direction, supplemented by additional reinforcement, known as drag bars, where necessary.

Reinforced concrete foundation beams are supported on a combination of bored concrete piles and steel screw piles (used primarily for tension loads). The ground floor is a reinforced concrete slab on grade.

The stairs and car stacker level ramp are detailed to slide to prevent overloading from inter-storey drift.

6.5.4.3 Performance and damage

A post-earthquake assessment and detailed engineering evaluation was carried out by CPG New Zealand Ltd. The performance and damage reported here has been taken from the CPG report.

The building was designed to NZS 1170.5[10] with a zone hazard factor, Z, of 0.22, as applied in Christchurch before the February earthquake. It was detailed as a limited ductile structure. Clifton et al[27] and CPG stated that the design ductility, μ, was close to 1.5 (due to standardised section sizes and to the conservative approach of adding the gravity shear component to the earthquake shear when sizing the active links). The building's calculated fundamental periods are 3.96 and 3.26 seconds in the north–south and east–west directions respectively. CPG noted that the performance of the building in the earthquake series showed that the structure was twice as stiff and strong as indicated by analytical models used in design.

CPG concluded that the building experienced earthquake shaking greater than the design-level earthquake for the recently revised seismic actions associated with a seismic hazard factor, Z, of 0.3.

Given the satisfactory performance of the building in these earthquakes, CPG concluded that the building would meet the criterion for 100 per cent of new building standard (NBS) once structural repairs were completed.

The building generally suffered minor structural and non-structural damage but isolated areas of significant damage were also recorded. The residual deformation measurements suggest the building twisted slightly, although mostly in the upper levels. These offsets were not of concern to CPG as they were within the displacements allowed for during design and construction.

The CPG investigation involved visually inspecting at least one side of all eccentrically braced frame links and moment resisting frame potential plastic hinge regions for yielding and any significant permanent offsets or fractures.

A number of active links in the eccentrically braced frames showed evidence of the onset of yielding, with some permanent deformation. Yielding was indicated by diagonal Luders' lines and paint flaking. One active link fractured, as shown in Figure 131. This link was located on the north-western frame at the underside of level 6.

Figure 131: Fractured active link in the Pacific Tower (source: Clifton et al.[27])

An assessment of damage accumulation by Professor Charles Clifton concluded the links had undergone 45 per cent damage (where 100 per cent damage means the link has fractured) and they would have sufficient capacity to resist a repeat of the Canterbury earthquake series or an Alpine Fault rupture without failure.

In its report, CPG stated that hardness testing of the lower links in this frame (without slabs attached) indicated strain hardening had occurred and that this would have resulted in a reduction of ductility capacity. While this has not been quantified in comparison to destructive testing of outside samples, it was CPG's opinion that it would be advisable to have these links replaced.

6.5.4.4 Conclusions

The performance of the Pacific Tower building appears to have been satisfactory. The level of redundancy in the building gives it a robust structure.

The Royal Commission has not carried out a detailed assessment of the Pacific Tower building but we consider that some aspects require further investigation. The concerns we have include:

1. CPG reported that the structure was twice as stiff and twice as strong as the analytical models had indicated. This would have reduced the fundamental periods of vibration from 3.96–2.8 seconds in the north–south direction, and 3.26–2.3 seconds in the east–west direction. The February east–west ground motions showed amplification in the period between 2.7 and 4 seconds. For this ground motion, the reduction in period would have reduced the acceleration demand on the building, as shown in Figure 132. It is highly desirable that the source of this increased stiffness should be accounted for.

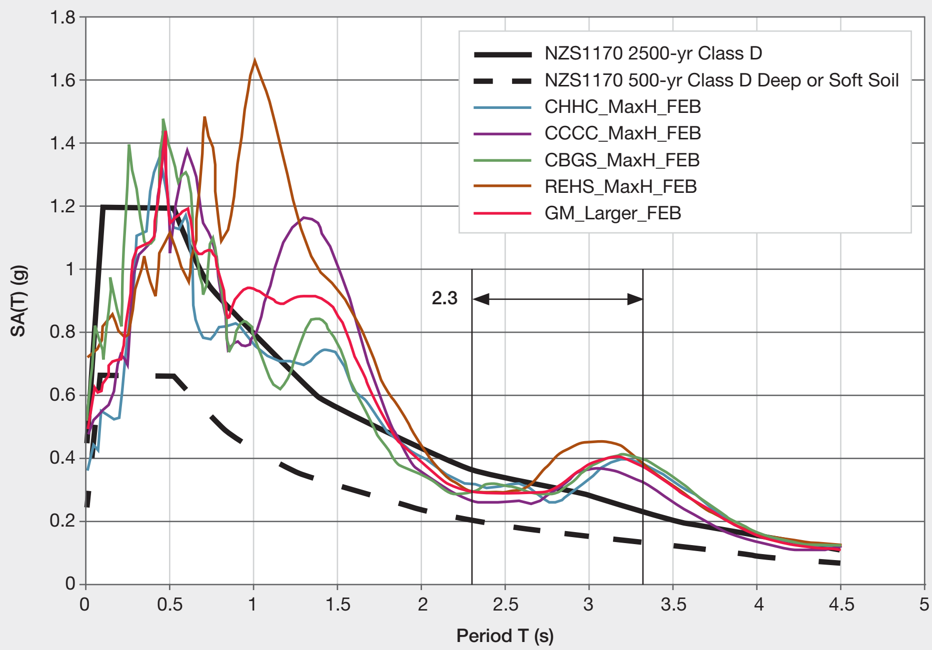


Figure 132: NZS 1170.5[10] spectra and largest horizontal direction recorded from the CBD strong motion records (source: Clifton et al.[27])

2. The required level of ductility used for design, *u*, was stated as 1.5 in both the paper by Clifton et al[27] and in the CPG report. Given the magnitude of the February earthquake, this would indicate the ductility demand would have been considerably less than 3, and even less than this if the building, in fact, was twice as stiff and twice as strong as indicated. This would imply that relatively small ductility demands would have been placed on the eccentrically braced frame active links. Hence, it was surprising that one failed and several sustained displacements well into the strain hardening range. It was important to note that building ductility was limited by the ductility of the active link and also to note that a relatively small increase in displacement ductility of the building can greatly increase the deformation ductility demand in the active link.
3. It was seen that some of the links had been strained well into the strain hardening range. These should be tested to see if there were any adverse impacts from both strain hardening and strain ageing. Strain levels of four per cent were in the range where strain ageing may be expected to have a significant influence on performance.
4. Owing to the mixed use of eccentrically braced frames and moment resisting frames in the building, the columns were considerably stronger and stiffer than would be required by NZS 3404:1997[28], or in a building where greater reliance was placed on eccentrically braced frames alone. This raised a question about the stability of a building if eccentrically braced frames were located in the boundary walls, with one braced bay in each wall. In this situation the loss of a single active link would be likely to result in a loss of torsional resistance, leading to a major overload of the remaining walls, which could lead to its collapse. The failure of the link in the Pacific Tower building highlights the need for a degree of structural redundancy in these buildings, which may be provided by requiring more robust columns.
5. The failure of the active link at what appeared to be a strain level well below the strain at which fracture may be expected to occur, highlights the need for very high quality detailing, construction and supervision of eccentrically braced frames.
6. We would encourage the steel industry to thoroughly assess the performance of eccentrically braced frames and demonstrate in tests that eccentrically braced frame units built under normal construction conditions, and tested under dynamic loading rates comparable with those induced in the February earthquake, do, in fact, have the level of reliability required by the Building Code.

6.5.5 52 Cathedral Square: Novotel Hotel building

Current status
To be repaired.

Figure 133: View looking east from Cathedral Square (source: CCC)

6.5.5.1 Introduction

The Novotel Christchurch building (Figure 133) is a 14-storey hotel located in the north-eastern corner of Cathedral Square. The building consists of 11 storeys of hotel accommodation with three levels of mixed retail and back-of-house space above a basement car park. A plant room floor is located at the top storey of the building. The building was designed and detailed in 2007 in accordance with AS/NZS 1170:2004[10] and was built in 2008 and 2009, with a formal opening in early 2010.

The Royal Commission has relied on the information provided by Lewis Bradford and Associates Ltd, Consulting Engineers, for the assessment of this building. The crack pattern observed in the southern shear wall above level 4 was of particular interest to us, as it illustrated the expected performance that was, unfortunately, not shown in many buildings. For this reason, the discussion is mainly limited to that part of the building, with some limited comment on the performance of the concrete panel cladding system.

6.5.5.2 Underlying ground conditions

The Novotel site is underlain by sandy gravel, sand and silt to a depth of about 24m below the existing ground surface and overlying dense to very dense sandy gravels. There is a low to moderate risk of liquefaction in the soils underlying the site in a future ultimate limit state event. Under an ultimate limit state event the estimated liquefaction-induced total settlement was about 0–10mm.

6.5.5.3 Gravity load system

The floors are built up from a metal tray (Traydec) with in situ concrete that varies in thickness from 125–170mm, depending on the location of the floor in the building. The floors are supported by steel beams, which in turn are supported on steel columns and the concrete walls.

6.5.5.4 Lateral load system

Lateral forces are resisted by four reinforced concrete in situ walls with thicknesses that vary from 400mm at their base to 200mm at the top of the building. The location of the structural walls is shown on Figures 134 and 135 for the podium (to level 4) and the tower, respectively.

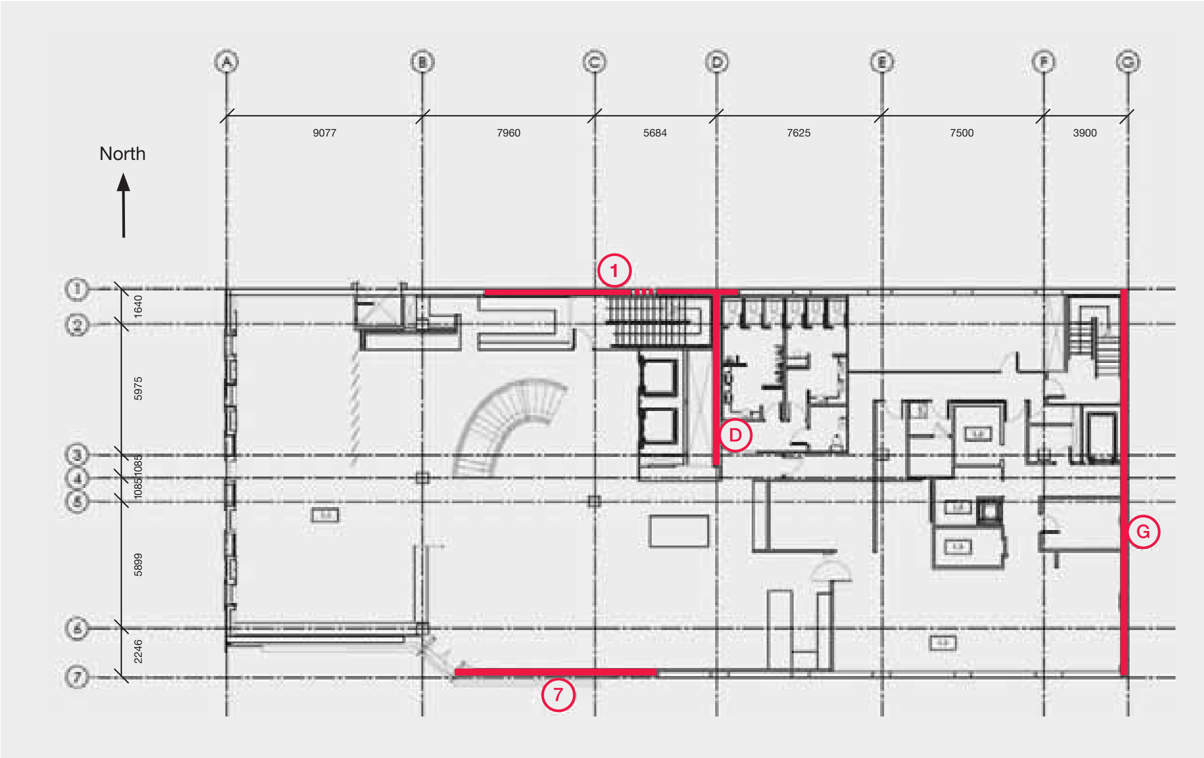


Figure 134: Floor plan at level 1 (similar up to level 4). The major structural walls are shown in red

