2017

CTV Investigation

Engineering Opinion Report

Beca

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Report

CTV Building Collapse - Engineering Opinion Report

Prepared for New Zealand Police

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Appendices

Glossary of Key Terms

The following key terms have been used throughout this report. The definitions provided are those that have been assumed to apply in the context of this report.

Acceleration – Displacement Response Spectra (ADRS)

A representation of the shaking experienced during a particular earthquake record where the peak acceleration and displacement responses of single degree of freedom oscillators of varying stiffness are plotted for defined levels of hysteretic damping.

Capacity

The ability of a building or element to resist stresses and strains due to applied loads/deformations. Capacity includes both the strength and deformation capability of a building or element.

Capacity Design

The design method in which elements of the seismic resisting system are chosen and suitably designed and detailed for energy dissipation. All other elements in the system are provided with sufficient strength to ensure the chosen elements are the only ones that dissipate energy, i.e. the other elements remain essentially elastic.

Catastrophic Collapse

Collapse of a building involving complete loss of separation between two or more floors of a building.

Design Earthquake/design spectrum

The earthquake shaking assumed for the design of the building or element. The design spectrum is a way of representing the design earthquake shaking where the design seismic load requirements are represented as a function of the building period(s) of vibration.

Diaphragm

A horizontal system (e.g. floor slab or structural steel roof bracing) that acts to transmit horizontal actions to and between the vertical elements of the primary seismic system.

Ductility/ductile

The ability of a building or member to maintain its strength resistance in repeated and reversing cycles once it has yielded, i.e. once it is deforming beyond yield.

Elastic

The range of deformations for which a section will return essentially to its un-deformed state upon removal of load. The elastic range is prior to yield. The nominal yield of an element may be associated with some small amount of permanent deformation.

Equivalent Static Method of Analysis

A method of analysis using static lateral forces to simulate the inertial forces resulting from earthquake shaking.

Hinge (plastic)

Regions in a member which yield under flexural actions while remaining ductile.

Inter-storey height

The overall height between two adjacent storeys.

Inter-storey drift

The relative lateral displacement between two adjacent storeys under lateral load, often reported either in units of length or as a percentage of inter-storey height.

Limited Ductile

A detailing provision of the concrete materials standard that ensures a member possesses a limited amount of ductility.

Modal Response Spectrum Method of Analysis

A method of dynamic analysis in which a given earthquake design spectrum is applied to a mathematical model of a building and the combined response of several modes of vibration is determined. This analysis assumes elastic (linear) behaviour, ie no elements have reached yield.

Natural period of vibration (period)

The time taken for a building to complete one full cycle when responding in its first lateral mode of vibration.

Non-linear Time History Analysis

A method of dynamic analysis, in which a mathematical model of the structure is subjected to time histories of actual earthquakes. The model typically allows for non-linear behaviour of the elements/members

Over-strength

The maximum strength of a member section calculated taking into account the main factors that may contribute to an increase in strength. This is used in the capacity design approach to provide confidence that significant non-linear behaviour is restricted to elements/members that have been detailed for it.

Primary seismic/lateral system/structure

The structural system that is assumed by the designer to resist the building's design earthquake forces. For the CTV building, these parts were the South Shear Wall and the North Wall Complex and their foundations.

Primary Gravity system/structure

The elements of the structural system that are assumed by the designer to form the basic load resisting system for vertical gravity loads such as dead and imposed live loads. These elements, if not adequately separated, will also be subjected to loading induced by deformations of the primary seismic structure even if this is not taken into account by the designer. The primary gravity structure (or elements of it) can have a dual function as the primary seismic structure. For the CTV building, the parts resisting gravity loads were the South Shear Wall, North Wall Complex, and the frames formed from the columns and beams. However, the frames were not separated from the primary seismic structure and therefore the effect of lateral deformations induced had to be included in their design even though the lateral loads they could resist were ignored.

(Structural) Resilience

The ability of a building structure to resist earthquake shaking of greater intensity than assumed for the purposes of design. This ability is inherent in seismic design codes, including those used for design of the CTV building.

Strength (seismic)

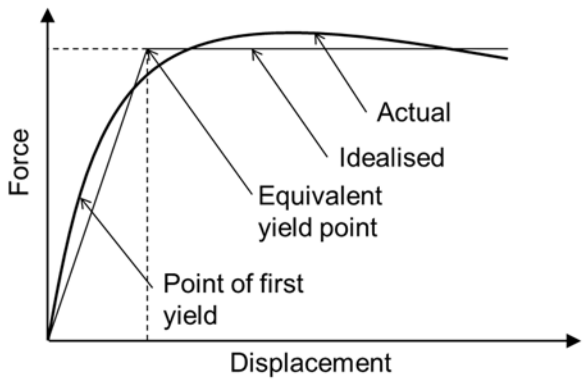
The value (e.g. kN) representing the resistance that a building or member provides against applied (seismic) loads.

Ultimate Limit State (ULS)

A defined stress and deformation state used for design.

Yield

The point at which an element begins to deform plastically. Up to this point, the element is in the elastic range. Often, yield is represented by an equivalent yield point which is an idealised point beyond which resistance will not increase with increasing displacement (deformation). The equivalent yield point is not necessarily coincident with the actual first yield point as shown in the diagram below.



Key Abbreviations

ARCE – Alan M. Reay, Consulting Engineer

ARCL – Alan Reay Consultants Limited

CBGS – Christchurch Botanic Gardens

CCC – Christchurch City Council

CCCC – Christchurch Cathedral College

CERC – Canterbury Earthquakes Royal Commission

CHHC – Christchurch Hospital

COM – Centre of Mass

COR – Centre of Rotation

CRC – Canterbury Regional Council

CTV – Canterbury Television

DBH – Department of Building and Housing (became part of MBIE)

ESM – Equivalent Static Method of Analysis

ETABS – Computer program to carry out a structural analysis of a building

HCG – Holmes Consulting Group

MBIE – Ministry of Business, Innovation, and Employment

NLTHA – Nonlinear Time-History Analysis

NWC – North Wall Complex

NZNSEE – New Zealand National Society for Earthquake Engineering

PGA – Peak Ground Acceleration

REHS – Resthaven

SSW – South Shear Wall

WCL – Williams Construction Limited

Summary and Conclusions

Beca has been commissioned by the New Zealand Police to provide an opinion on structural engineering matters related to the collapse of the commercial building at 249 Madras Street, commonly known as the CTV building, in which 115 people lost their lives as a result of the 22 February 2011 Christchurch earthquake. Part of the work includes an opinion on what was the “expected standard” of design, construction and consenting of buildings, particularly in Christchurch, during the mid-1980s and early 1990s when the design, construction and subsequent retrofit of the CTV building were carried out.

The information and reports produced by others during the Department of Building and Housing (DBH) and Canterbury Earthquake Royal Commission (CERC) inquiries have been used in the preparation of this report, together with our own analyses and investigations as described.

The collapse of the CTV building was catastrophic and sudden, with the floor slabs pancaking on top of one another leaving little space between. The CTV building was the only modern (post-1976) building to collapse in this manner in the earthquake sequence between September 2010 and February 2011.

The brief requires us to report on the following three questions:

1. Was there an omission by any individual to discharge their duty? Refer to Section 2 for the definition of “duty” as used in this report. In answering this question, we note that we are expressing our opinion as requested, but the issue of whether any particular individual has breached the assumed duty may ultimately be a matter for the courts to address.

2. Was that omission a substantial and operating cause of the deaths?

3. If so, was that omission a major departure from the expected standard?

The individuals whose actions we have been requested to consider were named by the Police in their brief (Refer Appendix A).

In preparing this opinion Beca has:

1. Reviewed the relevant parts of the evidence given at the CERC hearings and the CERC and DBH reports.

2. Carried out analyses of the building, using the codes and practices of the day.

3. Reviewed the calculations carried out by David Harding, an employee of Alan M. Reay Consulting Engineer (ARCE), and reviewed the drawings for the building.

4. Reviewed the calculations carried out by Geoff Banks, a director of Alan Reay Consultants Limited (ARCL), for the checking and design of the 1990/1991 retrofit works related to the diaphragm connections.

5. Undertaken interviews with consulting engineers, both within Beca and with other organisations listed in Appendix E, who were active in the design of multi-storey buildings at the time, to ascertain the relevant design standards, design approach, practices for overviewing and checking designs and expectations regarding council checking as part of the issuing of building permits in the mid-1980s.

6. Reviewed the design of other 1980s Christchurch buildings that we could find with similar structural systems and of similar size and scale, and compared the relevant parts of their structure with the CTV building structure.

7. Analysed the performance of the CTV building structure under the shaking experienced as recorded at other sites in Christchurch at the time of the earthquakes using the results of nonlinear time-history analyses (NLTHAs) undertaken jointly with [redacted]

8. Completed physical testing of full size replica concrete specimens of key parts of the CTV building structure at the University of Auckland.

9. Formed an opinion on what caused the collapse of the CTV building to be so severe that all floors pancaked to ground level resulting in heavy loss of life based on all of the information we have gathered.

In our opinion:

1. The collapse was initiated by loss of stiffness in one or more beam-column joints resulting in column axial failure, probably in the ground floor of an internal frame, probably on Grid 2. The capacity and stiffness of the beam-column joint regions associated with these columns was significantly reduced by the lack of transverse and shear reinforcement and this was a significant contributor to the collapse. The characteristics of axial failure in these relatively slender columns were such that their ability to support gravity loads would have been completely lost once it occurred. This would be either by progressive crushing/failure of the column concrete and/or sliding of the column to one side once at least one end became effectively detached from the structure. Once collapse of these columns was initiated there was no ability for the gravity loads to be redistributed to adjacent columns with the result the remaining columns were progressively overloaded, leading to the pancaking collapse of one floor on top of another.

2. If the building had been designed in accordance with and compliant with the codes of the day it would not have collapsed in the pancaking form it did, notwithstanding the level of shaking and number of earthquakes in the Canterbury sequence up to and including 22 February 2011.

3. The sudden pancaking nature of the failure occurred because the primary seismic structure had insufficient strength and stiffness to protect the primary gravity frames as detailed, and as a consequence the primary gravity frames were not sufficiently detailed for flexure or ductility to sustain the deflections imposed on them.

4. The primary seismic structure (i.e. the shear walls and their foundations) initially survived the earthquake, damaged but still standing, apart from the South Shear Wall which ultimately failed in its weak direction after the collapse of the floors because it lost north-south support provided by the floor slabs and the North Wall Complex. The intent of the codes of the day was that pancaking collapse would not have occurred in the level of shaking experienced on 22 February 2011, although the building could be heavily damaged. It is our opinion that, had the whole building complied with codes and practices of the day, the building, though damaged, would have had sufficient resilience not to collapse suddenly and in a pancaking fashion. All other buildings in Christchurch of a similar age with similar structural systems that experienced the 22 February 2011 earthquake had shear walls that provided more effective protection of the gravity structures and the gravity structures were detailed with more transverse steel that provided greater ductility.

5. There is no evidence that the collapse was triggered by ground subsidence/settlement.

6. The original 1986 structural analysis and design of the building was undertaken using appropriate codes and analysis techniques but contained a number of errors in relation to the application of the codes. There was also a significant mathematical error.

   The significant errors included:

   a. An incorrect calculation of the building's natural periods of vibration in both the east-west and the north-south directions leading to an under-calculation of the seismic design load.

   b. Incorrect reduction in the analysis results for design of the primary seismic elements by including an extra unjustified 0.8 factor, leading to a further lowering of the seismic design load compared with that required by the code. In combination with 6a), the net effect was that the North Wall Complex (in the north-south direction) was 25% weaker and the South Shear Wall (in the east-west direction) 40% weaker than required by the code.

   c. Failure to calculate correctly the building deflections under code-specified loading, particularly for loading in the east-west direction. This may have resulted in a lack of appreciation by the designer of the deflections required to be sustained by parts of the structure, which, in turn, led to the errors described in d) and e) below.

   d. Non-compliance with the code requirements for the spacing and amount of specified transverse reinforcement for beam-column joints, resulting in a lack of ductility/resilience.

   e. An incorrect and not justified decision to regard the beams, columns and their joints in the building as being required to resist only gravity loads. In doing so, the designer concluded incorrectly they were not required to be designed for seismic frame action or for ductility. This caused a lack of resilience in those elements which were then unable to sustain the required seismic deformations while maintaining their primary role of supporting the gravity loads.

   f. An incorrect units' conversion leading to a gross understatement of the shear to be transferred from the floor slabs to the North Wall Complex for loading in the east-west direction, which led to an under-designed connection that did not comply with codes of the day. This gave rise to the potential for the floors and North Wall Complex to separate with the result that the building would not behave as the codes intended.

   g. Failure to check the connection of the floor slabs to the North Wall Complex for loading in the north-south direction, leading to a connection that did not comply with the codes of the day. This gave rise to the potential for the floors to detach from the North Wall Complex with the result that that the building would not behave as the codes intended.

7. Apart from the lower design loads arising from 6a) and b) above, the primary seismic structure, comprising the shear walls and their foundations, was designed and detailed generally in accordance with codes and practices of the day.

8. The transverse beam-column joint steel specified on the drawings in the primary gravity frames did not comply with the code requirements. Away from the joints, the transverse column steel was light (i.e. small diameter and large spacing), but compliant with the minimum requirements of the code in the critical areas. Our investigations have shown that the requirements of the 1982 code, albeit light and less than required in later codes, would have been sufficient to prevent the building collapsing in the manner it did in this earthquake, if the significant design issues identified (understrength and noncompliant shear walls, and noncompliant beam-column joint transverse reinforcement) had not been present.

9. Although the transverse column steel in the CTV building complied with the minimum requirements of the concrete code it was very light compared with general practices of the day. We have found no other similar sized buildings of the era in Christchurch with as little transverse steel as was specified in the CTV building columns and beam-column joints.

10. The poor and non-compliant retrofitted connections between the North Wall Complex and the primary gravity structure, if present in an otherwise compliant building, would not have led to the collapse of the type that occurred in the 22 February 2011 earthquake.

11. The known construction errors (lack of transverse steel through the beam-column joints, changes to some beam reinforcing end anchorages, potentially low strength concrete, un-roughened precast to in-situ concrete joints), if present, in an otherwise compliant building, would not have led to the collapse of the type that occurred in the 22 February 2011 earthquake.

12. The structural design was carried out by Mr David Harding, an employee of Dr Alan Reay. According to Dr Reay's evidence at the CERC, neither he, nor any other engineer in his practice, or outside it, provided any oversight or checking. Mr Harding was a senior engineer and employee of Dr Reay, but inexperienced in multi-storey design. The stated lack of oversight by Dr Reay, or any form of review or checking by another experienced person, was contrary to accepted practice of the time. Our experience and the conclusion we reached following discussions with other practitioners of the day was that a principal of a design company, partner of a partnership, or sole practitioner with staff, would oversee a design such as this (often completing the concept design themselves), or organise another senior employee or external consulting engineer to review it. In our opinion, relevantly experienced oversight or review at concept stage would have questioned the low lateral seismic displacements and/or the decision not to detail the primary gravity frames with ductility. To appoint a newly employed senior engineer without experience in similarly sized structures and rely on him to ask questions if needed, was not in accordance with generally accepted and expected practice.

13. We agree with the CERC report conclusion that the review undertaken by Holmes Consulting Group in 1990 for a prospective purchaser of the building was "*never intended to be a full peer review of the design of the building*". The assessment did not identify that the gravity structure (i.e. the primary gravity frames) should have been detailed with ductility. This is not surprising given it would take an analysis of the whole building (both primary seismic and primary gravity structures) to confirm the requirement, and the short time available permitted only a relatively superficial review.

14. The Christchurch City Council (CCC) process for confirming the adequacy of the structural engineering aspects of building permits often relied on the designer, and included only spot checks otherwise. The other designers we spoke to who were practicing in Christchurch at the time said they would not rely on the building permit checking process as a review of their design.

15. Other factors have been raised, such as the cumulative effects of multiple earthquakes and demolition of the neighbouring building. In our opinion these factors were not of sufficient severity to be substantial contributors to the collapse.

[redacted]

In response to the questions posed by the Police, we make the following comments in relation to the named individuals they list in the brief:

Alan Reay – practicing as Alan M. Reay Consulting Engineer, ARCE and later a director of ARCL (post 1988): ARCE agreed to design the building. As Dr Reay was a sole practitioner this agreement was directly with him.

In agreeing to design the building, and as the only principal of ARCE, Dr Reay was obligated (had a duty) to his client, future building occupiers and the public in general, to deliver a design that met the expected standard of the day by allocating appropriately skilled and experienced staff to ensure this was achieved. The expected standard included meeting the design codes and standards, and the accepted practices of the day.

There were design errors and non-compliances with the design codes and standards of the day in the CTV building. The presence of these errors and non-compliances occurred because appropriately skilled staff were not allocated to the design and checking of the building. This was an omission in the discharge of Dr Reay's obligations to ensure the design of the building met the expected standard.

To meet his obligation (duty) regarding allocation of appropriate resources Dr Reay had a wide range of options including:

1. Design the building himself. In doing so he could elect the degree of review his design required based on his own assessment of his competence and recognising professional expectations.
2. Appoint an engineer experienced in building work of this type to carry out the design. If the engineer was suitably experienced then, from a technical point of view, Dr Reay could have delegated this task and expected the delegated engineer appoint another engineer to, or personally, undertake the level of checking and/or review to satisfy himself the design was sound.
3. Allocate the design task to a less experienced engineer. In this case Dr Reay was obligated to do sufficient checking and reviews to confirm that the design met the expected standard. He could have done this himself or delegated this to an engineer experienced in the design of buildings of this type to do on this behalf.

Accepted practice of the day was, and is, for a principal of a design firm (Dr Reay was the sole principal of ARCE) to determine the option to be followed based on the experience of the resources available and by an appropriate level of personal input to confirm that a design was delivered to the expected standard. It was not the accepted practice to completely delegate, without any supervision or checking process, all engineering for a significant multi-storey building structure to an engineer who had little or no relevant experience. It was also not the accepted practice of the day to rely on the consent checking process to provide any meaningful level of review.

To delegate the above tasks for the CTV building design, Dr Reay was obligated to satisfy himself that the engineer was experienced in the design of significant multi-storey buildings. The use of precast concrete frame members, innovative for the time, also warranted additional care. It was not sufficient to rely solely on age and years of experience in engineering without also considering the level of experience available that was directly relevant to the type and scope of building being designed.

It was clear from evidence provided to the CERC that Mr Harding did not have prior experience in the design of buildings of the type of the CTV building, nor did he misrepresent this lack of experience to Dr Reay. In his 13 years of experience, Mr Harding had only his first four years doing structural engineering which was limited to minor single storey domestic and industrial structures. Therefore, it was not reasonable for Dr Reay to assume that he could either delegate the design task to him (option 2 above), or to allocate the design task to him (option 3 above) without also carrying out suitable review himself or delegating this to another suitably experienced engineer.

[redacted]

Major departures from the accepted practices and, therefore, the expected standard of the day by Dr Reay were therefore:

- assuming that the design task could be delegated completely to Mr Harding, and
- allocating the job to Mr Harding, an engineer with virtually no experience in structural design of multi-storey buildings, without also providing appropriate checking himself or ensuring adequate review by others, and
- relying on the building consent process to identify errors in the design.

The building collapsed in a pancaking fashion and as a result caused multiple fatalities. Our investigations have shown a collapse of this type was due to errors in the design and would not have occurred if the design had complied with the design codes and design standards of the day. Dr Reay's omission to discharge his obligations in regard to the allocation of appropriate resources to the design were the reason why the design errors were not identified and corrected, and was therefore a substantial and operating cause of the deaths.

It does not appear from the available reports that Dr Reay was the principal of ARCL responsible for the retrofit carried out in 1990/1991. Geoff Banks, a principal of ARCL, appears to have taken this role.

It is our conclusion that:

1. Dr Reay, as the person who undertook to design the building, omitted to discharge his duty to allocate appropriately experienced personnel to the design, checking and review process of the building structure.
2. This omission was a substantial and operating cause of the collapse, and
3. The omission was a major departure from the expected standard.

David Harding – employee of ARCE and designer of the CTV building: Mr Harding made significant errors which meant the design of the building did not comply with the codes or standards of the day. There was a significant underestimation of the seismic load and reinforcing requirements in critical areas of the building in Mr Harding’s design. Mr Harding’s erroneous assumption that the beams, columns and their joints (the primary gravity frame) were not required to be designed or detailed for flexural actions, together with the low seismic load were the reasons why the building collapsed suddenly, floor-on-floor, in the pancaking manner. Other errors and omissions in the design compounded the effect of this decision by further reducing the strength and resilience available in the building.

Mr Harding also failed to note and instruct corrections to departures from the construction documentation by the contractor during construction of the building which further increased the deformations in the primary gravity structure beyond what they could sustain.

It is our conclusion that:

1. Mr Harding, as building designer, omitted to discharge his duty in relation to the design of the CTV building as the design did not comply with generally accepted practices and standards of the day.
2. The omission was a substantial and operating cause of the deaths, and
3. The omission was a major departure from the expected standard.

Geoff Banks - director of ARCL and designer of the drag bar retrofit in 1990/1991: Mr Banks made errors in the assessment of the design actions between the North Wall Complex and the floor slabs during the design of the retrofit works in 1990/1991. The strengthening works partially addressed the deficiencies in the north-south earthquake direction but not in the east-west direction. These errors resulted in an underestimation of the retrofit work required to provide a compliant connection between the floor slabs and the North Wall Complex. Mr Banks should have fully resolved the deficiency with the connection between the North Wall Complex and the floor slabs. However, Mr Banks would not have been expected to have

[redacted]

initiated a full structural review of the building at that time, or to have identified the beam-column joint issue, especially as Holmes Consulting Group's investigation indicated that the primary gravity structure was code compliant "in all respects".

Mr Banks' errors meant the slab to North Wall Complex connection, though improved, was still weaker than it should have been. While this was a potential contributing factor to the collapse, the error was not a major cause of the collapse.

It is our conclusion that:

1. Mr Banks omitted to discharge his duty in relation to the design of the strengthening works, but
2. The omission was not a substantial and operating cause of the deaths.

Graeme Tapper/Bryan Bluck - Assistant Building Engineer Christchurch City Council (CCC)/Chief Building Engineer CCC: The CERC identified that there had been a policy in the 1970s, which was perhaps still operating in the mid-1980s, for CCC to rely on structural engineering designers in lieu of a review by the council engineers when processing building permits. This is consistent with the fact that the CERC report indicated the building permit was issued three working days after the receipt of the structural drawings. We do not believe the council could have done sufficient work to identify the deficiencies in the structural design in this short period, especially with the limited resources they had available. It would appear they followed the practice for the council staff to rely on the assurances of the building structural engineer regarding compliance with codes. The practitioners operating in Christchurch at the time who we interviewed recognised this, and did not rely on the council checking process to provide sole confirmation of the adequacy of their design.

It is our conclusion that:

1. While the CCC may or may not have discharged their responsibilities under the relevant legislation of the time (we have not checked as it is a legal matter and beyond our brief), they did not depart from their usual practice of not doing a thorough structural check.

   The CCC may or may not have omitted to discharge their duty by not doing a thorough structural check and consequently issuing a building permit for a defective design.
2. The undetected defective structural design of the CTV building was a substantial and operating cause of the deaths, but
3. Issuing a building permit without a thorough structural check prior to issuing a building permit was not a major departure from the expected standard of the day in Christchurch.

Bill Jones/Gerald Shirtcliff – employees of Williams Construction managing construction of the building: The contractor failed to carry out the construction in conformance with the construction documentation in relation to the lack of preparation of construction joints at the beam-ends, and, failure to install the beam-column joint ties as detailed on the construction drawings. In our opinion, the degree of departure from the structural drawings represents an omission to discharge their duty. The construction deficiencies reduced the resilience of the primary gravity frames and hence potentially contributed to the collapse. However, it is our opinion that, in the absence of the design deficiencies, the construction omissions alone would not have resulted in the sudden and complete pancaking of the floors.

It is our conclusion that:

1. Messrs Jones and Shirtcliff as managing the construction work omitted to discharge their duties in relation to the construction of the CTV building by not constructing it in accordance with the plans and specifications, but
2. The omission was not a substantial and operating cause of the deaths.

1 Introduction

Beca Limited (Beca) has been commissioned by the New Zealand Police (Police) to provide an engineering opinion to assist them in investigating the potential criminal culpability of individuals following the collapse of the Canterbury Television (CTV) building in the 22 February 2011 earthquake.

The CTV building was the only modern (post-1976) building to catastrophically collapse in the 22 February 2011 earthquake. The collapse occurred quickly and led to all floors pancaking to ground level, causing the deaths of 115 people.

The Police provided terms of reference for the investigation and a list of individuals whose actions they instructed Beca to review in relation to the design, issuance of the building permit, construction and subsequent strengthening of the CTV building.

This report includes an outline of our approach, a description of the roles of the identified individuals, a structural description of the building, observations from the earthquakes, a summary of our opinion on the likely causes of catastrophic collapse of the building, whether or not the pancaking collapse was caused by omissions of the identified individuals to discharge their duties, and whether or not the omissions were a major departure from accepted practice of the day.

When undertaking this review, we have made extensive use of the reports prepared as part of the Department of Building and Housing's investigation into the collapse of the building and also the submissions to and deliberations of the Canterbury Earthquakes Royal Commission (CERC).

1.1 CTV Building Background

In 1986, Williams Construction Limited (Contractor) was invited by Prime West Corporation Limited (Developer) to submit a design-build proposal for an office building at 249 Madras Street, Christchurch. The building project was simply named 'Office Building – 249 Madras Street' at the time.

Mr Alun Wilkie of Alun Wilkie Associates (Architect) was engaged to draw up plans. Once this process was underway, Alan Reay, operating as a sole practitioner under the name Alan M. Reay Consulting Engineer (ARCE), was engaged as the structural engineer for the building.

David Harding, employed by Dr Reay as a structural engineer, was allocated the role of designing the building.

Alun Wilkie Associates and ARCE were formally engaged by Williams Construction in June 1986 and instructed to prepare drawings for permit and construction.

A building permit was obtained for the building in September 1986 with construction work beginning in October of that year. Construction of the building was completed in 1988.

In January 1990, Holmes Consulting Group (HCG) was engaged by Canterbury Regional Council (CRC), a potential purchaser of the building, to prepare a pre-purchase review as part of their due diligence. The main finding of their review was the identification of non-compliance in the connections between the floor slabs and the North Wall Complex (NWC).

CRC did not purchase the building and consequently a full review by HCG may not have been completed. The building was purchased by Madras Equities Limited in December 1990 who remained the owners until February 2011. Alan Reay Consultants Limited (ARCL) carried out the engineering design of drag bars to address the area of non-compliance identified in the HCG report and these were installed in October 1991. Geoff Banks, a director of ARCL at the time, carried out the design work.

From 1991 until September 2010, several minor additions and alterations were carried out, mostly concerned with tenancy fit-outs. One of these fit-outs was for Christchurch Television in 2000, a predecessor of Canterbury Television (CTV), the name by which the building became known.

On 4 September 2010, an earthquake of $7.1M_w$[1], known as the 'Darfield' earthquake, struck the Canterbury region. A series of significant aftershocks occurred on 26 December 2010, the first of which is referred to as the 'Boxing Day' earthquake. The CTV building endured both of these earthquakes, and the many minor aftershocks, without collapse, although signs of damage were observed during post-earthquake inspections.

On 22 February 2011, a devastating earthquake of $6.2M_w$ struck with an epicentre very close to the Christchurch CBD. The resultant shaking caused the sudden and complete pancaking collapse of the CTV building, claiming 115 lives.

In the wake of the February earthquake, the former Department of Building and Housing (DBH) conducted a technical investigation into the performance of four buildings that suffered serious structural failures, one of which was the CTV building. The New Zealand Government also established the Canterbury Earthquakes Royal Commission (CERC) to conduct an inquiry into matters to do with the earthquakes, including why these buildings failed as they did. The CERC was precluded from considering the liabilities of individuals for the failures.

The DBH appointed Dr Clark Hyland and Mr Ashley Smith to investigate the CTV building. Their findings were presented in the Hyland/Smith report. In addition to appointing Hyland and Smith, the DBH appointed an Expert Panel to oversee their work and review and approve their report.

Reports by these groups and others were presented at hearings of CERC. This led to a large amount of publically-available technical information related to the collapse of the CTV building.

In early 2014, the Police commissioned Beca to provide an engineering opinion to assist their investigation into the potential criminal culpability of individuals following the collapse of the CTV building.

[1] M_w is the symbol for moment magnitude, a scale used to measure of the amount of energy released at an earthquake's source

2 Terms of Reference and Scope of Work

The terms of reference for Beca's scope of work are as outlined in a Police letter, *CTV Building Collapse – 22 February 2011 Engineering Expert Opinion Briefing Document*, dated 14 February 2014. A copy of this letter is provided in Appendix A.

In summary, Beca is to provide an engineering opinion to address the following three questions in relation to the collapse of the CTV building in the 22 February 2011 earthquake:

1. Was there an omission by any individual to discharge their duty?
2. Was that omission a substantial and operating cause of the deaths?
3. If so, was that omission a major departure from the expected standard?

The brief requires Beca to provide evidence in relation to whether a person omitted to discharge a duty imposed by the Crimes Act. That duty requires anyone who has in his charge or under his control (or erects, makes, operates, or maintains) anything which, in the absence of precaution or care, may endanger human life, take reasonable precaution against and use reasonable care to avoid such danger, and is criminally responsible for the consequences of omitting without lawful excuse to discharge that duty.

As noted in the brief, it is a legal question as to whether a person is under that duty and therefore determining this is beyond the scope of this report. Accordingly, we have assumed that each of the individuals had such a duty, and that to discharge this duty they needed to perform their role in accordance with the generally accepted practices and standards of the day.

When evaluating if an omission to discharge a duty represented a major departure from the expected standard we assessed if it was a major departure from generally accepted practice of the day. Generally accepted practice was deemed to include the approach to professional matters (including ethical obligations), adherence to generally accepted design processes, interpretation of design standards of the day and generally accepted design office management practices to achieve the above.

In answering these questions, the focus is on those individuals identified in the Police letter and in assisting the Police consider whether evidence supports a criminal investigation. The roles of these individuals as they are understood to relate to their involvement with the CTV building are outlined in Section 4 of this report.

To avoid unnecessary duplication of effort, the information and reports produced by others during the DBH and CERC inquiries have been used where possible as a basis for this report. Beca has made no attempt to independently validate the facts presented in these reports.

3 Our Approach

3.1 Introduction

To achieve the objectives set out in the Terms of Reference, Beca adopted the following approach:

- Review existing information
- Identify the individuals involved in the design, construction, permitting and retrofitting of the building
- Review the performance of the building in the earthquakes
- Undertake analyses of the building as-designed to assist in determining likely failure and collapse mechanisms
- Review the 1980s design processes and requirements of the codes of the day
- Review the original 1986 design of the building
- Review the building permitting process
- Review the construction
- Review the 1990/1991 retrofit
- Establish the accepted practice of the day in relation to the original design and other issues deemed relevant
- Liaise and direct [redacted] to carry out nonlinear time-history analyses (NLTHAs) to inform on the impact the various issues had on the building collapse
- Carry out testing of full size replica concrete specimens at the University of Auckland
- Determine a likely collapse sequence
- Establish likely causes of collapse
- Evaluate the potential impact on the collapse of factors other than errors by individuals
- Consider the roles each identified individual played in the building collapse with regard to the Terms of Reference
- Identify areas where further investigation might be considered necessary.

These steps are discussed in more detail below.

3.2 Review of Existing Information

Through the DBH investigation and CERC inquiry, a large amount of information was produced and compiled relating to the history of the CTV building and its collapse on 22 February 2011.

We have reviewed the information that is publically available on the CERC website.

The documentation reviewed by Beca in producing this report is listed in Appendix B. This includes drawings and design calculations for the building, available permitting and construction information, calculations prepared for the 1990/1991 retrofit, and extensive commentary on the collapse and investigations completed and considered by CERC in reaching its conclusions.

Our understanding of the building configuration, how it carries loads from a structural perspective, and its history is presented in Section 5.

3.3 Identification of Individuals and Roles

The individuals to be considered in our investigation were identified in the briefing document provided by the Police.

The various individuals and their roles are described in Section 4.

3.4 Performance of the Building in the Earthquakes

A review of the damage that occurred was an essential part of gaining an understanding of how the building behaved in each of the significant earthquakes leading up to the building's collapse on 22 February 2011.

This included consideration of the Darfield earthquake (4 September 2010) and the Boxing Day earthquake (26 December 2010) in addition to the Christchurch earthquake on 22 February 2011.

It was also important to gain an understanding of the effect any damage the building may have sustained prior to 22 February 2011 had on its subsequent collapse.

These aspects are discussed in Section 6.

3.5 Establish the 1980s Design Practice and Process

The CTV building was designed in 1986. The relevant prevailing design codes were the loadings standard NZS 4203:1984 and the concrete structures standard NZS 3101:1982.

We reviewed these codes of practice to identify the process an engineer would have been expected to follow in order to design a multi-storey reinforced concrete building in compliance with the mandatory minimum standards set out in the design standards.

As part of our review we identified the techniques and tools typically available to structural engineers of the day.

We also established typical design office processes of the day as they related to selection of design team members, senior overview, and approach taken for technical review of building designs.

3.6 Review of Original 1986 Design

The design of the building was carried out in 1986 by David Harding, as an employee of Alan Reay.

We carried out a review of both the technical aspects of Mr Harding's design and the management practices of Dr Reay as a sole practitioner.

To investigate if the design completed by Mr Harding met the requirements of the codes of the day we completed parallel calculations for the building. In doing so, we carried out analyses of the building using the computer program ETABS in a fashion as close to what we believe was commonplace in 1986.

Technical aspects of our review of the original design are presented in Section 7. Issues associated with management of the design process and the responsibilities of those involved are discussed further in Section 3.11 and Section 7.

3.7 Review of Building Permitting Process

A building permit was issued for the building by the Christchurch City Council (CCC). The two CCC employees responsible for issuing the permit were Graeme Tapper and Bryan Bluck.

We carried out a review of available information relating to the permitting process followed for the CTV building. Most of the information available for review was contained within the CERC report.

The results of our review are discussed in Section 8.

3.8 Review of Construction

The Contractor responsible for the construction of the building was Williams Construction Limited (WCL). The two WCL employees identified by the Police as being responsible for the construction are Bill Jones (Site Foreman) and Gerald Shirtcliff (Construction Manager). ARCE were responsible for carrying out construction observation of the CTV building. Mr Harding was the engineer assigned to this role.

We carried out a review of available information relating to the construction of the CTV building. Most of the information available for review was contained within the CERC report.

The results of our review are discussed in Section 9.

3.9 Review of 1990/1991 Retrofit Design

The retrofit works involved strengthening the connections between the floor slabs and the North Wall Complex. This was designed by Geoff Banks, a director of Alan Reay Consultants Ltd (ARCL) at the time, in 1990 and 1991.

We carried out a review of the technical aspects of Mr Bank’s retrofit design.

The results of our review are presented in Section 10.