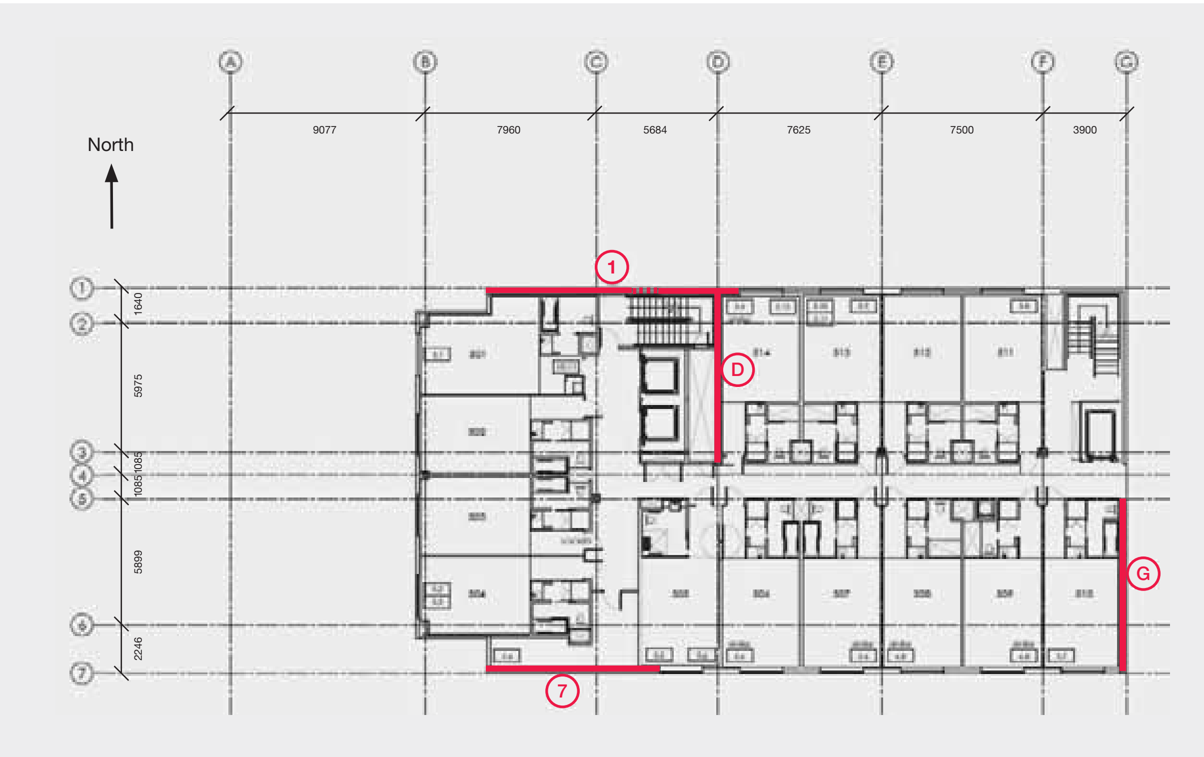


Figure 135: Typical floor plan for the upper levels. The major structural walls are shown in red

The details of the four walls are as follows: the wall on the eastern side of the building is 8m long above level 4 and below this level it occupies the full width of the building. The wall on the southern side of the building is 8m long above level 4 and below this level it is about 50 per cent longer. The wall on the northern side is more than 11m long and is continuous with an 8m wall on grid line D. These two walls act as an L-shaped structural member, with an appreciably higher lateral stiffness than the other two walls. With this arrangement the centre of stiffness is located about 5m from the northern wall and close to the intersection with grid line D. Owing to the change in length of two of the walls below level 4, the floors at levels 4 and 5 act as transfer diaphragms.

Drag bars in all the floors were cast into the topping concrete to carry the inertial and transfer forces to the walls.

6.5.5.5 Performance of structural walls

In the February earthquake the most intense shaking was in the east–west direction. As pointed out by Mr Lewis Bradford, because the centre of stiffness was in the northern half of the building, the greatest displacement imposed on the walls from this direction of earthquake actions would be in the southern wall. Inspection of the building showed that flexural cracking did occur above level 4 in this wall, with crack widths of 0.5–0.8mm. This zone was detailed as a potential plastic hinge region. The open cracks indicated that some plastic deformation developed in this zone. Of particular interest was a series of cracks developed at a spacing in the range of 150 to 500mm. Given the angle of the cracks and their reported widths, it was evident that they were due to flexure and axial loads. Because the cracks were inclined at less than 20^{o} to the horizontal it is unlikely that the horizontal reinforcement would have yielded.

The wall at level 4 is 300mm thick and 8m long. Above this level it extends to a height of about 27.5m. The wall is reinforced with longitudinal bars at 200mm centres on each face. The 1m strips at each end of the 8m long wall each contain 10 32mm grade 300 bars (2 by 5). In between these two strips were sixty 20mm Grade 500 bars (2 by 30). The area of concrete surrounding each group of 10 32mm bars is 240,000mm^2. Allowing for the transformed section, the stress the 32mm bars can induce in the concrete when they reach the yield point is 6.9Mpa, which should be more than adequate to ensure that secondary cracks can form. The corresponding tension stress that can be induced into the concrete surrounding the 20mm bars if they reach yield is 4.9MPa, clearly enough to cause the cracks initiated in the heavily reinforced end strips to extend into the mid-region of the wall.

Creep and shrinkage in concrete can influence crack formation in structural walls. Assuming typical values for the concrete, namely, free shrinkage strain of the concrete of 500 x 10^{-6}, a creep factor of 2.5 and an axial stress due to gravity (based on gross section) of 1MPa, the resultant stresses in the concrete are of the order of 0.13MPa in tension and 46MPa in compression in the reinforcement. In this case, the creep due to gravity loads almost cancelled out the tensile stresses induced in the concrete by shrinkage.

In the Lewis Bradford damage report, it was indicated that the cracks in the potential plastic hinge zone of the wall on the southern side of the building, just above level 4, were 0.5–0.8mm wide. On the basis of the cracks that can be observed in the diagram derived from a photograph provided by Lewis Bradford (see Figure 136), the material strain sustained by the potential plastic hinge can be assessed on the basis that the cracks were 0.8mm wide and the distance to the neutral axis from the extreme tension fibre was 7m. Based on these assumptions the material calculated, as detailed in NZS 3101:2006[12], was less than 15 per cent of the maximum permitted value given in that Standard.

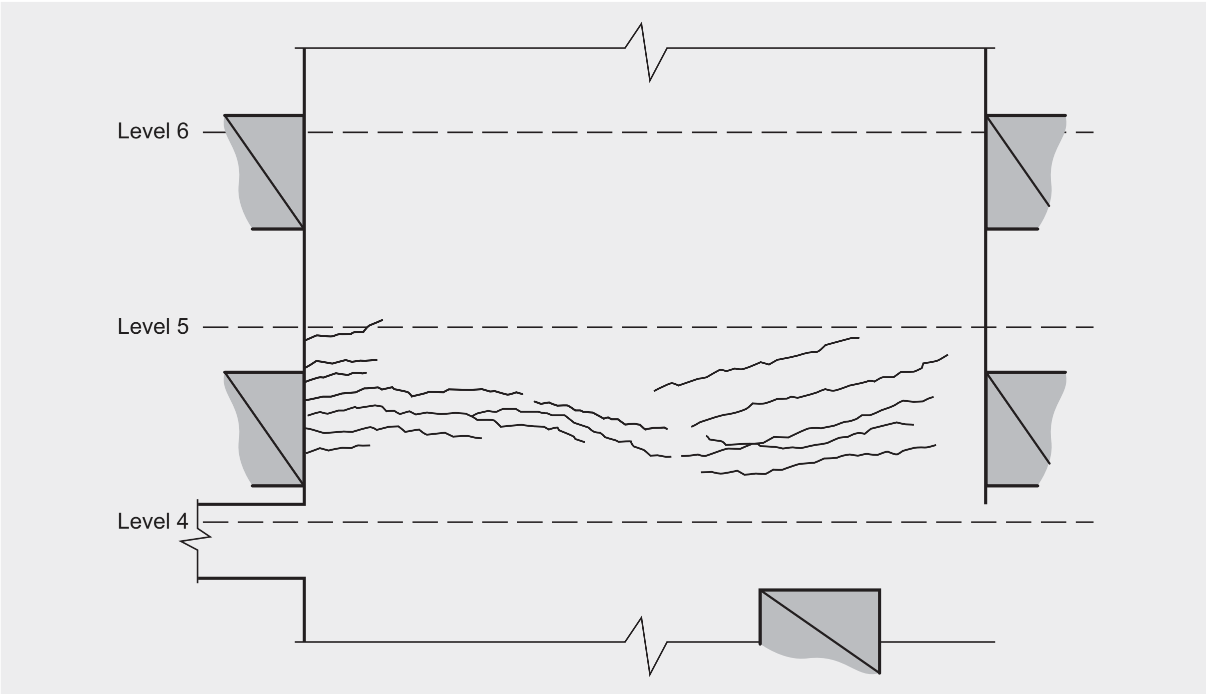


Figure 136: Crack patterns to the southern wall between levels 4 and 6

6.5.5.6 Performance of cladding panels

Under the very high ground motions that occurred during the February earthquake, the building could have moved horizontally 50mm or more at each floor. This level of inter-storey movement would be expected to cause some damage to the cladding panels owing to tightening of normal construction tolerances. The damage observed was significantly exacerbated where panel movement was impeded by solid high strength mortar joints between panels, butt joints or steel brackets between panels. This is an example of where the construction of the building has not met the intention of the design, and the seismic gaps and joints have not been maintained as intended. This has also been discussed in section 6.3.4 of this Volume with the western wall of the building at 151 Worcester Street.

Conclusions

We conclude that:

1. The cladding panels were installed in some cases so that the seismic gaps were compromised. Some damage was expected given the magnitude of the February earthquake, but the damage was exacerbated by the installation deficiencies.
2. Cracking in the walls, particularly in the potential plastic hinge region in the southern wall immediately above level 4, has been well controlled by the quantity and arrangement of reinforcement in the wall. The performance of this wall was excellent given the seismic ground motion to which the building was subjected. The distribution of reinforcement in the walls ensured reinforcement strains remained in an acceptable range and cracking was well controlled.

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Section 7: Cost implications of changing the seismic hazard factor

The Z factor is the seismic hazard factor that is applied to a location. It is a fundamental value used to determine the design seismic actions for buildings.

7.1 Background

NZS 1170.5 sets out hazard factors: Z in Table 3.3 for cities and towns, and in a contour map of New Zealand on pages 18 and 19. The Z factor calculation is underpinned by the Natural Seismic Hazard Model, as discussed in section 2 of Volume 1 of this Report. The highest Z factor is 0.6 (Otira and Arthur's Pass) while the lowest is 0.13 (a number of places including Auckland, Northland and Dunedin). Other examples are Wellington, Porirua and Hutt Valley at 0.40; Christchurch was 0.22 but it was raised to 0.30 after the earthquakes.

7.2 Introduction

The Z factor for Christchurch was increased to 0.30 after the earthquakes to take account of greater seismicity in the region. This change was made in consultation with GNS Science. Further work has been done by GNS since that time on the appropriate value for the seismic hazard factor for Christchurch, given the continuing seismic activity. This activity is expected to decrease over a period of decades (as discussed in section 2 of Volume 1). Further consideration is being given to the matter in the light of additional work on seismic hazards in the Christchurch region. At the time of writing the review of the Z value and the return factor for the servicability limit state are continuing.

The associated return period factor for the serviceability limit state was increased from 0.25 to 0.33. The return period factor and limit states are discussed in section 3 of Volume 1 of this Report.

7.3 Cost implications study

The Royal Commission considers it is important that the seismic hazard factor assigned to a place should provide an accurate reflection of the area's earthquake hazard. However, it is also important not to overstate the hazard and thereby impose unjustified additional construction costs.

We have reviewed the structure of eight buildings with a view to ascertaining the cost implications of an increase in the Z factor. When we commenced this review, the Z factor and the associated return period factor for the serviceability limit state had been increased. However, during the Commission's hearing on seismicity, GNS said further work on its seismic hazard model suggested a higher Z value, in the range of 0.34–0.39, with a corresponding return period factor for serviceability of 0.28.

For this reason we chose to assess cost implications for a hypothetical Z value of 0.35, except where a building had been redesigned already owing to the increase in Z factor to 0.30. In some cases, a major part of the cost increase came from the increased serviceability actions and the need to protect the building against damage associated with liquefaction in this limit state.

The buildings were all commercial structures ranging in size from one to 10 storeys, some with car parks. They were either built within the last 10 years or the subject of recently-issued building consents and not yet completed. They had a variety of foundation types and raised different geotechnical considerations. In some cases, it was apparent that a decision had been made to exceed the minimum requirements arising from the increased Z factor. A summary of the buildings' attributes and the increase in construction costs is shown in the following table.

Increases in construction costs associated with higher Z factors for eight Christchurch buildings			
Building	Characteristics	Z factor change	Increase in construction costs
1	Two levels: retail and car park	0.30 to 0.35	1%
2	Three-level office building	0.22 to 0.35	1%
3	10-level office building plus two-level car park	0.22 to 0.35	2.6%
4	Single-level educational block	0.22 to 0.35	1.1%
5	Three levels: two above ground, one basement	0.22 to 0.3	15.5%
6	Two-level government building	0.22 to 0.35	4%
7	Three-level hostel	0.22 to 0.3	7%
8	One-level supermarket plus basement car park	0.22 to 0.3	5%
Note: building 5 required a complete re-design of foundations, whereas the other buildings generally required only additional steel and concrete to comply with the higher Z factor			

7.4 Discussion

While this is not a statistically significant sample, it provides an indication that construction costs do not generally appear to be significantly increased as a result of increases in the seismic hazard factor of the magnitude we have considered.

Overall, the incremental construction cost of building to a Z factor increased from 0.22 to 0.35 was less than five per cent, and in half the cases less than two per cent.

7.5 Conclusion

It is important that the seismic hazard factors adequately account for risk, but that they do not overstate it. Overall construction costs do not appear to significantly increase as a result of increases in the seismic hazard factor, based on the limited sample of buildings that were reviewed by the Royal Commission.

Section 8: Discussion of representative sample issues

8.1 Introduction

8.1.1 General

In this section issues noted in our study of the representative sample of buildings are brought together to enable some overall observations to be made.

Structural design and material Standards are largely based on the results of structural testing, so it is desirable to relate the observed earthquake damage to the results of structural testing on individual structural elements. This aspect is discussed in section 8.1.2 of this Volume.

Before examining the observed performance of different structural elements in the Christchurch earthquakes the influence of strain ageing of steel, which can change the load-deflection characteristics of structural steel members and reinforcement, is briefly discussed in section 8.1.3 of this Volume. The method developed to assess the remaining strain capacity of steel in buildings is briefly outlined in section 8.1.4 of this Volume.

In subsequent sections the observed performance of individual structural elements is considered, together with the way in which these elements interact.

8.1.2 Comparison of observed behaviour and results of structural tests

In structural design a number of conservative assumptions are made. From an analysis of the proposed building, the design engineer determines the minimum required strength of the individual components to satisfy the ultimate and serviceability limit states. This is referred to in the New Zealand Standards as a design action. Generally the value will be conservative, as this analysis assumes the most adverse possible combination of actions. Having obtained a design action, such as the minimum flexural strength of a member, the design process then assures this minimum strength is achieved with a high level of certainty, given the variability of materials and workmanship, the reliability of design equations, etc. The design strength, which has to be equal to or greater than the design action, is taken as the nominal strength multiplied by a strength-reduction factor.

The nominal strength is a theoretical value calculated assuming that the materials in the member have their lower characteristic strengths. Consequently, for example, if the minimum quantity of reinforcement is used in a reinforced concrete member, the strength of that member will on average be greater than the nominal strength in more than 95 per cent of cases. The strength-reduction factor for reinforced concrete members subjected to flexural and axial load is 0.85.

In practice the nominal strength nearly always exceeds the minimum required value. This arises as the reinforcement sizes are not infinitely variable, and for simplicity it is important to maintain similar reinforcement arrangements for similar members. A typical ratio of nominal strength to the minimum required would be 1.15:1.0.

The nominal strength is calculated assuming the material properties have their lower strength characteristic values. Replacing the lower characteristic strength by average material properties would increase the strength by a factor of about 1.15.

The strength-reduction of 0.85 for flexure in reinforcement is equivalent to a factor of safety of 1.18.

Allowing for all of the factors indicates that the average strength will be around 1.5 times the design action.

In the design of a ductile structure a structural ductility factor of around 5 is likely to have been used, which if based on the likely average strengths would reduce the structural ductility factor to 3.3. Allowing for the conservative combination of actions assumed in the analysis, the actual ratio of average member strengths to the corresponding actions induced by an earthquake with an intensity equal to the design-level earthquake is likely to correspond to the values associated with a structural ductility factor between 2.5 and 3.0. A number of other factors such as energy dissipation in soils, increased damping associated with non-structural elements, etc., may further reduce the inelastic demands in an earthquake compared with design values.

In any comparison between damage sustained in an earthquake and that observed in structural tests, it is essential to recognise the inherent conservatism of structural design. Given all the variables that may occur this conservatism is essential if the risk of failure is to be kept low as specified for the ultimate limit state in our design codes and Standards. On the basis of the discussion above one could expect that on average the damage observed in the February earthquake would correspond to that observed in individual components tested to displacement ductility factors of 3.0–4.5.

It is also important to note that most structural tests apply an appreciable number of inelastic load cycles of increasing magnitude, all of which increase the damage sustained by the test specimen. Owing to the short duration of the February earthquake, very few inelastic load cycles would have been applied to the structures before the maximum displacement was imposed.

8.1.3 Significance of strain ageing

Strain ageing develops in reinforcing and structural steel over a period of a few weeks after it has been strained into the inelastic range. The extent to which strain ageing develops depends on the chemical composition of the steel, the temperature and the level of strain to which it has been subjected. The chemical composition and source of manufacture have varied over the years, so no single set of figures can be given for the influence of strain ageing in existing buildings. In structures where strain levels are assessed as critical we recommend that samples of reinforcement be broken out of the structure and tested to check for possible adverse effects of strain ageing. The Royal Commission understands that some batches of reinforcement used in the past are considerably more sensitive to strain ageing than is the case with structural steel members and concrete reinforcement being used at present.

Strain ageing causes:

- an increase in the yield stress;
- an increase in the maximum stress (though in many cases this increase is small);
- a reduction in the ductility of the reinforcement; and
- a decrease in the transition temperature at which the reinforcement ceases to behave as a ductile material[1].

Momtahan et al. (2009)[2] found from tests on New Zealand-manufactured reinforcement obtained from Pacific Steel in 2008 that strain ageing increased with:

- the strain level in the reinforcement in their tests, which varied from two to 15 times the yield strain; and
- the time interval between when the strain was induced and when the reinforcement was retested to measure the effect of strain ageing, which ranged from three to 50 days.

At a strain level of 10 yield strains (about 1.6 per cent) in Grade 300 reinforcement after a period of 50 days the yield stress had increased by about 13 per cent. No significant increase in the ultimate stress or decrease in ductility was noted in these tests. However, different values may be expected for reinforcement used in earlier decades or obtained from overseas.

8.1.4 Assessment of strain levels induced in reinforcement

To assess the strain levels in reinforcement that had crossed cracks in concrete, Leeb hardness measurements (surface hardness) were made on reinforcement extracted from damaged buildings in Christchurch. Reinforcing bars were broken out of buildings and tested for their remaining strain capacity. Tests were conducted on some bars that had been strained into the inelastic range at cracks, and some bars that had not been strained into the yield range. The latter were tested to establish the original properties of the bars that had not been yielded.

Leeb hardness measurements are made by a machine that fires an impact body at the surface and records the details of its impact (Allington, 2011)[3]. This process is repeated along the bar at close centres, enabling variations in surface hardness to be measured, and variations are related to the strain level in the reinforcement. With this information it is possible to assess the length of bar that has been strained into the yield range, and the maximum strain levels sustained at different locations along the bar. One issue with these tests that needs clarification is the possible significance of the change in material properties of the reinforcement caused by previous strain ageing of yielded reinforcement.

Tests on a number of bars have indicated that high strain levels were sustained by some reinforcement that crossed cracks in concrete. This raised concern that the reinforcement might not have the capacity to sustain additional inelastic deformation in the event of a further significant earthquake. Critical conditions were identified in:

- walls with only one crack or widely spaced cracks apparent after an earthquake;
- the potential plastic hinge zones of beams where only a single crack appeared to have formed in potential plastic hinge zones close to the face of the columns; and
- wide cracks in some floors.

Single cracks in potential plastic hinge zones in beams were seen in a number of buildings that were assessed for the Royal Commission, including 90 Armagh Street, the Christchurch Central Police Station and 151 Worcester Street. Single or widely-spaced cracks were seen in the structural walls of the Gallery Apartments, the PGC and IRD buildings. Wide floor cracks were seen in many buildings and they were common in floors constructed with precast prestressed floor units such as hollow-core or double-tee units.

8.2 Performance of reinforced concrete buildings

8.2.1 Beams in ductile moment resisting frames

The formation of single cracks up to 5mm wide in potential plastic hinge zones in beams is of concern, as it appeared to indicate that high strains had been induced in the reinforcement, limiting the strain capacity available for further seismic resistance. The observed behaviour of the potential plastic hinges in the beams appeared to be different from that observed in laboratory tests of beams, where multiple cracks formed in a radial pattern in plastic hinge zones, and yielding extended along the member for a length of about one beam's depth. However, we suggest that a closer examination of the evidence might have lead to a different conclusion.

The mechanism of a plastic hinge zone is shown in Figure 137, which is based on a beam in a ductile frame as described by CCANZ (2008)[4] with a clear span of 6450mm between columns. The beam depth is 900mm and it supports a floor that spans nearly 11m. The floor consists of 300mm hollow-core units spaced 750mm apart with timber infill and a 75mm concrete topping reinforced with 12mm Grade 300 bars in both directions. The ratio of bending moment to shear force at the column face corresponds to a length of 1.78m, as shown in Figure 137(b).