3.10 Other Factors

Although our focus has been on identifying and establishing the criticality of omissions by individuals, it has also been necessary to consider the role that other factors might have played in the type of collapse that occurred. These factors include:

- Severity of the earthquake shaking
- Cumulative effect of multiple earthquakes
- Demolition of neighbouring building
- Foundation softening
- Structural modifications made post-construction.

The assessed impacts of these factors on the collapse are discussed in Section 11.

3.11 Establish Accepted Practice of the Day

An important part of assessing whether an individual’s actions were a major departure from the standard expected of them was to establish what the accepted practice of engineers engaged in such work was at the time (i.e. mid-1980s for the original design and early-1990s for the design of the retrofit works).

Accepted practice relates to both the technical design practices involved in designing a multi-storey reinforced concrete building in Christchurch to the codes stated in 3.5, and the management practices of a design entity including technical direction, design guidance and design review/checking.

In order to establish accepted practice, we carried out three main pieces of work:

1. Interviewed relevant practitioners (external to Beca) active in building design in New Zealand in the mid-1980s and early 1990s.

2. Reviewed structural drawings of other relevant Christchurch buildings.

[redacted]

3. Reviewed the Institution of Professional Engineers New Zealand (IPENZ) Code of Ethics, and associated Code of Professional Practice for Consulting Engineers, current in mid-1986. A copy of these documents is provided in Appendices C and D respectively.

A brief description of these items is provided below.

The 1970s and 1980s were decades where rapid development in seismic design procedures took place. The 1980s was also a period that saw significant building activity in New Zealand, and therefore was a very busy period for building designers and council's with respect to permitting activities. The building market and environment was characterised by time being of the essence due to the financial climate.

Recollecting how things were done 25 to 30 years ago, and particularly during the 1980s, is naturally challenging. It is important, as far as possible, individual's recollections are not unduly influenced by the benefit of hindsight. Furthermore, there is a risk individuals will be inclined to recollect on the basis of what should have been the case, rather than what was actual practice at the time.

An argument could be made that practices within larger design companies were likely to have been different from those in smaller firms where systems may not have been as well-established.

To minimise the influence of beneficial hindsight, and to avoid undue dominance of a Beca (larger company) viewpoint, a process was adopted that involved surveying a number of practitioners who were active at the time. The practitioners were from a variety of design firms located throughout New Zealand, including Christchurch, and included those who were operating as senior engineers or principals of their companies.

Three specific areas were surveyed:

- Design practices in the mid-1980s, particularly in the areas applicable to the design of the CTV building such as the interpretation and handling of primary gravity structural elements.
- Design practices in the early 1990s, particularly around practices for design of diaphragms and their connections.
- Office practices around responsibilities, allocation of staff to projects, supervision of staff and design reviews.

The following individuals were interviewed. The company name indicated was the name of their employer at the time. We acknowledge their contribution to this exercise and thank them for participating.

- [redacted]
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- .

Beca staff carried out the interviews and recorded the responses. The responses were then sent to each for confirmation that they represented a correct record of the discussions and, if necessary, corrections were made. In some cases, drawings and calculations were referenced to improve the reliability of the responses. The questions asked during the interviews and the individuals responses are presented in Appendix E.

[redacted]

In addition to interviewing engineers of the time, drawings of six Christchurch buildings considered most relevant were reviewed. These drawings were shortlisted from a wider set obtained from the Ministry of Business, Innovation and Employment (MBIE) as a result of their investigation of buildings of this era. The six buildings in the shortlist were all of the buildings from the MBIE data set that were located in the Christchurch CBD, designed between 1984 and 1989, between 4 and 8 storeys in height and of reinforced concrete shear wall construction.

We reviewed the drawings of these buildings, including the specific detailing of relevant features, to obtain a more graphic representation of practice of the day in relation to the environment in which the CTV building was designed.

The six buildings and their designers were:

- 227 Cambridge Terrace, Christchurch, 1984 – [redacted]
- 79 Cambridge Terrace, Christchurch, 1985 – [redacted]
- 287 Durham Street (Landsborough House), Christchurch, 1985 – [redacted]
- 32 Oxford Terrace, Christchurch, 1986 – [redacted]
- 115 Kilmore Street, Christchurch, 1986 – [redacted]
- 9 Cathedral Square, Christchurch, 1989 – [redacted]

A summary of the characteristics and relevant features of each building is provided in Appendix G.

In the mid-1980s the relevant codes of ethics were covered by two documents:

1. The Institution of Professional Engineers Code of Ethics (dated February, 1986), and

2. The Code of Professional Practice for Consulting Engineers (published November, 1985, but formally approved by the Council on 3 July 1985).

These documents are reproduced in Appendices C and D respectively.

The Code of Professional Practice set out requirements for practice management and governance. It required members of the Institution of Professional Engineers New Zealand who practised as consulting engineers on their own behalf in consulting engineering to conform to the Institution's Code of Ethics and not become engaged in any act, activity or conduct which was contrary to the Code of Ethics or which was inconsistent with the integrity of the profession.

In addition there was a requirement of the member to provide professional engineering services in accordance with the Conditions of Engagement for Consulting Engineers. For the mid-1980s the relevant Conditions of Engagement would have been:

1. Conditions of Engagement for consulting Engineers when engaged as principal advisers, Document A, 1984, or

2. Conditions of Engagement for Consulting Engineers when engaged as secondary advisers Document B, 1986.

These Conditions of Engagement set out the terms under which the service was commissioned but required that Consulting Engineer's services also be governed by the Code of Ethics and the Code of Professional Practice for Consulting Engineers referred to above.

The Institution's Code of Ethics obligated each member to conduct himself so as to uphold the dignity, standing and reputation of the Institution and of the Profession. The code sets out seventeen specific requirements. The requirements considered particularly relevant to the activities associated with the original design of the CTV building were as follows:

1. *Each member shall exercise his professional and technical skills and judgement to the best of his ability and shall discharge his professional and technical responsibilities with integrity.*

6. *He shall not misrepresent his competence or without disclosing its limits, undertake work beyond it.*

8. *However engaged, he shall at all times recognise his responsibilities to his employer or client, others associated with his work, the public interest and his profession.*

13. *He shall encourage the further education and training of his subordinates particularly those who are candidates for corporate membership of the Institution.*

14. *He shall maintain and strive to improve his professional competence by attention to new developments relevant to his professional activity and shall encourage those working under him to do likewise.*

16. *When practising as a consulting engineer he shall conduct his affairs in accordance with this code and shall also observe the requirements of the Institution's current Code of Practice for Consulting Engineers.*

Our comments on accepted practices of the day, including the degree with which individuals complied with the Institution's Codes of Ethics and Professional Practice are contained within the relevant sections of the report, and are based on our own experiences, the results of the interviews, and our review of drawings of the buildings listed above.

3.12 Nonlinear Time-History Analyses

To inform on the role that different aspects of the design played in the pancaking collapse, two sets of nonlinear time-history analyses (NLTHAs) were run:

- A model to represent the as-designed building prior to 4 September 2010, excluding construction defects, referred to as the 'as-designed' model.
- A model to represent the building without identified design non-compliances, again excluding construction defects, referred to as the 'compliant' model.

The aim of these analyses was to test the following hypothesis:

If the compliant model did not reach a collapse state when subjected to the most appropriate record available for the site from the 22 February 2011 earthquake (the Christchurch Cathedral College record), but the as-designed model did reach a collapse limit state for the same input, the design omissions are a substantial cause of the pancaking collapse.

These analyses were performed by [redacted] under the instruction of Beca and extended the analyses completed as part of the CERC and DBH investigations into the building collapse.

Model details and key analysis inputs and outputs are presented in Appendix H.

The implications of the results of the NLTHAs are discussed, where relevant, throughout the report.

3.13 Testing at the University of Auckland

Due to lack of experimental evidence internationally on the expected behaviour of internal beam-column joint regions of the type used in the CTV building, a series of physical tests was carried out in the University of Auckland structural engineering testing laboratories.

Full size test specimens were constructed to replicate the arrangement of an internal beam-column joint and ground floor column to investigate the performance of three different configurations; as-designed (i.e. without construction deficiencies), compliant with codes of the day, and with construction deficiencies (but without design deficiencies.

We have considered the results and conclusions from these tests when reaching our conclusions on the possible sequence and cause of the collapse.

A description of the testing and a summary of the results and conclusions reached are provided in Appendix K.

3.14 Cause of Collapse

Investigators and researchers have previously proposed a number of possible scenarios for the collapse sequence of the CTV building. Several potential causes of initiation of collapse have also been proposed by others, but it has not previously been possible to be conclusive about either the initiation event or the subsequent sequence.

We carried out a review of the available information in terms of eyewitness reports, the photographic record and observations made post-collapse. Most of the information available for review in this regard was contained within the CERC report.

In addition to a review of available information, we also used the results of our NLTHAs and progressive collapse analyses to inform on a likely sequence and to assist in understanding how the building came to suffer the catastrophic pancaking collapse.

The physical tests of the internal column and beam-column configurations informed on the likely impact of the construction and design deficiencies and also the code of the day on the expected performance of the building on 22 February 2011 including the potential for pancaking collapse.

A likely collapse sequence is presented in Section 12 based on all of the available information.

Establishing a likely collapse sequence was considered important to evaluate the criticality of the various identified omissions and determine their relevance in terms of substantial and operating causes of failure.

Discussion of these aspects can be found in Section 12.

3.15 Conclusions

As a result of our investigations, we have reached conclusions regarding the criticality, with respect to the collapse, of omissions by the various individuals involved in the design, permitting, construction and subsequent retrofit of the building.

Our focus was on determining causes of the catastrophic pancaking collapse, as it was the nature of the collapse that led to the floor slabs lying on top of one another, leaving little space between for survival or rescue, and the resultant large loss of life.

Our conclusions, and the rationale for them, are discussed in Section 12.

4 Individuals Considered in this Investigation and their Roles

4.1 Introduction

Previous inquiries into the collapse of the CTV building conducted by others identified potential issues with the design, council permitting, construction and subsequent retrofit, as well as issues associated with post-earthquake activities between September 2010 and February 2011. There are a large number of individuals involved in these activities.

The current considerations by the Police as to the possibility of criminal culpability have been narrowed to the acts and/or omissions of those individuals associated with the original design, permitting, construction and subsequent retrofit of the CTV building.

The following section outlines the involvement of those individuals identified by the Police as 'Persons Whose Acts or Omissions are in Question regarding Design and/or Construction Faults.

Unless otherwise stated, the information presented in this section is taken from the CERC report without any independent checking.

4.2 David Harding

David Harding graduated from the University of Canterbury with a Bachelor of Engineering (Civil) degree with Second Class Honours in 1973. He became a Registered Engineer in 1976 which it is understood he maintained until and beyond the period of design of the CTV building.

His first period of employment after leaving university was with [redacted] where he remained for four years. During that period he was involved in the design of residential buildings and foundations, single-storey factories, offices, warehouses and school buildings, and the structural strengthening of brick buildings.

Mr Harding left [redacted] in 1978 to begin his first period of employment with Alan Reay, who was operating under the name Alan M. Reay Consulting Engineer (ARCE). While working for Dr Reay, he undertook the design of structural elements of residential buildings and one or two storey industrial and commercial buildings frequently of precast concrete construction.

Mr Harding ceased employment with ARCE to begin employment at the Waimairi District Council as its design engineer in order to gain experience in civil engineering. In that role he was responsible for the design office and supervised a traffic engineer, three civil engineering cadets and two draughtsmen. He was mainly involved in civil engineering, including the design of roundabouts and roads, but undertook some structural engineering related to annual surveys and maintenance of bridges.

After five years at the Waimairi District Council, Mr Harding was keen to move on and gain experience in multi-storey building design. He was approached by Dr Reay after Dr Reay was advised that his then employee and design engineer, John Henry, was leaving ARCE.

In November 1985, Mr Harding left the Council to begin his second period of employment with ARCE, this time in a senior position.

It is reported that Mr Harding had no experience in multi-storey building design up to this point and had not previously used the ETABS analysis program.

[redacted]

While working for ARCE during this second period of employment, but prior to beginning design of the CTV building, Mr Harding worked on the design of several buildings. Two of these were simple low-rise structures that did not require Mr Harding to use modal analysis, and the third was a nine-storey building (Westpark Tower) that he was handed by [redacted] partway through the design process.

In March 1986, Mr Harding began work on the CTV building as the design engineer. During the inquiry, Mr Harding said his involvement started after Dr Reay had met with the client and carried out some level of preliminary calculations and the concept design. It was agreed at the CERC that the CTV building design was the first time that Mr Harding had been fully responsible for an ETABS modal analysis.

Mr Harding carried out the design of the CTV building through to completion. During this time, he attended a seminar on the 'Design of Concrete Structures' at the University of Canterbury in July 1986. Mr Harding also carried out construction observation once construction commenced in late 1986.

Mr Harding was made an Associate of ARCE at some point after the design of the CTV building and left in 1988 to establish his own consulting practice.

4.3 Alan Reay

Alan Reay graduated from the University of Canterbury with a Bachelor of Engineering (Civil) degree with First Class Honours in 1965 and a PhD in Civil Engineering in 1970. His PhD thesis was concerned with the dynamic characteristics of civil engineering structures.

After completing his education and working for two years as a structural engineer with [redacted] in Christchurch, Dr Reay began operating as a sole practitioner in 1971 under the name Alan M. Reay Consulting Engineer (ARCE).

Initially, Dr Reay did not employ any structural engineers. But, as he took on more work, he employed other engineers including Mr Harding and Mr Henry. He also employed several structural draughtsmen.

At the time of the design of the CTV building in 1986, Dr Reay was still operating as a sole practitioner. It was not until 1988 that Alan Reay Consultants Limited (ARCL) was incorporated, with Dr Reay as a director.

In evidence, Dr Reay said that most of his work up to 1986 had related to, and focused-on, single-level precast concrete factories and cold-formed steel design. He was considered a very prominent designer in the area of tilt-slab design according to Mr Henry. Dr Reay also had varying levels of involvement in the design of three multi-storey buildings completed by ARCE before the CTV project came to the ARCE office,

The CTV building project came to ARCE in early 1986 through Dr Reay's prior relationship with the contractor, Williams Construction, which was already on board with the developer (Prime West). The architectural design was underway by this stage. Dr Reay stated during the CERC inquiry that he had an initial meeting with Williams Construction, but beyond this denied any further involvement in the design of the CTV building.

4.4 Graeme Tapper (deceased)

Graeme Tapper was a qualified engineer who was employed by the Christchurch City Council (CCC) as the Assistant Buildings Engineer at the time the documents relating to the CTV building were processed for a building permit. He was the deputy to Bryan Bluck. Together, their role was to assess the design of the CTV building for compliance with the structural aspects of Bylaw 105. Mr Tapper passed away prior to the February 2011 earthquake.

4.5 Bryan Bluck (deceased)

Bryan Bluck was a registered engineer who was employed by the Christchurch City Council (CCC) as the Chief Buildings Engineer at the time the documents relating to the CTV building were processed for a building permit. He was Mr Tapper's immediate superior. Together, their role was to assess the design of the CTV building for compliance with the structural aspects of Bylaw 105. Mr Bluck passed away prior to the February 2011 earthquake.

4.6 Geoff Banks

According to his brief of evidence at the CERC hearing, Geoff Banks graduated from the University of Canterbury with a Bachelor of Engineering (Civil) degree with First Class Honours in 1980. After finishing university, he joined Holmes Wood Poole and Johnstone (1982 to 1986) and subsequently Cambridge Consulting Engineers (1987 to 1988). Mr Banks joined ARCL in late 1988 and became a director in March 1989.

In 1990 and 1991, Mr Banks carried out checks and the engineering design of drag bars installed between the floor and the North Wall Complex on levels 4, 5 and 6 of the CTV building.

4.7 Bill Jones

According to his brief of evidence, Bill Jones was an employee of Williams Construction and held no tertiary qualifications. He entered the construction industry after leaving school at the age of 15 and had approximately 30 years' experience at the time of construction of the CTV building. He carried out the role of Site Foreman during the construction of the CTV building.

The CTV building was constructed between 1986 and 1988. During this time, Williams Construction was replaced by Union Construction Limited with Mr Jones moving to be employed by Union Construction.

4.8 Gerald Shirtcliff

Gerald Shirtcliff was an employee of Williams Construction. He carried out the role of Construction Manager during the construction of the CTV building. According to his statement of evidence, he was 40 at the time of construction of the CTV building.

The CTV building was constructed between 1986 and 1988. During this time, Williams Construction was replaced by Union Construction Limited with Mr Shirtcliff moving across to be employed by Union Construction.

We are aware of media commentary relating to Mr Shirtcliff's qualifications. It is widely reported that Mr Shirtcliff assumed the identity of another individual to use his Bachelor of Engineering degree to gain entry to New South Wales University in Sydney in 1972. Mr Shirtcliff, under the name William Fisher, obtained a Masters in Highway Engineering in 1974 by which time he was working as an engineer and had become a member of Engineers Australia, still using the Fisher name.

[redacted]

5 Description of the Building

The following section presents a description of the CTV building as it was designed in 1986 and modified over its life up to September 2010. This description is based on our interpretation of the available drawings of the building and reference to the available structural calculations. This section does not consider damage to the building suffered as a result of the earthquakes since September 2010. Refer to Section 6 for this content.

5.1 Original 1986 Building

The CTV building was a six-storey commercial office building located at 249 Madras Street, Christchurch, near the corner with Cashel Street (Figure 5.1). At the time of the February earthquake, the occupancy consisted of a car park on the ground floor and several tenanted spaces throughout the building including Canterbury Television (CTV), King's Education, The Clinic, and Relationship Services.

Figure 5.1: The CTV building in 2004, viewed from the south-east (Source: CERC Report 2012)

The plan dimensions of the CTV building floor-plate were 30.25m by 22.5m (Figure 5.2). The inter-storey height was typically 3.24m with a 3.7m high bottom storey (Figure 5.3). The convention used on the original structural drawings defines the ground level as level 1.

The building was founded on in-situ reinforced concrete pad and strip footings bearing on silt, sand and gravels (Soils and Foundations Ltd 1986 geotechnical report). The above-ground structure consisted of in-situ reinforced concrete walls including the 'North Wall Complex' and the 'South Shear Wall' and five suspended in-situ reinforced concrete floor slabs supported by a framing system of in-situ reinforced concrete columns and precast concrete beams. The walls and columns extended above level 6 to support a lightweight steel roof. The convention used in this report for column numbering is shown in Figure 5.4. This is the same approach as taken by the CERC in their report, so we have maintained it for consistency.

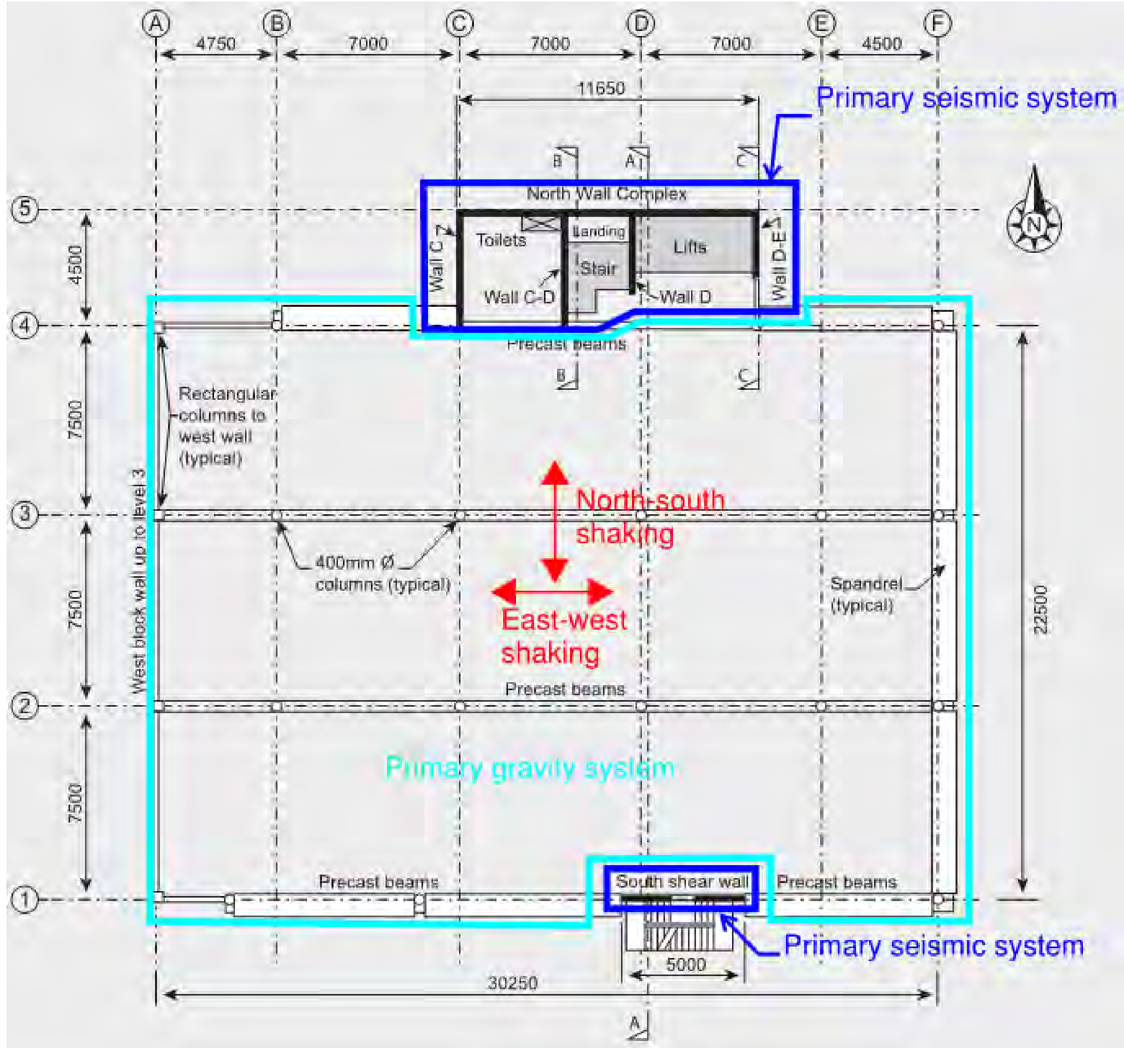


Figure 5.2: Typical upper floor structure of the CTV building (Source of floor plan: CERC Report 2012)

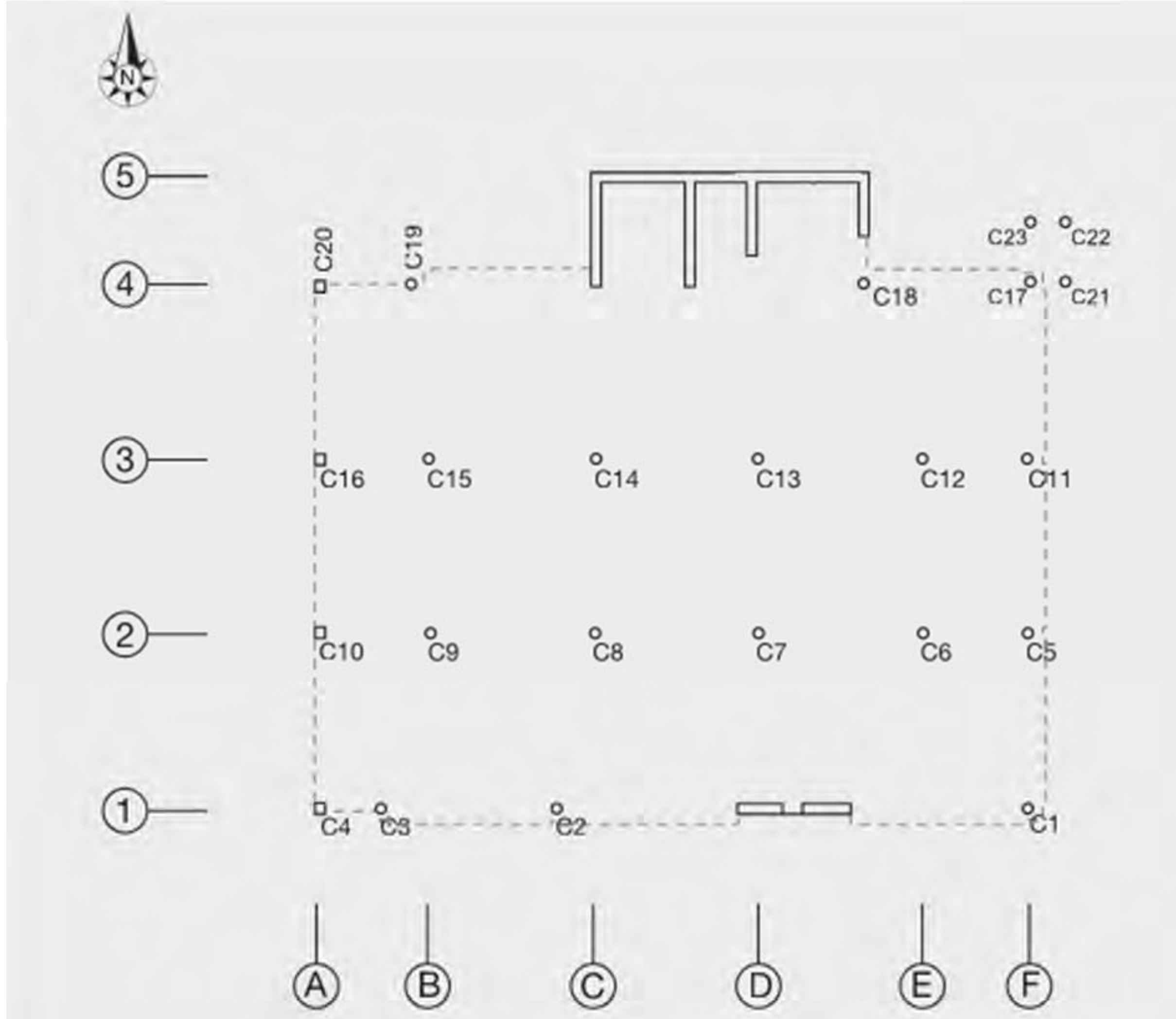


Figure 5.4: Column numbering system (Source: CERC Report 2012)

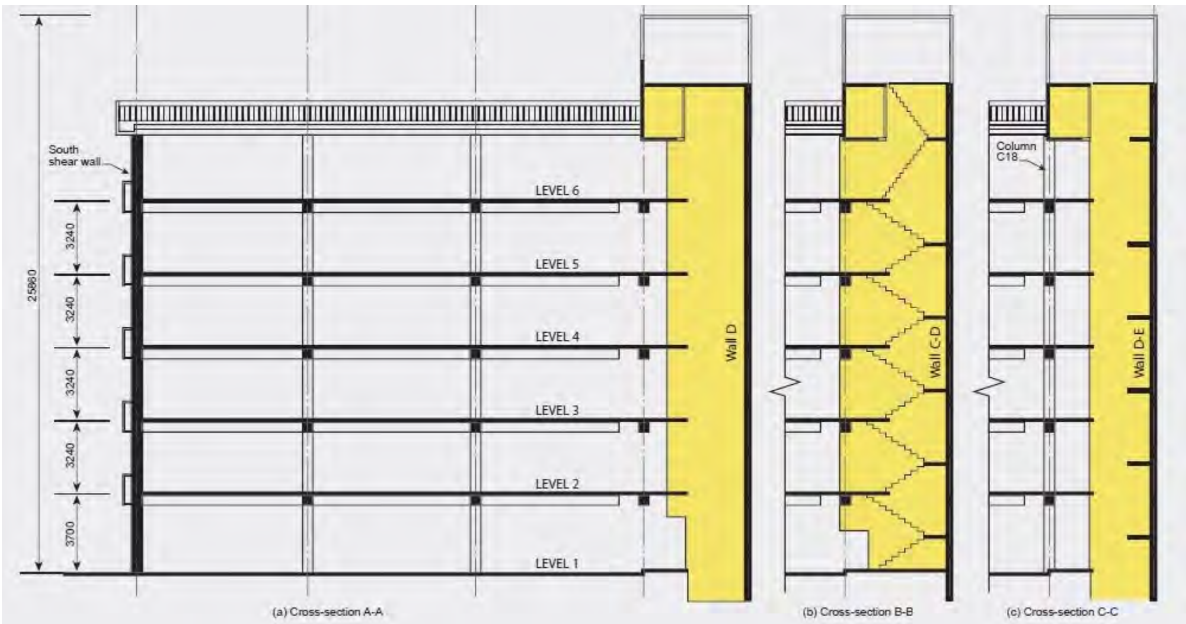


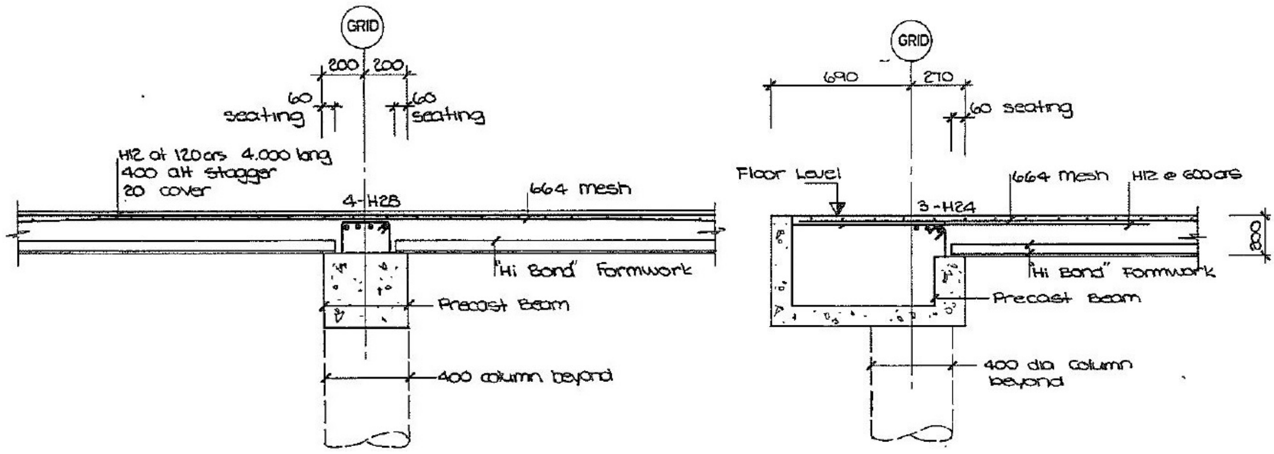
Figure 5.3: Cross-section through the CTV building looking west near grid D (Source: CERC Report 2012)

[redacted]

The seismic structural system of the building has been referred to as a ‘shear wall-protected gravity load system’. Such a system derives the lateral load resistance of the building in both principal directions (north-south and east-west) solely from the shear walls. These shear wall elements are referred to as the primary seismic structural elements with respect to seismic load resistance (Figure 5.2). In the CTV building, the beams and columns provided the majority of the gravity support to the floors and are referred to as the primary gravity structural elements. The primary gravity structural elements were not required to contribute to the strength of the seismic load resisting system, only to remain capable of carrying axial load as the building deformed under lateral seismic loads. The reinforced concrete floor slabs acted as relatively rigid diaphragms to distribute seismic loads to the shear walls, including any torsional (twisting) effects resulting from the eccentricity between the centre of mass and the centre of stiffness/centre of strength.

The suspended floor slabs up to and including level 6 comprised a profiled metal deck on which concrete was poured during construction to form the floors, generally with an as-detailed overall thickness of 200mm. The composite arrangement of metal decking and concrete was the proprietary product Dimond Hi-Bond. The floor slabs were supported by reinforced concrete beams.

The internal beams were precast to the underside of the floor slabs (Figure 5.5a). The perimeter beams on grids 1, 4 and F were shell beams which were filled with in-situ concrete when the floor slab was cast (Figure 5.5b). On grid A at levels 2 and 3 there were precast beams spanning between the columns in the north-south direction.



a) Internal beam

b) Perimeter beam

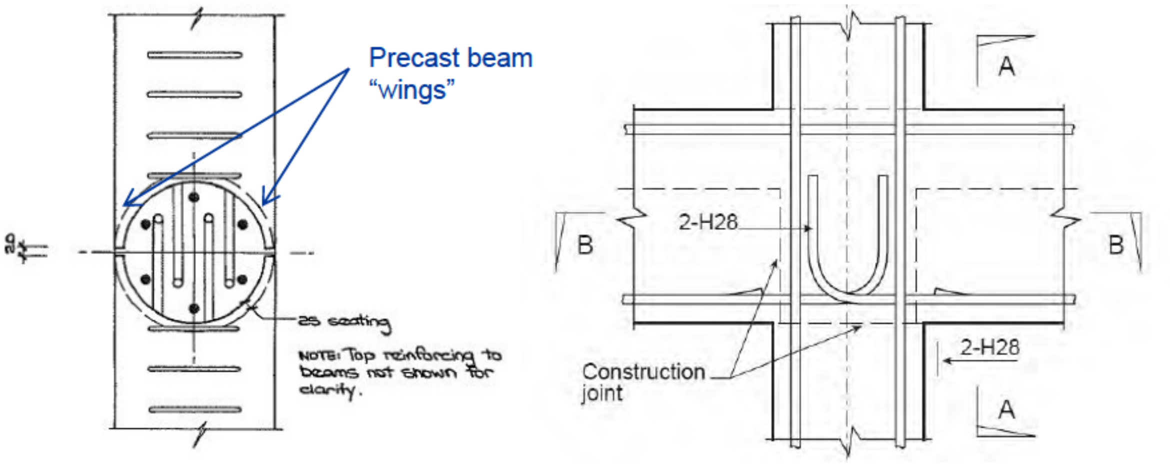
Figure 5.5: Typical internal and perimeter precast beam sections supporting Hi-Bond flooring system (Source: ARCE Structural Drawings 1986)

The beam-column joints generally consisted of two bottom beam bars, 24mm (H24) or 28mm (H28) in diameter, protruding beyond the precast beam section and bent up with a 90 degree hook inside the column. For an internal joint on grids 2 and 3, the bottom beam bar 90 degree hooks were located in the opposite half of the column from where they entered, giving them an overlap with the beam bars entering from the opposite face (Figure 5.6). This was also the case for the external joint on grids 2 and 3 at grid F (Figures 5.7 and 5.8a). However, for an internal joint on grid F, the bottom beam bar hooks were located in the same half of the column from where they entered, giving them no overlap (Figures 5.7b and 5.8b). Due to the circular shape of the columns, the ends of the precast beams had a semi-circular shape, termed “wings” (Figure 5.6a).

Top beam bars were either 24mm or 28mm in diameter and were placed after the precast beam sections were installed. These bars were subsequently encased in in-situ concrete during pouring of the floors and beams.