Under seismic actions a primary crack forms in the beam close to the face of the column. This crack reduces the tensile stresses in the concrete in the hatched area shown in Figure 137(c) for a distance of L_1 along the beam. If the bending moment transferred across the primary crack is of sufficient magnitude a second primary crack may be initiated at a distance of between L_1 and $2L_1$ from the first primary crack. The location of these cracks is independent of the bond characteristics of the reinforcement. As shown, the spacing of these cracks is generally about 1.5 times the distance from the extreme tension fibre to the neutral axis. However, if the bending moment increases sufficiently an additional primary crack may form between the more widely spaced primary cracks.

Secondary cracks may form between the primary cracks if the reinforcement crossing the crack can transfer sufficient tension force to exceed the direct tensile strength of the concrete surrounding the reinforcement (see commentary to NZS 3101:2006[5], section 5 for the difference between the direct and flexural tensile strengths of concrete). This critical area of concrete is shown hatched in Figure 137(d). Secondary cracks are generally spaced at about three times the distance of the centroid of the bars from the extreme tension fibre in the beam. However, stirrups tend to act as crack initiators so the spacing of the cracks is often equal to that of the stirrups.

In the beam shown in Figure 137 the stirrup spacing is 150mm, which indicates that a secondary crack may be expected to form 150mm from the primary crack. If a secondary crack forms during an earthquake it will afterwards close to a small width unless the reinforcement at this crack has yielded, in which case it will remain open. If the reinforcement does not yield, the crack will be difficult to see unless the concrete surface is examined closely with a magnifying glass.

In Figure 137 the secondary crack is located at a distance of 150mm from the primary crack. The bending moment at this secondary crack is 91.5 per cent of the bending moment at the primary crack. The tension force at the primary crack must increase by at least 1/0.915, or 9.3 per cent, to cause the reinforcement to yield at the secondary crack. Note that diagonal cracks and the associated tension lag in the reinforcement generally develop after extensive flexural cracks have formed.

Figure 137(a) shows the measured stress-strain response of a deformed 24mm Grade 300 bar (Matthews, 2004)[6]. The ratio of ultimate stress to yield stress in this bar was close to the maximum permissible value of 1.5 for Grade 300 reinforcement in the Steel Reinforcing Materials Standard, AS/NZS 4671:2001[7]. The corresponding minimum ratio in the Standard is 1:15. In Figure 137(a) the stress-strain relationship in the strain hardening range has been scaled to correspond to the minimum ratio of peak stress to yield stress. While the test carried out by Matthews (2004)[6] shows that strain hardening is initiated at a strain of close to 1.25 per cent, other tests (Allington, 2011)[3] show initiation at strain levels above 2.5 per cent.

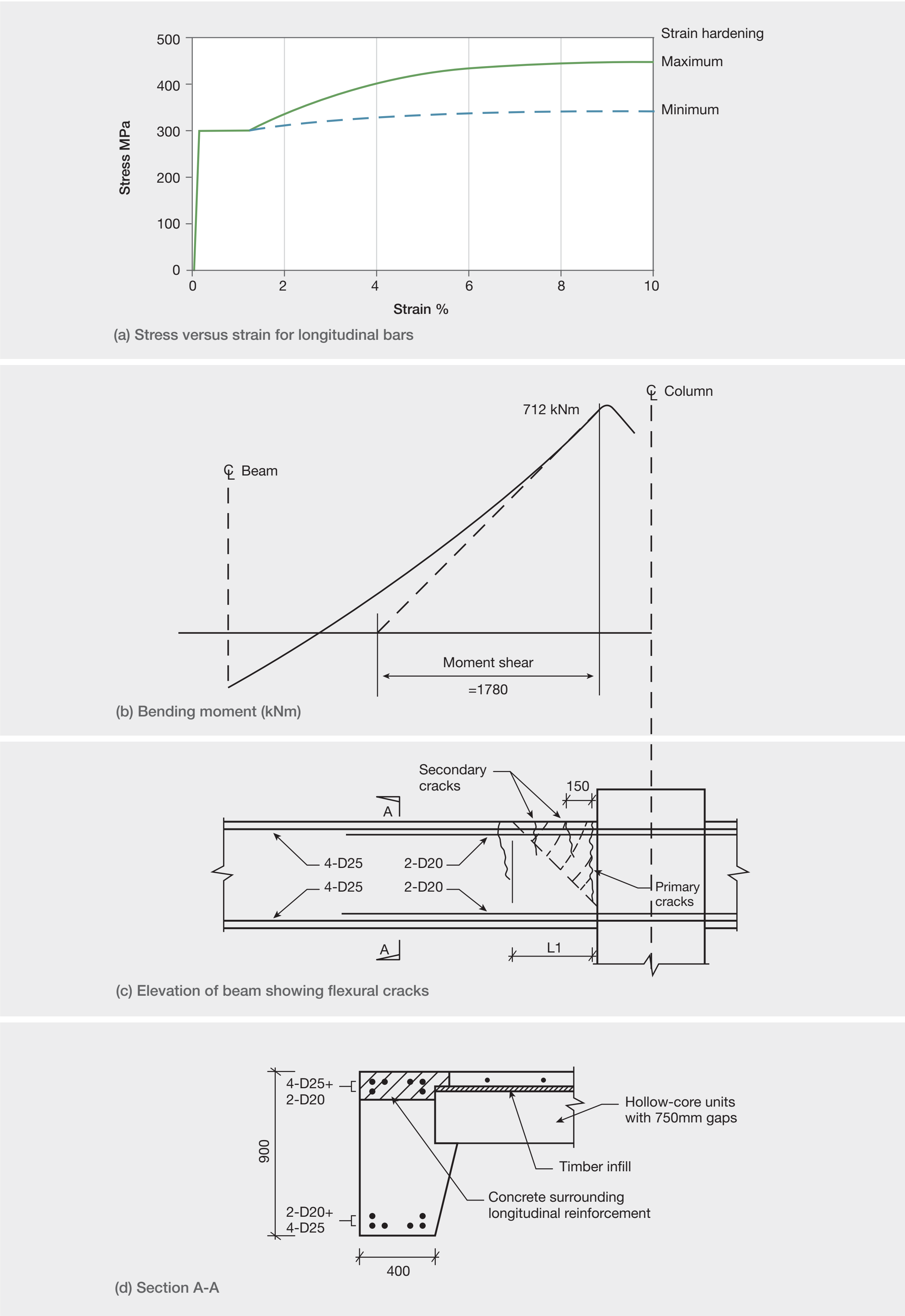
From the stress-strain relationships shown in Figure 1(a) it can be seen that to reach a strain-hardening level of 9.3 per cent requires strain levels of 1.8–4.0 per cent. Now assume that the reinforcement yields over a distance of around eight bar diameters and that the strain varies linearly between the crack and locations where the yield strain is reached. Then the width of the primary crack will be in the range of 1.8–5.2mm, which gives material strain levels of 15–33 per cent of the maximum design values given in NZS 3101:2006[5].

Once diagonal cracks form in the plastic hinge, the length over which the reinforcement yields is typically extended by 40 per cent of the beam depth, owing to tension lag. Generally, however, relatively high strains are induced in the reinforcement before this stage is reached, as discussed above.

Our conclusion is that numerous observations of plastic hinge zones with only one crack several millimetres wide do not contradict test results obtained from beam-column sub-assemblies. Relatively high strains need to be sustained at the critical section of a plastic hinge before yielding can spread along the beam. This process may be assisted for actions associated with aftershocks by moderate strain ageing of the highly strained reinforcement near the face of the column, or another critical section in a plastic hinge. The high localised strain in the longitudinal reinforcement close to the column face does not necessarily indicate a significant decrease in seismic capacity of the plastic hinge zone. Once strain hardening has taken place, yielding of the reinforcement extends along the beam. Then, as plastic hinge deformation further increases, the rate of increase of strain diminishes in the reinforcement close to the critical section.

One aspect of the behaviour of plastic hinges in beam and beam-column sub-assemblies requires further investigation. Generally in tests the loading sequence involves applying displacement cycles of gradually increasing magnitude until failure occurs. Each time the reinforcement yields and a crack opens up, the reinforcing bars are displaced relative to the concrete to enable them to span the crack. This movement reduces the bond resistance. Consequently, the distance along which yielding penetrates into the beam-column joint zone increases with each inelastic load cycle. In the February earthquake the major displacement cycles took place without the multiple small cycles that have been applied in structural tests. Consequently, the yield penetration of the bars into the beam-column joint zones of buildings in Christchurch is likely to have been appreciably less resulting in higher peak strains than was the case in laboratory tests.

Research is needed into the influence of different loading sequences on yield penetration of reinforcement into beam-column joints.



(a) Stress versus strain for longitudinal bars

(b) Bending moment (kNm)

(c) Elevation of beam showing flexural cracks

(d) Section A-A

Figure 137: Crack formation in a beam

8.2.2 Structural walls

8.2.2.1 Crack control in lightly reinforced structural walls

In the Gallery Apartments building a single relatively fine crack was seen in one of the walls. On further investigation it was found that the reinforcement crossing this crack had failed in tension. This gave rise to concern that the strains arising in reinforcement crossing other cracks might be much higher than was anticipated from laboratory tests.

It is likely that the flexural crack in the wall was a primary crack induced by seismic actions exceeding the flexural cracking moment of the wall. As the wall was several metres in length any further primary crack would be several metres away from the first one. However, such a crack could only form if the bending moment that could be sustained at the section containing the first primary crack was of sufficient magnitude to enable the bending moment induced at the location of the second primary crack to exceed the flexural cracking moment. As indicated in the next paragraph, this is unlikely to have happened in the walls of the Gallery Apartments building (see section 6.5.1.1 of this Volume), because of the low proportions of longitudinal reinforcement in the walls.

Inspection of the drawings indicates that typically the proportion of longitudinal reinforcement in the walls was 0.17–0.22 per cent, which satisfied the minimum of 0.14 per cent for Grade 500 reinforcement in NZS 3101:1995[8]. With 0.22 per cent longitudinal reinforcement stressed to 1.15 x 500MPa (where the factor 1.15 allows for strain hardening), the average tensile stress induced in the concrete is equal to 1.25MPa. As noted in our discussion of the Gallery Apartments building, the concrete compression strength in the walls was measured on two cores taken from the walls and assessed from a number of Schmidt Hammer tests. The assessed strength was of the order of 50MPa. Two further cores were taken from the wall and the direct tensile strength was assessed from split cylinder tests as 2.4 and 3.4MPa. The calculated average tensile strength given in NZS 3101:2006[5] (see commentary to section 5 of this Volume) is 4.1MPa, with upper and lower characteristic strengths of 2.8 and 5.4MPa. The clear indication from the assessed and measured tensile strengths of the concrete is that there was insufficient longitudinal reinforcement to initiate secondary cracks. Consequently yielding of reinforcement was confined to the immediate vicinity of the single crack, which induced high tensile strains in the reinforcement and may account for the observed failure of the reinforcement at the crack.

The minimum longitudinal reinforcement proportion given in the present NZS 3101:2006[5] is defined by the expression $0.25\frac{\sqrt{f'_c}}{f_y}$, which allowing for strain hardening gives an average tensile stress in 50MPa concrete of close to 2MPa. The minimum reinforcement content recommended in SESOC (2012)[9] is

$$0.4\frac{\sqrt{f'_c}}{f_y}$$

for which the corresponding average tensile stress is 3.2MPa. Cracking would probably develop at tensile stress levels below the average tensile strength, owing to stress concentrations close to the bars and eccentric actions in the wall. However, it is clear that current minimum design specifications are inadequate to ensure that cracking will spread over a number of secondary cracks, allowing ductile behaviour to develop. Research into crack control in walls is highly desirable. Increasing the minimum longitudinal (vertical) reinforcement has the disadvantage of increasing the strength of the walls and hence increasing the cost of the foundations.

There are two potential approaches that can be used to improve crack control, both of which have advantages and disadvantages:

1. A proportion of the longitudinal (vertical) reinforcement may be concentrated at the ends of the wall. This would ensure sufficient reinforcement in these zones to initiate secondary cracks in the immediate locality. A lower proportion of reinforcement is required between the zones of concentrated reinforcement, to ensure that the secondary cracks formed in the end zones can spread over the remainder of the wall. A further advantage of this reinforcement arrangement is increased strength for the serviceability limit state, as the strength increases more rapidly with displacement than is the case when reinforcement is uniformly distributed. However, concentrated reinforcement in the compression zone in walls with low axial load ratios $({}^{N}\!/_{A_g f'_c})$ increases the potential for elongation of the wall, which may reduce the lateral resistance to sliding shear. Concentrated reinforcement may also increase interaction with other structural elements due to increased elongation of the walls.

2. At the critical section of a potential plastic hinge longitudinal reinforcement may be de-bonded for a length by wrapping the bars in a grease-impregnated tape. This will ensure that the bar yields over the de-bonded length and that a single wide crack can form under critical loading conditions. This has implications for shear transfer across the crack and may cause more of the shear force to be resisted by the compression zone of the wall, and loss of torsional resistance where the wall is part of a shear core. This was one of the potential failure mechanisms identified for the PGC building.

8.2.2.2 Shear core walls

In some buildings, such as the PGC and Bradley Nuttall House buildings, walls surrounding stairwells, lifts and sometimes toilets formed a rectangular core of walls (shear core) acting as a unit to provide lateral force resistance to the building. In the PGC building this core was subjected to both torsion and flexure. Bending action may have induced a primary tension crack that extended along the wall. An open crack of a few millimetres may then have resulted in a loss of shear transfer by aggregate interlock action, leading to a redistribution of torsional actions that initiated failure. The longitudinal reinforcement in the walls had a cover of 92mm, which created good bond conditions, so it is likely that in the PGC building the opening up of the crack led to tensile failure of the bars. The same outcome could have been achieved by a single wide crack forming where reinforcement was de-bonded. Consequently there are situations where de-bonding reinforcement might not achieve the desired outcome.

If sufficient shear stress caused by torsional moments acting on the shear core can be transferred across tension cracks by aggregate interlock, diagonal cracking may occur. In this event the shear stresses are resisted by diagonal compression forces in the concrete acting together with tension forces in both the horizontal and vertical wall reinforcement. However, when the flexural bending moment and its associated tension force increase, there is a decrease in the tension available to resist the longitudinal component of the tension force associated with the torsional moment. As a result the torsional resistance decreases as the flexural bending moment increases, and when the longitudinal reinforcement yields because of the flexural moment, torsional resistance is minimal. In assessing the potential seismic performance it is important to understand this interaction. The commentary to NZS 3101:2006[5] discusses it but the equations for the interaction of flexure and torsion are not included in the Standard.

We recommend that interaction equations for flexure and torsion be added to NZS 3101:2006[5] and that the significance of wide cracks in members as an influence on shear and torsion be identified in documents used to design or assess the potential seismic performance of buildings.

8.2.2.3 Walls under high axial loads

In the assessment of the HGC it was found that wall D5–6 failed, possibly by buckling. We noted that the criteria for buckling of a compression zone of a wall in NZS 3101:2006[5] were based on the assumption that the compression zone had been subjected to extensive yielding in tension during a previous half-cycle of loading. Prior tensile yielding of reinforcement in a compression zone reduces the buckling stability of the compression zone and hence the buckling stability of the wall. Consequently the stability criterion for walls subjected to low axial load ratios $\left({}^{N}\!/_{A_g f'_c} \right)$ is well founded. However, buckling in walls subjected to moderate and high axial load ratios is not covered, as in these walls extensive tensile yielding of reinforcement may not occur.

We recommend that suitable equations be developed to define the minimum slenderness ratios for these walls, with allowance made for the axial load ratio and the lateral displacement imposed on a wall. Until this work has been carried out it is recommended that in a ductile detailing length where the axial load ratio exceeds 0.10 the ratio of clear height to thickness should be equal to or less than the smaller of the ratios given by current design criteria in NZS 3101:2006[5], or 10.

A number of unexpected failures occurred in structural walls, apparently caused by a combination of axial load, bi-axial bending moments and shear forces leading to buckling of reinforcement. This buckling was probably associated with compression being imposed on reinforcement after it had been subjected to tensile yielding in a previous half-cycle of loading.

It is noted that the axial force acting on a structural wall is difficult to determine with any degree of accuracy. When bending moments act on a wall, elongation occurs. The relative vertical movement is partially restrained by floors and other vertical structural members. In some cases, this restraint can result in additional high axial compression forces being imposed on a wall. In a large-scale structural test this action has been observed to increase very significantly the lateral load resistance of the wall[10].

In other situations this restraining action might reduce the ductility of the wall or change its load-deflection characteristics. It should be noted that current standard methods of analysis do not predict elongation, so axial loads determined by these methods can be significantly in error in terms of the critical axial forces acting on walls.

In design it has been standard practice to determine where compression zones form in walls subjected to their maximum design bending and axial loading. Confinement reinforcement in the compression zones is provided for this condition. Between these zones no confinement or restraint against bar buckling has been required. However, in a half-cycle of loading, high tensile strains may be induced in the vertical reinforcement in the mid-region between the confined regions of the wall. When the bending moment decreases and starts to reverse in direction, the reinforcement in the mid-region is subjected to compression by the axial load. As the crack is initially still open the vertical reinforcement has to yield back in compression before the crack can close to enable the concrete to act. Under these conditions the buckling resistance of vertical reinforcement is reduced.

In a number of cases, including the Westpac Tower building, crushing of concrete and buckling of the reinforcement occurred in walls outside the confined end zones. This problem is identified in the draft proposals[9], and we support the suggestion made in the document in regard to the ductile detailing length, that:

- the full length of the compression zones associated with the ultimate limit state be confined, rather than the limited portion of the compression specified in NZS 3101:2006[5]; and
- anti-buckling ties be added to restrain all the longitudinal (vertical) reinforcement in the wall between the confined zones.

8.2.2.4 Coupled structural walls

Since the mid-1970s coupled shear walls have been proportioned so that yielding is confined to the base of the walls and to the coupling beams. The over-strength actions in coupled walls, and in particular the axial forces induced in the individual walls, are calculated from the over-strengths of the coupling beams. However, one aspect of behaviour has been ignored in this process: the flexural and shear capacity of the coupling beams increases when axial compression is imposed on these members. With the formation of plastic hinges in the coupling beams, elongation occurs and pushes the walls apart. However, the walls are almost invariably tied into the floors, which will partially restrain this movement so that coupling beams are compressed and floors tensioned. This action may result in either a significant increase in the strength of the coupled shear wall or the development of a wide crack and failure of the reinforcement in the floors.

The IRD building (section 6.5.3) provides an example of where the interaction of a coupled shear wall with the floors is likely to have increased the strength of the walls. That increase may also have increased the forces acting on the foundations. If so, it may have contributed to the differential settlement of the piles under the shear core relative to the piles under the perimeter walls.

The potential influence of floor slabs on coupled shear walls needs to be identified and the significance of this action assessed in a research project.

8.2.3 Floors as diaphragms

8.2.3.1 Design actions for ground acceleration

The current Earthquake Actions Standard NZS 1170.5:2004[11] does not give a clear method of determining diaphragm forces in the floors of multi-storey buildings, but does provide equations to determine design accelerations of floors at different levels. These could be used to estimate the maximum inertial force acting on a floor, although they do not appear to allow for the likely increase in floor accelerations in parts of floors where significant torsional displacements occur. The commentary to NZS 1170.5[11] indicates that these equations can be applied to a wide range of structural and non-structural items attached to floors or other structural elements. However, nowhere does it suggest that these equations can be used to calculate the total forces due to inertia forces acting on the floors as a whole.

Floors acting as diaphragms are required to:

- transfer forces between lateral-force-resisting elements;
- resist self-strain forces such as those that arise from in-plane deformation of walls that have different strengths for lateral displacement in the forward and backward directions (see section 8.4); and
- transfer inertial forces caused by gravity loads on the floor to the lateral-force-resisting elements.

Drag bars are required in many buildings to tie diaphragms into lateral-force-resisting elements. This particular aspect of design appears to have been inadequately considered. Given the lack of treatment of this problem in the design Standards, this omission is not surprising.

The need to tie floor slabs into individual lateral-force-resisting elements depends on the position of the elements used to resist lateral force. For a shear core surrounded by floors the situation is generally not critical as the force can be resisted by lateral pressure between the floor slab and one or more of the walls of the shear core (as in the IRD building), or the indirect route of shear transfer between the walls and the floor may provide continuity reinforcement between the floor and the wall (as in the Bradley Nuttall building). The situation is more critical when lateral-force-resisting elements are on the perimeter of the building. In this situation beams need to be tied into walls and columns so the forces can be transferred, or drag bars may be required to pick up the necessary forces from the floor slabs.

All columns and walls need to be adequately tied into floors to provide restraint against buckling and ensure that they do not separate from the floor slab.

From observations of our representative sample of buildings we consider that structural engineers need additional guidance on how to assess the magnitude of membrane forces and design for membrane actions in floors.

8.2.3.2 Elongation of reinforced concrete beams

The Royal Commission examined a number of cases where elongation of reinforced concrete beams caused wide cracks to form in the floors. In one case (Clarendon Tower) the cracks were so wide that the floors were in danger of collapse through the precast floor units being pulled off their support ledges. Elsewhere cracks in the floors were wide enough to cause the mesh reinforcement to fail in tension (Craigs Investment building, Clarendon Tower, 151 Worcester Street) and again, in the case of the Clarendon some of the columns were separated from the floors. The development of wide cracks can reduce a floor's ability to transmit diaphragm forces to walls and columns on its perimeter, and can result in failure of the reinforcement tying these elements to the floor.

The cast-in-situ concrete floors that we examined had behaved well. With these, elongation of beams generated a number of fine cracks that are not a concern in terms of seismic performance. Concrete slabs cast on metal trays formed from metal sheeting (Traydec, Hibond) appear to have performed well where they were used with steel beams. Their performance with reinforced concrete beams is likely to depend on the type of reinforcement used in the in situ concrete topping. The use of mesh reinforcement can result in the floors sustaining a brittle failure mode, so this is not suitable for resisting membrane forces, particularly where elongation induces wide cracks in the floor.

The situation where floors are built up using precast prestressed concrete units (double-tee and hollow-core) with in situ concrete topping differs from that where the floors are fully cast-in-situ. In the case of the precast units the prestressing prevents or restrains crack widths from opening in a direction normal to the span of the unit. As a result nearly all of the elongation in the plastic hinge closest to the support position of the precast units opens up a single crack between their ends and the structural element supporting them. Under sufficient elongation the reinforcement crossing the crack may fail in tension, particularly where non-ductile mesh has been used. The loss of support length for the precast units, caused by beam elongation and spalling of the concrete (behind the precast unit and from the front face of the support ledge) can endanger the stability of the floor supports unless an adequate ledge length has been provided to allow for these actions[12].