[redacted]

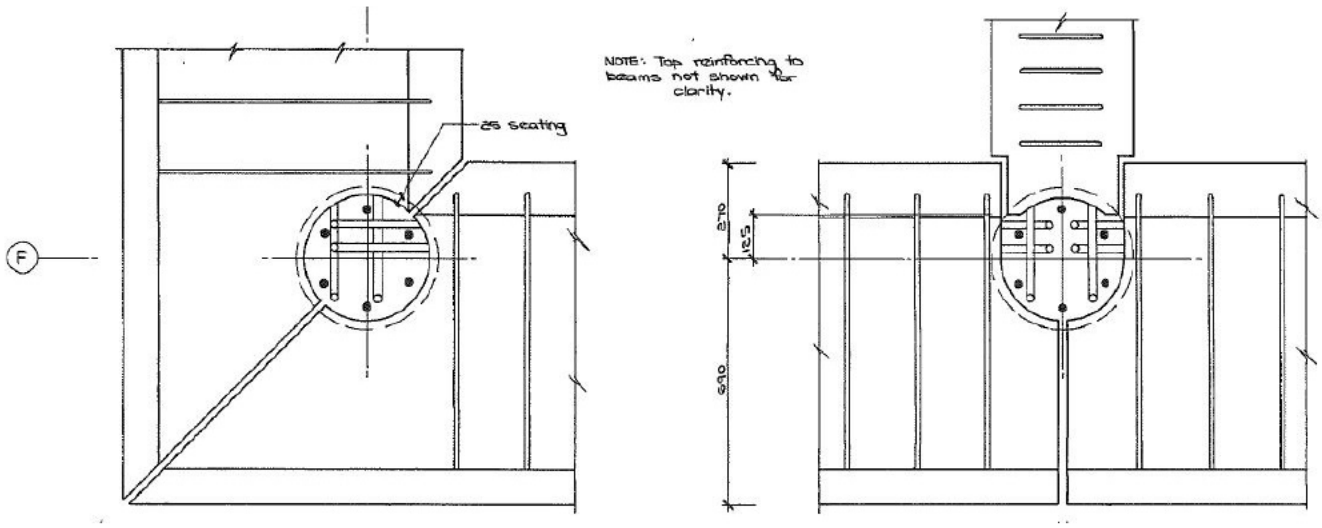
For internal joints, straight top beam bars were used (Figures 5.6b and 5.8b) and for external joints it is likely that 90 degree hooks were used, similar to the detail indicated for the bottom bars (Figure 5.8a). These top beam bar hooks are not specifically detailed on the drawings.



a) Plan (Source: ARCE Structural Drawings 1986)

b) Elevation (CERC Report 2012)

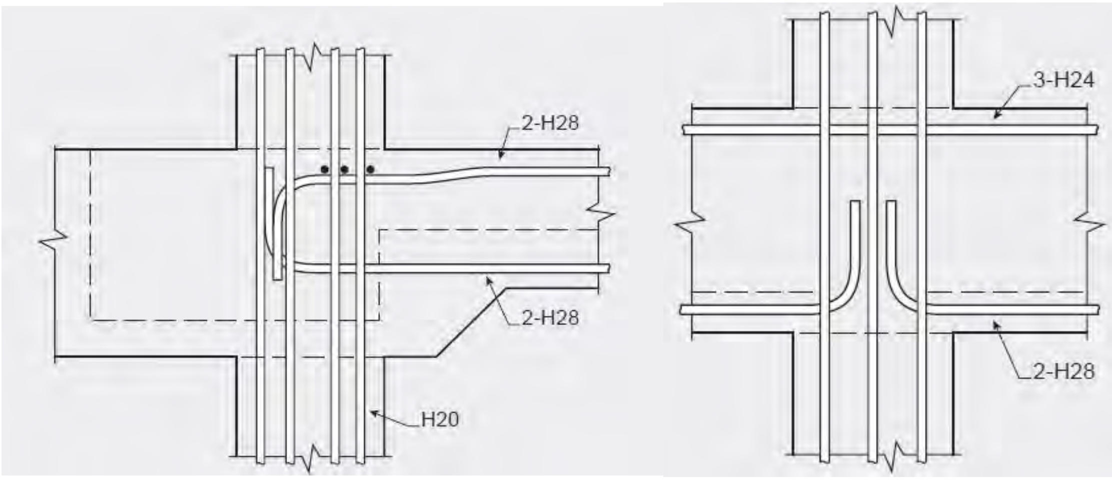
Figure 5.6: Typical internal beam-column joint details on grids 2 and 3



a) Plan of external corner joint on grids 1 and F

b) Plan of internal joint on grid F

Figure 5.7: Typical perimeter beam-column joint plans on grid F (Source: ARCE Structural Drawings 1986)



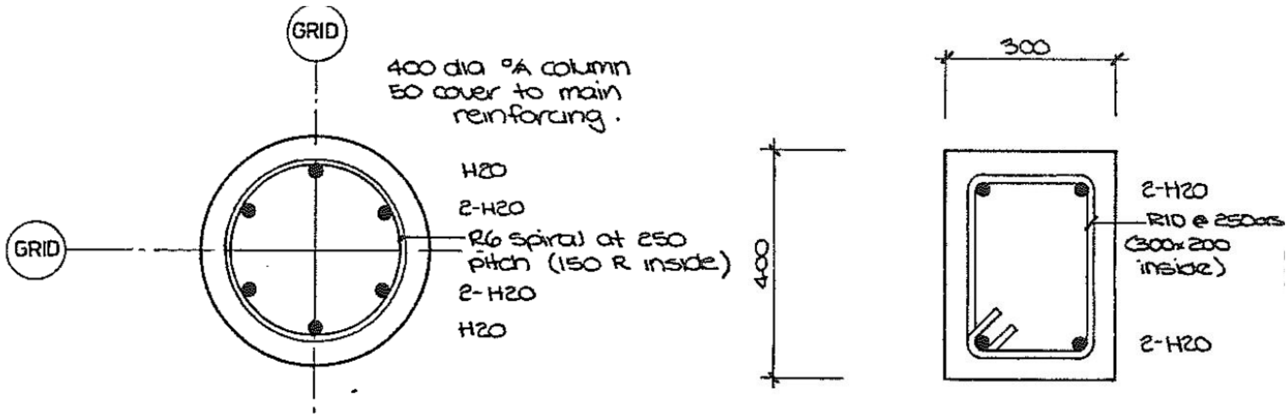
a) Elevation of external joint on grids 2 and 3

b) Elevation of internal joint on grid F

Figure 5.8: Typical perimeter beam-column joint elevations (Source: CERC Report 2012)

Reinforced concrete masonry infill walls were constructed between the columns and beams on the first three levels on the western side of the building (grid A). These infills were detailed with 25mm gaps to the concrete structure.

The majority of beams were supported by circular in-situ reinforced concrete columns of 400mm diameter (Figure 5.9a). The columns were typically circular and detailed the same over the full height of the building with six 20mm diameter vertical steel reinforcement bars surrounded by 6mm diameter horizontal steel spirals at 250mm centres. On grid A, the columns were rectangular (Figure 5.9b) and one interior column (C18) was square, but only on level 1.



a) 400mm diameter column section

b) 400 x 300mm column section

Figure 5.9: Typical column details (Source: ARCE Structural Drawings 1986)

The roof structure was lightweight steel comprising cold-formed steel purlins supported on rolled steel joists which were in turn supported on the reinforced concrete columns and shear walls that extended above level 6. Lateral earthquake resistance to loads at roof level was provided by a combination of the cantilevering reinforced concrete columns and the shear walls. Roof cross-bracing comprising horizontal steel flats was provided to distribute loads to the columns and walls.

Reinforced concrete spandrels were placed between the perimeter columns at each level above level 1 on grids 1, 4 and F. These spandrels were detailed with 10mm gaps to the columns at each end.

5.2 Structural Modifications after 1986

The following section outlines the structural modifications to the building after completion of construction noted by the CERC. Many of these were minor including:

- Holes drilled in floor slabs for services. Some were in locations where, after the collapse, the slab was seen to have pulled away from the North Wall Complex
- Addition of block walls in some parts of the ground floor, which were built in 1991 and largely removed in 2000
- Alteration to the entry foyer and canopy at the north-east corner
- Telecommunication equipment added to the roof
- Addition of a Lundia shelving system
- Lightweight partitions on all floors.

In our opinion, these minor modifications would not have had a material influence on the manner in which the CTV building collapsed.

There were two significant structural modifications that had the potential to affect the performance of the building in an earthquake. Their significance will be expanded upon in subsequent sections of this report. These modifications were:

Installation of Drag Bars

Steel angle drag bars were fitted to the floor slabs and walls at grids D and D-E of levels 4 to 6 in October 1991 (Figure 5.10). The drag bars consisted of steel angle sections bolted to the underside of the slabs and to the shear walls.

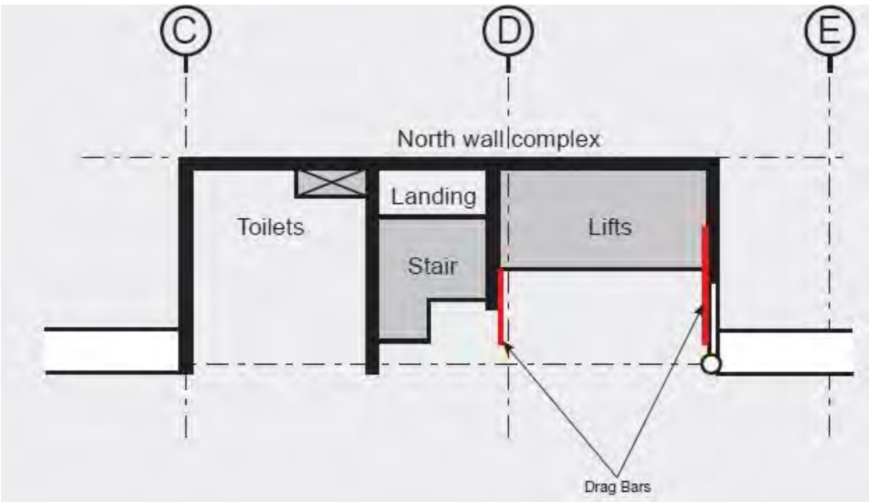


Figure 5.10: Locations and extent of drag bars installed on levels 4, 5 and 6 at walls D and D-E (Source: CERC Report 2012)

Stair Penetration

A stair penetration was cut in the floor of level 2 during a fit-out in 2000 (Figure 5.11).

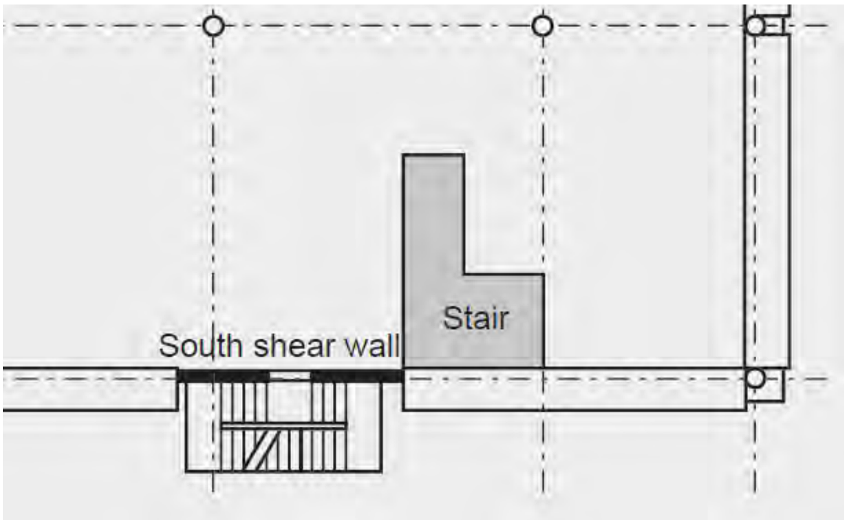


Figure 5.11: Location of stairwell penetration cut in level 2 floor slab in 2000 (Source: CERC Report 2012)

6 Observations from the Earthquakes

6.1 Introduction

The Canterbury region was struck by an initial 7.1M_w earthquake near Darfield on 4 September 2010, followed by a large number of aftershocks. The most significant of these, over the period up to the 22 February 2011 (Christchurch) earthquake, included an earthquake on 26 December 2010 (Boxing Day earthquake).

The CTV building endured the September and Boxing Day earthquakes without collapse but collapsed catastrophically in the February 2011 earthquake, with the loss of 115 lives.

In this section of the report we briefly outline the characteristics of each of the three significant earthquakes as well as a description of the observations of damage to the CTV building resulting from each event. Our opinion regarding the likely sequence of collapse is outlined in Section 12 of this report and an explanation of the significance of the size of the three earthquakes is provided in Section 11 of this report.

Much of the content regarding post-earthquake observations has been taken from the CERC report.

6.2 Darfield Earthquake (4 September 2010)

6.2.1 Earthquake Characteristics

On 4 September 2010, at 4:35am, an earthquake of 7.1M_w struck Christchurch and the surrounding Canterbury region. Its epicentre was near Darfield, about 40km west of Christchurch on a previously unknown fault beneath the Canterbury Plains. GNS Science advised in its report to CERC that this was a rare event that had occurred in an area where previous seismic activity was relatively low for New Zealand. The predominant direction of shaking was reported to be approximately north-south in this event (CERC 2012).

6.2.2 CTV Building Observations

The CTV building endured the earthquake on 4 September 2010 without collapse.

In addition to observations made by occupants of the building, formal inspections of varying levels of detail were carried out on the CTV building between 4 September and 26 December 2010. These included:

- CCC-initiated Level 1 Rapid Assessment carried out on 5 September 2010 by a CCC building consent officer, a Chartered Professional Engineer, a USAR officer and an unidentified person
- CCC-initiated Level 2 Rapid Assessment carried out on 7 September 2010 by three senior CCC building officers
- Damage-assessment carried out in late-September 2010 by a Chartered Professional Engineer, initiated by the CTV Building Manager.

A green placard was posted for the CTV building following both the Level 1 and 2 Rapid Assessments.

Observations from the inspections and occupants generally indicated:

- Cracks in plasterboard linings
- Cracks and/or separation along joints between block walls and the surrounding structure
- Cracked glass around the perimeter
- Cracks in plasterboard of internal partition walls

- Sagging floors
- Horizontally-oriented cracks in the concrete column at level 6 outside the lifts (Column C18)
- Fine cracking in isolated columns
- Cracking in the walls and stairwells of the North Wall Complex at several floor levels
- Fine diagonal cracking in some beams
- Fine diagonal hair-line cracking in the plaster coating on the inside of the South Shear Wall at level 2
- A fine diagonal crack on the outside of the South Shear Wall at level 1.

In addition to the observations of damage, some occupants reported concerns about floor movements having increased following the earthquake. Demolition of the neighbouring building was also carried out from October 2010 which produced vibrations in the building felt by the occupants.

6.2.3 Conclusion

Our summary of the damage indicated above is that the building suffered some minor damage to non-structural elements indicative of building movements during the 4 September 2010 earthquake. Some minor cracking was observed to structural elements which is also indicative of building movements, but there were no observations that indicate major inelastic demand in the structural system. With respect to potential damage to the internal columns in the ground floor, our physical testing has suggested that these columns were unlikely to have exhibited visible cracks following the 4 September 2010 earthquake.

It is common following significant earthquakes for building occupants to have a heightened sensitivity to vibrations and awareness of building distortions (e.g. sags in floors) and damage (e.g. cracks). It is possible that the observed higher vibrations and floor sagging after September were at least in part a manifestation of this. If the observations did indeed correspond to a change in building behaviour following the September earthquake, this could be attributed to cracking near the supports of the floor slabs. This type of cracking would increases the flexibility of the floor system that in turn could lead to increased sag and potentially more easily perceptible movements.

6.3 Boxing Day Earthquake (26 December 2010)

6.3.1 Earthquake Characteristics

On Boxing Day 2010 a sequence of aftershocks struck directly under the Christchurch CBD. Although the magnitudes of these aftershocks were relatively small, the epicentre was within the CBD and mostly occurred at depths of 3.7–7.0km. The sequence began with a moment magnitude (M_W) 4.7 earthquake at 10:30am, followed by magnitude (M_L) 4.6 and 4.7 events later that day. The initial earthquake was the most significant and is referred to as the Boxing Day earthquake. The epicentre was located 1.8km north-west of the Christchurch Cathedral at a depth of about 4km.

6.3.2 CTV Building Observations

The CTV building endured the earthquake on 26 December 2010 without collapse.

In addition to observations made by occupants of the building, formal inspections were carried out on the CTV building between 26 December 2010 and 22 February 2011. These included:

- CCC-initiated Level 1 Rapid Assessment carried out on 27 December 2010 by a CCC building inspector and an unknown team
- USAR Rapid Visual Assessment carried out on 27 December 2010 by a member of the New Zealand Fire Service and a colleague
- Inspections to provide a quotation for concrete crack repair by an employee of [redacted] in early-2011

- Inspections to provide a quotation for concrete crack repair by an employee of [redacted] in February 2011.

A green placard was posted on the CTV building following the Level 1 Rapid Assessment. The observations of the USAR members are also consistent with the allocation of the green placard from the CCC inspection.

Observations from the inspections and occupants generally indicated:

- Further cracking of glass around the perimeter
- Potentially further cracking to plasterboard of internal partition walls and linings
- Gaps between the steel window frames and the surrounding concrete structure that had potentially widened
- Potentially further cracking in the walls and stairwells of the North Wall Complex at several floor levels
- Potentially further cracking of isolated internal columns
- Potentially further damage to the lintel of wall D-E at level 7 near its connection to column C18 (Figure 6.1).

Figure 6.1: Cracking to column C18 and lintel of wall D-E at level 7 (Source: [redacted])

In addition to the observations of damage, some occupants reported concerns about even more movement of the floors following the Boxing Day earthquake.

6.3.3 Conclusion

Our summary of the damage indicated above is that the building suffered some further minor damage to non-structural elements indicative of building movements during the Boxing Day earthquake. Some further cracking to structural elements may also have occurred, including the area of connection between column C18 and the lintel of wall D-E. This area was placed under stress as the building displaced with components in the north-south direction. However, it is not believed that the lateral strength of the building was significantly compromised, even if this connection between column C18 and the wall above had been completely lost.

As for the September earthquake, there were no observations that indicate major inelastic demand in the structural system as a result of the Boxing Day earthquake.

6.4 February Earthquake (22 February 2011)

6.4.1 Earthquake Characteristics

The most destructive of the earthquakes to strike Christchurch occurred at 12:51pm on 22 February 2011 on what is now commonly referred to as the Port Hills Fault. Of magnitude 6.2M_w, the rupture occurred on a north-east/south-west oriented fault at a shallow depth, reaching to within one kilometre of the surface. The resulting ground motions were high. The existence of this fault was unknown before the February earthquake, but there had been some aftershock activity in this area prior to the 22 February event. This earthquake led to the collapse of the CTV building. The predominant direction of shaking was reported to be approximately east-west in this event (CERC 2012).

6.4.2 CTV Building Observations

The effect of the February earthquake on the CTV building was sudden and catastrophic. The building collapsed rapidly and almost completely, effectively ‘pancaking’.

Following the collapse on 22 February 2011, examinations of the building debris were made by engineers who were part of the USAR effort. This started immediately when the primary objective was rescue and recovery but continued in the days afterwards in an effort to gather evidence about the manner in which the building collapsed. It should be noted that much of the photographic evidence gathered by these engineers was taken after debris had been shifted during the rescue and recovery effort. However, it gives a useful indication of the state of the building post-collapse. A summary of this follows below.

6.4.2.1 Overall Building State

After collapse, the remnants of the building were resting mainly within its own footprint, with the North Wall Complex still standing, the floor slabs lying on top of each other, and the South Shear Wall having fallen inwards on top of the roughly 4m high debris pile (Figure 6.2). Remnants of floor slabs were leaning up against the North Wall Complex.

Figure 6.2: Photo taken immediately following collapse (Source: [redacted])

The only areas where debris appeared to fall outside the main footprint of the building were on the east and north sides, with debris having fallen on parked cars in Madras Street (Figure 6.2) and several metres north of their original position on grid 4 (Figure 6.3).

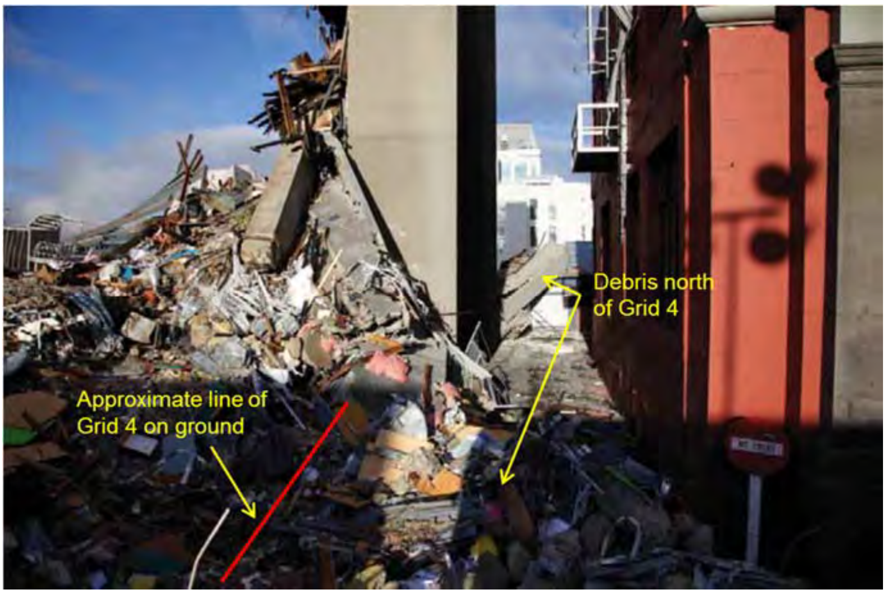


Figure 6.3: North-east corner of the building looking west showing debris lying north of grid 4 (Source: [redacted])

6.4.2.2 Columns

Due to the pancaking nature of the collapse, the majority of the internal columns were not sighted after collapse, and it is likely they were reduced to rubble during the collapse sequence. However, a number of perimeter columns and interior columns above level 6 remained intact. Therefore, those segments of columns that were photographed were likely to be from the perimeter of the building. One investigator gave evidence to CERC that the observed column segments generally had damage only at their ends with the central section largely intact (Figure 6.4).

Figure 6.4: Perimeter column with intact central portion (Source: [redacted])

Column C4 in the south-west corner of the building remained standing up to what appears to be level 3, with concrete missing from the joint zone at level 2 (Figure 6.5).

Figure 6.5: Column C4 standing post-collapse up to level 3 (Source: Hyland/Smith 2012)

Column C19 near the north-west corner was observed having fallen towards the north with a segment resting up against the neighbouring building (Figure 6.6).

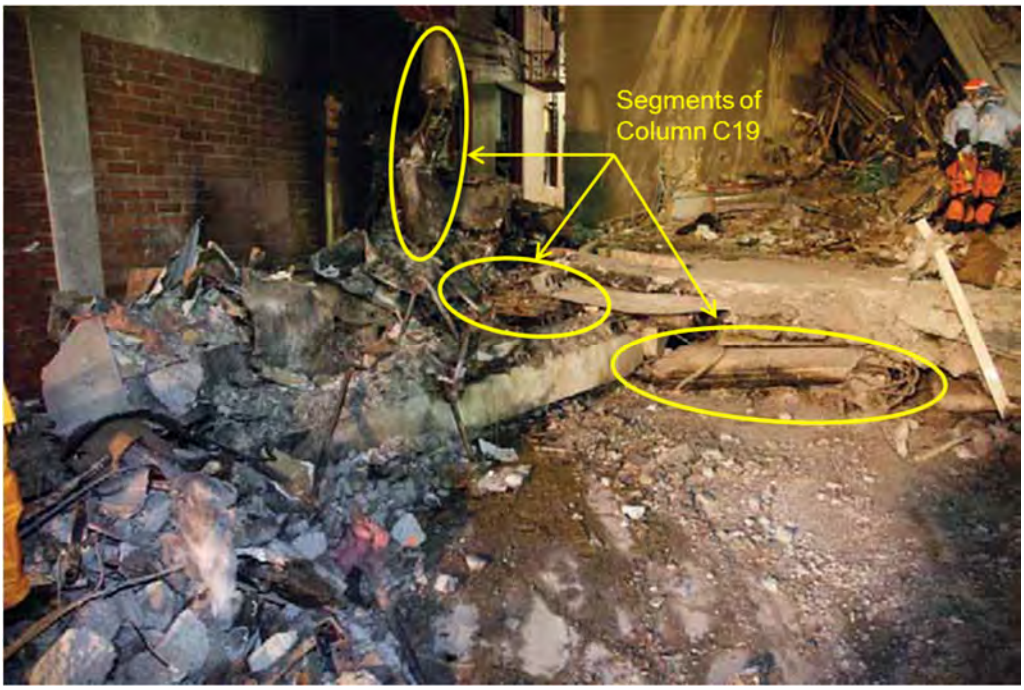


Figure 6.6: Column C19 fallen towards the north (Source: [redacted])

6.4.2.3 Beams

The beams were largely intact with the damage typically concentrated at the ends, and the central section intact. Generally, the floor slabs had detached from the beams with the failure plane aligning with the tops of the beam stirrups and the top of the precast section (Figure 6.7).

Figure 6.7: Interior beam showing failure plane around top of stirrups (Source: [redacted])

6.4.2.4 Beam-Column Joints

The investigators reported that they did not see any beam-column joints intact except at level 6. They also did not see any evidence of transverse reinforcement in the joint zones (Figure 6.8). In observing the beam-columns joints, the investigators also noticed the wings at the ends of the precast beams had mostly broken off and the beam ends were smooth where they met the in-situ face of the column.

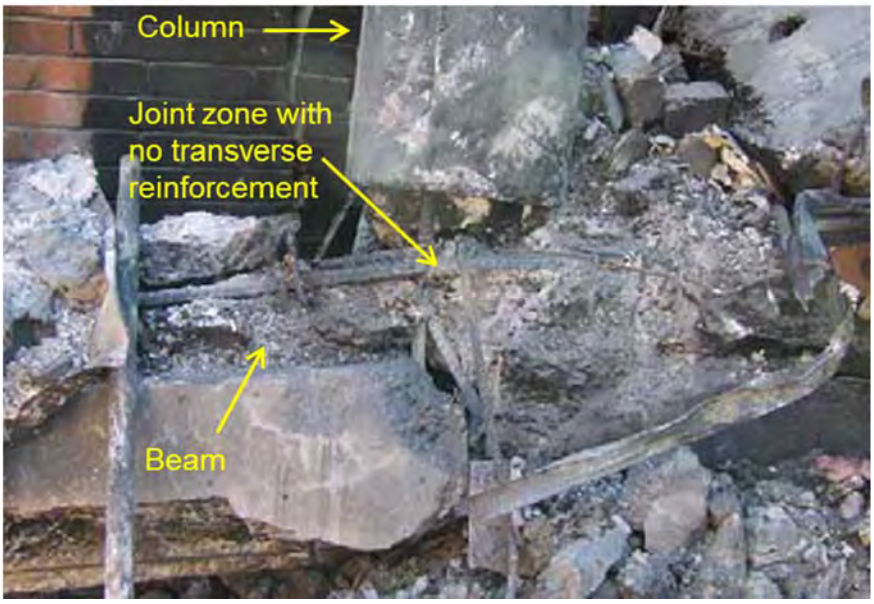


Figure 6.8: Beam-column joint region with no transverse reinforcement (Source: [redacted])

6.4.2.5 Floor Slabs

In their collapsed state, the floor slabs were generally observed to have disconnected from the Hi-bond metal decking. Near the north side, the floor slabs came to rest leaning against but disconnected from the North Wall Complex. Near the south side, the floor slabs had disconnected from the South Shear Wall and come to rest near the base of the wall.

6.4.2.6 South Shear Wall

The South Shear Wall was found lying on top of the debris pile, having bent over out-of-plane (i.e. about its weak axis) at level 2. Horizontal and diagonal cracks were observed in the piers near the base of the wall in the section that remained upright (Figure 6.9). Diagonal cracks were also observed in the piers from level 2 to level 4 and in the coupling beam below level 3. Only very minor cracking was observed in the coupling beam below level 2.

The CERC report presented evidence from a USAR engineer who concluded that the cracks near the base of the wall were indicative of out-of-plane bending and that, due to the lack of significant damage to the coupling beams, surmised that the South Shear Wall saw little in-plane loads. This is contrary to the Hyland/Smith conclusions that the cracks near the base of the wall were indicative of the wall behaving as a flexural cantilever in-plane. The CERC report made an assessment of the coupling beam strengths relative to the base of the wall and concluded that the South Shear Wall would have acted in-plane predominantly as a single unit, which is consistent with the observed cracking.

Figure 6.9: South Shear Wall cracking near the base (Source [redacted])

6.4.2.7 North Wall Complex

The North Wall Complex remained standing after the collapse of the remainder of the building. It was in good condition with little damage, except where the beams on grid 4 had pulled away from the walls. Some horizontal cracking was observed at the base of the wall and vertical cracking in the north-south oriented wall segments adjacent to the grid 5 segment.

6.5 Conclusions

The CTV building endured two significant earthquakes in September and December 2010 without collapse, but collapsed catastrophically as a result of the shaking experienced in the February 2011 earthquake.

During the September earthquake, the CTV building suffered minor damage to non-structural and structural elements, but there were no signs to indicate major inelastic demand in the primary seismic structural system.

During the Boxing Day earthquake, the CTV building suffered possible further minor damage to non-structural and structural elements, but again there were no signs to indicate major inelastic demand in the primary seismic structural system.

As a result of shaking experienced in the February earthquake, the CTV building collapsed in a pancake fashion with the floors lying on top of each other, the majority of internal columns completely destroyed, with little apparent damage to the primary seismic structural elements, noting the South Shear Wall collapsed once it was no longer supported in the out-of-plane direction. A likely sequence of collapse is provided in Section 12 of this report.

7 Original 1986 Design

7.1 General

In 1986, Alan Reay (operating as a sole practitioner under the name Alan M. Reay Consulting Engineer (ARCE)) was invited to be the structural engineer for the building, referred to at the time as the 249 Madras Street development.

ARCE was formally engaged by Williams Construction in June 1986 and instructed to prepare drawings for permit and construction. Dr Reay assigned Mr Harding, a structural engineer employed by him, to the project and provided him with preliminary drawings from the architect.

A permit for the building was granted by the Christchurch City Council (CCC) on 30 September 1986. Included in the documentation submitted to the CCC was a set of structural drawings signed only by Mr Harding. This set of drawings was stamped ‘Approved Subject to the By-Laws’ by CCC.

The following section discusses the approaches taken by Mr Harding and Dr Reay and the requirements of the codes and other prevalent literature of the day. Identification is made of any errors/failings in their respective approaches before an assessment of whether these represent an omission to discharge their duty, and whether or not the omissions were major departures from accepted practice of the day.

In order to inform on errors in relation to technical aspects of Mr Harding’s design, we carried out analyses and calculations using codes of the day and techniques that were common and available to designers of the mid-1980s, and similar to what we understand were used by Mr Harding. Key inputs and results of our analyses are presented in Appendix F.

7.2 Approach by Individuals Involved

This section outlines the approaches taken by Mr Harding as the designer of the building, and Dr Reay as the sole principal of ARCE.

7.2.1 Approach by Mr Harding

Our review of the original design included review of the available 1986 design calculations prepared by Mr Harding and the structural drawings that were approved for building permit. The calculations do not include details of the computer analyses that were completed, but they do contain the derivation of some input parameters we have assumed were used in the analyses, and also some of the results of the analyses.

We note the form of the building as assumed in the original CTV calculations varies slightly from that shown on the drawings, particularly the height of the first storey. Where we consider it relevant in subsequent sections we have noted the effect of any differences we have observed between the final building configuration and that assumed in the calculations.

We have assumed the version of the structural drawings available to us, which is stamped by Christchurch City Council as “Approved Subject to the Bylaws”, is that which was used for the construction of the building.

7.2.1.1 Conceptual Intent

While not explicitly stated, Mr Harding’s calculations identified two separate systems with respect to seismic load resistance; the primary seismic structure and the primary gravity structure. He also separated out the foundations into their own design package as well as miscellaneous secondary elements.

Primary Seismic Structure

The building's primary seismic structure comprised of the North Wall Complex and the South Shear Wall, as shown in Figure 5.2. These elements were designed to resist the entire design lateral seismic load.

It is apparent the in-situ floor slabs at levels 2 to 6 were assumed to form (effectively) rigid diaphragms that distributed lateral seismic forces at each level to the vertical primary seismic elements (the North Wall Complex and South Shear Wall). This was (and still is) the common design practice of the day and the computer analysis programs (such as ETABS) were also based on this assumption.

For north-south shaking, the entire lateral seismic load was assumed to be carried by the North Wall Complex. The North Wall Complex in this direction consisted of four relatively slender walls of similar length (walls C, C-D, D and D-E as defined in Figure 5.2) that were designed to develop flexural hinges at their base (i.e. acting as ductile cantilever shear walls). Due to their similar length, the base hinges in each wall could be expected to form almost simultaneously. Any torsion induced by lateral loading in the north-south direction would then be resisted by the couple created by the North Wall Complex and South Shear Wall separated by a lever arm of approximately 25 metres.

For east-west shaking, the lateral seismic loads were shared between the North Wall Complex and the South Shear Wall, with the distribution of loads resisted dependent on the relative location of the centre of rotation to each. In the elastic range, the North Wall Complex was much stiffer than the South Shear Wall, which would normally mean it would attract a higher proportion of the load. However, due to torsional effects, redistribution of elastic actions from the North Wall Complex to the South Shear Wall could be expected to occur. The South Shear Wall was designed as a ductile coupled shear wall with capacities reasonably close to the design demands, whereas the North Wall Complex in this direction consisted of an 11.65m long C-section wall with strength capacity well in excess of the calculated demand.

For loading in both directions, the floor diaphragms and their connections were required to distribute forces between the primary seismic elements (the North Wall Complex and South Shear Wall) to resist torsional effects.

Storey forces were designed to be transferred between the floor slabs and the North Wall Complex for loading in the east-west direction through the connecting piece of toilet slab east of grid C. Storey forces were intended to be transferred between the floor slabs and the South Shear Wall for loading in the east-west direction through the approximately 30m combined length of grid 1 perimeter beams and the wall itself, with the beams acting to drag the forces to the wall. There are no calculations available to convey on the designer's intention regarding transferring storey forces to the North Wall Complex for loading in the north-south direction.

Foundations

Below ground level, seismic loads were designed to be resisted by a series of reinforced concrete pads and ground beams. A 2m deep inverted-tee in-situ ground beam was provided under the South Shear Wall. The ground beam extended 7.5m either side of the South Shear Wall to pick up columns C2 and C1 from above. This ground beam was designed for the overstrength actions of the South Shear Wall above. The North Wall Complex was supported on a 500mm deep raft and four 1m wide by 1.8m deep ground beams that extended from under the ends of walls C, C-D, D and D-E to pick up the columns on grid 3. The ground beams were designed to be ductile in outrigger action between the North Wall Complex and grid 3 columns in the north-south direction. For north-south shaking, yielding was allowed for in both these ground beams and the above-ground potential wall hinge zones. The raft was designed to remain elastic for loading in the east-west direction.