8.2.3.3 Support of precast floor units

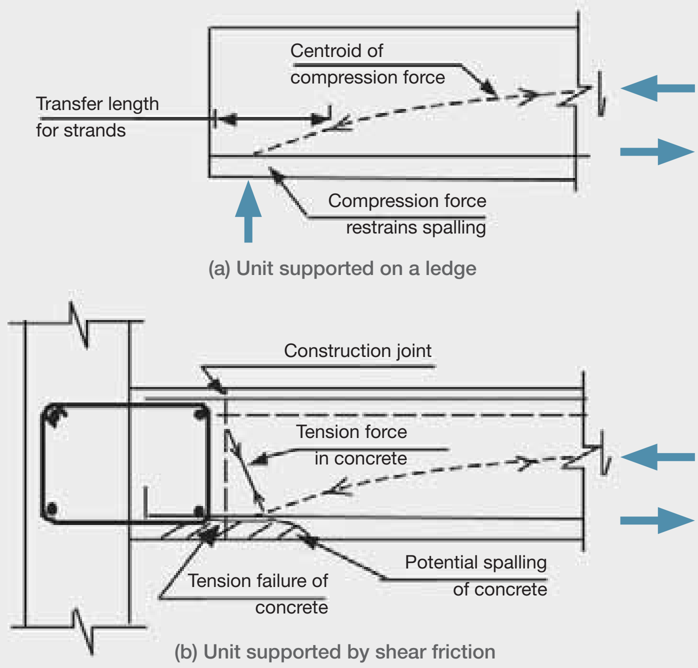


Figure 138: Support of precast floor units.

The usual arrangement is to support precast floor units on a ledge. With this arrangement the reaction from the support balances the transfer force from the pretension strands and the inclined compression force in the unit, as shown in Figure 138(a). The spalling of concrete below the strands in the transfer length is prevented by the compressive reaction from the supporting ledge.

In two structures (the IRD and Gallery Apartments buildings) some units were supported by shear friction between the back face of the unit and the face of the supporting structural element. One detail is shown in Figure 138(b). In this case there was no compression force from the support to suppress tension stresses in the transfer length, and consequently the spalling resistance was diminished in the concrete below the pretension strands.

A potential problem arises if sway of the structure causes relative rotation to develop between the precast unit and the supporting element. This rotation generates a prising action and any reinforcement near the bottom of the unit is likely to be either subjected to high yield strains or pulled out of the supporting element or precast unit. The crack width at the end face of the precast unit increases with the magnitude of rotation, and with this the capacity for shear transfer by aggregate interlock action decreases sharply.

This decrease may be even more pronounced if the direction of sway reverses several times, as this increases the elongation caused by yielding of reinforcement on both sides of the member. With a wide crack, of a millimetre or two, aggregate interlock action is negligible and only dowel action remains. Dowel action in bars is generally limited by the tensile strength of the concrete at the level of the bars, and failure can be brittle. If the precast unit is mounted directly against the face of a wall, tension failure caused by dowel action is suppressed. True dowel action is limited, especially if the bars are simultaneously subjected to high axial tensile strains. In this situation, kinking of the bar to about 30° can occur and potentially prevent complete failure. However, this mechanism is associated with a vertical displacement in the order of the bar diameter[13]. Relying on this action is not recommended, as a few cycles of loading may result in a low-cycle fatigue failure.

8.2.3.4 Punching shear failure

In the Hotel Grand Chancellor (HGC) and Grant Thornton building at 47 Cathedral Square, punching shear failures were seen in floor slabs (Figure 139). The HGC punching shear may have been due to the shock loading associated with the collapse of the wall D5–6, or alternatively due to high vertical accelerations associated with vertical ground motion. Another possible cause was the bending moments transferred to the slab by column deformation associated with inter-storey drift.

To ensure safety of slabs against punching shear failure, the design of flat slabs should follow a capacity design approach to ensure they can resist the maximum bending moment that can be transferred to them by the columns. Punching shear failures due to these actions are brittle in character.

Figure 139: Punching shear failure: Grant Thornton building (source: Holmes Consulting Group Ltd)

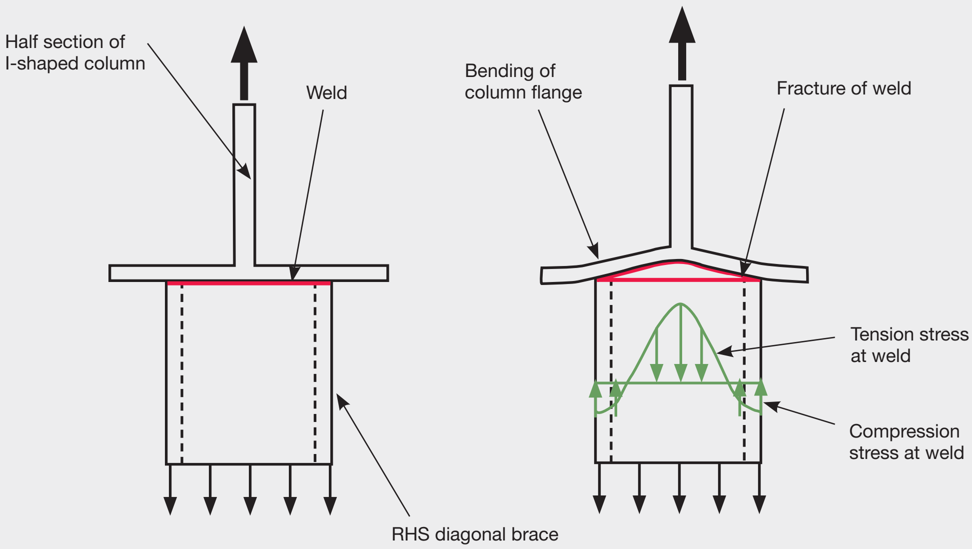
8.3 Performance of structural steel buildings

The damage to low-rise structural steel industrial buildings was relatively minor[14]. Damage tended to be limited to bracing elements, which needed either to be replaced or re-tightened because they had yielded. In some cases the connection details between the bracing elements and the main structure needed repair.

Load tracking is important when designing structural steel and concrete structures alike. The example in Figure 140[14] shows a case where load tracking was not used.

In an earthquake the lateral displacement of the building concerned caused the rectangular hollow section (RHS) brace, shown in Figure 140(a), to be loaded in tension or compression, with the forces being transferred into the column. When the brace was in tension, the transfer of forces through the weld caused the column flange to bend. The incompatibility of the flange displacement relative to the end of the RHS brace resulted in the stress distribution, shown in Figure 140(b). Concentrations of high-tension stress caused the weld to fracture. If a stiffener plate had been welded in the column it would have reduced flange bending and suppressed the weld failure.

(a) Failed brace (source: Clifton et al., 2011[14])



(b) Failure mechanism due to poor detailing

Figure 140: Failure of brace welded to a column (source: Clifton et al., 2011[14])

Figure 141 shows one of the two link fractures that occurred in a concrete parking building with eccentrically braced steel frames acting as the lateral load resisting elements. This structure had at least six eccentrically braced frames in each of its principal directions at each level. This significant redundancy gave a satisfactory seismic performance despite the fracture of two links. Clifton et al.,[14] noted that the fractures might not have been discovered had they been hidden by non-structural finishes. The failed link in the Pacific Tower is an example of this happening, as it was not discovered in the initial inspections.

Figure 141: Example of a fractured EBF near the active link: car park building on the corner of St Asaph and Antigua Streets (source: Clifton et al., 2011[13])

A high standard of workmanship is especially important in structural elements containing highly stressed components. The active links are designed for high inelastic demands, so poor construction quality can lead to compromised load paths or material defects. The failure in Figure 141 was attributed to the offset of the diagonal brace flange from the stiffener plate. This offset meant that when the brace was loaded in tension, the axial tension load in the brace was fed into the web adjacent to the active link rather than directly into the stiffener. This led to the failure of the beam web outside the zone of the active link.

We note that local stress concentrations may be induced by inclusions or gaps in welding, or by localised spot-welding. These stress concentrations can act as fracture-initiation points when the steel is subjected to cyclic inelastic actions. An example of this may occur where shear studs have been welded to the top flanges of beams to obtain composite action between the beam and floor slab. If the studs are welded directly above the active link region, this may act as an initiation point for premature failure. Long welds generally heat the full steel section and cool more uniformly, so the stress is less concentrated.

Multi-storey steel buildings in the CBD generally performed well, with some fit for reoccupation after repairs. Christchurch has relatively few structural steel buildings and most of these have been designed and built during the last 20 years. Consequently they have been designed to recent Standards.

The Pacific Tower has been discussed in section 6.5.4 of this Volume. The fracturing of the active link in eccentrically braced frames (EBF) has also been observed in another building, as described above. Clifton et al.,[14] note that these fractures are a particular concern as they are the first of their type to be recorded in EBFs worldwide.

Our major concern is with the behaviour of the active links and the lack of redundancy seen in some buildings that rely on EBFs for their seismic resistance. It appears possible to design such buildings to comply with relevant standards while still lacking redundancy. To prevent this, consideration should be given to amending NZS 3404:1997[15] to require some measure of redundancy to be designed into these buildings. This might be achieved by requiring the columns to have sufficient strength and stiffness so that they contribute to an alternative load path if a single active link fails in an EBF.

8.4 Application of basic concepts in seismic design

8.4.1 Ratcheting

Our investigation into the seismic performance of buildings in Christchurch has indicated that a number of designers have overlooked some fundamental concepts of structural design. Response spectra provide a basis for much of current practice in seismic design. However, response spectra are based on the assumption that the strength and stiffness of single degree of freedom oscillators are equal for displacements both forwards and backwards. In the HGC building this condition was not satisfied because the eastern-most bay was cantilevered off the remainder of the building, causing the structure to displace preferentially towards the east in the February earthquake. This situation, known as ratcheting, could have been simply avoided by redistributing the flexural reinforcement in the beams to equalise the strengths for both forward and backward displacements (see the discussion of the HGC building, section 3 of this Volume).

Ratcheting can also occur in structures that contain walls or columns that have different strengths forwards and backwards. A typical example of this occurs when T-shaped walls are built into buildings, as shown in Figure 142. Generally there will be less longitudinal (vertical) reinforcement in the flange than in the leg, meaning that when lateral seismic forces act on the wall it is weaker when the leg goes into tension and the flange goes into compression, than when the reverse is the case. In the example shown in Figure 142 the walls would tend to move apart, inducing tension in the region between the walls. If both walls were turned around so that the flanges were close to each other, they would tend to move together in an earthquake, potentially crushing or tending to crush the structure between them. This situation was seen by the NZSEE/EQC teams observing damage from the 27 February 2010 earthquake in Chile[16].

Ratcheting can also occur in cantilevers and other transfer structures where gravity loads act in conjunction with vertical forces induced by vertical ground motion. Where this situation can arise, designers should ensure that under the combined gravity and seismic actions either the transfer structure remains elastic or the bending moments shake down into a stable configuration.

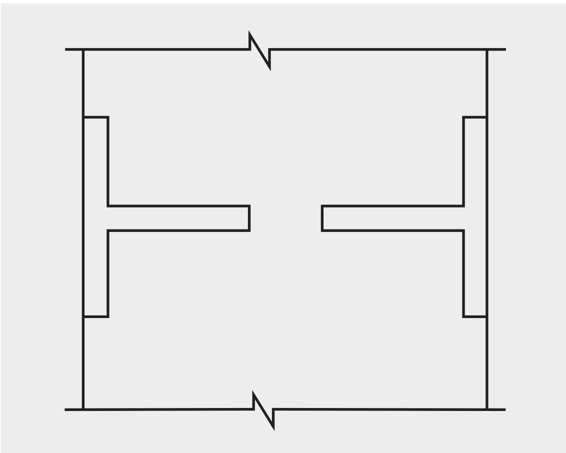


Figure 142: Two T-shaped walls in a building

8.4.2 The 'what-if' approach

Analyses for earthquake actions are invariably based on assumptions that cannot be validated before the analysis has been completed. The 'what-if' approach requires the designer to assess, review and check for significant potential sources of error. Two examples are given here of the failure to follow this approach.