[redacted]

Primary Gravity Structure

All of the building's columns and the beams framing into them made up the primary gravity structure. The flooring system consisted of Hi-bond metal decking with 200mm thick in-situ concrete spanning in a north-south orientation onto beams on grids 1, 2, 3 and 4. The beams spanned between the columns and walls that ultimately transferred the gravity loads to the ground through the pads and strip footings.

The drawings indicate the beams generally consisted of a precast lower portion with longitudinal bars that protruded from their ends and bent up in a 90 degree hook within the joints. The upper portions of the beams were detailed as a zone of in-situ concrete, the same depth as the slab, with top longitudinal beam bars running continuously through the joint. The columns were detailed as cast in-situ concrete.

As detailed on the drawings, construction would have involved placing these precast portions on the columns that had been previously poured up to the level of the beam soffit, with the longitudinal column bars passing up through the joint region and extending above the finished floor level as starters. The metal decking, top beam reinforcement, slab reinforcement and joint reinforcement was to be placed before in-situ concrete was poured to form the slab and beam-column joint.

During the CERC hearings, Mr Harding gave evidence that:

"The beams were designed to be continuous beams and as such were designed to be moment resisting only between adjacent beams for gravity loading. The columns were not intended to be part of a moment resisting frame, and the ends of the columns were designed as pin joints. Consequently the beam-column joints were not designed to carry any bending moment from the columns, and any contribution which these columns may make toward the building lateral stiffness was not relied upon."

Mr Harding assumed that the primary gravity structure did not contribute to the strength or stiffness of the seismic force resistance of the whole building, and that the beam-column joints and columns themselves would not attract any significant lateral seismic shears.

Miscellaneous Secondary Elements

The concrete masonry infill walls on grid A up to level 4 were not designed to contribute to the lateral seismic force resistance of the building. 25mm gaps between the columns and the infill walls were detailed to achieve this separation.

The concrete spandrel panels on grids 1, F and 4 were not designed to contribute to the lateral seismic force resistance of the building. 10mm gaps to the columns at both ends of the panels were detailed to achieve this separation.

7.2.1.2 Design of Primary Seismic Structure

The primary seismic structure consisted of the North Wall Complex and the South Shear Wall. In order to carry out analyses of the primary seismic system for seismic design, Mr Harding used the computer program ETABS. Since ARCE did not have the facilities to do so at their offices, this was carried out at the University of Canterbury.

It is implied in the original calculations that Mr Harding carried out the analysis for earthquake loads assuming that these were carried by the North Wall Complex and the South Shear Wall only (i.e. ignoring the contribution to the stiffness of the lateral force resisting system from the primary gravity framing members). This was a valid approach at the time.

It is also implied the primary seismic system was modelled as fixed at the ground level, with no consideration made in the seismic section of the original calculations for any flexibility from foundation beams or soil supports. This was a valid approach and normal practice at the time.

The steps taken by Mr Harding to design the primary seismic structure, as observed from the original calculations, involved determination of:

1. Basic seismic coefficient using assumed period.
2. Seismic weight of each level.
3. Building geometry.
4. Stiffness properties of the various primary seismic structural elements.
5. Typical floor centre of mass and centre of rigidity.
6. Distribution of equivalent static forces up the height of the building.
7. Ground floor shears and moments in the primary seismic elements.
8. Check on assumed periods.
9. Building deflections.
10. Dynamic and static analyses scaling.
11. Detailed design of primary seismic elements.

We will provide a brief summary of Mr Harding's approach, including key values, for each step.

As noted earlier, we have carried out our own analyses and calculations of the primary seismic system. We have used the structure as detailed on the drawings and techniques as close as possible to what was available and common at the time, and what is understood to have been used by Mr Harding. This included building a simple model in ETABS. This has allowed us to provide comparisons with the results of Mr Harding's analysis. Details of our analyses are provided in Appendix F.

Basic Seismic Coefficient

Loadings were derived from the loadings code NZS 4203:1984. Based on an assumed natural period of vibration (period) for the building of less than or equal to 0.7 seconds in each direction of loading, Mr Harding obtained a basic coefficient of C_d = 0.1.

Based on an assumed period of 0.7 seconds, we obtain the same value.

Seismic Weight

Mr Harding calculated a value for the total seismic weight of the building of 33,014kN.

Our value for the total seismic weight of the building is 31,500kN (i.e. within 5% of Mr Harding's value).

Building Geometry

Mr Harding set out the building geometry for use in the analysis of the primary seismic structure.

Our analyses used the geometry as shown on the drawings. One area of discrepancy identified is the height of the first storey (3.9m in Mr Harding's calculations versus 3.7m on the drawings). This most likely occurred as a result of development of the building layout after the structural calculations were prepared. The net effect of this discrepancy on the analysis results is not considered significant.

Stiffness Properties of the Primary Seismic Elements

Mr Harding used an effective section modifier of $0.6I_g$ to account for cracking for all primary seismic elements.

We used $0.6I_g$ for all wall elements and $0.4I_g$ for the coupling beams. The net effect of this discrepancy on the analysis results is not considered significant.

Note: I_g refers to the moment of inertia of the gross (or total) concrete section about an axis through its centroid. This property of a section, together with the elastic modulus of the material it is made from, determines its flexural stiffness.

Centre of Mass and Rigidity

Mr Harding calculated the coordinates of the centre of mass (COM) for a typical floor. We have not seen calculations for the coordinates of the centre of rigidity (COR), but a location is indicated on page S3 of the original calculations.

ETABS provided values for the COM and COR for our model of the primary seismic structure. Figure 7.1 shows the range of locations of the COM and COR from our model. Mr Harding's typical COM and COR values are within our ranges indicated.

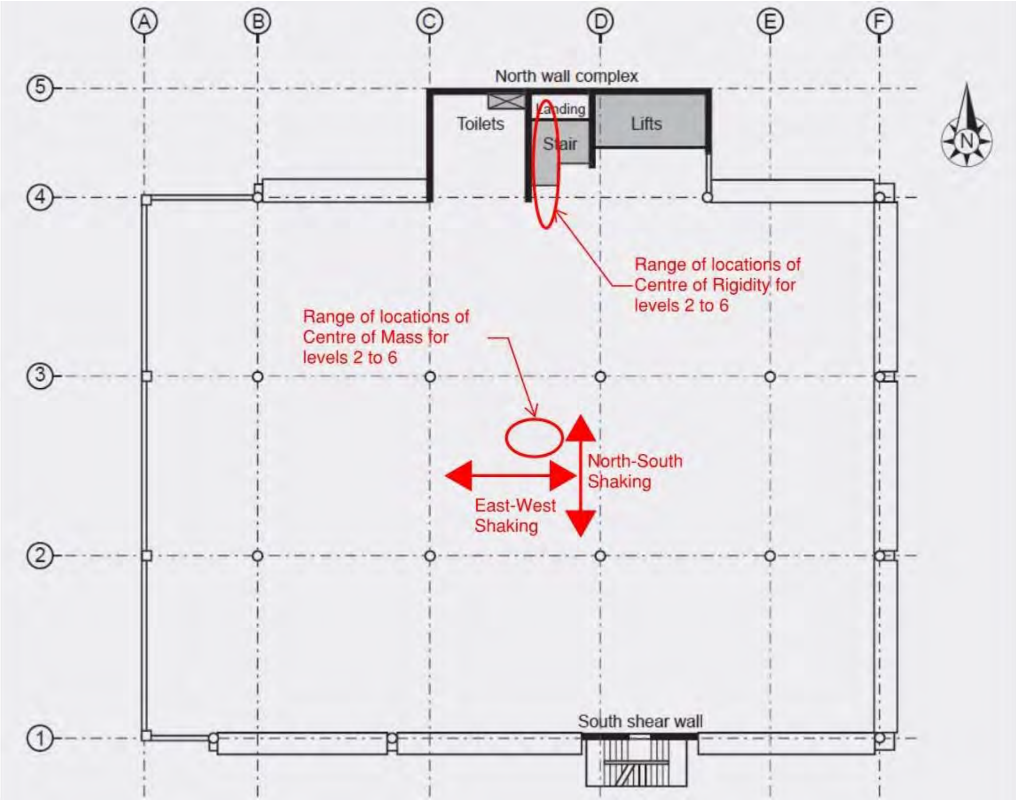


Figure 7.1: Locations of the COM and COR of Beca ETABS model

Base Shear and Distribution over the Height of the Building

Using a basic seismic coefficient of 0.1 in both directions, Mr Harding obtained a base shear of 3,301kN. This base shear was distributed over the height of the building in accordance with the code for the equivalent static method, applying an additional 10% of the base shear at the roof.

In our analyses, we considered level 6 to be the uppermost significant level of seismic weight and applied the additional 10% at this level. The approach taken by Mr Harding is conservative with respect to overturning moment demands on the shear walls.

Ground Floor Shears in Primary Seismic Elements

Mr Harding elected to carry out both a 3D equivalent static and 3D spectral modal analysis in each direction. The effects of accidental eccentricity were considered in line with the code by offsetting either the centre of mass or the point of application of storey force by 0.1b (10% of the building dimension in the direction perpendicular to the direction of loading) depending on the type of analysis.

Ground floor shears in the primary seismic elements from each load case were presented in the calculations. In the north-south direction, the North Wall Complex takes the entire load. This is consistent with our analyses. In the east-west direction, the North Wall Complex takes between 40 and 60% of the total load depending on the direction of accidental eccentricity considered. This is also consistent with our analyses and suggests the torsional behaviour of our model and Mr Harding's was similar.

Check on Assumed Periods and Scaling of Base Shears

Mr Harding presented periods for the north-south and east-west directions of loading. It is not clear where these have been obtained from, but it is likely that they come from the ETABS dynamic analysis output.

Mr Harding reported periods of 1.06 seconds in both the north-south and east-west directions. Since these reported periods are greater than the assumed period of 0.7 seconds, he calculated a period scaling factor for static loads, equal to the ratio of seismic coefficients calculated for the two different periods. This factor was calculated as 0.712, and was used to scale the equivalent static method results reported thereafter. This led to a base shear in each direction of loading of 2,350kN.

Our analyses indicate fundamental mode translational periods of 0.8 seconds and 0.3 seconds in the north-south and east-west directions respectively. This leads to equivalent static base shears of 2,840kN and 3,150kN. The value reported by Mr Harding for both directions (2,350kN) is between 75 and 85% of these values.

An explanation for the period we calculated being lower (0.8 versus 1.06 seconds) in the north-south direction can be explained by the lower seismic weight and stiffer structure we used in this direction (due to the lower bottom storey). However, there is a large discrepancy in periods for the east-west direction (0.3 versus 1.06 seconds). This suggests that either Mr Harding obtained the period for this direction incorrectly from the ETABS output, or the structure used in his analysis was far more flexible. A review of the building deflections (carried out below) suggests the former.

Building Deflections

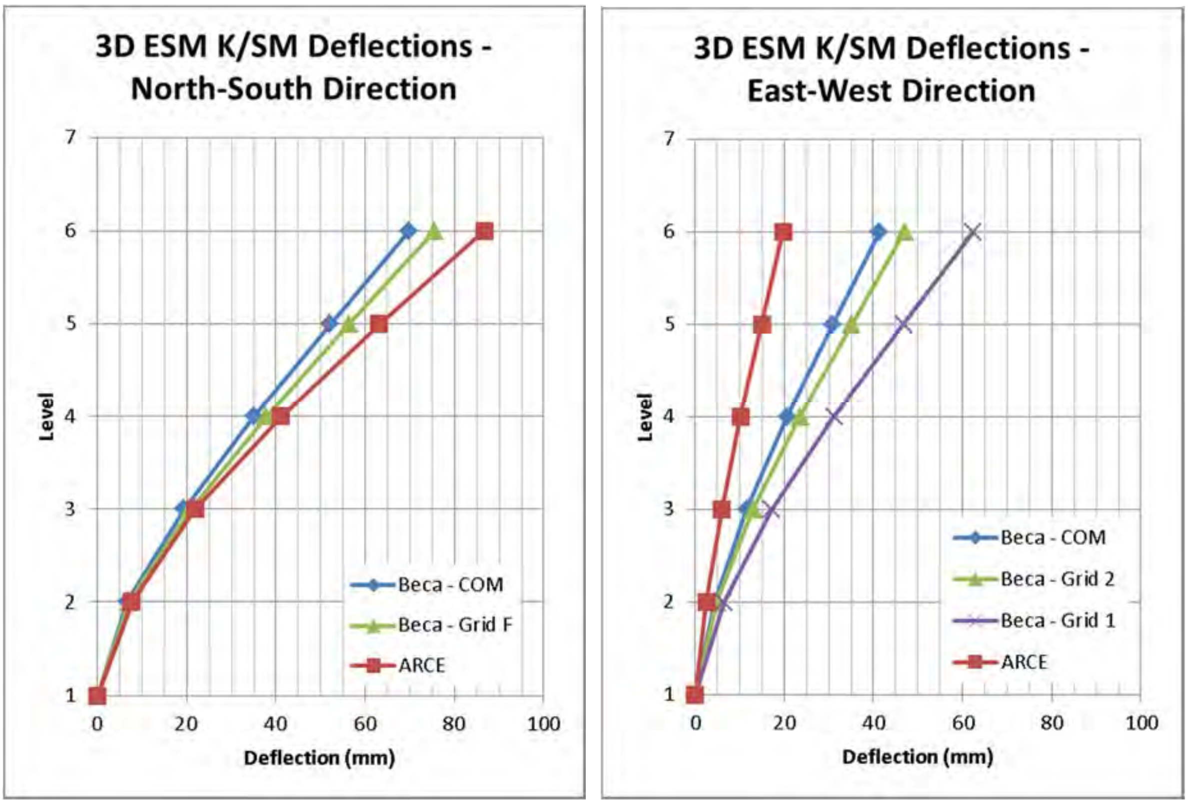
Mr Harding reported building deflections obtained from analyses for both the north-south and east-west directions of loading. For each direction, deflections are reported for two cases representing the positive and negative directions of COM offset (COM offset is a requirement of the loadings code to account for torsional effects that may arise accidentally). It is not clear at what point on the structure the deflections are being reported. It is also not clear from what analysis (equivalent static or spectral modal analysis) the deflections were obtained.

In the absence of this information, we have assumed Mr Harding was reporting deflections at the COM of each level from the 3D equivalent static analyses, as this is the most likely scenario.

[redacted]

Mr Harding initially reported deflections using the loadings derived from a base shear of 3,300kN. He subsequently applied the period scaling factor of 0.712 and an amplification factor of 2.5 (as required by the code representing K/SM) to obtain relevant deflection values.

We have calculated the deflections of our model using 3D analyses in the same manner. We monitored deflections at the COM as well as grids F (north-south loading) and grids 1 and 2 (east-west loading). Comparisons of our deflections with those of Mr Harding's are shown in Figure 7.2. For north-south loading we have only considered the case where the centre of mass is offset to the east. For east-west loading we have only considered the case where the centre of mass is offset to the south.



a) North-south direction

b) East-west direction

Figure 7.2: 3D Equivalent static method K/SM scaled deflections

Figure 7.2a shows that the deflections calculated by Mr Harding in the north-south direction are higher than our calculated values for both the centre of mass and grid F. Our results also show that the building is not very susceptible to torsional rotations for loading in the north-south direction because the centre of mass and grid F deflections are similar. Overall, the deflections calculated by Mr Harding in the north-south direction are higher than those we have calculated.

Figure 7.2b shows that the deflections calculated by Mr Harding in the east-west direction are much lower than our calculated values at the centre of mass (by a factor of 2). Our results also show that the building is susceptible to torsional rotations for loading in the east-west direction because the deflections at grid 1 are significantly larger than at the centre of mass. Mr Harding was reporting a much higher period in the east-west direction than our analysis shows, and yet had smaller deflections, which is the reverse of what would have been expected (i.e. higher period, greater implied flexibility, higher deflections expected). This suggests that the period reported by Mr Harding for the east-west direction is likely to be in error, and hence the period scaling factor of 0.712 was not justified. Overall, Mr Harding has significantly underestimated the deflections of the building in the east-west direction. Mr Harding also failed to report values at grids offset from the centre of mass which would have produced even larger deflections.

[redacted]

Dynamic and Static Analyses Scaling

Before embarking on the detailed design of the primary seismic elements, Mr Harding calculated scale factors for both the 3D modal and 3D equivalent static analysis results.

For the 3D modal analysis in each direction, Mr Harding scaled the modal base shear to 90% of the equivalent static base shear (for C_d = 0.1 and an assumed period of 0.7 seconds) and then applied the period scaling factor and an additional 0.8 factor to obtain a base shear of 1,690kN. We have already mentioned the period scaling factor determined by Mr Harding was too low, but the extra 0.8 is also unjustified. The net effect is that the modal base shears obtained by Mr Harding were between 60 and 65% of the values we calculated.

For the 3D equivalent static analysis in each direction, Mr Harding scaled the equivalent static base shear (for C_d = 0.1 and an assumed period of 0.7 seconds) by the period scaling factor and also by an extra 0.8 factor to obtain a base shear of 1,880kN. The net effect is that the equivalent static base shear obtained by Mr Harding (1,880kN) for both directions was also between 60 to 65% of our values (3,150kN and 2,840kN).

Detailed Design of Primary Seismic Elements

After scaling the load cases as above, Mr Harding obtained the ground floor shears in the primary seismic elements and determined that the reduced equivalent static loads governed for each case. The primary seismic elements were then designed on this basis.

When designing the South Shear Wall, Mr Harding further reduced the design actions by a factor of 0.8 equivalent to the S factor for a coupled shear wall, while the North Wall Complex was designed for S of 1.

We have not carried out a detailed review of the design of the primary seismic elements. From a brief review, the extent of reinforcement and detailing designed by Mr Harding is generally appropriate to meet the demands he obtained and the ductility he assumed was required. However, due to errors in calculating the demands as noted above, we believe the walls as-designed (with the exception of the North Wall Complex in the east-west direction) had strengths between 60 to 75% of that required by the code (i.e. the walls were weaker than required by the code).

7.2.1.3 Design of Diaphragm Connections

Mr Harding appears to have considered only the east-west direction of loading for the design of the diaphragm connections to the North Wall Complex and South Shear Wall.

For this direction, Mr Harding took the maximum storey force from the equivalent static method (879kN at level 6) and reduced this by the period scaling factor (0.712) and an additional 0.8 factor to obtain a design storey force of 501kN. He then considered that 60% of this could go to either of the North Wall Complex or the South Shear Wall to give a connection design force of 300kN. He did not consider overstrength actions.

For the South Shear Wall connection on grid 1, Mr Harding calculated the required connection force per metre length. He then calculated the strength provided by the slab concrete and the 664 mesh and concluded that the force could easily be transferred. Mr Harding did not check the ability of the grid 1 beams to act as drag beams to take these forces to the South Shear Wall at each level. In particular, he did not provide calculations for the connection of the beams into the South Shear Wall on each side.

For the North Wall Complex connection, Mr Harding calculated the shear demand in a 1.4m length of slab using a force of 30,000N and concluded this to be “OK” (Figure 7.3). It is not clear why he used 30kN instead of 300kN, which suggests a possible numerical error. It is also not clear why he used a slab length of 1.4m, when the drawings indicated a longer length was available. It is possible the layout of the toilet slab may have changed subsequent to these calculations being carried out.

He then calculated a bending moment demand in the 1.4m length of slab using a force of 30kN and a lever arm of 1m. Again, it is not clear why he used 30kN instead of 300kN. It also not clear why he used a lever arm of 1m when the drawings indicated a longer length (approximately 2.4m) should have been used.

A discussion of the appropriate method for determining design diaphragm connection forces and the values we obtained follows in Section 7.3.

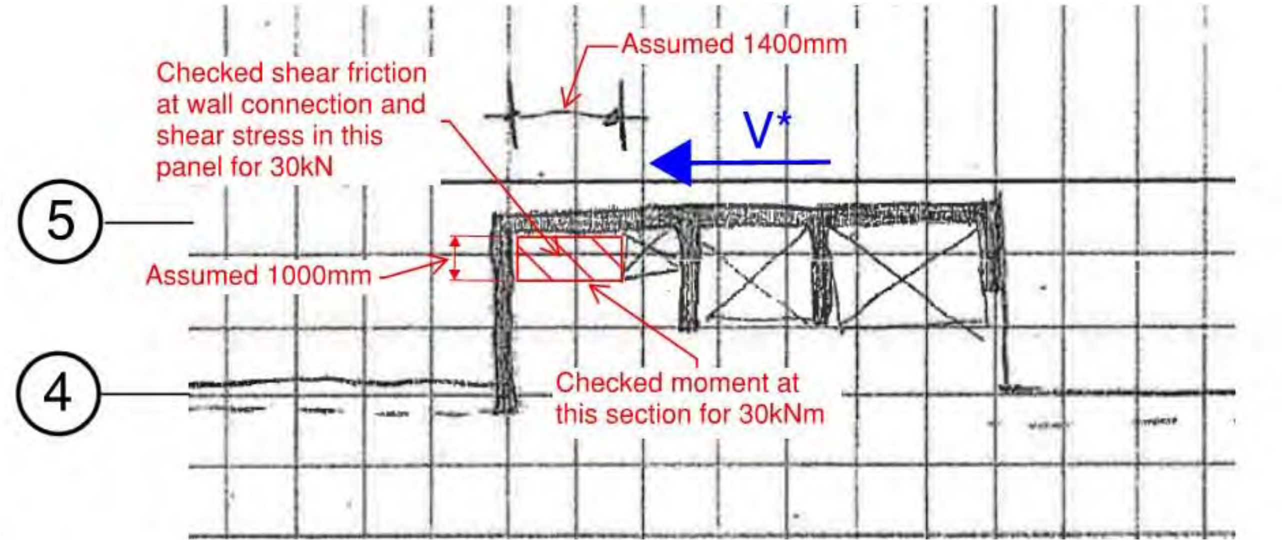


Figure 7.3: Approach by Mr Harding to design NWC diaphragm connection (Source: ARCE Structural Calculations 1986)

7.2.1.4 Design of Foundations

We have focussed only on the approach taken by Mr Harding to design the foundations to resist seismic loads.

In general, Mr Harding designed the foundations in a two-step process:

1. Checks on allowable bearing pressures, using code-specified seismic actions, to size the footings.
2. Detailed design of the footings themselves, including determining the required reinforcement.

For the South Shear Wall, Mr Harding designed a 2m deep inverted tee ground beam that extended 7.5m either side of the wall to pick up columns C1 and C2. He used the overstrength moment of the coupled shear wall above as an input and designed the ground beam accordingly.

For loading of the North Wall Complex in the north-south direction, Mr Harding designed ductile ground beams that extended from under walls C, C-D, D and D-E to a grid 3 ground beam. These ground beams were required to perform as outriggers and had a flexural capacity similar to the wall above. The shear design of the ductile ground beams was based on the overstrength capacities of the ground beams themselves.

For loading of the North Wall Complex in the east-west direction, Mr Harding designed the raft extensions beyond the extent of the above-ground wall complex for the bearing pressures that were obtained from code-specified overturning moments, with an additional 40% "*to provide a factor of safety against stamping action*".

We have not carried out a detailed review of the design of the foundation elements to resist seismic demands as obtained by Mr Harding. In general, particularly for east-west loading on the South Shear Wall and north-south loading on the North Wall Complex, Mr Harding appears to have followed through overstrength actions from the superstructure appropriately to design the foundation elements. However, due to errors noted above, we believe the demands Mr Harding had calculated to be between 60 to 65% of those required.

7.2.1.5 Design of Primary Gravity Structure

The primary gravity structure included the beams, columns and beam-column joints.

Mr Harding's design of these elements was carried out in a section of his calculations entitled 'Gravity Elements'. Mr Harding began by designing the floor system and continuous beams on grids 1, 2, 3 and 4 for gravity loads. He then proceeded to consider the columns.

The process Mr Harding followed to design the columns involved:

1. Axial load take-down for all columns.
2. Evaluation of slenderness effects for columns braced against sidesway.
3. Determination of required column longitudinal reinforcement and concrete strength for 1.4D + 1.7L load case using column design charts, up the height of the building.
4. Design of the typical column spiral using the non-seismic provisions of the concrete code, obtaining a R6 at 250mm pitch.
5. Comparison of the spiral design to what was required using the seismic provisions of the concrete code, obtaining a R6 at 40mm pitch.

After determining what was required for the spiral design from the seismic provisions, Mr Harding made the comment:

"*these do not apply as columns are non-seismic".*

He then elected to remain with the spiral design from the non-seismic provisions.

There are no calculations available for the design of the beam-column joints. The only indication of what was intended for the reinforcement in the beam-column joints was on drawing S14 where the R6 spiral at 250mm pitch is shown to extend down into the joint zones from the columns above.

7.2.1.6 Design Documentation

The structural drawings have a space in the title block to record sign-off, labelled 'Approved'. In the case of the signed permit drawings, the initials appear to be those of Mr Harding. There are no spaces to record sign-off by those who were responsible for drafting the drawings or checking either the drawings or the engineering.

7.2.2 Approach by Dr Reay

Our review of the approach taken by Dr Reay has been based on evidence presented at the CERC hearings.

Dr Reay was engaged to carry out the structural design of the building. In this role he was responsible for the service to the client, including delivering a code compliant design for the building structure. Since he employed staff to assist with delivery of the project, his duty included ensuring the allocation of appropriate staff to the project, including the provision of technical guidance as necessary and ensuring adequate review of the design and its documentation.

The extent to which Dr Reay carried out these duties in relation to the CTV building cannot be independently determined with certainty. Summaries can only be gleaned from the testimonies of individuals involved, with some discrepancies and differences in recollections to be expected.

Our summary of Dr Reay's involvement with the project, as inferred from the CERC report, is as follows:

1. Commissioned the job through a prior relationship with the Contractor.
2. Allocated Mr Harding as the engineer to the project and introduced him to the Contractor.
3. Allocated a draftsman and any other staff as required to assist the design team.
4. A possible involvement in the decision to include the South Shear Wall as a structural element for resisting lateral loads.
5. Being available for informal consultation with Mr Harding.

In total, Dr Reay recorded 3.5 hours of time against the CTV building (CERC Report 2012).

In his statement of evidence, Dr Reay makes the following comments regarding his involvement:

"*With my approval as manager and following an agreement being reached with Williams in terms of a professional services contract, Mr Harding took on the responsibility for this project for ARCE*."

"*Mr Harding would have carried the project through, including dealing with Council, any site visit and any other necessary attendances. I would not have been involved in any of these stages in more than a minor way*."

"*I do not recall reviewing the drawings, calculations or specification and I would not have expected to have done so.*"

Dr Reay also makes the following comments regarding review processes within ARCE:

"*There was no review procedure in place at that time*."

"*Traditionally ARCE did not include a design review and, as a small firm, relied on the Council review process"*

The CERC report also states:

"*Dr Reay also said that he remembered making sure Mr Harding understood that he did not have the time or knowledge to be able to assist him in depth…"*

7.3 Code Requirements and Other References

The following section outlines the codes and prevalent literature structural engineers were using to inform their designs of multi-storey reinforced concrete buildings in New Zealand in the mid-1980s. Together, these are construed as forming the minimum standard to which engineers were expected to design.

The CTV building was designed using the then current loadings standard, NZS 4203:1984, and concrete materials standard, NZS 3101:1982.

[redacted]

7.3.1 Codes and Prevalent Literature

The 1970s and 1980s represented a period of significant development in the understanding of the behaviour of buildings in earthquakes and in the practices that should be applied to the structural design of buildings for earthquake effects.

Modern earthquake design procedures/philosophies were taught in the New Zealand universities from the late 1960s, but anyone graduating prior to that date was reliant on technical papers, seminars and contact with the universities to develop their knowledge and skills in the new methodologies. This required a significant commitment to maintain currency from structural engineers who found themselves in this position.

The earthquake loadings standard, NZS 4203, which was a significant departure from what it followed, had been published in 1976 but materials standards compatible with the new philosophies were being developed throughout the 1970s and 1980s and not available (other than in draft) until the 1980s and 1990s. The concrete standard NZS 3101 was not published until 1982 and a fully compatible steel standard not until 1992. NZS 4203 was revised in 1984 and introduced further changes to design procedures.

As a consequence of this period of rapid change, designers were reliant also on sources other than the current standards to keep up to date. We recall that a major source of knowledge of the latest developments in seismic design methodologies was the Bulletin of the New Zealand National Society for Earthquake Engineering (NZNSEE). Our practitioner interviewees all confirmed that the NZNSEE bulletin was a well-thumbed resource of the time for designers of reinforced concrete multi-storey buildings. Articles of particular relevance to the design of the CTV building would have been:

- Shear walls – *The Analysis and Design of and the Evaluation of Design Actions for Reinforced Concrete Ductile Shear Wall Structures* by Paulay and Williams, published in Volume 13, No. 2, June 1980
- Foundations – *Foundations for Shear Wall Structures* by Binney and Paulay, published in Volume 13, No. 2, June 1980
- Diaphragms – *Diaphragms in Seismic Resistant Buildings* by Kolston and Buchanan, published in Volume 13, No. 2, June 1980
- Frames – Various papers in the late 1970s
- Beam-column joints – Various papers in the late 1970s

Publications such as *Reinforced Concrete Structures* by Park and Paulay (1975) also provided structural designers with background material.

When reviewing the original design of the CTV building and the retrofit design in the early 1990s, we have considered that the design standards and these seminal papers and texts together define accepted design practice of the time for multi-storey building designers. This was confirmed by the practitioners of the day who we interviewed.

7.3.2 Code Intent

The intent of shear wall protected gravity load systems, as described by Paulay and Williams (1980), was:

"*To provide adequate structural ductility and capability to dissipate energy for the case when the largest disturbance to be expected in the region does occur. Extensive damage, perhaps beyond the possibility of repair, is accepted under these extreme conditions, but collapse is prevented*."

The intent of the code was that all structural elements must be ductile, as stated in NZS 4203:1984 clause 3.2.1:

"*The building as a whole, and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life, shall be designed to possess ductility...*"

The level of ductility required was specified in both the loadings standard NZS 4203 and in the materials standards of the time. The concrete standard NZS 3101 allowed minimal detailing for ductility in some circumstances when certain requirements were met (refer 7.3.6).

7.3.3 Structural System Symmetry

The first section in the earthquake provisions of the loadings code (NZS 4203:1984 clause 3.1.1) concerned structural system symmetry, and advised that the primary seismic resisting elements should be located symmetrically about the centre of mass of the building.

The commentary to this clause had a warning relevant to systems such as that of the CTV building:

"*Notwithstanding the availability of modern computers, considerable uncertainty exists in selecting a mathematical model representing the true behaviour of complex arrangements such as combinations of geometrically dissimilar shear walls and unsymmetrical combinations of shear walls and frames.*"

7.3.4 Separation of Primary Seismic and Primary Gravity Structure

The concrete design standard, NZS 3101:1982, allowed designers to choose between what was primary and 'secondary' structure with respect to seismic force resistance. Note: reference in NZS 3101:1982 to 'secondary structural elements' is taken to apply to 'primary gravity structure' as used in this report, and is contradictory to the definition of 'primary elements' in NZS 4203:1984 which includes beams and columns. Primary seismic structure was designed to carry all of the design lateral seismic loads for the structure as a whole. The primary gravity structure, within prescribed constraints, could be assumed to carry the gravity (vertical) loads only with no requirement to carry a specific seismic load. However, if this distinction was made, NZS 3101:1982 required the primary gravity structure to be able to undergo the lateral seismic displacements of the primary structure while maintaining its ability to carry vertical loads as required by NZS 4203.

This was stated in NZS 4203:1984 commentary clause 3.2, including a non-mandatory suggestion to check the primary gravity system framing elements at four times the specified loading:

"*It will be necessary to check that members not required by design to be part of the horizontal force resisting system have adequate vertical load carrying capacity when the seismic resisting system deforms to resist the earthquake. Such checks should include a check on column and beam adequacy at four or more times the distortion from the specified loading*."

We note that the NZS 4203:1984 code requirement (clause 3.8.1) was to consider deformations twice the distortion from the specified load and, therefore, half this recommendation.

7.3.5 Design of Foundations

The design of the CTV building allowed for ductile yielding in the foundation. There were no limitations in the codes of the day regarding the provision of ductile foundation structures.

The loadings code also allowed for the foundation to be capacity designed, as clause 3.3.7.2.1 of NZS 4203:1984 states:

"*… foundation systems shall be designed to preclude foundation failure, or uplift of an entire foundation element, at loadings corresponding to yielding of the earthquake dissipating elements...*"

NZS 4203:1984 clause 3.3.7.3 also limited the load the foundations were required to be designed for:

"*… no foundation system need be designed to resist forces and moments greater than those resulting from a horizontal force corresponding to SM=2.*"

7.3.6 Design of Primary Gravity Structure

Those primary gravity elements which were not detailed for separation, and were subject to loadings induced by deformations of the primary seismic system, were classified as 'Group 2'.

Once the allocation was made, Group 2 primary gravity elements were required to be detailed to allow for the level of ductile behaviour implied. NZS 3101:1982 provided the choice of three levels of ductile detailing; the non-seismic provisions, limited-ductile provisions or seismic provisions. Clause 3.5.14.3 set out the method for determining which provisions should apply with respect to Group 2 primary gravity elements.

Our interpretation of clause 3.5.14.3 is as follows:

- The non-seismic provisions could be used when the actions in the primary gravity element being considered were determined by imposing the computed lateral seismic deformations of the primary seismic structure (i.e. amplifying by K/SM), and the primary gravity element remained elastic (i.e. the actions did not exceed dependable strengths).
- The limited ductile provisions could be used when the actions in the primary gravity element being considered were determined by imposing half the computed lateral seismic deformations of the primary seismic structure (i.e. amplifying by 0.5*K/SM), and the primary gravity element remained elastic.
- The seismic provisions had to be satisfied when the primary gravity element being considered did not remain elastic when sustaining the actions determined by imposing half the computed lateral seismic deformations.

If elements were required to be detailed for ductility, limited capacity design could be used to ensure that other elements meeting at a joint would remain elastic.

7.3.7 Diaphragm Connections

The method to be used for the calculation of actions in the connections of diaphragms to primary seismic elements was not well described in the codes of the day.

Clause 3.4.6.3 of NZS 4203:1984 reads:

"*Floors and roofs acting as diaphragms and other principal members distributing seismic forces shall be designed in accordance with clause 3.4.9. Allowance shall be made for any additional forces in such members that may result from redistribution of storey shears*."

Clause 3.4.9 relates to the use of the parts and portions provisions to determine diaphragm forces. However, there is no further guidance in NZS 4203:1984 on how to allow for additional forces from the redistribution of storey shears.

Clause 10.5.6.1 of NZS 3101:1982 reads:

"*Diaphragms … shall be designed for the maximum forces that can be resisted by the vertical primary load resisting system, or for forces corresponding with the seismic design coefficients specified by NZS 4203 for parts and portions of buildings, whichever is smaller*."

This clause has introduced the concept of capacity design for the design of diaphragms, but still does not require the forces to be larger than those given by parts and portions.

In addition, the well-regarded 1980 bulletin paper on diaphragms by Kolston and Buchanan refers to two types of diaphragms:

- Type I: Simple Diaphragms - regular diaphragms with no redistribution
- Type II: Transfer Diaphragms - diaphragms that transfer forces due to a vertical irregularity.