With the Gallery Apartments, in the analytical model used to assess design actions the flexural stiffness of the walls was taken as 0.25 times the properties based on the gross sections as recommended in the Concrete Structures Standard, NZS 3101:1995[8]. On this basis the fundamental periods of the building in the north–south and east–west directions were assessed as 3.4 and 3.9 seconds respectively. However, an inspection of the magnitudes of bending moments induced and the level of reinforcement required to resist these actions would have indicated that cracking would be limited to near the base of the wall. Furthermore, given the low reinforcement content, the 'what-if' approach would have shown that secondary cracking could not form; yielding would therefore be confined to one crack, meaning that ductile performance of the building could not be achieved. If this approach had been followed it would have revealed that the building should be re-analysed with increased wall stiffness values and designed (if the distribution of cracking was still limited) as a nominally ductile structure.

The PGC building was analysed at least twice using the inelastic time history method. The details used on the first occasion are not known. However, in the analysis for DBH, the shear core wall section properties were taken as 0.4 times the values based on gross section properties (the current Standard recommends 0.25). The origin of the 0.4 value is not known. Multiplying gross section properties by a factor of less than one is a common practice to allow for stiffness reduction caused by flexural cracking.

However, again if the predicted bending moments and axial forces had been assessed following the 'what-if' approach, it would have been evident that few cracks would have been expected to form and consequently the reduction in stiffness of the walls assumed in the analysis was not appropriate. It should have been clear that secondary cracks could not be expected to form and that virtually all the inelastic deformation would be concentrated at one crack, which could have indicated a potential brittle failure location.

8.4.3 Compatibility

In a number of buildings it was clear that no consideration had been given to compatibility of displacements of different structural elements. Floors were attached to walls that deflected independently of one another under lateral forces, applying incompatible displacements to the floors or beams connecting the walls (Gallery Apartments, Bedford Row Car park building) so that the differential deflections damaged the floors.

8.4.4 Flexural torsional interaction

Flexural torsional interaction was not considered in the analysis of the PGC building and it is not dealt with in the Concrete Structures Standard[5], although it is discussed in the commentary to the Standard.

We recommended that torsional flexural interaction be introduced into the Concrete Structures Standard.

8.4.5 Irregularity in buildings

From the post-1960 buildings we have assessed, it is clear that the performance of the buildings in the Christchurch earthquakes was strongly influenced by their degree of irregularity and the magnitude of the eccentricity of the centre of mass from the centre of lateral stiffness and strength of each building. The latter factor is referred to as the ‘eccentricity’ of a given building in this Report.

The Christchurch Central Police Station and the CCC Civic Offices buildings are both regular structures with low eccentricity and both performed well in terms of the objectives of the design standards at the time when they were designed. The Forsyth Barr building was also regular, had a relatively low eccentricity and performed well except for the stairs. At the other extreme the PGC, HGC, Gallery Apartments and 151 Worcester Street buildings all were highly irregular with high eccentricities and all performed poorly. The Bradley Nuttall House building also had a high eccentricity and its performance was marginal.

In terms of regularity and eccentricity, the IRD building lay between the two groups described above and performed well except for the differential settlement of its foundations.

The major exception to this trend was the Clarendon Tower. This structure was relatively regular and the eccentricity was low. However, in this case the high level of elongation associated with the structural details used in the northern and southern external moment resisting frames resulted in a rapid stiffness degradation of the northern frame. This resulted in the building developing high eccentricity in the February and subsequent earthquakes (see section 6.3.5 in this Volume). The loss of stiffness would have had less effect on the torsional response of the building if the perimeter frames in the eastern and western walls had been of similar lateral stiffness to those in the northern and southern sides.

We recommend that the current method of allowing for irregularity and eccentricity in building design should be revised to allow more realistically for the adverse effects of these two factors.

8.4.6 Vertical seismic ground motion

Further research is required into the potential effects of vertical ground motion on buildings. A ratcheting action is possible in beams and slabs, caused by gravity load acting simultaneously with the vertical excitation. However, the high frequency of the vertical motion may limit this interaction. More research is required to determine possible adverse effects of vertical ground motion and to establish where high vertical ground motion is likely to occur.

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Section 9: Conclusions and recommendations

In this section we recommend that a number of changes be made in the design of buildings for earthquake resistance. These recommendations include changes in the way that seismic design is undertaken and changes to structural Standards. In many cases additional research is necessary to identify specific values that are appropriate for design codes and Standards.

9.1 Recommendations related to the Earthquake Actions Standard, NZS 1170.5[1]

9.1.1 The current values for the response spectral shape factor, C(T), for deep alluvial soils found under Christchurch appear to overestimate horizontal accelerations in the short period range and underestimate accelerations in the range of 2.0–4.0 seconds when compared with the derived spectra for the Christchurch earthquakes.

Recommendation

We recommend that:

32. The response spectral shape factor, *C(T)*, for deep alluvial soils under Christchurch, should be revised. The likely change in spectral shape with earthquakes on more distant faults also needs to be considered.

9.1.2 The current spectral values for vertical ground motion are too high in the long period range and may be too low in the short period range for structures located close to some faults.

Recommendations

We recommend that:

33. The shape of response spectra for vertical ground motion should be revised.

34. The implications of vertical ground motion for seismic design actions should be considered and locations identified where high vertical accelerations may be expected in earthquakes.

9.1.3 Regularity of structures in both plan and elevation and eccentricities between the centres of mass and the centres of lateral stiffness and strength have been shown to have a major influence on seismic performance.

Recommendation

We recommend that:

35. The requirements for regularity in buildings, and for torsion due to the distance between the centre of mass and the centres of stiffness and strength, should be revised to recognise the implications of these parameters on observed behaviour.

Recommendation

We recommend that:

36. Design actions for floors acting as diaphragms need to be more clearly identified in the Standard. This includes actions that arise from:
    - the weight of the floor and its associated gravity loading and the acceleration of the floor;
    - shear transfer between the lateral-force-resisting elements;
    - self-strain forces induced by elongation and bending of beams; and
    - local forces induced by structural elements such as T-shaped walls that have differing strengths for displacement in the forward and backward directions.

9.1.4 The magnitude weighting factor recognises the influence of duration of shaking on the damage potential of earthquakes (see Seismicity, Volume 1, section 2).

Recommendation

We recommend that:

37. A more rational theoretical basis should be developed for magnitude weighting, which is used in the development of the design response spectra for structures.

9.1.5 There is an inadequate understanding of;

- the difference between design inter-storey, and peak inter-storey drifts; and
- the influence of ductile behaviour on the shape profile of a multi-storey building. This adjustment is made with the 'drift modification factor' in the Standard.

Recommendations

We recommend that:

38. Explanation should be added to the commentary to the Standard to explain:
    - the difference between design inter-storey, and peak inter-storey drifts; and
    - the influence of ductile behaviour on the shape profile of a multi-storey building.
39. The Standard should be amended to require that the supports of stairs and access ramps be designed to be capable of sustaining 1.5 times the peak inter-storey drift associated with the ultimate limit state, together with an appropriate allowance for construction tolerance and any potential elongation effects.

Attention is also drawn to section 9.6 of this Volume, where we discuss the design of means of egress from buildings.

9.2 Recommendations related to the Concrete Structures Standard, NZS 3101:2006[2]

9.2.1 Literature research is required into the influence of the rate of loading on seismic performance of reinforced concrete structures. This topic has been examined in the reports on a number of projects with varying conclusions. A number of papers have indicated that the influence of loading rates associated with earthquakes has little significant influence on behaviour, while others report that loading speeds consistent with earthquakes can reduce ductility.

We suspect that ductility is reduced in lightly reinforced members but not in members with moderate or high reinforcement content.

Recommendation

We recommend that:

40. A comprehensive study of the existing literature on the influence of the rate of loading on seismic performance of reinforced concrete structures should be undertaken to address the inconsistencies in the published opinions, and to make appropriate recommendations for design.

9.2.2 In many structural tests the loading sequence has involved use of gradually increasing cycles of displacement. This may have led to an overestimate of the yield penetration compared with that sustained in an earthquake where the major displacement occurs near the start of the shaking. This overestimate of yield penetration may have resulted in overestimates of available ductility of lightly reinforced and walls and beam-column subassemblies.

Recommendation

We recommend that:

41. Research into the influence of the sequence of loading cycles on yield penetration of reinforcement into beam-column joints and the development zones of reinforcement is desirable.

9.2.3 The reinforcement content and arrangement in a number of structural walls has been shown to be inadequate to ensure that yielding of reinforcement can extend beyond the immediate vicinity of a single primary crack. Improving ductility may be achieved by:

- the use of higher minimum reinforcement contents;
- changes in the distribution of reinforcement in the wall; and
- de-bonding bars in critical zones. Where the de-bonding option is used the potential negative implications of this action on shear and torsional behaviour in T-shaped walls and in walls forming a shear core in a building should be identified.

Recommendation

We recommend that:

42. Changes should be made to the Standard to ensure that yielding of reinforcement can extend beyond the immediate vicinity of a single primary crack, and that further research should be carried out to refine design requirements related to crack control in structural walls.

9.2.4 A number of structural walls did not perform in the earthquakes as well as anticipated. There are a number of possible reasons for this:

- the walls sustained greater axial forces than were anticipated in the design owing to the restraint that other structural elements provided against elongation when the wall developed a plastic hinge;
- vertical reinforcement in a wall in the region between confined compression zones is subjected to compression when the bending moment decreases and reverses in direction. Under these conditions the longitudinal reinforcement may yield in compression, which can result in buckling; and
- the majority of structural tests on walls that have been made to establish design criteria have been tested with in-plane loading only. The effect of bi-axial loading has received little attention, and this aspect needs further research.

Recommendation

We recommend that:

43. The Standard should be modified to include requirements related to confinement of ductile walls.

    For the ductile detailing length of ductile walls, transverse reinforcement shall be provided over the full length of the wall as follows:

    - confinement of boundary regions shall be provided in accordance with NZS 3101:2006, clause 11.4.6, modified to provide confinement over the full length of the compression zone; and
    - transverse reinforcement in the central portion of the wall shall satisfy the anti-buckling requirements of NZS 3101:2006[2], clause 11.4.6.3.

    We note that earlier this year, the Structural Engineering Society New Zealand Inc. (SESOC) published a draft recommendation to this effect.[3]

9.2.5 Suitable provisions to prevent buckling of walls subjected to moderate and high axial load ratios are currently not considered in the standard.

Recommendation

We recommend that:

44. As a short-term measure, where there is a ductile detailing length in a wall and the axial load ratio, $N/A_g f'_c$, equals or exceeds a value of 0.10, the ratio of the clear height between locations where the wall is laterally restrained to the wall thickness should not exceed the smaller of 10, or the value given by clause 11.4.2 in the Standard.

    Research should also be carried out to establish more rational expressions for limiting the ratio of clear height to thickness, allowing for both the loading and the imposed deformations on walls.

9.2.6 In a number of buildings occupants reported that after the September earthquake the building was more lively than it had been before the earthquake. There are a number of potential explanations for this. Stiffness degradation caused by yielding in the structure and elongation of the plastic hinges is one possible cause and is supported by a limited examination of test results on structural frame tests made in laboratories (see Figure 113 in section 6.3.5 of this Volume). Knowledge of potential loss of stiffness due to these actions could be of value in assessing the required level of performance for a damage limit state.

Recommendations

We recommend that:

45. Research should be carried out into stiffness degradation due to yielding in the structure and elongation of the plastic hinges, as this could be of considerable value in establishing acceptable design criteria.

46 Guidance should be given in the Standard on the expected magnitude of elongation that occurs with different magnitudes of material strain and structural designers should be required to account for this deformation in their designs.