The paper recommends using the parts and portions provisions for Type I diaphragms and a more rigorous approach for Type II diaphragms considering the shear demand at a level in a primary element and factors for dynamic magnification and moment capacity overstrength (i.e. a capacity design procedure).

We consider the diaphragms in the CTV building for loading in the north-south direction to be Type I, appropriate for the use of parts and portions.

Due to the offset of the COM and COR in the east-west direction of loading, the system is horizontally irregular in this direction and requires significant horizontal redistribution of shear through the diaphragm. This is not consistent with the definitions for either Type I or Type II diaphragms and represents a different case again.

In the absence of specific guidance for the case of a horizontally irregular system, we suggest good practice for a structural engineer of the time would have been to adopt a rational method of analysis. This is particularly so, given the warnings in both NZS 4203 and the Kolston and Buchanan paper relating to diaphragms required to redistribute forces. One such method would have been to consider the change in shear from one level to the next obtained from a static analysis at code-specified levels of loading, factored up by the overstrength capacity of the wall mechanism. Mr Harding, in effect, adopted this method, but neglected to consider overstrength actions and did not make a comparison with the forces calculated using the parts provisions.

However, given the clause in NZS 3101:1982 (10.5.6.1) suggesting diaphragm forces need not be taken to be larger than that determined using parts and portions, and this was the quickest and most common method available, we believe it reasonable to have based the design of the CTV building diaphragms on the values obtained from this method.

The values we calculated using parts and portions for the forces to be used to design the diaphragm connections to both the North Wall Complex and South Shear Wall are shown in Table 7.1 In the east-west direction, total storey force was split 60% to each side to represent consideration of accidental eccentricity in the least favourable direction for the connection being considered.

Table 7.1: Required diaphragm connection forces from NZS 4203:1984 Parts and Portions compared to ARCE values

Level	Beca Calculation)		ARCE 1986 Calculation	
	NWC North-South (kN)	NWC/SSW East-West (kN)	NWC North-South (kN)	NWC/SSW East-West (kN)
6	1170	700	-	300
5	950	570	-	300
4	780	470	-	300
3	800	480	-	300
2	810	490	-	300

7.4 Did the Individuals Involved Omit to Discharge Their Duty?

The following section makes an assessment of the degree to which Mr Harding, as designer of the building, and Dr Reay, as the sole practitioner commissioned by the client to design the building, discharged their duties. This includes identification of any errors or failings in their approach.

7.4.1 Identified Errors/Failings by Mr Harding

Our review of the design identified the following errors/failings by Mr Harding:

1. The natural period of vibration of the primary structure in the north-south direction was too high (1.06 seconds versus 0.8 seconds), contributing to a reduction in base shear compared with our calculation of the code requirements for base shear.

2. The natural period of vibration of the primary structure in the east-west direction was too high (1.06 seconds versus less than 0.7 seconds), contributing to a reduction in base shear compared with our calculation of the code requirements for base shear.

3. Failure to calculate building deflections at locations other than the centre of mass.

4. Significant underestimation of building deflections in the east-west direction. This was inconsistent with the higher period he had calculated in this direction which should have resulted in a calculation of higher deflections.

5. Use of an extra, unjustified, 0.8 factor to reduce the base shears for design of the primary structure, contributing to a further reduction in base shear compared with our calculation of the code requirements. Combined with the contributions from points 1 and 2, the base shears used to design the primary structure were between 60% (east-west) and 65% (north-south) of our calculation of the code requirements.

6. A follow-on effect of points 1, 2 and 5 was that the South Shear Wall had a design strength that was 60% of the required value and the North Wall Complex in the north-south direction had a design strength that was 75% of the required value (i.e. the walls were weaker than required by the code).

7. A follow-on effect of point 6 is that the ductile ground beams acting in outrigger action for the North Wall Complex under north-south loading were under-designed.

8. Incorrectly assumed that the primary gravity frames were unable to attract actions under lateral building displacements (i.e. invalid assumption of pin joints). This assumption had multiple follow-on effects with respect to the design of the beams, columns and beam-column joints. A more detailed

explanation of the resulting non-compliances in the primary gravity frame design is provided in Section 7.4.1.1 below.

9. Failure to design the diaphragm connections between the floor slabs and the North Wall Complex for north-south loading. This led to the connection being non-compliant, having approximately 70% of the strength required by the minimum interpretation of the code (800kN versus 1,170kN, refer Appendix F and Table 7.1). This non-compliance was later identified with remedial work carried out in an attempt to address the issue (see Section 10 of this report).

10. Use of a diaphragm connection force for designing the connections between the floor slabs and the South Shear Wall and North Wall Complex in the east-west direction (300kN) that was less than half that required for the upper levels by the minimum interpretation of the code (i.e. 700kN, refer Table 7.1).

11. When checking the diaphragm connection between the floor slabs and the North Wall Complex under east-west loading, the designer appears to have made a numerical error that reduced the 300kN connection design force by a factor of 10 to 30kN. The resulting connection was non-compliant, particularly in shear-friction at grid 5 and in its ability to transfer the moment generated by transferring the load from grid 5 to grid 4.

7.4.1.1 Code Non-Compliances Identified in Primary Gravity Frame Design

As mentioned above, Mr Harding's assumption that the primary gravity frame was not required to sustain flexural action due to lateral displacements of the building was incorrect. Due to the inherently monolithic nature of the joints as detailed and the lack of specific detailing of pins in the frame members, the frames had to sustain flexural action under the lateral displacements imposed by the primary seismic system and therefore attract seismic shears, moments and axial loads.

There was no evidence that Mr Harding considered anything other than gravity actions in the design of the beams and columns, and no evidence of any design of the beam-column joints.

The transverse reinforcement provided, particularly in the columns and beam-column joints, was non-compliant with the minimum requirements of the code in many areas of the building. In addition, the detailing of aspects such as the termination of beam bars in the joints illustrated no expectation that the frames would have to sustain flexural action under lateral loads.

We have carried out the checks described in Section 7.3.6 that should have been carried out by Mr Harding in order to identify if there were elements of the primary gravity frame that did not remain elastic and therefore were required to be detailed for ductility.

Those elements that did not remain elastic when the deformations of the primary seismic structure were imposed, amplified by the factor K/SM, are shown in Figure 7.4.

From an inspection of Figure 7.4, the regions that did not remain elastic generally included the beams and columns of the perimeter frames on grids 1 and F above level 2, the columns of the internal frame on Grid 2 in the top floor and the Grid 2 beams at exterior joints. Beams and columns on grids 3 and 4, and the lower floors of Grid 2, generally remained elastic.

When the deformations of the primary seismic structure were imposed, amplified by the factor 0.5*K/SM, the majority of the areas indicated with red circles in Figure 7.4 remained elastic, with the exception of some beams near the corners, and non-corner perimeter columns from level 5 to 6 on Grids 1 and F.

[redacted]

The conclusion from the code clauses is that either the columns or the beams in these areas, whichever were weaker meeting at a joint, were required to be designed to at least the limited ductile requirements of NZS 3101:1982. In general, at an interior junction, this required the columns to be detailed for limited ductility. The beams could have been designed to remain elastic using capacity design procedures.

For the beam-column joints, if any adjacent member framing into a joint did not remain elastic, the joint was to be designed using the seismic provisions of the code. If all members framing into a joint remained elastic, the non-seismic provisions only needed to be satisfied.

Column Reinforcing Steel Non-Compliances

The circular columns were provided with an R6 spiral at 250mm pitch throughout. This complies with the non-seismic provisions of NZS 3101:1982. However, it does not comply with the limited ductile provisions. Significantly more transverse reinforcement was required in the potential plastic hinge regions of those columns shown not to remain elastic in Figure 7.4. Therefore, the transverse reinforcement specified in these column potential plastic hinge regions did not comply with the appropriate provisions of the concrete code.

Beam Reinforcing Steel Non-Compliances

The limited ductile provisions were required to be satisfied for potential plastic hinge regions of beams on grids 1 and F at exterior joints and on grid 1 at their connections to the South Shear Wall. The beams in these locations were provided with R12 stirrups at 200mm centres. The provided spacing exceeds the limit in the limited ductile provisions of d/4 = 125mm. Therefore, the beam transverse reinforcement in these areas was non-compliant.

Beam-Column Joint Reinforcing Steel Non-Compliances

The design of the beam-column joints of all circular columns specified an R6 spiral at 250mm pitch. This exceeds the spacing limit of 200mm for joints of any type (clause 9.4.8 of NZS 3101:1982), rendering all circular beam-column joints non-compliant.

We carried out an assessment of the horizontal joint reinforcement required in the interior joints at level 2 on grid 2 to satisfy the strength requirements of the non-seismic provisions. We found that either an R6 spiral at 85mm pitch or an R10 spiral at 200mm pitch would be required at this location. This is significantly more reinforcement than provided by Mr Harding's design. The amount of transverse reinforcement required in the joints increases for the upper levels and the grids on the perimeter.

Reinforcing Steel Detailing Non-Compliances

All bottom beam bars terminated in 90 degree hooks within the joint zone with inadequate development length to develop the yield strength of the bar in tension. This is non-compliant in the areas where the beams were required to be detailed for limited ductility, namely those framing into the corners of the building.

7.4.1.2 Conclusion on Discharge of Duty by Mr Harding

We have identified a number of errors and failings in Mr Harding's design of the building, including a significant mathematical error. The effect of these errors and failings is that the design which Mr Harding carried out was not compliant with the relevant codes of the day.

Mr Harding had a duty to design the building to the codes of the day. His failure to do so represents an omission to discharge his duty in relation to the design of the building.

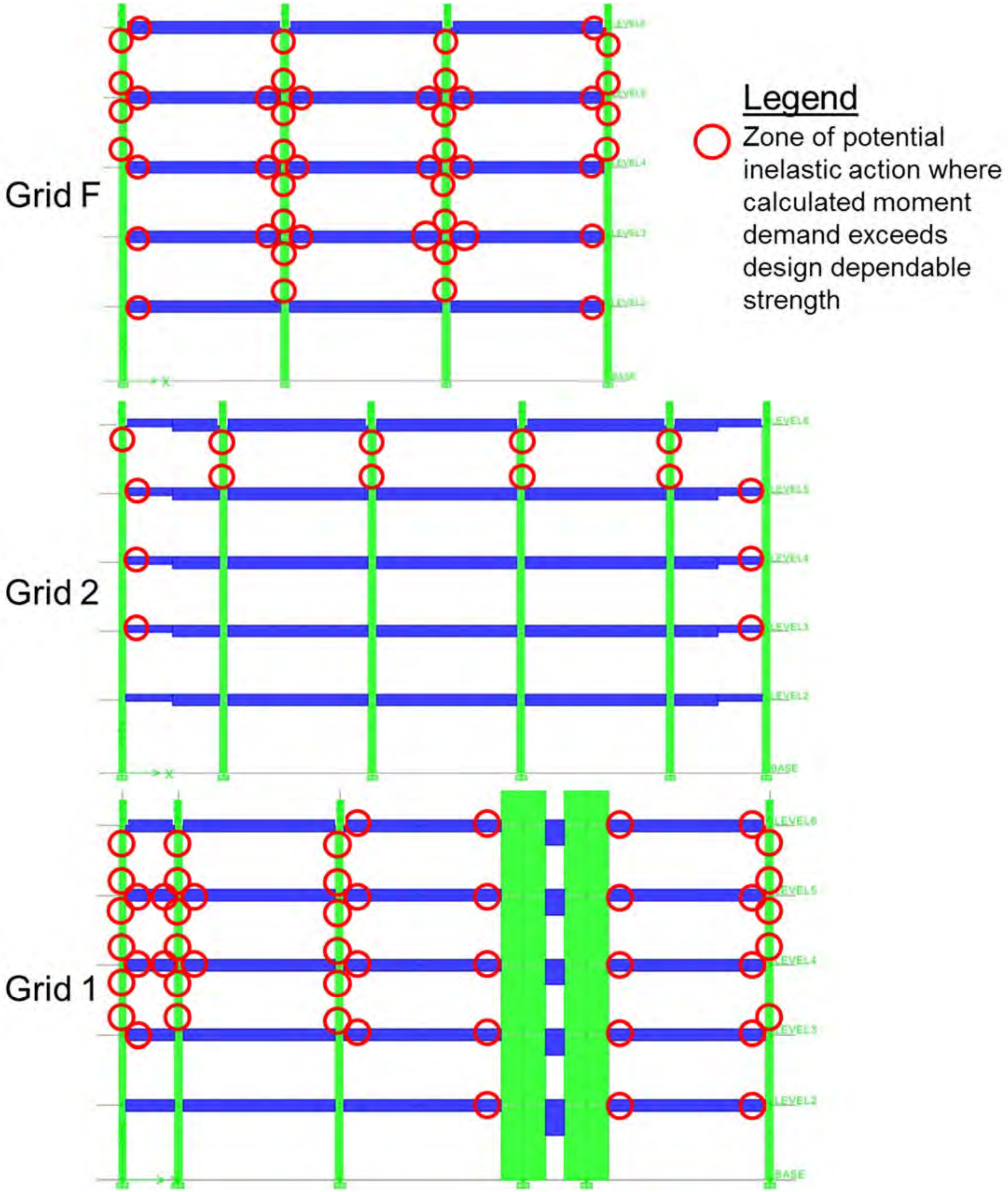


Figure 7.4: Potential areas of inelastic action in the primary gravity elements from 3D equivalent static analyses including accidental eccentricity and scaled by K/SM=2.5

[redacted]

7.4.2 Identified Failings by Dr Reay

Identified failings by Dr Reay are as follows:

1. Failure to check that Mr Harding, the person he allocated to undertake the design of the CTV building, was sufficiently experienced to design such structures.

2. Failure to provide, or arrange any form of checking or review of Mr Harding's work, either by himself or by another suitably experienced engineer.

7.4.3 Conclusion on Discharge of Duty by Dr Reay

We have identified two failings in respect to Dr Reay's approach to his role as the principal of ARCE responsible for the design the building. The effect of these failings is that the design that Dr Reay was commissioned to deliver was not compliant with the relevant codes of the day.

These failings, and particularly in combination, were major omissions in the discharge of his duty in relation to the design of the building.

7.5 Accepted Practice of the Day

In this section, we assess the extent of departure of the specific errors/failings identified in 7.4 from the accepted practices of the day for designers and design entity principals. Our comments on accepted practice of the day are based on a combination of recollections of the time by practitioner's within our own consultancy, the responses of the practitioners interviewed, our review of six relevant 1980s Christchurch buildings, and our review of actions of members of the Institution of Professional Engineers New Zealand against the Codes of Ethics and Professional Practice for Consulting Engineers of the day. Refer to Appendix E for response sheets relating to our practitioner interviews and Appendix G for a summary of the review of the Christchurch buildings. The Codes of Ethics and of Professional Practice for Consulting Engineers that were current at the time of the design of the CTV building are provided in Appendices C and D respectively.

7.5.1 Designers

Mr Harding's general approach to the design was in line with practices of the day. However, the lack of compliance of the design with the loadings and concrete codes was a major departure from generally accepted practice.

Several of Mr Harding's errors were either mistakes or simply due to inexperience, and do not relate to specific aspects of practice of the day. These include the overestimation of the building period in the east-west direction, the extra 0.8 factor when scaling base shears for design, the mathematical error when designing the diaphragm connections and the underestimation of the building deformations in the east-west direction.

Many of the other omissions relate to specific aspects of practice of the day and are commented on below:

Splitting of Primary and Primary Gravity Structure

It was common for engineers to identify the primary and the primary gravity structure, and up until the mid-1980s, relatively common to consider them separately from an analysis perspective.

However, by the mid-1980s, many engineers carrying out multi-storey building design had access to 3D analysis computer programs and were modelling primary gravity structure, together with the primary seismic structure, in one analysis. There was an incentive to do this as the primary gravity structure carried a small proportion of the lateral earthquake load thus reducing the design loads on the primary system. This is an option that was not available if the structures were considered separately.

Mr Harding had access to such a program, as he was using a 3D version of ETABS at the University of Canterbury.

However, since the codes of the day allowed consideration of primary and primary gravity structure, and theoretically it was conservative not to include the primary gravity frames in the analysis of the primary seismic structure, Mr Harding's decision to consider primary and primary gravity structure separately is not considered a departure from the accepted practice.

Design of the Primary Gravity Structure

It was widely recognised that the parts of the structure not included within the assumed primary seismic structure still had to sustain the deformations imposed by the primary lateral structure unless they were separated from the primary seismic structure.

Generally accepted practice of the day when dealing with detailing of the primary gravity framing elements included:

- Not including the primary gravity frames in the analysis of the primary structure, and using the seismic provisions of the concrete code to detail for ductility. The cost of additional transverse reinforcement was small, and this approach saved time in not having to specifically assess the effects of the primary structure deformations on the primary gravity frames.
- Detailing the primary gravity frames for ductility where required, but reducing this where yielding was not shown to be occurring. This entailed an analysis of the primary gravity frames to derive design actions generated by the imposed displacements.
- Including the primary gravity frames in the analysis of the primary seismic structure to directly assess the effect of primary seismic structure deformations on the primary gravity structure. The level of detailing to be provided for the primary gravity framing elements could then be easily determined.

It was a major departure from accepted practice of the day to use the non-seismic provisions to detail the primary gravity frame elements without also confirming this was appropriate by carrying out a check on the effect of primary seismic structure deformations.

We have determined that the column transverse reinforcement met the minimum (non-seismic) requirements of the concrete code in many locations in the building. However, we have been unable to identify other buildings of the period that had columns with quantities of transverse reinforcement as low as provided in the CTV building columns, particularly the lower interior columns. With reference to Appendix G, no other design included specification of 6mm diameter transverse reinforcement in columns (all were 10mm diameter or greater) or specification of 6mm diameter transverse reinforcement in beam-column joints.

We have concluded, therefore, that other designers of the time were not prepared to reduce the transverse steel requirements to the minimum specified in the code.

Mr Harding's decision to regard the primary gravity frames as non-seismic based on an assumption that they would not attract any actions as a result of lateral displacements, was a major departure from the accepted practice of the day. That assumption led to further non-compliances in the design that were also major departures from accepted practice. Furthermore, where the provided transverse reinforcement in the columns was code compliant, it was less than what other engineers of the time were specifying.

Diaphragm Connections

Due to the uncertainty at the time regarding the appropriate method for determining diaphragm connection forces, we consider it was accepted practice to use either the parts and portions provisions of the loadings standard or a capacity design approach based on the overstrength capacity of the wall mechanism, whichever gave the lowest actions. If Mr Harding had followed either approach, he would have calculated a significantly larger design connection force.

It was well recognised that diaphragm connection forces must be adequately developed into both the floor slabs and the primary wall elements. Mr Harding failed to ensure this in several areas including the North Wall Complex for north-south loading and the North Wall Complex for east-west loading.

7.5.2 Code of Ethics and Code of Professional Practice for Consulting Engineers

The obligations of the Code of Ethics for members of the Institution of Professional Engineers New Zealand and of the Code of Professional Practice for Consulting Engineers are outlined in Section 3.11 and reproduced in Appendices C and D.

At the time of the design of the CTV building both Dr Reay and Mr Harding were members of the Institution of Professional Engineers. The Institution's Code of Ethics was not controversial and represented the expected practice of the time. Therefore both Dr Reay and Mr Harding would have been expected to follow it.

In addition Dr Reay, who was practising as a consulting engineer on his own behalf was also bound by the Code of Professional Practice for Consulting Engineers which referred back to the Code of Ethics.

These codes represent some of the measures that constituted accepted standards of practice of the day.

Our review and the work undertaken for CERC identified a number of serious errors in Mr Harding's analysis and design of the building. Also the drawings were signed off by him. Therefore, Mr Harding breached either or both:

- Clause 1 of the Code of Ethics which required him to exercise his professional and technical skill and judgement to the best of his ability and discharge his professional and technical responsibilities with integrity;
- Clause 6 of the Code of Ethics which required him not to misrepresent his competence nor, without disclosing its limits, undertake work beyond it.

As a result of this breach he also breached Clause 8 as he failed to recognise that his actions did not meet his responsibilities to his employer, the public interest and his profession.

It does appear that Mr Harding strove to improve his professional competence by attendance at seminars and discussions with Professor Paulay that were relevant to the design of a building such as the CTV building, but he overestimated the level of competence that he had achieved.

Dr Reay accepted the commission for the design of the CTV building from his client, Williams Construction, acting on his own behalf as a consulting engineer. He delegated the design to Mr Harding.

During the CERC hearings, Dr Reay maintained that he was simply the manager and was entitled to rely on Mr Harding's competence to produce a compliant structural design for the CTV building without independent checking. This reliance was by virtue of Mr Harding's age, years of experience and the fact that he was a Registered Engineer. We disagree that these attributes were sufficient. Dr Reay acknowledged that Mr Harding had minimal experience in the design of multi-storey buildings which should have been a critical aspect of his evaluation of whether or not he could completely rely on Mr Harding's competence for the design this significant building.

By not personally checking, or organising independent review of Mr Harding's work by someone with the appropriate experience, to satisfy himself of its competency, Dr Reay breached Clauses 1 and 6 of the Code of Ethics and therefore Clause 11 of the Code of Professional Practice for Consulting Engineers which required a consulting engineer not to engage in any act, activity or conduct which was contrary to the Code of Ethics of the Institution.

As a result of these breaches, Dr Reay also breached Clause 8 of the Code of Ethics as he failed to recognise that his actions (or lack of action) did not meet his responsibilities to his client, the public interest and his profession. These responsibilities are discussed further in Section 7.5.3.

Therefore, our view is that, in the design of the CTV building, neither Mr Harding nor Dr Reay met the requirements of the code of ethics of the Institution, and also Dr Reay did not meet the requirements of the Code of Practice for Consulting Engineers. Not meeting the then current requirements of the codes of conduct was outside the accepted practice of the day.

In reaching the conclusion outlined above we have not had the benefit of hearing representations from either Dr Reay or Mr Harding beyond those given to the CERC hearings.

7.5.3 Design Entity Principals

Responsibilities

Accepted practice of the time was that a sole practitioner, a partner in a partnership, or a principal of a company took responsibility for an engineering design engagement and oversaw the work of his or her staff during its progress through design or undertook the work personally. The sole practitioner or partner developed or reviewed, the structural concepts, and approved the drawings prior to issue for construction. We have referred to this role as the principal's role in the discussion below.

Dr Reay stated during the CERC hearings that although he accepted overall responsibility on behalf of ARCE (which was Dr Reay himself), he considered responsibility for the design of the project to have been handed over to Mr Harding by allocating him as the staff engineer, without the need for on-going oversight or review before the design documentation was issued for building permit or construction purposes. This represents a major departure from the accepted practice of the time which was for a principal to arrange for, or personally provide, oversight during the design and, typically, review the completed design prior to construction.

Allocation of Staff to Projects and Oversight Provided

A principal was responsible for the allocation of staff and being aware of the capability of individuals.

It was not uncommon at the time for an inexperienced engineer to be given the job of designing a building such as the CTV building. However, it was general practice for the principal to recognise and assess this inexperience and ensure appropriate design guidance was provided, whether or not it was sought.

The more inexperienced an engineer, the more contact a principal or appropriate senior member of the team would have with the engineer.

The principal would ensure proactive checks were carried out and not await questions.

It was reported in the CERC report that Dr Reay made Mr Harding aware that he did not have the time or knowledge to be able to assist him, nor was there anyone else available in the office to provide this support. In these circumstances, Dr Reay should have arranged for a suitable external person or organisation to fulfil this role in order to discharge his responsibilities.

Dr Reay's decision to allocate Mr Harding, who he acknowledged as having little experience in the design of this type of building, to the role without any real guidance or overview was a major departure from the accepted practice of the day of a principal.

Checking Processes

Internal checking processes varied between design organisations and between particular projects. Almost all of the principals of the time who were interviewed indicated that they had involvement during the design process and reviewed the drawings before they were issued. However, they did not always carry out, or organise others to carry out, independent checks of the calculations.

The fact that Dr Reay did not review the drawings before they were submitted represents a departure from the accepted practice of the time. Furthermore, if he considered himself unsuitable to review the drawings from a technical perspective, he should have arranged for a suitable external person or organisation to fulfil this role.

All of the principals of the time who were interviewed indicated that they would not rely on the Council review process to validate the design outputs of their firm. Sole reliance on the Council to check the design, as Dr Reay indicated during the CERC hearings he had done, would represent a major departure from the accepted practice of the day.

Design Documentation

Although it was common for design companies to record design, drawing and checking responsibilities on their drawings with dedicated spaces made available, it was not a universal practice. The CCC did not require more than one 'responsible' space to be signed when the drawings were submitted for building consent. Therefore, the fact that there was only one signature on the drawings was not a departure from the generally accepted practice of the day.

7.6 Conclusions

Mr Harding's technical errors/failings can be categorised into the following:

- Underestimation of the actions for which the primary seismic elements (the North Wall Complex and the South Shear Wall) and their foundations were designed
- Diaphragm connections between the floor slabs and both the North Wall Complex and the South Shear Wall that were under the strength required to transfer the design loads specified by the minimum interpretation of the codes of the day
- Non-compliant detailing of all beam-column joints and many beam and column potential plastic hinge regions that led to primary gravity frames with very little resilience beyond yield (i.e. non-ductile detailing).

Mr Harding was an engineer experienced in single storey residential and industrial buildings and roading, but had very little experience in the structural design of multi-storey buildings.

Mr Harding's calculations show that he made a surprising and incorrect decision to regard the beams, columns and joints in the CTV building as gravity only structure that was not required to be designed for seismic frame action or for ductility, without providing specific detailing to achieve this (e.g. pin-ended elements).

It is possible this judgement call was made because his underestimation of drifts in the east-west direction misled him as to the likely magnitude of the demands on the primary gravity structure. The lack of experience of Mr Harding in multi-storey building design was a factor in his decision-making on this issue.

The code required the seismic flexural action of the primary gravity structure to be confirmed and it included a method of checking the justification for the lack of ductility. The original calculations for the CTV building show that no such checks were carried out by Mr Harding.

We undertook the checks required by the code and concluded that the decision not to design the primary gravity structure for seismic frame action was invalid, at least in some areas. Practices of the day were that such checks were done and the appropriate parts detailed for ductility, or the primary gravity structure was either included in the analysis of the primary seismic structure or detailed for ductility. This would have been especially so for a building of this configuration with relatively flexible walls in one direction (north-south) and primarily on one side of the building in the other direction (east-west) resulting in eccentricities and twisting in plan. Also, if the checks had been done, an experienced designer of the day is likely to have designed the primary gravity elements for ductility if any elements at a floor level required it, especially for this building with relatively flexible walls and a high degree of eccentricity.

The resulting design of the building contained significant non-compliances with the codes of the day. This represents an omission by Mr Harding to discharge his duty in relation to the design of the building.

Notwithstanding the code specified minimum reinforcement requirements, we found no other examples of similarly sized buildings in Christchurch of the period where designers were prepared to adopt the low levels of column and joint transverse reinforcement used in the CTV building. It is this reinforcement that provides the ductility.

Mr Harding's design of the primary gravity structure was a major departure from the expected standard of the day.

The commission for the design of the CTV building was awarded to Alan M. Reay Consulting Engineer. Dr Reay was a sole practitioner.

The CTV building was a six storey building of reasonable size incorporating innovative features for the time (i.e. pre-casting of frame elements). It warranted careful consideration during design as if it was not designed correctly and failed it had the potential to cause multiple fatalities.

Dr Reay chose to delegate the design to Mr Harding who had not previously undertaken design of structures of this type. He failed to ensure that adequate experience or oversight, review and checking were applied to the design, with the result that errors in the building design were not identified. A suitably experienced designer would not have made the errors that Mr Harding did, or adequate oversight, review and checking of the design concept and detailing by a suitably experienced engineer would have identified and corrected them. The lack of either was not in accordance with the accepted practices of the day. Dr Reay's reliance on the consent checking process to identify errors was also not in accordance with the accepted practice of the day.

His failure to allocate a sufficiently experienced engineer to the design of the building and to provide any form of checking or review of Mr Harding's work, were, in combination, major omissions by Dr Reay to discharge his duty as a consulting engineer undertaking the design of a significant building. His approach to these responsibilities, was a major departure from the expected standard of the day.

[redacted]

8 Building Permit

8.1 Introduction

The CERC considered all of the evidence presented to it, some of it undocumented recollections of discussions and actions at the time, and concluded that a building permit should not have been issued.

The documentation available for review was not complete in some critical areas (e.g. the structural design drawings included with the initial application for building consent were not available). We have reviewed the documentation that was available and the discussion on it presented in the CERC report. Refer to Appendix B for a full list of documentation reviewed by Beca.

Our review has been from the perspective of identifying whether the individuals involved with the permitting process omitted to discharge their duty and, if so, whether or not the omissions were major departures from the accepted practice of the day.

In this section, our focus has been on the actions and/or inactions of the CCC personnel involved with the process of reviewing and approving the structural aspects of the project that ultimately resulted in the building permit being granted. The CCC personnel were:

- Mr Graeme Tapper
- Mr Brian Bluck.

8.2 Building Permit Process for CTV Building

Based on the available information, the process for granting of the building permit for the CTV building is understood to have proceeded as follows:

- The project architect filed an application for building permit with CCC on 17 July 1986 without the structural drawings.
- Structural drawings were submitted on 26 August 1986. This set of drawings was not available to Beca or the CERC for review.
- Mr Tapper reviewed these drawings and responded with a letter on 27 August 1986 requesting further clarification and information and noted that the drawings had not been signed by the designer.
- On or about 5 September 1986 revised sets of signed structural drawings and calculations relating to the fire rating of the Hibond structure were submitted to CCC.
- Mr Tapper signed off the structural aspects of the project on 10 September 1986.
- The building permit was granted on 30 September 1986.

A copy of the structural drawings stamped as the building permit set (dated August 1986) were available for our review. It is unclear if the ARCE general structural design calculations, other than those covering the Hibond structure fire rating, were submitted to CCC.

The process as outlined above was very typical of the time. It was a period of significant activity in the building industry and time was important due to the high financial holding costs and earning potential for building projects. It was not unusual for incomplete structural documentation to be provided at the time of initial submission for a building permit, and be followed by further submissions of completed construction documentation before the permit would be granted. This allowed initiation of council review on matters other than structural while the structural documentation was being completed. This process was generally accepted by councils and resulted in the minimisation of the overall time to gain a building permit.

Guidelines for structural checking engineers prepared by Mr Bluck and presented to the CERC noted his instructions that the checking engineer was *“entitled to rely on the recognised expertise of a Professional Designer who is prepared to certify under his signature that a specific design for a conventional or innovative structure (or detail) complies in all respects with the intent of the provisions of NZS 1900 Chapter 8”*. The reference to NZS 1900 dates this memorandum somewhat earlier than the mid-1980s but nevertheless indicates the approach that is likely to have prevailed at the time the CTV building permit application was being reviewed.

8.3 Capacity of CCC to Review the Structural Design

The CCC review of the structural aspects was carried out over one day in August (arguably with incomplete drawings) and approximately five days in September (from the 5^{th} to 10^{th} which included a weekend) with the revised drawings. It is unlikely that Mr Tapper could have dedicated all his time over this latter period to the CTV building project.

The seismic structural system comprising two shear walls of very different structural characteristics was not straight forward. It is questionable if a period of only three days was sufficient time for a reviewing engineer to become familiar with the structural configuration of the CTV building, complete or check a structural analysis of the structure and then confirm compliance with structural standards. Also, the reviewing engineer would not have had the resources to carry out the modal analyses that had been completed by the designer. To complete the review over this short period, especially if he did not have the calculations, we believe Mr Tapper would have been reliant on the approach that had been outlined by Mr Bluck in the memorandum referred to above. Therefore, it would appear that Mr Tapper relied on the designer’s signature on the drawings as certification that the design complied in all respects with the codes of the day,

Notwithstanding the limited time available, we would have expected that any gross omissions in the structural design of the building would have been picked up during a review of the structural drawings by a structural engineer experienced in multi-storey building design. It is unclear if Mr Tapper possessed that experience.

8.4 Did the Individuals Involved Omit to Discharge Their Duty?

It is apparent from the evidence presented to the CERC that Mr Tapper was unhappy with aspects of the structural drawings of the CTV building initially submitted for permitting. What is not clear is the particular issue(s) causing these concerns. While the Tapper letter dated 27 August 1986 refers to the floor connection to the shear wall system, this can be construed as a particular reference to a lack of notes on the drawing relating to this detail and not necessarily an indication of concerns with the design of the connection itself which, because of the lack of notes, he may not have been in a position to evaluate. The exact circumstances around this may never be known unless the original permit application documentation can be located.

Notwithstanding the above, Mr Tapper signed off the structural aspects for the building permit once completed signed structural drawings were provided by the designer.

There are three issues that could be classified as failings in the structural review process carried out by CCC. These relate to the non-identification and/or acceptance of three significant deficiencies in the design:

- The very light spiral reinforcing in the columns, although arguably complying with the letter of NZS 3101:1982 in many areas of the building, was not typical of the time and warranted at least a comment from a reviewer.

- The beam-column joint reinforcement shown on the drawings was significantly less than the minimum required by NZS 3101:1982, and significantly less than provided in other similar buildings constructed in Christchurch at the time.
- The important attachment of the North Wall Complex to the floor slab at each floor was significantly under-designed and obviously so. A review of the structure should have identified that there was at least an issue with this detail that required comment from the designer. These deficiencies were identified during the HCG review in 1990, but they were not identified by the original designer nor were they fully resolved during the later retrofit of the connection (see Section 10).

It has been suggested during the CERC hearings that Dr Reay convinced CCC that the design was adequate. Whether or not discussions between Dr Reay and Mr Bluck occurred, Beca is of the view that the failure to highlight these particular deficiencies represents an omission by both Mr Bluck and Mr Tapper to discharge their duties in relation to the permitting of the building.

8.5 Conclusion

There were failings by the individuals involved in the CCC building permit review process to identify critical structural deficiencies in the building design.

Questions on the design were raised by Mr Tapper at the time he was reviewing the initial design documentation that had been submitted for building permit, but it is not clear if he identified the specific issues that we have noted, nor how comprehensive was his review of the final submitted documentation.

It is also unclear how much experience Mr Tapper had in multi-storey building design and, therefore, if it was reasonable to have expected that he should have identified the issues given the limited time spent on review.

Identification of the design deficiencies during the building permit review ideally should have occurred and should have led to them being addressed in the design and construction of the completed building. However, advice from the practitioners group with which we consulted, was that the generally accepted practice of the time was that a substantial structural review may not happen and could not be relied upon.

Verification of the design during the building permit process should not have been expected by the designers of the CTV building because of the limited time and resources available to the CCC building permit approval team.

Therefore, we have concluded that while Mr Tapper and Mr Bluck omitted to discharge their duty in relation to the permitting of the building, their omissions did not represent a major departure from accepted practice of the time.

9 Construction

9.1 Introduction

We have relied on the reporting of others to identify defects in the construction of the CTV building. In particular, we have referred to the investigation report prepared by Hyland/Smith, the site examination report prepared by Hyland and the discussion on these matters in the CERC report.

We have made our own assessment of the importance of the construction defects when responding to the questions posed to us in the brief for this review.

The construction of the CTV building was carried out by contractors Williams Construction and Union Construction. The key construction personnel were the construction manager, Gerald Shirtcliff and the construction foreman, Bill Jones.

Construction monitoring was carried out by Mr Harding for ARCE, and also by CCC inspectors.

9.2 Identified Construction Issues

The Hyland/Smith report identified the following issues that could be characterised as construction defects:

- Potentially low concrete strength in columns
- Un-roughened construction joints at beam ends, both in the gravity frame and in the connections to the South Shear Wall
- Bent-back bottom reinforcement in the beams connected into the North Wall Complex
- Omission of spiral reinforcement in the beam-column joint regions
- Lack of separation between columns and spandrel precast panels
- Lack of separation between columns and the western block wall infills
- Missing bar in the attachment of column C18 to the North Wall Complex
- Offset column cage.

We also considered the impact of the:

- Experience of the contractor
- Adequacy of construction monitoring by the designer
- Adequacy of inspections by CCC.

Discussion of these issues follows.

9.2.1 Low Concrete Strengths in Columns

Hyland/Smith carried out testing of concrete cores taken from remaining segments of the structure including the only remaining in-situ pieces of the ground floor columns. From the distribution of the results of these tests, extended by Schmidt hammer testing, they concluded that some of the concrete may have been understrength.

We agree with the findings of the CERC that there are insufficient results to provide a reliable basis to conclude that the concrete in the building was significantly below the specified design strength. The limited data would likely result in the broader distribution of the Hyland/Smith test results when compared with the results that would be targeted during concrete production. We also agree that the low results obtained from column C18 should be considered as an “outlier” warranting further investigation before it is concluded that low concrete strength is an issue for the CTV building.

[redacted]

We note that the specified concrete strength for the columns was reduced towards the top of the building. Therefore, the tested mean strength of approximately 30MPa from the test results for all columns is also sufficiently close to the specified strength of 35MPa for the level 1 columns to suggest that the concrete in the columns was not significantly understrength. Our calculations also confirm that some degree of understrength concrete could be tolerated in the ground floor columns without having a significant effect on their propensity for loss of axial load carrying capacity under lateral displacements. This is provided the reinforcement in the frame as a whole (i.e. including in the beam-column joints) had been in accordance with the codes of the day.

9.2.2 Un-Roughened Construction Joints at Beam Ends

In our view this was a construction defect that was primarily the responsibility of the contractor. However, it should have been picked up during the inspection process by the designer and/or CCC, and therefore cannot be considered solely the responsibility of the contractor.

By citing compliance with NZS 3109:1980, the specification required roughening of all construction joints by the contractor. Irrespective of whether or not a copy of NZS 3109:1980 was available to the contractor, the preparation of construction joints was fundamental to concrete construction in the mid-1980s and general practice. The references on the drawings to roughening of the inside face of the precast beam shell was for that specific area. The lack of a specific beam end preparation detail on the drawings is not necessarily an omission in the drawings and, therefore, not sufficient to imply that the designer did not expect or require the interface to be roughened.

The ends of the beams could have been roughened in accordance with NZS 3109:1980 and it would have been possible to do this on site if not done prior to delivery to site. The lack of preparation of the beam end construction joints was a construction defect.

The designer and/or the CCC inspectors should have picked up this defect while on site.

Our analyses have shown that poor preparation of the gravity beam-end construction joints, while not in accordance with the specification, and not good or accepted practice, would not have significantly increased the propensity for sudden pancaking collapse.

The poor preparation of at least some of the beam end pockets in the South Shear Wall is also a construction deflect, although the photographic evidence would suggest these joints remained predominantly intact until the collapse was significantly advanced.

9.2.3 Bent-Back Bottom Reinforcement in the Beams Connected into the North Wall Complex

The observed bending back of the beam bottom bars in the connection between the floor beams and the North Wall Complex was a construction defect. The bending back of these bars was difficult to explain given that it didn't seem to be typical and would have been difficult to achieve on site without application of heat. If heat had been applied we might have expected that there would have been evidence this had occurred.

We agree with the CERC conclusion that connectivity at this joint was provided by the top reinforcement and this defect would therefore have made only a minor contribution, if at all, to the building collapse.

9.2.4 Omission of Spiral Reinforcement in the Beam-Column Joint Regions

The omission of joint reinforcement was a construction defect. The joint reinforcement should have been placed as required on the drawing and, if this was not possible, the contractor should have consulted the engineer.

We would have expected this defect to have been picked up during the observation/inspection activities during construction.

We agree with the conclusion of the CERC that inclusion of the joint reinforcement as detailed would have had little effect on preventing joint failure as it was so minimal and light that it would have been ineffective in restraining the vertical reinforcing bars and would have added minimal additional shear capacity to the joint.

9.2.5 Lack of Separation between Columns and Spandrel Precast Panels

The CERC concluded that if the clearance had been less than specified or if any gap had been taken up during the 22 February earthquake, the strength of the panel and connection to the structure would have been unlikely to have been sufficient to significantly influence the columns.

We agree with this conclusion.

9.2.6 Lack of Separation between Columns and the Western Block Wall Infills

The evidence suggests that the block wall infills were constructed in accordance with the construction drawings (i.e. with gaps to the concrete frame elements).

Even if the gap between the block wall infills and the concrete structure had been less than detailed, there appears to be no evidence that the block wall infills contributed to the collapse in any significant way. This was also the conclusion of the CERC.

9.2.7 Missing Bar in the Attachment of Column C18 to the North Wall Complex

The missing vertical bar in column C18 is a construction defect.

It does not appear that the contribution of column C18 to the lateral resistance of the building (i.e. as a vertical tie to the wall outrigger) was included in the lateral load calculations prepared by ARCE. Therefore, although the additional bar may have reduced the propensity for damage to the column and top of the wall, the lack of this bar would not have significantly weakened the overall structure.

9.2.8 Offset Column Cage

The photographic evidence indicates that in at least one column (location uncertain) the reinforcing cage was displaced in the cross section of the column, resulting in very little cover on one side of the column.

In general, the cages were most likely to have been located correctly at the floor levels where the main bars passed through the beam-column joint, and also into the foundation. It is likely the displacement occurred due to the difficulty in holding the lightly bound cage plumb in the form. This would mean the cage was likely to have been displaced in the mid height region of the column.

The effect on the building as a whole of the displacement of the column reinforcing cage in the mid height region of the column would not have been significant.

9.2.9 Experience of the Contractor's Personnel

The responsibility of a contractor is the construction of a building in accordance with the construction drawings and specifications, and seeking clarification regarding ambiguities, omissions and discrepancies in the construction documentation. The contractor cannot be expected to take responsibility for structural design.

An experienced contractor can assist by identifying possible structural deficiencies and referring these to the engineer for clarification and/or comment. However, it was not, and is not accepted practice for the designer or council inspectors to rely on the contractor to do so.

The CERC was advised the construction manager for Williams Construction and Union Construction, Mr Shirtcliff, was infrequently on site and therefore did not provide the guidance that the foreman, Mr Jones, required. It was also subsequently revealed that Mr Shirtcliff's engineering qualifications may have been misrepresented at the time. These suggestions, whether accurate or not, had little relevant impact, as the type and extent of construction defects that have been identified did not have a significant effect on the collapse.

9.2.10 Adequacy of Construction Observation by the Designer

We have seen no documentation that defines the intended role of the designer during construction.

This was a design-build contract. Under such a contract in the 1980s, the contractor would typically engage the designer for construction "observation". Construction observation was defined in standard industry documentation of the time as: *"Observation comprises such inspections as the Consulting Engineer considers necessary to ascertain whether his design is being interpreted correctly and whether the works for which he is the professional adviser are being carried out in general accordance with the contract documents"*.

It appears likely that ARCE assumed this role during construction. In discharging such a role only periodic visits to site would be carried out, and it would not be expected that the designer would necessarily pick up all defects, or see all parts of the building. It would be expected that the inspections would identify systemic issues such as poor preparation of construction joints or incorrect implementation of a common and important detail that was repeated several times, such as a beam-column joint.

Our understanding is that Mr Harding carried out the construction observation on behalf of ARCE. If so he should have picked up the reported systemic lack of preparation of the beam-end construction joints and the lack of beam-column joint reinforcement, but not necessarily, for example, the bent-up bottom beam bars which do not appear to have been a general defect.

9.2.11 Adequacy of Inspections by CCC

It was common practice at the time for the Council to require inspections by the design engineer. Notwithstanding, the Council would have been expected to also carry out its own inspections. It would appear from evidence given to the CERC that the inspections by CCC during erection of the concrete structure were infrequent.

The CCC inspectors should have picked up issues such as the poor preparation of construction joints but, in our opinion, would have been unlikely to have identified particular design issues (e.g. that there were no joint spirals).

We have no knowledge of the identity of the CCC inspectors for the CTV building, or of the detail of any inspections carried out.

9.2.12 Findings from the Physical Testing

The physical testing of the concrete specimen that included the missing joint reinforcement and lack of beam end construction joint preparation in the internal bay of the primary gravity structure indicated that these deficiencies, on their own (i.e. in the absence of the design deficiencies in the primary seismic structure), would not have been sufficient to lead to pancaking collapse.

9.3 Conclusion

Although the construction defects identified represent omissions by the relevant individuals to discharge their duty in relation to the construction and inspection of the building, and are departures from expected practice of the day, the identified defects on their own or together would not have contributed significantly to the collapse of the building.

[redacted]

10 1990/1991 Retrofit

10.1 General

Holmes Consulting Group (HCG) carried out a pre-purchase review of the CTV building in 1990 for Canterbury Regional Council. One of the main findings in their report was the identification of a "*vital area of non-compliance with current design codes*".

The HCG report expands on this finding in a later section:

"*An area of concern however has been discovered in the connections of the structural floor diaphragm to the shear walls. While this is not a concern on the coupled shear wall to the south of the building, connections to the walls at the North face of the building are tenuous, due to penetrations for services, lift shafts and the stairs, as detailed on the drawings*".

The HCG report makes the following comment about the design of the gravity structure:

"*From our perusal of the drawings, and our investigation of the building, it appears the gravity structure is sound and complies in all respects with the appropriate design loading and materials codes*".

The HCG report makes the following comment about the design of the shear walls:

"*The shear walls themselves appear to have been generally well designed to the requirements of the correct design loading and materials codes*".

The original design entity for the building, ARCE, by this time reformed as Alan Reay Consultants Limited (ARCL), undertook to design a retrofit to remediate the identified non-compliance. Geoff Banks, a director of ARCL at the time, carried out this work. In his brief of evidence to the CERC, Mr Banks states that he was only provided with the body of the HCG report (the ten typed pages) and not the accompanying calculations. Therefore, Mr Banks carried out his own calculations with respect to the connections between the floor diaphragms and the North Wall Complex.

In his brief of evidence, Mr Banks also states he was focussed on the detailed aspects of the retrofit works and Dr Reay had an oversight role. In Dr Reay's statement of evidence, he stated he had little direct involvement in the retrofit works.

Based on his calculations, Mr Banks concluded that the scope of the non-compliance was the connections between the floor slabs and walls D and D-E for earthquake loading in the north-south direction and subsequently designed a remedial solution.

The remedial work that was eventually installed in 1991 involved the installation of two steel angles extending beyond the ends of walls D and D-E at levels 4, 5 and 6.

The following section discusses the approach taken by Mr Banks and the requirements of the codes and other prevalent literature of the day. Identification is made of any errors/failings in Mr Banks approach before an assessment of whether these represent an omission to discharge his duty, and whether or not the omission was a major departure from accepted practice of the day.

[redacted]

10.2 Approach by Mr Banks

Mr Banks carried out calculations to check the adequacy of the existing connections between the floor diaphragms and the North Wall Complex as detailed on the available structural drawings. He did not appear to carry out any calculations, nor document any checks, on any other aspect of the building.

Mr Banks began his calculations in 1990 by determining the required diaphragm connection forces for the North Wall Complex for each loading direction. He initially made reference to the 1986 calculations by Mr Harding, whereby a static storey design force of 501kN was used with no account made of overstrength.

Instead of using the value from the original calculations, Mr Banks carried out his own calculations for the design forces to be used for the diaphragm connections. He calculated the required diaphragm connection force for the uppermost suspended floor (level 6) using the requirements for parts and portions from NZS 4203:1984. This resulted in a total diaphragm force at level 6 of 1,241kN. This value is nearly 2.5 times larger than the relevant value used by Mr Harding in the original calculations (501kN).

10.2.1 East-West Direction

For the east-west direction of loading, Mr Banks allocated 58% of this force (724kN) to the connections to the North Wall Complex, based on the relative base shear taken by the complex and the South Shear Wall. Again, this value is nearly 2.5 times larger than the relevant value used by Mr Harding in the original calculations (300kN).

Mr Banks then carried out a check on the adequacy of the provided connection to the wall on grid 5 for loading in the east-west direction for the 724kN load. He checked shear-friction in the interface connection and shear in the toilet slab and concluded the connection in this direction was sufficient.

10.2.2 North-South Direction

For the north-south direction of loading, Mr Banks allocated the full 1,241kN to the connections to the North Wall Complex.

He identified there was little or no existing connection between walls D and D-E and the floor slabs for this direction of loading. He also assumed the existing connections to walls C and C-D were sufficient.

Consequently, Mr Banks broke down the required storey force into components on the two non-compliant connections with 19% (231kN) allocated to the connection to wall D-E and 22% (279kN) to wall D.

To address the non-compliant connections, Mr Banks proceeded with a scheme that involved connecting walls D and D-E to the floor slabs using steel drag bars. In a letter to HCG attempting to confirm the scope of the retrofit works, Mr Banks stated the maximum design load per drag bar would be 300kN (i.e. slightly greater than the maximum value calculated of 279kN).

The design concept for the drag bars involved installing steel angle sections under the floor slabs and bolting them to both the shear walls and the slabs. The drag bars were to extend 800mm beyond the end of wall D and 1,700mm beyond the end of wall D-E.

When designing the drag bars, Mr Banks reduced the 300kN value for the lower floors in proportion to the reduction in the K_x parameter with floor height (NZS 4203:1984 clause 3.4.9.2). He obtained 240kN at level 5 and 184kN for levels 4, 3 and 2. He then identified two H12 bars in the end of wall D and one H12 bar in the end of wall D-E. He calculated the strength of these bars in tension and further reduced the design connection forces on the proposed drag bars for all levels by these values. Drag bars and their connections to the walls and floor slabs at levels 4, 5 and 6 were subsequently designed for the resulting loads.

[redacted]

At levels 2 and 3 he carried out checks to determine whether these forces could be transferred to walls C and C-D. He found there was reserve strength (shear and flexure) in walls C and C-D such that the additional forces could be transferred adequately.

Sketches were produced by ARCL to convey the design of the drag bar scheme to a contractor for construction.

10.3 Code Requirements and State of Knowledge

The applicable standards to be used for the design of diaphragms and their connections in 1990/1991 were the same as those at the time of the original building design in 1986. It is our understanding that there was also no further significant literature published on the design of diaphragms between 1986 and the early-1990s such that the state of knowledge was similar to that outlined in Section 7.3. This is understandable when considered in context with the economic situation prevailing at the time. By the late-1980s and early-1990s, the number of multi-storey buildings being designed in New Zealand had significantly reduced compared with the mid-1980s.

The important statements made in Section 7.3.7 are that the codes were not entirely clear about how to determine forces for the design of diaphragm connections, particularly in a building with a significant plan irregularity.

Our interpretation of the codes and prevalent literature of the time is that the most suitable method for determining design forces for the connections of diaphragms to ductile shear walls in a horizontally irregular system was to use capacity design procedures taking into account the overstrength capacity of the shear wall mechanism.

For the design of connections of diaphragms to ductile shear walls in a regular system (i.e. the north-south direction of the CTV building), it was suitable to use the requirements for parts and portions from NZS 4203:1984.

10.4 Did Mr Banks Omit to Discharge His Duty?

We reviewed Mr Banks' calculations in relation to the remedial works in two respects; checking of the east-west direction and the design of the drag bars in the north-south direction.

10.4.1 Checking of East-West Direction

We do not agree with Mr Banks' approach to checking the adequacy of the connection between the floor slabs and the North Wall Complex wall element on grid 5. This finding is based on what would have been expected from a structural engineer following fundamental principles of reinforced concrete design.

Figure 10.1 shows all the critical areas that should have been checked by a structural engineer when checking the ability of the connection to transfer a shear force, V*, from grid 5 to grid 4, and adequately develop it into the floor slabs south of grid 4.

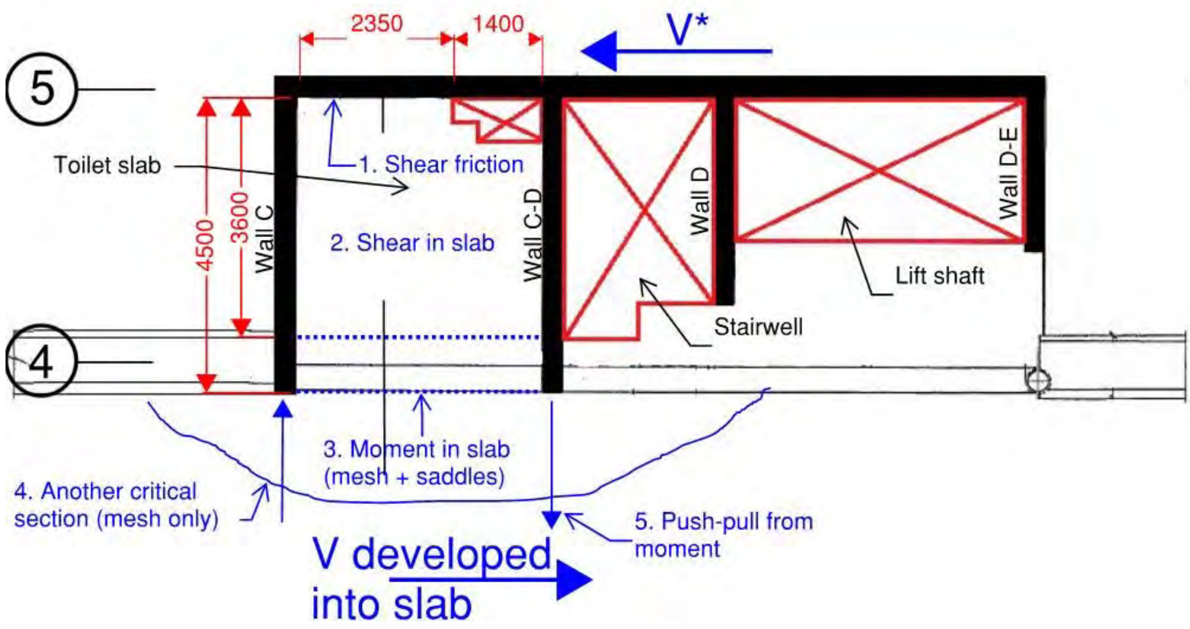


Figure 10.1: Critical areas to be checked in the design of the NWC diaphragm connection for east-west loading

In his approach, Mr Banks:

1. Included the contribution of two layers of 664 floor mesh in the calculation of the shear-friction capacity of the toilet slab to grid 5 wall connection, even though the drawings indicate the mesh is not sufficiently anchored into the wall to justify its full inclusion.

3. Failed to check the moment action effect generated by the 4.5m lever arm between grids 5 and 4 (critical areas 3 and 5 in Figure 10.1).

4. Failed to check a critical section beyond the grid 4 starters towards the middle of the floor slab (critical area 4 in Figure 10.1).

Due to these errors and failings, Mr Banks failed to identify that the connection of the floor slabs to the North Wall Complex for east-west loading was not compliant with the design codes and therefore also required remedial works.

10.4.2 Design of Drag Bars for North-South Direction

Notwithstanding Mr Banks' use of connection force values in the north-south direction that were of the order of what was expected for the time, we have identified that Mr Banks failed to check the development of tension forces in the drag bars into the floor slabs (Figure 10.2). The drag bars only extend a short distance beyond the end of walls D and D-E and create a critical section beyond their termination point that wasn't checked.

Due to this omission, the retrofit design by Mr Banks was flawed and the drag bars unlikely to be capable of fully developing the loads for which they were intended.

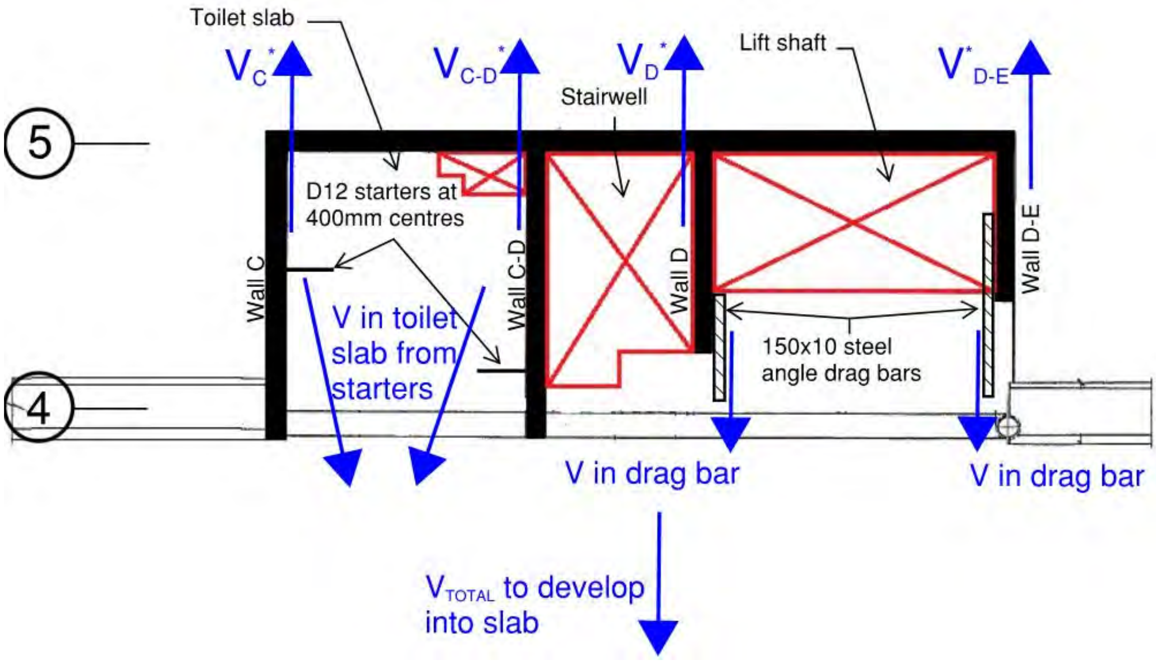


Figure 10.2: Diaphragm connection to North Wall Complex for north-south loading

10.4.3 Conclusion on Discharge of Duty by Mr Banks

We have identified a number of failings in Mr Banks' approach to his role as the engineer responsible for remedying the connection between the floor slabs and the North Wall Complex. The effect of these failings is that the connection for which Mr Banks was responsible for remedying was still not compliant with the relevant codes of the day.

Mr Banks had a duty to ensure the connection of the floor slabs to the North Wall Complex was compliant with the codes of the day. His failure to do so represents an omission to discharge his duty in relation to the retrofit of the building.

10.5 Accepted Practice of the Day

We make comment on the accepted practice of the day in four respects; project team personnel responsibilities, design of diaphragm connections, the retrofit of an identified non-compliant building and pre-purchase reviews.

10.5.1 Project Team Responsibilities

As outlined earlier, Mr Banks was a director of ARCL and carried out the majority of the work involved with the retrofit. He was also a structural engineer experienced in the design of similar structures.

From the responses given during our practitioner interviews, it was common for design firms to allocate one principal or director to be in charge of each project. This director would be ultimately responsible for the project, even if they consulted another director during the project.

Therefore, since Mr Banks was a director of ARCL, and he was a structural engineer sufficiently experienced in this type of work, it was general practice of the day for him to be considered as the principal responsible for the design of the retrofit works, not Dr Reay.

10.5.2 Design of Diaphragm Connections in Early-1990s

As intended by the codes of the day, diaphragm actions and forces in the connections of diaphragms to primary seismic structure were to be determined from a rational analysis. However, through our practitioner interviews, it is apparent there were different practices to determining the level of seismic load that diaphragms should be designed for, especially when seismic loads were required to be distributed between lateral load resisting elements in eccentric buildings. This situation still exists today, albeit to a lesser extent.

From our review of the calculations of several buildings in the mid-1980s, it was noted that many practitioners were using the parts and portions approach to determining diaphragm connection forces, even for irregular buildings when other more suitable methods may have given higher values.

Therefore, we have determined it was accepted practice of the day to calculate diaphragm connection forces for a building such as CTV using the parts and portions provisions from NZS 4203:1984.

To this end, we do not consider Mr Banks' use of parts and portions a departure from the accepted practice of the day.

However, we consider the failure of Mr Banks to check there was adequate development of forces from the floor slabs into the walls of the North Wall Complex for loading in both directions a major departure from the expected standard of the day with respect to fundamental principles of reinforced concrete design.

10.5.3 Retrofit of Identified Non-Compliant Building

HCGs pre-purchase inspection report brought to the attention of ARCL, the successor to ARCE and original engineering designer of the CTV building, that their design was not compliant with the codes of the day.

The HCG report identified the area of non-compliance as the connections of the floor slabs to the North Wall Complex for diaphragm action under lateral loads. With respect to other aspects of the building's design such as the gravity structure and the shear walls, the HCG report stated that the design was either compliant or well-detailed.

However, it has been identified by both the CERC and Beca's investigations that other aspects of the building's design were non-compliant.

In these circumstances, we consider it accepted practice for the original design firm to have focused only on remedying the identified non-compliances and not on other aspects, particularly when the same report has made general statements that the remaining key structural elements were "*well designed to the requirements of*.." or "…*complies in all respects with the appropriate design loading and materials codes*".

10.5.4 Pre-Purchase Reviews

The other question that arises because of the review by HCG of the design is whether the finding that "*the gravity structure is sound and complies in all respects with the appropriate design loading and materials codes*", is a second opinion which reinforces the claim that the original design was compliant (apart from the diaphragm connections).

The CERC found that the HCG review was never intended to be a full peer review of the design of the building, and was carried out in a short timeframe.

A pre-purchase review such as this would, as a general rule, comprise a general review of the drawings and spot checks of some key elements. It would have been clear to the reviewer that the assumed structural concept was for the central gravity structure of frames and columns to be protected from seismic loads and deflection by the shear walls.

Without doing significant analysis, the reviewer would not have been able to check the validity of the assumption that the shear walls were adequately stiff to protect the "*gravity structure*". It does not appear that HCG was given the timeframe to carry out such analyses and therefore would have been unable to validate the conclusion discussed in Section 7 that the frames should have been detailed for ductility.

While it might be argued that a reviewer should have spotted the issue from a "*drawing perusal*", it is not surprising the reviewer did not do the necessary deflection checks under the circumstances of limited time and the fact the reviewers were instructed by their client to stop work. It is noted that no one relied on the result of the review, except the then current owner and ARCL, who relied on the fact that the one error identified was the only one.

10.6 Conclusion

Mr Banks, in his role as a director of ARCL, was responsible for the design of the retrofit works.

It was not a major departure from accepted practice of the day for an original design firm, when addressing an identified and specific non-compliance with their design, to focus only on the identified issue and not be expected to review the whole building.

We identified that Mr Banks made errors in his checks and retrofit design of the diaphragm connections to the North Wall Complex. These errors meant the floor diaphragm connections to the North Wall Complex were still non-compliant with the codes and accepted practice of the day after the retrofit.

These errors represent an omission by Mr Banks to discharge his duty in relation to the remedying of the identified non-compliant connections. We consider this omission to be a major departure from the accepted practice of the day.

The NLTHAs reported on in Appendix H indicated that the response of an otherwise compliant building model was not greatly affected by whether or not the ties were modelled as retrofitted or as fully compliant. The conclusion from these analyses is that the non-compliance of the retrofitted drag bars was not a significant factor in the collapse of the building on 22 February 2011.

[redacted]

11 Other Contributing Factors

11.1 Introduction

Apart from design and construction issues, several other factors have been identified that could have influenced whether the building collapsed or not. They are:

- The code-defined minimum requirements for columns
- The size of the earthquake
- Cumulative effect of multiple earthquakes
- Demolition of neighbouring building
- Foundation softening
- Structural modifications made post-construction.

These factors are discussed below together with our opinion on their likely relevance to the collapse.

11.2 Code-defined Minimum Requirements for Columns

The minimum requirements for the level of transverse reinforcement in gravity-only columns specified by the concrete materials code at the time, NZS 3101:1982, were low relative to the minimum requirements that both preceded and superseded it, and also relative to what was required internationally at the time.

We have concluded that:

- The building collapsed in the manner it did because the columns could not maintain axial load carrying capacity while simultaneously sustaining the lateral deformations (drifts) imposed on them during the February 2011 earthquake. They disintegrated once the demands on their end-regions exceeded the available capacity.
- The transverse reinforcement specified by the designer for the identified critical gravity columns (grid 2 in the ground floor) was compliant with the minimum requirements of NZS 3101:1982.
- If the critical columns had been provided with the minimum level of transverse reinforcement, required by subsequent versions of the design codes, it is unlikely the building would have collapsed in the manner it did during the February 2011 earthquake.

As a result, it is our belief that the low minimum requirements for transverse reinforcement for gravity columns in the concrete materials standard of the day made the primary gravity structure in the CTV building less resilient than it might otherwise have been, and less resilient than it would have been if it had been designed to comply with the 1995 code.

However, our investigations have shown that the requirements of the design codes of the day, albeit less stringent than later required, would have been sufficient to prevent the building collapsing in the manner it did during the February 2011 earthquake.

The use of the low transverse reinforcement option for gravity-only columns in the concrete design code was only permissible if the columns were well protected by the primary lateral structure. To confirm this was the case, the code required calculations to be carried out whereby the assumed primary gravity structure was subjected to the lateral deformations from the primary lateral structure. No such calculations by the designer of the CTV building have been found and it is assumed they were not done. If they had been it would have been apparent to the designer that the primary gravity structure had to sustain flexural actions, requiring at least a minimum level of transverse reinforcement in the joint regions between the beams and the columns.

The levels of transverse reinforcement specified by the designer for the beam-column joint regions were much less than this minimum requirement.

Our investigations have shown that the lack of the minimum level of joint reinforcement resulted in significantly higher inelastic demands on the lightly reinforced columns than would otherwise have been the case. The investigations showed that these demands were sufficient to cause the columns to disintegrate whereas they would not have, by a reasonable margin, if the correct joint reinforcement had been present.

In addition, the designer under-estimated the design actions on the primary lateral structure which led to the specification of a primary lateral structure that was significantly undersized when compared with the minimum required by the design codes. Our investigations have shown that this deficiency also placed significantly higher demands on the primary gravity structure, and in particular the lower level columns, than would have been the case if the primary lateral structure had met the code requirements.

Therefore, while acknowledging that the code compliant columns lacked the resiliency required in later codes and the catastrophic nature of the collapse was as a result of the disintegration of the lower level primary gravity columns, our investigations have shown the collapse would not have occurred in the 22 February 2011 earthquake if the requirements of the design codes of the day had been met in full. We have concluded as a result, that the principle cause of the building failure, in this earthquake, was the design errors/deficiencies that resulted in demands on the columns beyond their available capacity, rather than their low resilience which could be considered a consequence of the code.

11.3 Size of the Earthquake

11.3.1 Introduction

In this section we address the issue of whether the shaking experienced during the 22 February 2011 earthquake was of sufficient severity to cause the collapse of the CTV building if the design and construction non-compliances we have identified in the structure had not been present.

This requires consideration of the characteristics of the earthquake shaking, the performance objectives and requirements of the codes of the day, and how they were generally applied, the performance of other buildings designed in accordance with those codes, insights gained from the performance of the CTV building during this earthquake and also the results of the NLTHA.

11.3.2 Earthquake Characteristics

The characteristics of the 22 February 2011 earthquake are provided in Appendix I. We consider that the CCCC recording site is the most representative of the CTV building site (refer Appendix J for the reasoning). It is the site most similar geologically and geotechnically, of equivalent distance from the epicentre, and the closest site where earthquake shaking was recorded.

The relevant aspects of the CCCC earthquake record are:

Duration: Less than 7 seconds of strong motion was recorded and less than 3 seconds when the amplitude of shaking (measured as ground acceleration) was greater than 65% of the peak values.

Directionality: The indication overall from the records throughout Christchurch city was that the motion had a bias to the east-west direction with the CCCC record approximately northwest-southeast.

Intensity: At the CCCC recording site the following were experienced:

■ Peak horizontal ground acceleration	0.48g
■ Peak vertical ground acceleration	1.6g
■ Peak elastic spectral response	1.1g and 550mm in N64E (structural period ~1.5s)
■ 5% damped acceleration and displacement	0.8g and 420mm in N26W (structural period ~ 1.5s).

The peak elastic response was several times greater than the minimum elastic actions required to be considered for compliance with the Ultimate Limit State (ULS) defined in the then current loadings code, NZS 4203:1984, but the duration of strong shaking was significantly less.

The intention of NZS 4203:1984 was that it allowed for multiple reversals (cycles) of strong motion (8 cycles at the equivalent elastic capacity is referred to in the code clause C3.2). The short duration of strong motion in this earthquake gave fewer cycles than this.

The higher intensity but short duration of this earthquake as recorded generally across the city makes it difficult to compare it with the ULS design loading expectations defined in the code. However, we believe that it would not be unreasonable to consider it to be a severe earthquake, but not as severe as the design loading as a cursory inspection of the elastic spectral response might suggest.

The peak vertical ground acceleration was also high during this earthquake. Vertical ground accelerations have not typically been explicitly designed for in multi-storey buildings. The NLTHAs for both the CCCC and REHS records have shown that the vertical accelerations have a high frequency characteristic (so the axial loads in columns are not sustained for long periods) and also that they occurred early in the records and had significantly reduced by the time the building was subjected to the main horizontal pulses (refer Appendix H).

While vertical accelerations could have resulted in axial loads in columns that were higher in the initial stages of the earthquake than would have resulted from lateral frame action alone, our analyses show they were not a significant contributor to the CTV building collapse as they were not sustained for long enough, nor did they add significantly to the lateral load generated axial loads when the lateral frame actions were at a maximum.

11.3.3 Performance Objectives

The seismic performance objectives behind design codes (loadings and materials) for a building designed at the time, in New Zealand and internationally, could be generally summarised as:

- Able to resist minor to moderate intensity earthquakes without damage, and
- Able to resist major earthquakes without collapse, but with structural as well as non-structural damage expected.

The general intention was to protect property and life for minor to moderate earthquakes, and to protect life in major earthquakes. These objectives, were set out in the commentary to the earlier 1965 code (MP12:1965), and were well understood by practitioners but were not stated specifically in the codes of the mid-1980s. A technical paper (Glogau 1976) describing the philosophy behind the earthquake provisions of NZS 4203:1976 indicated that the objectives in that standard differed little from those outlined in MP12:1965. NZS 4203:1984 was a second edition and largely unchanged from NZS 4203:1976.