9.2.7 Elongation in plastic hinges in beams can have a significant influence on the behaviour of other structural elements. For example:

- it can reduce seismic isolation gaps in structures;
- in coupled structural walls, elongation in the coupling beams may be restrained by floor slabs that are tied into the walls. This action has the potential to increase significantly the seismic actions induced in the coupling beams, the coupled walls and the foundations;

- in building bays containing stairs, elongation of beams can reduce the effective width of support ledges for precast stairs or, alternatively, can result in the stairs and associated platforms being subjected to axial forces; and
- in buildings with precast panels allowance should be made for elongation in the design of the fixing of the panels.

Recommendation

We recommend that:

47. Structural designers develop a greater awareness of the interactions between elements due to elongation so that allowance for adverse effects can be mitigated in the design and guidance on these matters should be given in the commentary to the Standard.

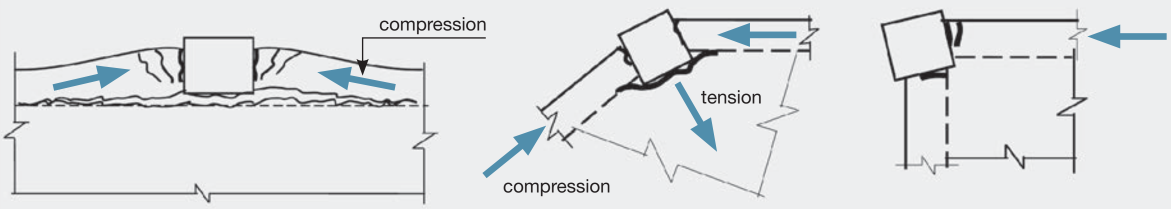
9.2.8 Elongation of plastic hinges in beams has a direct effect on the performance of floor slabs, particularly where the floors have been constructed using precast prestressed floor units. The Standard currently indicates the strength enhancement that may result from this interaction (Clause 9.4.1.6.2 of the Standard). However, some other aspects with important implications for seismic performance are not covered. Research papers have already been published[4] that may be of assistance to develop this guidance.

Recommendation

We recommend that:

48. The Standard should be revised to provide guidance on elongation of plastic hinges in beams. This should include:
    - the width and location of cracks that may be induced in floor slabs at the junction of the floor and supporting beams and the disruption that these cracks may cause to membrane forces that transfer seismic forces to the lateral-force-resisting elements; and
    - details of reinforcement required to ensure that the bars do not fail in tension at the cracks.

9.2.9 The restraint provided to beams by floor slabs, particularly where the floor slab contains prestressed precast floor units, can induce significant axial compression force in beams. This can cause the beams and associated columns to separate from the floors as illustrated in Figure 143(a). This type of separation occurred in the Clarendon Tower building. It would have been prevented if there had been a beam framing into the column at right angles to the perimeter beam. Alternatively, reinforcement that ties the column into the floor can be provided, as detailed in clause 10.3.6 of the Standard. Figure 143(b) shows the form of deformation seen in the Westpac Tower building. In this case the deformation cannot be practically restrained as very high forces would be required. Some form of ductile tie could be used to enable any cracks that are generated to be repaired. The column rotation shown in Figure 143(c) was observed in structural tests by Peng.[5]



(a) Beam and column separate from floor slab

(b) Beams elongate and push column away from floor slab

(c) Column rotates due to elongation in one beam

Figure 143: Compression forces induced by elongation and boundary elements

Recommendations

We recommend that:

49. In the commentary to the Standard attention should be drawn to the significant axial compression force that may be induced in beams by the restraint of floor slabs.
50. Low-friction bearing strips should be used to support double-tee precast units to isolate the precast units and the supporting structure from friction forces.

9.2.10 In one of the large transfer beams in the Hotel Grand Chancellor (HGC), extensive spalling occurred in the cover concrete at the mid-depth region of the beam. This was the location where U-shaped stirrup pairs, proportioned to enclose the top and bottom longitudinal reinforcement in the beam, lapped each other in cover concrete. The transfer of tension between the stirrup legs in the lap zone created significant tension in the concrete, and evidently it was this tension force that caused the spalling. The loss of this concrete would have left the stirrups ineffective and it is fortunate that collapse did not occur. The detailing that was used satisfied current requirements in the Standard.

Recommendation

We recommend that:

51. Where clause 8.7.2.8 in the Standard permits the use of stirrups in the form of overlapping U-shaped bars, the proportion of these bars lapped in cover concrete should not exceed 0.5.

9.3 Issues related to the Structural Steel Standard, NZS 3404:2009[6]

The Standard does not require redundancy in a building that relies on eccentrically braced frames for seismic resistance to ensure that collapse cannot occur in the event of one or two active links failing. We consider there should be a requirement for redundancy in such buildings. This requirement might be satisfied by providing columns with sufficient strength and stiffness to provide an alternative load path for a portion of the lateral force resisted by the eccentrically braced frames in each frame.

Recommendation

We recommend that:

52. The Standard should be amended to require a level of redundancy to be built into structures where eccentrically braced frames are used to provide seismic resistance.

9.4 General issues related to structural design

These recommendations are directed to design engineers, and should be considered by the Structural Engineering Society New Zealand Inc., the New Zealand Geotechnical Society, the New Zealand Society for Earthquake Engineering Inc., the Institution of Professional Engineers New Zealand, and other interested bodies. They should also be addressed in continuing education courses. In some cases information should be added to the commentary to NZS 1170.5.[1]

9.4.1 Problems associated with foundation soils have been a major issue in Christchurch. These are discussed in detail in Volume 1, section 4 of this Report.

Recommendation

We recommend that:

53. There should be greater cooperation and dialogue between geotechnical and structural engineers.

9.4.2 Load paths need to be defined to ensure that the details have sufficient strength and ductility to enable them to perform as required. For example, inertial forces from the floor slab need to be transmitted to lateral force restraining elements. To protect against very high but short-term forces associated with higher mode effects it is important that the load paths have some ductility.

Recommendation

We recommend that:

54. Designers should define load paths to ensure that the details have sufficient strength and ductility to enable them to perform as required.

9.4.3 The validity of basic assumptions made in analyses should be assessed as a part of structural design. The 'what if' approach should be used, with examples including assessing:

- whether ratcheting may occur, and if so what steps can be taken to prevent it; and
- whether an assumed section property, say 0.25 x gross section for a lightly loaded wall, is appropriate for the building and limit state being considered. Values of section properties recommended in NZS 3101:2006[2] are based on the assumption that the member will have developed flexural cracks at relatively close centres. A check on the magnitude of bending moments may indicate that the extent of flexural cracking is limited, in which case the analysis should be repeated with more appropriate section properties. This process can help to identify the potential ductility of the building and indicate the appropriate detailing that should be used.

Recommendation

We recommend that:

55. Structural engineers should assess the validity of basic assumptions made in their analyses.

9.4.4 Potential problems may arise from ratcheting in structures where:

- gravity loads are resisted by cantilever action;
- structures or structural elements have different lateral strengths in the forward and backward directions; or
- transfer structures are incorporated in buildings.

Recommendation

We recommend that:

56. Appropriate allowance should be made for ratcheting where this action may occur.

9.4.5 Current widely used methods of analysis do not predict elongation associated with flexural cracking and the formation of plastic hinges. This aspect can be of particular concern when assessing axial forces induced in structural walls. The formation of flexural cracking causes the wall to elongate and this is greatly increased if a plastic hinge develops. Elongation can be partially restrained by floors that connect the wall to other vertical elements. This can result in the wall being subjected to much higher axial forces than was indicated in the structural analysis. For this reason care is required in proportioning and detailing walls and other structural elements that support the walls.

Recommendations

We recommend that:

57. Structural engineers should be aware that current widely used methods of analysis do not predict elongation associated with flexural cracking and the formation of plastic hinges.
58. In designing details, compatibility in deformations is maintained between individual structural components.

9.4.6 To understand how the tensile strength of concrete can influence structural behaviour, it is essential to have an understanding of basic concepts relating to crack control. This is necessary to avoid the adverse effects of tensile strength on ductility of buildings.

Recommendation

We recommend that:

59. Structural engineers should be aware of the relevance of the tensile strength of concrete and how it can influence structural behaviour.

9.5 Particular issues relating to assessment of existing buildings

These recommendations are directed to design engineers, and should be considered by the Structural Engineering Society New Zealand Inc., the New Zealand Society for Earthquake Engineering Inc., the Institution of Professional Engineers New Zealand, and other interested bodies. They should also be addressed in continuing education courses.

Recommendation

We recommend that:

60. Training or guidance should be provided so that structural engineers are aware of the following issues when assessing existing buildings:
    a In a number of reinforced concrete buildings designed using Standards published prior to 1995, the columns that were provided primarily to support gravity loading had inadequate confinement reinforcement to enable them to sustain the inter-storey drifts associated with the ultimate limit state. There are a number of reasons for this:
        - first, it was not until 1995 that a requirement was introduced for all columns to have confinement reinforcement;
        - second, design inter-storey drifts calculated using Standards in use prior to 1995 gave smaller inter-storey drifts than the corresponding values found using current Standards. The difference arises from the use of stiffer section properties, the lack of a requirement for drifts associated with P-delta actions to be included, and the practice of taking the design inter-storey drift as 50 per cent of the peak value ($^{2}/_{SM}$) while the ductility was calculated on the basis of ($^{4}/_{SM}$).

b There are a number of structural weaknesses in existing buildings due to aspects of design not being adequately considered in earlier design Standards. The report by MacRae et al.[7] identifies many of these aspects.

c In assessing the potential seismic performance, particular attention should be paid to ensuring that seismic gaps for isolating stairs or separating buildings, or parts of buildings, have been kept clear.

9.5.1 Non-ductile mesh was widely used as reinforcement in the in situ concrete topping on floors containing precast units. This mesh has been found to fail at crack widths of the order of 2mm in width, which in some cases results in a major loss of the ability of the floors to perform as diaphragms.

Recommendation

We recommend that:

61. Where mesh has been used to transfer diaphragm forces that are critical for the stability of a building in a major earthquake, retrofit should be undertaken to ensure there is adequate ductility to sustain the load path.

9.6 Issues raised in our Interim Report related to structural design-means of egress

A number of recommendations were made in the Royal Commission's Interim Report. All these have been addressed in greater detail in this report except the following.

It was proposed that a maximum considered earthquake limit state be introduced into the Earthquake Actions Standard, NZS 1170.5:2004[1]. The intention was that this limit state be considered for the design of stairs, ramps and egress routes from buildings to ensure that these remained useable following a major earthquake. Having given further consideration to this issue, we now consider that the same objective can be achieved by a different approach that might better fit the existing framework of NZS 1170.5[1].

Recommendation

We recommend that:

62. Critical elements such as stairs, ramps and egress routes from buildings should be designed to sustain the peak for inter-storey drifts equal to 1.5 times the inter-storey drift, in the ultimate limit state. In calculating this inter-storey drift appropriate allowance should be made for elongation in plastic hinges or rocking joints with an appropriate allowance for construction tolerance. NZS 1170.5:2004[1] and the relevant materials Standards should be modified to provide for this requirement.

9.7 Building elements that are not part of the primary structure

Recommendations

We recommend that:

63. The principles of protecting life beyond ultimate limit state design should be applied to all elements of a building that may be a risk to life if they fail in an earthquake.
64. In designing a building, the overall structure, including the ancillary structures, should be considered by a person with an understanding of how that building is likely to behave in an earthquake.
65. Building elements considered to pose a life-safety issue if they fail should only be installed by a suitably qualified and experienced person, or under the supervision of such a person. The Department of Building and Housing should give consideration to the necessary regulatory framework for this.

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Canterbury Earthquakes Royal Commission
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