The material design standards, such as NZS 3101, the design code of concrete structures, contained detailing and design requirements to meet these objectives.

MP12:1965 stated (clause 5.3):

"The principal concern of the Loadings Committee in arriving at appropriate design standards has been to endeavour to remove the hazard to life as far as practicable. In addition, the standards have been devised for the purpose of minimising damage, both structural and non-structural, for earthquakes of intermediate intensities but the committee has accepted the probability of non-structural damage and of appreciable plastic strain to the basic structures in the event of major earthquakes."

Significant structural damage was therefore expected by the code writers in major earthquakes but it was well recognised that the earthquake code provisions (loadings and materials) were intended to provide buildings with resilience (toughness) to prevent pancaking collapses of the nature that occurred in the case of the CTV building. The observed damage of all other similar modern multi-storey buildings in Christchurch following the 22 February 2011 earthquake demonstrated the resilience that was expected, notwithstanding the high accelerations recorded. (Refer to Section 3.11 and Appendix G).

It would be misleading to consider the minimum design load levels specified by the code for the ULS as being representative of the maximum intensities of shaking which could occur in the "major earthquakes" referred to in MP12:1965. The shaking expected in major earthquakes was deliberately not defined in NZS 4203 as it is not in current codes. The experience from observing the performance of buildings in New Zealand (including in the Christchurch earthquake) and internationally has been that buildings will survive, without collapse, levels of shaking that are much larger than implied by the minimum ULS code loading that they have been designed for, albeit with possible significant structural and non-structural damage. This is recognised by code writers when setting the minimum design requirements other than earthquake loads and minimum requirements for the various construction materials. The earthquake loads in codes are set to a level that *"are reasonably representative of those <u>found adequate</u> in earthquakes"* (MP12:1965) rather than those actually expected in major earthquakes.

The assumption in NZS 4203:1984 was that adherence to the requirements of the codes (loading and materials) would deliver the objectives outlined above. This was achieved through requiring a minimum level of reliable strength in the structure and a level of ductility, by special detailing, in the structural components to allow earthquake motions while retaining the integrity of the elements that were essential to maintain gravity load support.

The requirements for ductility specified in NZS 4203:1984 are not as severe as specified in current codes, particularly the level of displacements buildings are required to be designed for. Therefore, modern buildings could be expected to exhibit greater resilience (ability to sustain earthquake shaking beyond design levels) to resist catastrophic earthquake shaking than buildings designed to earlier standards. Nevertheless, a building designed to the requirements of NZS 4203:1984 and NZS 3101:1982 should have been able to survive the shaking experienced during the series of earthquakes from 4 September 2010 to 22 February 2011 without catastrophic collapse. This is confirmed by the performance of other similar buildings in the Christchurch earthquake and also from the results of the NLTHAs and physical testing completed during this investigation.

11.3.4 Observed Performance of Other Buildings

It is relevant to this discussion that the CTV building was the only modern (post 1976) building to collapse in this earthquake.

We have identified what we understand to be all of the other concrete shear wall multi-storey buildings in Christchurch city that were designed over the period 1984 to 1989. These are listed in Section 3.11. All of these buildings were in the vicinity of the CTV building, and together with the CTV building, are located on the map in Figure 11.1. The characteristics of these buildings are provided in Appendix G. Two of these buildings were designed by ARCE.

A review of Google Earth shows that all of these buildings were still standing as at November 2012. This is not to say that these buildings were not structurally damaged, but it is evidence that they did not collapse during the 22 February 2011 earthquake. These buildings met the objectives set out in 11.2.3 whereas the CTV building did not.

What is of interest is that these buildings differed from the CTV building in two main respects. In all of these buildings the primary gravity structure appeared to have been detailed as primary gravity frames meeting the requirements of the codes, particularly in the level of detailing in the beam-column joints and in the columns. None of these buildings had columns with such low levels of tie reinforcement (referred to as lateral or transverse reinforcement) as the CTV building columns and beam-column joints.

The ability of buildings to resist earthquake shaking considerably in excess of minimum design strength values is not restricted to the Christchurch experience. Internationally, the experience has been that very few buildings that have been specifically designed for earthquake resistance using the philosophy of the two levels of earthquake performance similar to that in the New Zealand codes have collapsed in major earthquakes. Even fewer have collapsed in the catastrophic manner of the CTV building.

Figure 11.1: Location of Six Example Buildings Relative to the CTV Building (Source: Google Earth)

11.3.5 Performance of the CTV Building

The primary seismic structure, being the North Wall Complex and the South Shear Wall, withstood the earthquake forces without collapse. The North Wall Complex remained standing, and the South Shear Wall, which was designed to resist forces in the east-west direction only, fell in a northerly direction over the top of the pancaked slabs only after it lost its support in its very weak direction. It would have been pulled in that direction by the collapsing slabs. The eyewitness accounts indicate that the building was still standing after the most intense shaking had passed, which implies that the primary seismic structure did experience the most intense shaking from the earthquake and would have survived if other parts of the structure had not failed.

The primary gravity structure, being the internal and external beams and columns and the floor slabs, collapsed. It was subjected to horizontal deformations that introduced moments and shears (and axial loads in some columns) that had not been allowed for in the design as required by the codes. The gravity structure failed in a brittle and sudden manner because there was not the shear and binding steel reinforcement the code required to provide the ductility and resilience to be able to survive such movement.

Had the CTV building met the minimum requirements of the codes of the day, our calculations, NLTHAs and the results from the physical testing indicate that it would have had sufficient resilience to survive the earthquake shaking (as recorded on the CCCC recorder) without the catastrophic collapse that occurred.

11.3.6 Conclusions

The available evidence suggests:

- The duration of strong shaking in the 22 February 2011 earthquake was short, and arguably shorter than the code writers were envisaging when preparing the code provisions.
- The elastic spectral response determined, using the ground motion recorded at the CCCC site, was in excess of the minimum design loads defined by the codes of the day. However, elastic spectral response, even significantly beyond design load levels, is not an indication that collapse is imminent or likely even though the building structure may be significantly damaged.
- The vertical accelerations were high but the NLTHAs indicated that these occurred earlier than the strongest horizontal shaking and unlikely to have significantly influenced the collapse.
- The primary seismic structure of the CTV building performed as expected and would have survived this earthquake if the primary gravity structure and its attachment to the primary seismic structure had not failed in the manner they did.
- The primary gravity structure did not comply with code requirements of the day. If it had complied it would have been expected to have had sufficient resilience to have survived this earthquake without collapse even though it may have been severely damaged. The full scale testing described in Appendix K supports this opinion.
- The CTV building was the only post-1976 building to collapse in Christchurch, and therefore also the only building of its structural type to collapse. Other comparative buildings had binding and shear steel in the beam-column joints in the primary gravity structure in accordance with the code. The CTV building did not.

We have concluded that the size of this earthquake, on its own, was not a substantial and operating cause of the collapse.

11.4 Cumulative Effect of Multiple Earthquakes

During the CERC analyses the CTV building was subjected to three earthquakes that caused significant shaking in Christchurch city between 4 September 2010 and 22 February 2011; the Darfield earthquake (4 September 2010), the Boxing Day earthquake (26 December 2010) and the Christchurch earthquake (22 February 2011). Characteristics of these earthquakes are provided in Appendix I.

Although confirmation that the building was not significantly damaged by any of the earthquakes prior to 22 February 2011 is not possible, significant structural damage was unlikely for the following reasons:

- Inspections of the building between September 2010 and February 2011 did not reveal damage that would suggest that the primary seismic structure of the building had been subjected to a significant number of cycles causing yielding of reinforcing steel in either of the earlier earthquakes.
- The combined duration of all three earthquakes is less than that expected for the design earthquake by the code writers.
- The NLTHAs indicated no significant difference in behaviour when analyses were run with and without the September and Boxing Day earthquakes preceeding the February earthquake.
- Observation of the ‘Compliant design/construction errors’ specimen during the physical testing following excursions to drifts representative of the maximum experienced in the September 2010 earthquake indicated no visible damage in the ground floor internal column that was modelled.

The ability of the building to survive the 22 February 2011 earthquake should not have been significantly impaired by the prior earthquakes if it had been properly designed.

11.5 Demolition of Neighbouring Building

We have seen no evidence to suggest that the demolition of the neighbouring building at 213 Cashel Street had any structural effect on the CTV building. There was no excavation and the energy input available from demolition activities would not have been sufficient to cause vibrations in the CTV building of a magnitude that would have resulted in steel yielding or structural damage.

The ability of the CTV building to survive the 22 February 2011 earthquake was not impaired by the demolition of the neighbouring building.

11.6 Foundation Softening

The foundations detailed for the CTV building were substantial, including those under the South Shear Wall and the North Wall Complex. Mr Harding’s calculations show the foundation under the North Wall Complex was designed to yield for earthquake loading in the north-south direction and was found in our assessment to be sufficient to transfer the capacity actions in the walls as detailed. For the east-west direction, the foundation under the North Wall Complex was not detailed for the over-strength of the north wall but nevertheless we found that it met minimum code requirements. We found that the South Shear Wall foundation had sufficient capacity to resist the over-strength actions of the South Shear Wall as detailed, acting in the east-west direction.

There is no evidence that the foundations were damaged during the earthquake, but to our knowledge, neither have they been fully inspected for damage or for soil deformations, during the post collapse investigations to date.

[redacted] at Canterbury University, has proffered softening of soils as a possible explanation for large inter-storey drifts in the primary gravity structure in what he perceives to be in the absence of significant damage to the South Shear Wall. [redacted] has set out his reasons for the view that *“a possible and feasible cause for the building’s collapse could have been foundation movement*

Sections of this document have been redacted to protect the privacy of individuals.

under the southern shear wall leading to a substantial loss of stiffness in the wall" in an affidavit affirmed on 13 August 2013.

Our investigations (including review of the photographic evidence) indicate cracking at the base of the South Shear Wall that was consistent with significant damage having occurred prior to collapse, and consistent with the drifts in the building that have been predicted by our analysis, not allowing for the mechanism postulated by [redacted] Any additional foundation rotation, than already allowed for in our analyses, would have led to less damage at the base of the South Shear Wall.

We have concluded that the mechanism postulated by [redacted] is not supported by the evidence and therefore is not a contributing factor in the collapse of the building.

11.7 Structural Modifications Made Post-Construction

The effect of minor modifications to the structure carried out after 1986 was discussed in Section 5.2 and discounted as having a material influence on the way in which the building collapsed.

The most significant alteration was the cutting of a penetration in the level 2 floor slab in the location indicated in Figure 5.11. The placing of the penetration in this location reduced the length of slab attached to the grid line 1 beam and therefore the ability to transfer loads to the South Shear Wall at level 2. However, as shown in Section 7.3.7, we have calculated the load to be transferred at level 2 to be significantly lower at level 2 than at the upper levels of the building. Even with the reduction in slab-to-beam length, the level 2 slab is calculated to be less stressed than at the upper levels. The penetration is expected to have minimal effect on the seismic load paths through the level 2 slab. Therefore, we have concluded that the introduction of the penetration in this location did not increase the collapse potential of the building.

11.8 Conclusions

There is no evidence to suggest that the factors discussed in this section of the report were a substantial cause of the collapse of the CTV building.

While the loads on the structure during the earthquake shaking on 22 February 2011 were significant, and likely to have been several times greater than the minimum design levels specified in the code of the day, our assessment is that the primary seismic resisting system of the building was adequate to survive the earthquake without collapse. If the building as a whole had been able to exhibit resilience, as did all other similar Christchurch buildings (i.e. with shear wall protected gravity systems), we believe it would not have collapsed in the catastrophic way that it did. The main differences between the CTV building and the other similar buildings in the CBD, was in the detailing of the primary gravity system and the strength of the primary seismic system. The non-compliances and deficiencies in the design of the primary gravity system of the CTV building left it unable to sustain the deformations that would be imposed on it during a large earthquake such as that experienced on 22 February 2011.

The shaking on 22 February 2011 was intense, but the satisfactory performance of other similar buildings and the satisfactory performance of a compliant CTV building during our NLTHAs and the physical testing indicates that the building would have survived the earthquake sequence had it been compliant with the design standards of the day.

12 Cause of Collapse

12.1 Introduction

In this section we provide our opinion on the cause of the collapse and why it occurred as a pancaking failure.

The approach we have followed is to determine a likely collapse sequence based on the available evidence, consider the implications from the results from the NLTHAs and the physical testing we have completed as part of this investigation and then bring these together into an overall conclusion as to the cause of the building collapse.

The impact of the code-defined minimum requirements for columns, size of the earthquake, the cumulative effect of multiple earthquakes, demolition of neighbouring buildings, foundation softening and structural modifications made post-construction on the cause of collapse were discussed in Section 11. It was concluded in Section 11 that these factors were not a substantial cause of the collapse and therefore they are not discussed further here.

12.2 Likely Collapse Sequence

12.2.1 Introduction

Investigators and researchers have proposed a number of possible scenarios for the collapse sequence of the CTV building.

The initiator of the collapse is of interest, but the primary objective is to understand why the collapse occurred as it did whereby the floor slabs suddenly pancaked on top of one another leaving little space between for survival or rescue. Such a failure was not expected in a modern (post-1976) building.

In this section of the report we briefly review and summarise the available information as it relates to the collapse sequence. This includes eyewitness reports, the photographic record, observations made post-collapse, our own analyses of the structure, the NLTHAs completed as part of the CERC, and the NLTHAs and the physical testing we have completed as part of this investigation. Informed by these, we present our conclusions on the initiation and the sequence of the collapse.

The source of the eye witness accounts and photographic record referred to in this section is the CERC report on the collapse.

12.2.2 Eyewitness Accounts

The eyewitness accounts of the collapse are summarised as follows (Level 1 is at ground level):

- The building appeared to lurch to the east and then move east-west and slightly to the north
- Motion tended to be anticlockwise
- Glass on the south face on every floor shattered
- From the outside the collapse appeared to initiate around level 2, although collapse of Level 5 onto Level 4 was also observed
- Viewed from the inside, collapse occurred from below Level 4 on the south side of the building
- Collapse occurred after a pause in shaking.

12.2.3 Photographic Record

Section 5 of the CERC report contains the photographs of the collapsed building that we have referred to in reaching our conclusions. A selection of these is included in Section 6 of this report.

12.2.4 Observations Post-collapse

The following observations were made by witnesses after the collapse. Our interpretation from them, relevant to determining a probable collapse sequence, are shown in italics:

- Floor slabs leaning on North Wall Complex *indicating the floor slabs collapsed in the central bays first and/or the floors collapsed from the south towards the north. This is also consistent with eyewitness accounts from within the building.*
- South Shear Wall draped over the top of the building debris *indicating that it fell after the floor slabs collapsed and was pulled over to the north.*
- Little debris beyond the building to the south *indicating the building was not moving towards the south as it collapsed.*
- Debris extended onto the Madras Street pavement *indicating building was moving towards the east as it collapsed.*
- Pancaked slabs with some relative displacement in the north and east directions *indicating collapse towards the north and east consistent with our interpretation of other observations listed above.*

12.2.5 Computer Analyses

12.2.5.1 General

A catastrophic collapse of the nature that occurred in the CTV building was not expected in a modern building. To investigate why the collapse occurred and why it resulted in the pancaking of the floor slabs we undertook two sets of analyses.

One was a series of static 3D analyses to investigate the possible reasons for the rapid progressive nature of the collapse and the second series involved the completion of several NLTHAs to investigate the overall performance of the building, and in particular the demands on the primary gravity structure during the earthquake(s). These NLTHAs supplemented those which were completed by CERC.

12.2.5.2 Investigation of Progressive Collapse

To inform on the likely reasons for the catastrophic collapse, several 3D analyses of the building were completed. In these analyses the likely effect of removal of a first storey column in each column location was tested. The objective was to determine the ability of the structure to redistribute the gravity loads to other elements and/or the propensity for progressive failure throughout the whole structure if a particular column was removed.

We interpreted from these analyses that failure of an individual column was likely to be tolerable except for the four internal columns C7, C8, C13 and C14. While failure of any one of the other columns could lead to collapse we determined that this was likely to be localised to the immediate region of the failed column. However, failure of any one of the central internal columns would have resulted in progressive overloading of adjacent structure, including adjacent columns with the resulting rapid progression of the collapse.

We cannot be certain that more than one column did not fail simultaneously. However, these analyses indicated that it was the integrity of the central interior columns that was critical to maintaining vertical support for the structure overall and avoiding a pancaking collapse.

12.2.5.3 Non-linear Time History Analyses

The NLTHA results are reported in Appendix H. The step by step behaviour/response of the building during the earthquake shaking as indicated by the NLTHAs is summarised below. The earthquake strong motion used in these analyses was that recorded at CCCC which, although not actually at the CTV site, is believed to be the most appropriate of those available (refer Appendix J).

The sequence of significant movements the building experienced during the NLTHA for the as-designed model using the CCCC record is shown in Figure 12.1. Note: the horizontal building movements have been exaggerated for viewing purposes and are not representative of the scale experienced by the actual building during the 22 February 2011 earthquake.

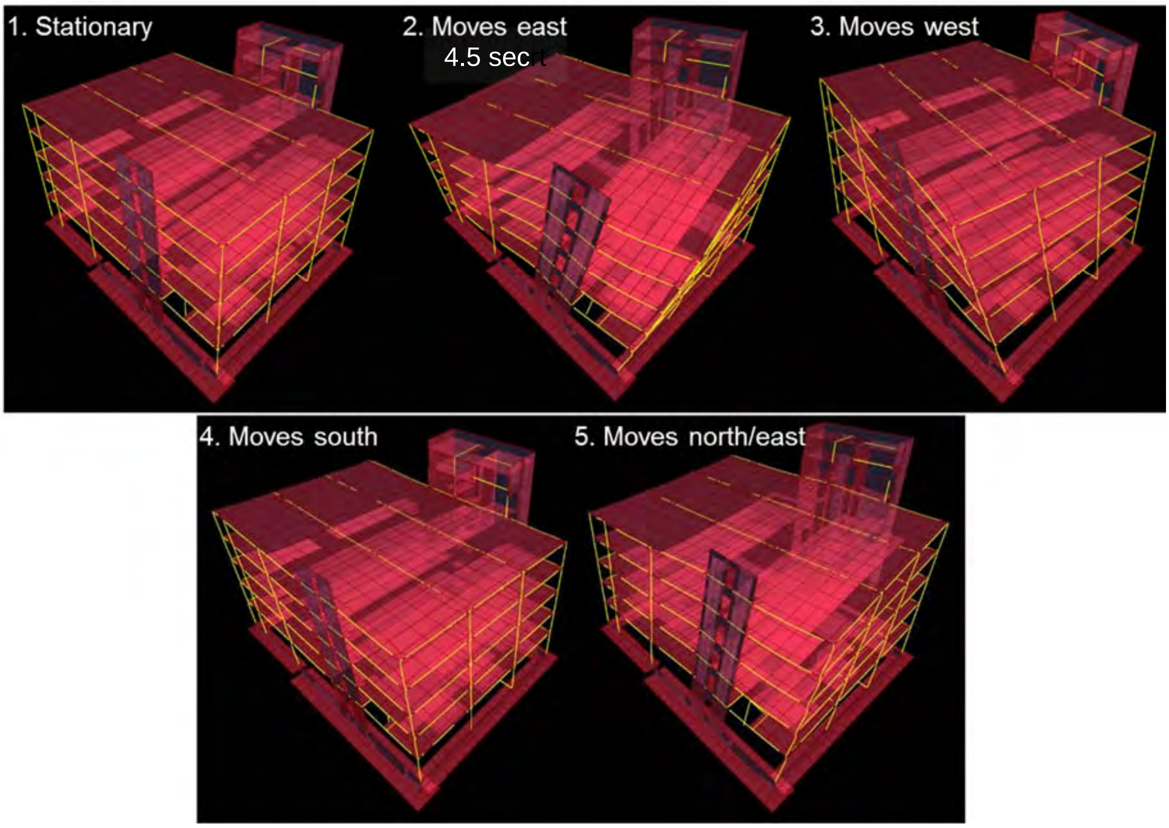


Figure 12.1: Building movement sequence from the NLTHA using the CCCC record for the February earthquake (Source [redacted]). Note the deformations are shown to an exaggerated scale. A time lapse video showing these movements has been produced.

In summary the responses indicated were:

- Approximately 4.5 seconds into the earthquake the building moved significantly to the east followed by movement to the west and north. This was sufficient to cause yield at the base and in the coupling beams of the South Shear Wall and at the column end regions below the soffit of the internal beam on Grid 2 at Level 2 and pivoting of the building, in plan, about the North Wall Complex. This results in high inter-storey drift demands along Grids 1 and 2.
- Between 5 and 6 seconds the building swayed towards the north-west and then south-east with predominant movement in the east-west direction.
- Between 6 and 8 seconds the building movements also included a significant component in the north-south direction.

The in-plane frame demands on the members (columns and beam-column joints) within the primary gravity structure reached a maximum between 4.5 and 6 seconds into the earthquake.

12.2.6 Physical Testing

The physical testing confirmed that loss of axial load capacity in the internal columns C7 and C8 at Level 1 would occur under the peak lateral deflections predicted in the NLTHAs (as-designed model) and that this would be caused by loss of stiffness and strength of the beam-column joints at Level 2 on Grid 2.

12.2.7 Possible Collapse Sequence

The following collapse sequence for the building as it existed on 22 February 2011 fits the observations of eye witnesses during and following the collapse and also the results of the analyses and the physical testing that have been completed as part of our investigation. We acknowledge that the precise initiation and sequence of the failure may never be known with certainty.

1. Building lurches to the east.
2. Movement to the east is followed by movement to the west
3. The relative difference in stiffness between the North Wall Complex and the South Shear Wall results in rotation in plan about the North Wall Complex (movement to the east would appear from within the building to be twisting anticlockwise and from Madras Street to be twisting towards the east and slightly in the northerly direction). The South Shear Wall yields above the foundation and in the coupling beams. Once yield in the South Shear Wall occurs, the resistance to twist for any additional loading is only available from the frames in the primary gravity structure and the torsional resistance of the North Wall Complex. The frames were not detailed for ductility and their ability to carry load would quickly deteriorate once yield occurred. The North Wall Complex was not well configured to resist torsion.
4. The building is now twisting about the North Wall Complex. The south face and Grid 2 (refer Figure 5.2 for grid position) experience significant lateral drift.
5. North-south connection between the North Wall Complex and the floor slabs, at least at some levels, is lost and results in additional twisting causing higher drifts along Grid 2 and on the building perimeter. Vertical load carrying capacity and the shear connection in the east-west direction of the floor to the North Wall Complex is still maintained.
6. Drift in east-west direction is sufficient to:
    a. Shatter the glass on the south face at all levels.
    b. Cause yielding in the internal columns in the primary gravity structure. Yielding occurs at the bottom of the column(s) immediately above the floor slab and also at the top of the column(s) in the region below the soffit of the beam for both east and west directions of motion.
    c. Crack the internal beam-column joints on Grid 2 at lower levels. This results in an increase in the in-plane bending deformations at the base of the internal columns on Grid 2.
7. Building is shaken north-south.

8. Column C18 (refer Figure 5.4 for location) possibly exceeds its capacity in tension at its connection to the North Wall Complex at the top of the building, further reducing the stiffness of North Wall Complex in the north-south direction.

9. Shaking pauses but the building is still moving (twisting) north.

10. Concrete in the interior columns at the lower levels (most likely at the bottom of the ground floor storey) cannot sustain the gravity loads and axial failure is initiated.

11. The building collapses from the centre of the floor slab (as the internal columns collapse).

12. Other interior columns cannot sustain the ever-increasing loads as vertical loads are redistributed. The collapse progresses rapidly throughout the primary gravity structure from Grid 2 through to the North Wall Complex. The floor slabs are left partially hung up on the North Wall Complex and South Shear Wall.

13. Floor slabs fall adjacent to the South Shear Wall leaving the top of South Shear Wall to be pulled north over the top of the debris pile as the upper floors are pulled to the north.

14. The North Wall Complex remains standing but with evidence of yielding.

We believe the internal columns C7 and C8 (refer Figure 5.4 for location) between Levels 1 and 3 were potentially the most vulnerable elements in the primary gravity structure. These columns were the most heavily vertically loaded and our analyses indicated that if these columns had failed the adjacent columns would have been unable to carry the additional redistributed load. Progressive failure of the primary gravity structure therefore followed failure of either, or, both of these internal columns.

While the above is suggested as a probable sequence based on our interpretation of the available evidence, there can be no certainty. Other sequences that also include major floor displacements in the north-south and east-west directions and of a twisting nature could have caused significant damage in the C7 and C8 columns and associated beam-column joints, sufficient for the gravity structure to lose vertical load carrying capacity.

12.3 Implications for Cause of Collapse from the NLTHAs

The NLTHA results using the CCCC strong ground motion record and the subsequent post processing of the results indicated:

- If the primary gravity system had continued to support gravity loads and remained effectively attached to the primary seismic system (North Wall Complex and South Shear Wall), the primary seismic structure would have survived the shaking notwithstanding the underestimation of the required capacity in the original design.
- The under capacity of the primary seismic structure (South Shear Wall) resulted in larger lateral deformations in the building and therefore larger demands on the primary gravity structure than would otherwise have occurred. These were sufficient for the interior columns to have lost axial load carrying capacity.
- The potential for axial failure of the columns within the primary gravity structure was unlikely if the internal beam-column joints had been reinforced as required by the design standards of the day (and thereby remained essentially elastic), and the primary seismic structure had the required capacity.
- The presence of full ductile connection between the North Wall Complex and the floor slabs did not significantly change the response of the building as a whole compared with that when the building was modelled with the retrofitted connections.

- The axial loads due to high vertical accelerations did not have a significant influence on the potential for axial load failure of the internal columns because these had reduced significantly before the occurrence of the peak axial loads due to sway of the building.

The implication of these results is that if the beam-column joints had not degraded in stiffness (ie had been reinforced in accordance with the design standards of the day), the building had sufficient capacity to survive the CCCC shaking. Also if the primary seismic structure had the required capacity, the deformations in the primary gravity structure would not have been sufficient to fail the columns, notwithstanding the non-compliant levels of binding steel in the beam-column joints.

The conclusion from the NLTHAs is that in the absence of the design deficiencies pancaking collapse would not have occurred.

12.4 Implications for Cause of Collapse from the Physical Testing

The physical testing indicated:

- If the internal beam-column joints in the lower levels of the building had been compliant in accordance with the design standards of the day in terms of the level of binding reinforcement (horizontal spirals) provided, column axial failure would have been unlikely in the CCCC earthquake motion
- If the capacity of the primary seismic structure (North Wall Complex and South Shear Wall) had been compliant with the Standards of the day, sufficient protection (control on lateral deformations) would have been provided for the primary gravity structure (and in particular the internal ground floor columns on Grid 2) to prevent column axial failure
- The construction deficiencies (non-roughened beam end construction joints and no joint spiral) were not, on their own, sufficient to lead to axial column failure, if the primary seismic structure had the capacity required of the design standards of the day.

The conclusion from the physical testing is that in the absence of the design deficiencies pancaking collapse would not have occurred.

12.5 Cause of Collapse - Conclusion

The CTV building collapsed on 22 February 2011 because the primary gravity structure was not detailed for ductility and could not sustain the lateral deformations it was subjected to during the earthquake and at the same time continue to carry its vertical gravity loads.

Therefore, the fundamental assumption of the designer that this building had a shear-wall protected gravity system was not justified and not realised in this earthquake.

The primary cause for the catastrophic nature of the collapse, and the large number of fatalities, was the decision by the designer to assume that the primary gravity structure was fully protected by the primary seismic structure and did not need to be designed for seismic actions. The result was a gravity structure that was particularly vulnerable to earthquake shaking. It had little integrity and little resilience once the seismic deformations were imposed on it and once failure had initiated.

Premature failure of the primary gravity structure meant that the primary seismic structure was not able to function as intended by the designer. The observed damage to the North Wall Complex and South Shear Wall and the manner of the failure of the South Shear Wall on top of the debris of the floor slabs (as the final stage of collapse) indicates that the primary seismic structure of the building was adequate to sustain the loads arising from the series of earthquakes, including that on the 22 February 2011.

The collapse was sudden and catastrophic, characterised by pancaking of the floor slabs except where these were “held up” by the North Wall Complex. The nature of the collapse, whereby each slab collapsed onto the slab below leaving very little room between for survival, was the reason for the large number of fatalities.

We have postulated a possible collapse sequence based on our interpretations of the information we had available from the CERC report and our own analyses and physical testing. This is one possible sequence and others have been suggested (e.g. Holmes, Priestley), but the overall cause is consistent with the CERC conclusions.

We believe that collapse was initiated by failure of one or more internal columns in the lower levels of the building. Once these columns failed, the adjacent columns also failed, as they were unable to sustain the additional building gravity loads that were progressively redistributed to them. The lack of resilience and redundancy in the primary gravity structure resulted in the catastrophic failure that occurred.

The beam-column joints had insufficient transverse (binding) spiral reinforcement over the height of the beam-column joint regions and we believe this is the primary reason why the columns failed. The contractor failed to provide the specified steel for these regions but the detailed reinforcing steel would not have been sufficient even if it had been placed.

The lack of adequate transverse reinforcement in the interior beam-column joint regions in the lower levels meant that, once the concrete in these regions cracked, the loss in shear stiffness of the joint led to significantly higher demands at the base of the internal columns for the same imposed inter-storey drift once the column had reached yield. The higher demands at the column base significantly reduced the deformation capacity of the column as a whole compared with that available if the joint had retained its stiffness. These demands were sufficient to lead to axial failure in the interior columns.

The strength of the primary seismic structure and the quantity of horizontal interior beam-column joint reinforcement did not comply with the requirements of the design standards of the day. If these aspects had been compliant, our investigations indicate collapse on 22 February 2011 would not have occurred.

Whether or not the building failed in precisely the manner we have postulated, it is clear that the lack of any significant resilience in the primary gravity structure made this building very vulnerable in severe earthquake shaking, and very susceptible to pancaking of the floor slabs. The lack of resilience was a direct consequence of deficiencies in the design.

[redacted]

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Appendix A

New Zealand Police Briefing Letter

Sections of this document have been redacted to protect the privacy of individuals.

[redacted]

New Zealand POLICE
Nga Pirihimana O Aotearoa

SM 017

14 February 2014

[redacted]
BECA
PO BOX 3942
WELLINGTON 6140

Dear Sir

CTV BUILDING COLLAPSE - 22 FEBRUARY 2011
ENGINEERING EXPERT OPINION BRIEFING DOCUMENT

Introduction

The Police are currently investigating the potential criminal culpability of individuals in connection with the deaths of 115 people, following the collapse of the Canterbury Television Building (CTV) during the 22 February 2011 earthquake.

Following a Police review of the New Zealand Department of Building and Housing Expert Panel Report, and the findings made by the Canterbury Earthquake Royal Commission regarding the collapse of the CTV Building, the Police have identified the potential charges that could be laid under the Crimes Act 1961. One of these charges is manslaughter. Proof of this charge would, to a large degree, depend on expert engineering evidence.

The Law

Under New Zealand criminal law there is no ability for a corporate body to be prosecuted for manslaughter, only an individual (referred to legally as a defendant).

It is important to note that in a criminal prosecution police are required to prove the elements of a crime to the standard of 'beyond reasonable doubt'. The Royal Commission was not required to find facts proved to this high standard but rather only to the lesser civil law standard of the 'balance of probabilities' (more likely than not).

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[redacted]

Beyond reasonable doubt is explained to a jury as being a very high standard that is only met if they are sure a defendant is guilty. It is not enough for the prosecution to persuade them that a defendant is probably guilty or even very likely guilty. On the other hand it is virtually impossible to prove anything to absolute certainty when dealing with the reconstruction of past events and the prosecution does not have to do that. A reasonable doubt is an honest and reasonable uncertainty left in their mind about guilt after careful and impartial consideration of all of the evidence.

Any prosecution of an individual(s) on manslaughter would be based on a breach of a duty imposed by the Crimes Act. That duty requires that anyone who has in his charge or under his control (or erects, makes, operates, or maintains) anything which, in the absence of precaution or care, may endanger human life, take reasonable precaution against and use reasonable care to avoid such danger, and is criminally responsible for the consequences of omitting without lawful excuse to discharge that duty.

To prove manslaughter (which is culpable [ie blameworthy] homicide), the prosecution would have to prove the following elements:

1. the defendant was under that duty (NB this is a legal question and does not require expert evidence) **and**
2. he omitted to discharge that duty **and**
3. that the deaths were **caused** by that omission.
4. that, in the circumstances of the case, the omission was a **major departure** from the standard of care expected of a reasonable person to whom that duty applied.

In considering whether this charge could be proved against any individual the elements which are the more difficult to assess and which the Police require expert engineering input on is whether it can be proved beyond reasonable doubt that:

- any individual omitted to discharge this duty

- any such omission caused the deaths.

- any such established omission was a major departure from the expected standard.

"Major departure" is not defined in the Crimes Act, however it is considered by the courts to be akin to the concept of "gross negligence" under U.K. law. The test for a major departure is objective and the defendant's state of mind is not a pre-requisite to proving a major departure. All the circumstances of the case must be considered and the defendant's state of mind (eg if he knowingly ran a risk or was indifferent to an obvious risk of death) may be relevant in deciding whether there was a major departure.

[redacted]

Determining what amounts to a "major departure" is a question of degree and a value judgement in the circumstances of a particular case.

It could be a "major departure" if it was inattention or failure to advert to a serious risk which went beyond "mere inadvertence" in respect of an obvious and important matter which the defendant's duty demanded he should address.

In defining major departure, a jury is likely to be directed by a judge that they should only convict if satisfied that, having regard to the risk of death involved, the conduct of the defendant was so bad as to amount, in their judgement, to a crime.

Proof that the omission "caused" the deaths requires that the prosecution prove that the omission was a "substantial and operating" cause of the deaths. It need not be the only cause. There can be other contributing causes, independent of the omission in question, provided that the omission by the defendant was a substantial and operating cause of the deaths.

There could also be other contributing causes which could be attributed to the omissions of others to perform a similar duty, provided the omission by the defendant was still a substantial and operating cause of the deaths.

Therefore, the Police require expert engineering opinion, essentially to address these three issues:

1. Was there an omission by any individual to discharge the duty?

2. Was the omission a substantial and operating cause of the deaths?

3. If so, was that omission a major departure from the expected standard?

In forming an opinion, consideration needs to be given to whether the relevant elements could be established to the required standard. In determining the degree of certainty that can be attached to an opinion reached, the strength and reliability of the evidence available needs to be considered. Additionally, contradictory expert opinions, considered valid or otherwise, need to be objectively assessed and considered. Ultimately, if there is a contrary expert opinion on a particular element that cannot be reasonably dismissed, it is unlikely that element would be able to be proved beyond reasonable doubt.

The Department of Building and Housing report and the Royal Commission's findings considered a number of matters which may have to be taken into account in forming an opinion on these three issues. They include:

- Practices in 1986 (including building codes of the day and their interpretation and technology available)

[redacted]

- The Christchurch City Council's permit approval process in 1986 (including the fact that Council engineers checked and approved permits)

- Can one be sure that the building would have collapsed in the way it did if the construction defects (e.g. failure to roughen, presence of bent–back bars, concrete strength) were not present?

- What effect does the fitting of the drag bars in 1991 have on the issue of whether the collapse was caused by design faults and/or construction faults?

- The February 2011 earthquake is acknowledged to have been approximately two times the design level earthquake contemplated by the 1986 building codes. The September 2010 earthquake is acknowledged to have been at that design level earthquake. What affect does that have on the standard a reasonable design engineer could have been expected to meet in 1986?

CTV Building Design Related Issues - Focus of Police Considerations

Many contributory factors have been identified in relation to the collapse of the CTV Building. A range of views are held by experts and opinion is divided as to the contributory factors leading to the building collapse as well as the actual collapse sequence itself. Furthermore it is noted that the relative weight that can be placed on any one or group of contributory factors, to the exclusion of other possibilities, is the subject of divergent expert opinion.

It is acknowledged that inquiries conducted into the collapse of the CTV Building have identified issues with the design, council permitting, construction (materials and workmanship, as well as supervision by the council, designer and engineer), as well as issues associated with post earthquake events in September and December 2010 including assessment, monitoring and subsequent response or lack thereof. There is also the fact that assessments made of the building, completed on behalf of prospective purchasers in 1990, lead to some major non-compliant design issues being identified.

The focus of considerations now being made by the Police as to the possibility of criminal culpability have however been narrowed down to the acts and/or omissions of persons associated with the design and to a lesser degree the construction of the CTV Building. This focus has resulted from the issues associated with the non-compliance of the design with required standards and City Council Bylaws of the day (subject to variable interpretation of the Building Code).

[redacted]

Persons Whose Acts or Omissions are in Question regarding Design and/or Construction Faults

David Harding:

Structural Engineer employed by Alan M Reay Consulting Engineer. He was severely criticised by the Royal Commission who noted that he:

- had no experience in designing multi-storey buildings
- worked unsupervised (and did not ask for supervision)
- failed to carry out certain important calculations
- was not aware that he needed to calculate deflections at the corner of the building
- failed to track load paths
- failed to ensure the floors were adequately tied to the north shear core

The Royal Commission criticised the specifications in his drawings and, implicitly, that he did not adequately or competently supervise the construction.

He had left Alan M Reay Consulting Engineer by the time a Holmes Consulting Report was completed in 1990. He was made aware of the issues identified (as Reay discussed the report with Harding, apparently to see if the problem had been resolved during construction).

Dr Alan Reay:

Dr Reay recruited David Harding to replace previous employee John Henry. Henry was experienced in multi-storey buildings, including Landsborough House (a shear core design). Although Dr Reay must have known that David Harding did not have any experience in the design of a multi-storied building such as this one, he essentially left Harding to work unsupervised on the CTV project. This is the subject of strong criticism in the Commission's report.

The Royal Commission found Dr Reay had a role in convincing the Christchurch City Council Engineer (Bluck, Tapper's supervisor) to grant a permit for the CTV Building in circumstances where, on his own evidence, he had little knowledge of the project and its structural design.

Dr Reay was made aware of the HCG 1990 report. He distanced himself from the (lack of) permit application for the drag bar retrofit. Both he and Geoff Banks resisted the idea that the nature of the problem HCG identified meant all of Harding's design work on the CTV Building should have been reviewed.

[redacted]

Graeme Tapper (deceased):

The Christchurch City Council Reviewing Engineer who identified defects with the structural plans (including the north shear core connections) but signed off on the structural aspects of the building permit application.

Bryan Bluck (deceased):

Graeme Tapper's boss. Hearsay evidence (of Peter Nichols, former Christchurch City Council (CCC) employee) indicates that after Graeme Tapper's letter setting out issues with the structural plans had been received, Dr Reay went over Tapper's head directly to Bluck and convinced him the building was sound. A further inference is drawn that Bryan Bluck influenced Tapper, which resulted in Tapper signing off the structural aspects of the permit application when Tapper did not agree with that decision (but, by inference, was directed to do so).

The Christchurch City Council (CCC) accepts, as it did in opening, that the CTV building did not comply in certain respects with Bylaw 105. However the CCC did not go so far as to expressly concede the permit should not have been issued. The Royal Commission findings on whether CCC Reviewing Engineer should have identified the column confinement and beam column joint non-compliance is not clear.

Geoff Banks:

He completed calculations for the 1991 drag bar retrofit following receipt of the Holmes Consulting Group report. Banks played a significant role in relation to the (lack of) building permit application for the 1991 work.

The Commission identified a significant weakness in the connections to the north wall complex, but concluded that for the north-south direction this was remedied by the drag bar retrofit in 1991 (even though not on all floors). However it identified that the defect in the east-west direction was not remedied. The consequence of the defect was that "The design forces that were used to resist seismic forces in the east-west direction were approximately half of the values that would have been consistent with NZS 4203:1984." (pg 261)

Banks' evidence [TRANS.20120817.83] was that Harding must have used the static equivalent method as opposed to those derived from the parts and portions section of the code, and that Harding's figure for the required loading connections in the east-west direction was 300 kN but that Banks' figure using the parts and portions provisions was 724 kN.

The Commission held that: "The failure to apply for a permit was a clear omission which meant that the inadequacy of the connections to the north wall complex in the original design was not drawn to the attention of the Christchurch City Council in 1991." (pg 304)

[redacted]

The report begs the question whether, if Banks' structural plan had been submitted to the Christchurch City Council for a permit, a permit should have/would have been granted.

Construction (Bill Jones/ Gerald Shirtcliff):

Bill Jones was the foreman for the majority of the construction of the CTV building. Gerald Shirtcliff was employed by Williams Construction Ltd as a Construction Manager for part of the period of construction (although the evidence at the Commission showed that he had very little to do with the construction of the building).

Construction defects highlighted by the Commission included: failure to roughen pre-cast surfaces, potential low concrete strength and bent-back bars in some of the beams adjoining the north core.

These are the main issues identified during the Police review of the evidence presented and considered at the Commission of Inquiry and the Building and Housing investigation.

There may be other issues that have been identified that you recognize as requiring your consideration. The list above is by no means restrictive.

[redacted] will be available to meet with you on Wednesday the 19th of February to discuss any issues that arise from this letter. He will also provide you with other documentation that you may require.

Please feel free to contact me at any time if there are issues in respect of this matter that you need to discuss.

Yours faithfully

Peter Read
Detective Superintendent:
Southern

[redacted]

Appendix B

Documentation Reviewed by Beca

Appendix B Documentation Reviewed by Beca

The following table lists the documents produced by others in relation to either the CTV building or other relevant buildings that Beca have reviewed in producing their opinion report on the collapse of the CTV building.

Title	Description	Originator	Beca's Source
Amuri Chambers	1985 signed structural drawings of Landsborough House (287 Durham Street)	Alan M. Reay Consulting Engineer	CERC website
New Office Building - Madras St for Williams Development	1986 structural calculations of CTV building	Alan M. Reay Consulting Engineer	CERC website
Office Building - 249 Madras Street	1986 signed permit set structural drawings of CTV building	Alan M. Reay Consulting Engineer	CERC website
Office Building for Mair & Co. Ltd	1985 signed permit set structural drawings of 79 Cambridge Terrace	Alan M. Reay Consulting Engineer	MBIE
Structural Specification	1986 structural specification for construction of CTV building	Alan M. Reay Consulting Engineer	CERC website
249 Madras St - Diaphragm Check	1990/1991 calculations relating to the retrofit of the CTV building	Alan Reay Consultants Limited	CERC website
Secondary Frame Design Review Report	2012 report submitted to the CERC	Alan Reay Consultants Limited	CERC website
Seismic Analysis Report	2012 report submitted to the CERC	Alan Reay Consultants Limited	CERC website
Seventh Statement of Evidence of Ashley Henry Smith, comments on the evidence of Douglas Alexander Latham	2012 statement of evidence submitted to the CERC inquiry	Ashley Smith	CERC website
CTV Elastic Response Spectra Analyses	Report by Athol Carr summarising ERSA panel review for the CERC	Athol Carr	CERC website
CTV Non-linear Time History Analyses	Report by Athol Carr summarising NLTHA panel review for the CERC	Athol Carr	CERC website
CTV Building Drag Bar Capacity Assessment	2012 report for the CERC	Beca	CERC website
Supplementary Statement of Evidence of Brendon Archie Bradley	2012 supplementary statement of evidence submitted to the CERC inquiry	Brendon Bradley	CERC website
Amended Composite Statement of Evidence of Alan Michael Reay - Code Compliance	2012 statement of evidence submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay	CERC website
Composite Statement of Evidence of Alan Michael Reay - Design Issues	2012 statement of evidence submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay	CERC website
Composite Statement of Evidence of Alan Michael Reay - Holmes Report and Retrofit Issues	2012 statement of evidence submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay	CERC website
First Statement of Evidence of Alan Michael Reay	2012 statement of evidence submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay	CERC website
Second Statement of Evidence of Alan Michael Reay	2012 statement of evidence submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay	CERC website
Third Statement of Evidence of Alan Michael Reay - Evidence in Reply	2012 statement of evidence submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay	CERC website
Closing Submissions for Alan Reay Consultants Limited and Dr Reay	2012 closing submissions submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay Consultants Limited and Alan Reay	CERC website
Memorandum of Counsel for Alan Reay Consultants Limited and Dr Reay, 24 July 2012	2012 memorandum of counsel submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay Consultants Limited and Alan Reay	CERC website
Memorandum of Counsel for Alan Reay Consultants Limited and Dr Reay, 25 July 2012	2012 memorandum of counsel submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay Consultants Limited and Alan Reay	CERC website
Opening Address for Alan Reay Consultants Limited and Dr Reay	2012 opening address submitted to the CERC inquiry	Buddle Findlay on behalf of Alan Reay Consultants Limited and Alan Reay	CERC website
CTV Building - 249 Madras Street - Investigation of Foundations	2013 letter to MBIE highlighting a soil deformation phenomenon with attached affidavit of [redacted]	[redacted]	NZ Police
Statement of Evidence of Brendon Archie Bradley	2012 statement of evidence submitted to the CERC inquiry including attachments	Buddle Findlay on behalf of Brendon Bradley	CERC website
Canterbury Earthquakes Royal Commission Final Report, Volume 6, Canterbury Television Building (CTV)	2012 volume of the CERC report relating to the CTV building	Canterbury Earthquakes Royal Commission	CERC website
Critique of CTV Site Examination and Materials Tests, and CTV Building Collapse Investigation	2012 critique report for the CERC of Hyland/Smith reports	Cement & Concrete Association of New Zealand	CERC website
CTV Building, 1986 Code Compliance ETABS Analysis Report	2012 report for the CERC	Compusoft Engineering Limited	CERC website
CTV Building, Non-linear Seismic Analysis Report	2012 report for the DBH	Compusoft Engineering Limited	MBIE website

[redacted]

Title	Description	Originator	Beca's Source
CTV Building, Non-linear Time History Analysis (NLTHA) Report	2012 report for the CERC	Compusoft Engineering Limited	Compusoft Engineering Limited
Five Storey Office Building, 115 Kilmore Street	1986 signed permit set structural drawings of 115 Kilmore Street	[redacted] Consulting Engineer	MBIE
Submissions of Holmes Consulting Group and John Hare in respect of the hearing into the collapse of the CTV building	2012 submissions to the CERC inquiry	DLA Phillips Fox on behalf of Holmes Consulting Group	CERC website
Brief of Evidence of Geoffrey Nigel Banks	2012 brief of evidence submitted to the CERC inquiry	Duncan Cotterill on behalf of Geoff Banks	CERC website
Closing Submissions on behalf of Geoffrey Nigel Banks	2012 closing submissions submitted to the CERC inquiry	Duncan Cotterill on behalf of Geoff Banks	CERC website
CTV Building - 249 Madras St	2012 letter from [redacted] for Alan Reay Consultants Limited regarding modulus of subgrade reaction values at the CTV site	Geotech Consulting Limited	CERC website
Structural Report, Office Building, 249 Madras Street	1990 pre-purchase inspection report of CTV building including calculations	Holmes Consulting Group	CERC website
CTV Building Site Examination and Materials Tests	2012 report for the DBH	Hyland Fatigue + Earthquake Engineering	MBIE website
Second Brief of Evidence of John Henry	2012 brief of evidence submitted to the CERC inquiry	John Henry	CERC website
Statement of Evidence of John Henry in relation to the CTV building collapse	2012 statement of evidence submitted to the CERC inquiry	John Henry	CERC website
An Alternative Collapse Scenario for the CTV Building	Submission to the CERC	John Mander	CERC website
Statement of Evidence of Michael John Nigel Priestley in relation to the CTV Building	2012 statement of evidence submitted to the CERC inquiry	Nigel Priestley	CERC website
New Development for Central City Estates	1989/1990 signed permit set structural drawings for 9 Cathedral Square	[redacted]	MBIE
Oxford Tce. Project	1986 signed structural drawings of 32 Oxford Terrace	[redacted]	MBIE
Brief of Evidence of David Harding in relation to the CTV building	2012 brief of evidence submitted to the CERC inquiry	Saunders & Co on behalf of David Harding	CERC website
Closing Submissions of Counsel for Mr Harding in relation to the hearing into the collapse of the CTV building	2012 closing submissions submitted to the CERC inquiry	Saunders & Co on behalf of David Harding	CERC website
Supplementary Brief of Evidence of David Harding in relation to the CTV building	2012 supplementary brief of evidence submitted to the CERC inquiry	Saunders & Co on behalf of David Harding	CERC website
Site Investigation Report, Proposed Development, 249 Madras Street	1986 geotechnical report for CTV building site	Soils & Foundations (1973) Limited	CERC website
Closing Submissions of Counsel Assisting in respect of the hearing into the collapse of the CTV building	2012 closing submissions submitted to the CERC inquiry	[redacted] Counsel Assisting	CERC website
CTV Building Collapse Investigation	2012 report for the DBH	StructureSmith and Hyland Fatigue + Earthquake Engineering	MBIE website
CTV Building Geotechnical Advice	2011 letter report for the DBH including 2012 addendum	Tonkin & Taylor Limited	MBIE website
CTV Building Geotechnical Advice - Addendum 2	2012 letter addendum for the DBH	Tonkin & Taylor Limited	CERC website
Development Cambridge Terrace, Methodist Trust Association	1984 signed permit set structural drawings of 227 Cambridge Terrace	[redacted]	MBIE
Peer Review, CTV Building Collapse Investigation	2012 peer review of Hyland/Smith report for the CERC	William T. Holmes	CERC website

[redacted]

Appendix C

IPENZ 1986 Code of Ethics

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PROFESSIONAL INFORMATION

For Members

OF THE

Institution of Professional Engineers New Zealand Incorporated

A special issue in the IPENZ Transaction series

1986

[redacted]

CHAPTER 4

The Institution of Professional Engineers Code of Ethics

Rule 18.2 of the Rules of the Institution of Professional Engineers New Zealand states:

"Each member shall so conduct himself as to uphold the dignity, standing and reputation of the Institution and of the profession".

In furtherance thereof:

1. Each member shall exercise his professional and technical skill and judgement to the best of his ability and shall discharge his professional and technical responsibilities with integrity.

2. He shall refrain from and discourage criticism in public of the work of another member. This does not preclude an engineer in his professional capacity from providing responsible comment on the work of another engineer when called upon to do so in the course of his employment.

3. He shall not review the work of another member without taking reasonable steps to ensure that such member is informed.

4. He shall not attempt to supplant another member already engaged except where a duly constituted appeal system operates.

5. He shall not improperly solicit work either directly or through an agent nor shall he reward any person, by commission or otherwise, for the introduction of work.

6. He shall not misrepresent his competence nor, without disclosing its limits, undertake work beyond it.

7. When called upon to give an opinion in his professional capacity he shall give an opinion that is objective and reliable to the best of his ability.

8. However engaged, he shall at all times recognise his responsibilities to his employer or client, others associated with his work, the public interest and his profession.

9. When his professional advice is not accepted he shall take reasonable steps to ensure that the person overruling or neglecting his advice is made aware of the possible consequences.

10. In respect of his professional relationship, whether with employer or client, a member shall disclose any financial or other interest he may have which might impair his professional judgement.

11. He shall not disclose any confidential information or matter related to his work or the business of his client or employer, without the express authority of that client or employer.

12. He shall not accept from nor give to any third party anything of substantial value.

13. He shall encourage the further education and training of his subordinates particularly those who are candidates for corporate membership of the Institution.

14. He shall maintain and strive to improve his professional competence by attention to new developments relevant to his professional activity and shall encourage those working under him to do likewise.

15. He shall strive to relate his work to the preservation or enhancement of the environment and to make effective use of available resources of manpower, machines, materials and money.

16. When practising as a consulting engineer he shall conduct his affairs in accordance with this Code and shall also observe the requirements of the Institution's current Code of Practice for Consulting Engineers.

17. In connection with work in another country where established professional practices differ from this Code he may adopt local practices but shall be guided by the principles contained in this Code. He shall observe Rule 18.2 at all times.

BY ORDER OF THE COUNCIL
Wellington, February 1986

[redacted]

Appendix D

ACENZ 1985 Code of Professional Practice

Code of Professional Practice for Consulting Engineers

A great deal of work has been done in the last 12 months by the Association of Consulting Engineers, and members of Council, on the Code of Professional Practice for Consulting Engineers. The final draft was formally approved by the Council on July 3, 1985 and is shown below in full.

Members of the Institution of Professional Engineers New Zealand who practise on either a temporary, part-time or permanent basis as consulting engineers, whether on their own behalf or as partners of a professional practice or as shareholders or directors of an incorporated practice engaged in consulting engineering, are required to conform to the Institution's Code of Ethics and thus to the following Code of Professional Practice for Consulting Engineers.

1. The entity in which a member practices shall:
   (a) carry on the business of Consulting Engineers and such other professions which are directly incidental to the profession of Consulting Engineering and no other;
   (b) be so constituted that all decisions of the practice shall be controlled by corporate members of the Institution, or a person professionally qualified in accordance with Appendix IV to this Code; and
   (c) be independent of any interest that could reasonably be expected to control or diminish independence of judgement in any act, activity or conduct.
2. (a) A member shall not practise in a partnership which claims as a partner one who is not a corporate member of the Institution or a person professionally qualified in accordance with Appendix IV to this Code;
   (b) In the case of an incorporated practice in which a member is engaged as a shareholder or director the Memorandum and/or Articles of Association of such a company shall comply with the requirements of Appendix I to this Code and any other requirements approved by the Institution from time to time.
3. Notwithstanding the provisions of Clause 2 hereof a member shall not be restricted from entering into a specific arrangement or association provided that the arrangement:
   (a) does not interfere with the independence of the member;
   (b) does not impede the observance of the Code of Ethics;
   (c) is limited in duration and limited to the execution of one single project; and
   (d) a description of the arrangement is notified to the Institution at or prior to its commencement.
4. The name of an entity in which a member practises shall not include any word or names which are misleading or adjectives other than factual ones.
5. The names of all partners in a partnership in which a member practises shall appear on the firm's letterhead.
6. A member shall provide professional engineering services in accordance with the Conditions of Engagement for Consulting Engineers approved by the Institution from time to time, except that it is permissible for the member to act in an honorary capacity for any charitable, religious, educational, or sporting body in which he is interested.
7. A member shall not accept an engagement from a client in replacement of another Consulting Engineer without first ascertaining from the other Engineer that his appointment has been properly terminated in writing.
8. A member may properly use circumspect advertising to announce the member's practice and availability. The form of communication used, and the content of the announcement, shall be dignified and free from any matter that could bring disrepute to the profession. Advertising must be informative and not comparative, and must be free from ostentatious or laudatory expressions and implications. Requirements for certain particular forms of advertising are set out in Appendix II to this Code.
9. A member shall pay all monies, except tender document deposits, received for or on behalf of clients into a trust account maintained by that member and shall comply with the requirements of Appendix III to this Code and any other requirements approved by the Institution from time to time.
10. A member shall not act as a medium for payments made on behalf of a client to a contractor.
11. Without limiting any of the foregoing, a member shall at all times ensure that in carrying on the profession of consulting engineering, whether as a member of a firm or as a director or shareholder of an incorporated practice, such firm or company does not do or permit to be done or become engaged in any act, activity or conduct which is contrary to the Code of Ethics of the Institution or this Code of Professional Practice or which is inconsistent with the integrity of the profession.

Appendix I to Code of Professional Practice for Consulting Engineers Requirements for a company limited by shares engaged in consulting engineering

1. The right to vote at general meetings shall be on the basis of one vote per voting share.
2. Not less than 51 percent of the issued share capital and not less than 75 percent of the shares carrying a right to vote at general meetings shall be held by:
   (a) a person or persons who are corporate members of the Institution or persons professionally qualified in accordance with Appendix IV

of this Code; or

(b) a company or companies:

(i) of which the total shareholding is held by a person or persons who are qualified in (a) above, and

(ii) which otherwise comply with this Appendix.

3. Not more than 25 percent of the shares carrying the right to vote at general meetings may be held by members of the company who have professional qualifications directly incidental to the profession of consulting engineering, and who are not corporate members of the Institution, or persons not professionally qualified in accordance with Appendix IV to this Code.

4. Not more than 10 percent of the shares carrying the right to vote at general meetings may be held by members of the company having non-professional qualifications.

5. No fewer than 75 percent of the directors shall be corporate members of the Institution or persons professionally qualified in accordance with Appendix IV to this Code.

6. No fewer than 50 percent of those directors present at and having the right to vote upon any matters at a properly constituted meeting of directors shall be corporate members of the Institution or persons professionally qualified in accordance with Appendix IV to this Code. The chairman of such a meeting shall be a corporate member of the Institution or a member of other professional bodies recognised for this purpose by the Institution.

7. No share which, under Clause 2 of this Appendix, is required to be held by a corporate member or members of the Institution or persons professionally qualified in accordance with Appendix IV to this Code may be held by any persons other than the beneficial owner or by a member or members of the Institution, or a company which complies with Clause 2 hereof.

Appendix II to Code of Professional Practice for Consulting Engineers Requirements for particular forms of advertising

1. Journals, directories and newspapers

Advertising shall be limited to a professional card stating:

— name and address of the member or firm

— the professional services offered

— the names of the principals of the firm and their professional qualifications

— emblem and name of their professional associations.

A card in a news feature associated with a specific project in which a member has been involved may include a brief factual statement describing the professional services provided on that project.

2. Material for publication in newspapers or technical or business journals

Statements or articles shall be based upon the member's professional knowledge and experience, and may cover topics relating to the member's own work or topics of general engineering interest. Publication of the name of the member or firm, professional qualifications and associations shall be the only credit received.

3. Radio and television

A member shall not advertise on radio or television.

4. Material issued by other parties

A member shall not be a party to, nor contribute to advertisements, statements or articles issued by other parties, unless the member is able to exercise control over the content of such statements or articles as to compliance with the requirements of the Code of Professional Practice for Consulting Engineers.

Appendix III to Code of Professional Practice for Consulting Engineers Requirements relating to the establishment of trust accounts

1. Monies received for or on behalf of clients shall not be paid into a member's own practice bank account but paid into a trust bank account maintained by that member for that purpose.

2. The funds shall be held exclusively for the client for whom or on whose behalf they are received and shall be paid to that client or disbursed in accordance with arrangements made between the member and the client. No member shall draw a cheque in his own favour on a trust or management account for fees, disbursements, etc other than with the support of an invoice previously rendered to the client and countersigned by him.

3. At all times, there must be sufficient funds in the trust account to cover all client balances and on no account should such a bank account be operated on an overdraft basis.

4. The trust account must be suitably named or identified and any interest received on clients' monies in a trust bank account shall be accounted for to the client or clients concerned.

5. No payments shall be made from a trust bank account to or on behalf of clients in excess of the amount held to the credit of the clients.

6. Transactions through a trust account should be adequately recorded on a trust ledger which should be balanced monthly and show transactions on behalf of and balances held to the credit of individual clients.

7. Separate ledger records shall be maintained for each client.

8. A reconciliation statement shall be submitted to each client at intervals of not more than three months.

9. No funds shall be invested on behalf of a client without his specific authority.

10. A client shall receive a copy of all certificates issued by the member, endorsed to the effect that payment has been effected on his behalf.

Appendix IV to Code of Professional Practice for Consulting Engineers Recognised professions

In terms of the Code of Professional Practice persons holding the following qualifications may be partners in consulting engineering practices, or directors/shareholders in terms of Appendix I to the Code of Professional Practice —

Architects registered under the Architects Registration Act of New Zealand

Surveyors registered under the Surveyors Act of New Zealand.

Other qualifications—partnerships

In the case of partnerships other qualifications may be submitted to the Council of the Institution, whose approval must be obtained before making any commitment.

In general, qualifications of prospective partners should be in fields related to consulting engineering and be of similar standard to those required for corporate membership of the Institution. □

[redacted]

Appendix E

Record of Beca Interviews on Practice of the Day

Sections of this document have been redacted to protect the privacy of individuals.

Appendix E

CTV Building Collapse - Engineering Opinion Report

Contents

[redacted]

E.1 [redacted]

Date/Location of Interview: 28 March 2014, [redacted].

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	26 or 27 total staff in 1986. Approximately 10 technicians and 15 engineers plus a couple of admin staff.
A2. Was the company multi-disciplinary at the time or just structural?	Predominantly structural, small amount of civil.
A3. What locations did the company typically do work in?	Office only in Wellington, majority of work in Wellington but some in Lower North Island (Hawkes Bay, Wanganui), maybe one in Hamilton, and one in Christchurch (96 Hereford Street).
A4. What type of structural work was the company typically carrying out during this time?	Full range from small domestic to multi-storey commercial. A bit in health sector and government.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Limited Liability.
A6. What was your position/role?	Director from 1985.
A7. What was your experience at the time (years, type of work, etc)?	Started work in 1970/71, went back to uni to complete a Masters in 1971/72 then [redacted], then CIT, before [redacted].
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	No personal relationship. Met but not acquaintances.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system?	Hard, tried to make as regular as possible.
B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Not as such, although did make reference to "secondary frames" in calculations. Included both secondary frames and primary frames/walls in analysis models.

[redacted]

B3. How was the primary seismic system typically analysed? Please include response on: Which computer programs were used? Were secondary elements included in the analysis of the primary seismic force-resisting system? Was an equivalent static method used or modal response spectrum, or both? Was accidental eccentricity considered? If Yes, in what way? What allowance was made for cracking in the primary system?	Up to 1983 used an in-house program developed by [redacted] that carried out 2D linked-frame analyses incorporating shear deformations (walls), rigid end blocks, but no graphical user interface. Bought ETABS around 1983 which did 3D and modal. Carried out both ESM and modal analyses. Was including the secondary frames in the analysis model including the contribution from their stiffness. For modal, scaled to 0.9CdWt, checked modal storey shears at least 80% of ESM storey shears, etc. Probably did modal at COM initially, then once storey forces were determined applied them at the offset locations +/-0.1b in a static analysis. For primary frame members, generally used gross properties for the columns, 0.5Ig for rectangular beams and other values if L or T-shape beams.
B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	Checked secondary framing elements for yielding at μ times the deflection. Generally used gross properties for the secondary beams and columns as assumed not a lot of cracking. This may attract a bit more load but its effect was allowed for in design. Through this process, generally by default the secondary framing members contained at least limited ductile level of detailing.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	Imposed displacements as part of overall model, used proportion of gross properties as above for B4.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	Imposed displacements as part of overall model, were using end blocks with infinite rigidity but varying length of block, which required an adjustment to obtain column face moments. Choice of length generally made by looking at aspect ratios of beams and columns, half the joint width was a typical assumption. Were even using rigid end blocks prior to ETABS in 1983.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	In interests of efficiency (design and draughting time), would apply to all similar frames, i.e. made global decisions.
B8. What was your inclination to choose non-seismic or limited ductile detailing?	As discussed above, by default most were probably at least limited ductile.
B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	Not immediately familiar with how choice was made but made the comment that would not take advantage of it and that his philosophy was that the code was a minimum requirement, not a maximum.

B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	Mentioned that they weren't doing many shear wall buildings but when they were, recalled using diaphragms with boundary beams to increase the length of connection.
B11. How far beyond the end of the wall would you take the connections?	Recalled using drag bars.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	Probably yes, had a copy of the Bulletins and treated them as best practice to follow.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	[redacted] generally not doing many shear wall buildings and also went through a downturn during the early 90s where very little new-build design was carried out. But, made comment that to his mind no new knowledge on diaphragms came out since 1980 so the thinking would have been similar in early 90s and mid-80s.
C2. How far beyond the end of the wall would you take the connections?	See C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	See C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project?	[redacted] were a small office, Directors (or Senior Associates) in charge of all projects from beginning to end. A junior was likely to be doing the work under their supervision but with regular discussion (roughly daily if they were very junior or a couple of times a week if they were more experienced). Seniors did the high-level scheming and determined what was expected. Irrespective of the experience of the person doing the work, a Director or Senior Associate would have been actively involved.
D2. What checking procedures and/or supervision did you employ?	Director would be involved regularly, using other engineers in the office to check selected aspects (independently of the design team).
D3. What reliance would you have placed on the Council Consent review process?	Did not rely on it, aware they had to get through it but knew Council didn't do a good job of checking so couldn't rely on it.

[redacted]

D4. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	Regarding ensuring engineers had requisite skills, Directors made sure staff stayed up to date, regularly referring papers to them to read. When asked if the buck stops at the Director - Absolutely. The bigger the office the less hands-on a Director might have been in a particular project but still took an active involvement in ensuring its quality. May not be responsible for every bit of technical detail but certainly responsible for those that they employ. Mid-1980s [redacted] drawings had boxes for 'Designed', 'Drawn', 'Traced', 'Checked' and 'Director in Charge'.

[redacted]

E.2 [redacted]

Date/Location of Interview: 14 April 2014, [redacted].

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	a) Approx 40 b) Approx 20 structural engineers, 15 technicians and 5 others
A2. Was the company multi-disciplinary at the time or just structural?	Mostly structural with a small amount of project management.
A3. What locations did the company typically do work in?	Had offices in Auckland and Wellington and did work mostly in these areas. Other offices established overseas.
A4. What type of structural work was the company typically carrying out during this time?	Multi-storey commercial new-builds, plus industrial buildings. Strengthening of URM buildings.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Limited Liability.
A6. What was your position/role?	Started as a graduate in 1980, progressed to an Associate in the mid-1980s.
A7. What was your experience at the time (years, type of work, etc)?	Zero in 1980, up to seven in 1987 when left to go to a [redacted] overseas.
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	None.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system? Generally, was there pressure from the architects and/or developers?	Fairly strong push for regularity which had multiple benefits including economy of design and easier to build. Focus on designing simple, buildable structures. More complex structures such as hotels involved greater pressure from architects for less regularity.
B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Yes. Took a Group 2 secondary elements approach for gravity frames. Was mostly designing frame structures of approximately 8 to 14 storeys with reasonably small footprints that had two to six internal gravity columns.

[redacted]

B3. How was the primary seismic system typically analysed? Please include response on: Which computer programs were used? Were secondary elements included in the analysis of the primary seismic force-resisting system? Was an equivalent static method used or modal response spectrum, or both? Was accidental eccentricity considered? If Yes, in what way? What allowance was made for cracking in the primary system?	ETABS was used as the primary modelling program. In the early stages of using ETABS (i.e. before approx 1985), typically separated out into lateral system and secondary gravity sub-assembly for separate analysis. Subsequently included secondary frames in the 3D ETABS model of the building with the main objective being to help shed some load from the primary system (whether it be 5 or 10% anything helped). Recalls doing dynamic analyses as a check on the periods and the modal hierarchy as well as modal storey shears, but generally designed the strength of the lateral system for equivalent static loads applied offset 0.1b from the calculated COM. Podiums were the main source of irregularity for the type of buildings he was designing. Recalls allowing for cracking in the beams in accordance with common allowances, but unsure of the values used.
B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	Where earthquake actions were obtained for secondary frames, sought to ensure that the members remained elastic in seismic load combinations, such that the seismic provisions need not be applied. Seismic induced actions not always obtained where secondary systems not included in ETABS model.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	No comment given.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	Was including them when the subframes were included as part of overall structure model, recalls using the industry standard recommendation for the percentage of joint length (25 or 50% mentioned) but unsure of exact values used.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	Generally designing to remain elastic.
B8. What was your inclination to choose non-seismic or limited ductile detailing?	Tried to use the non-seismic provisions where gravity frames could be kept elastic.
B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	Recalls using 0.7 in cases where the columns were not heavily loaded (i.e. sizes reasonable to accommodate axial loads) so could make flexural strength and axial capacity work at this level of strength reduction, and then make use of the less stringent requirements on transverse reinforcement detailing. In cases of smaller columns and higher axial load ratios, designed lower levels of gravity columns for phi of 0.9 and detailed accordingly, and upper floors for phi of 0.7.

B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	Mainly dealing with regular frame buildings, not shear wall structures, nor complex transfer structures, so typically didn't focus on this.
B11. How far beyond the end of the wall would you take the connections?	Recognised that ETABS used a rigid diaphragm which just gives an envelope with respect to Type 2 transfer diaphragms, i.e. assumed transfer occurs all at one level but may spread over multiple levels since the diaphragm is not rigid but has some flexibility.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE? What other reference documents were you or your designers using or aware of?	Yes, told to read them as soon as he started at [redacted] as a graduate. Generally read all the NZSEE Bulletin articles that were applicable, especially the late 1970s papers on frame structures.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	No comment on 1990s design approaches.
C2. How far beyond the end of the wall would you take the connections?	See C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	See C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project (i.e. is consideration made of their experience, etc)?	Never one person doing the whole project. Senior did the scheme then passed on to a more junior team member to execute the design, maintaining active overview. On large projects may have had someone doing the analysis and passing on to a team to do the design component.
D2. What checking procedures and/or supervision did you employ?	Design engineers checked the drawings meticulously before going out. Not much specific independent checking of designs (other than for some critical elements, e.g. walls); relied on fact that senior member had done the scheme and was overviewing design. Not as sophisticated QA processes as today.
D3. How would your checking and/or supervision procedures change depending on the experience of the person carrying out the work?	Depended more on the size of job, e.g. if larger job with different components would break it down and give simpler tasks to the less experienced team members. Small jobs tended to have a more senior design engineer involved and doing the majority of the design so the job is still economical.

D4. What reliance would you have placed on the Council Consent review process?	Low - aware that they typically weren't doing comprehensive checking, which had a positive effect as it made them design more diligently knowing the council checking was not to be relied upon.
D5. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	Yes, consequence of failure sat with them.
D6. Did you review the drawings before they were issued and who was responsible for signing them?	Drawings would be checked by the design engineer prior to issue 'For Construction', signed as 'Checked' then an Associate or Director would sign as 'Approved'. Buildings submitted to Council for building permit (consent) were typically not checked and signed.

E.3 [redacted]

Date/Location of Interview: 17 April 2014, [redacted].

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): Total number of employees Breakdown by number of engineers, draughties and other	a) 120 by 1987 across 5 offices b) Half engineers, half technicians Aimed for 1 Director for every 15 technical employees. Every job had an allocated Director.
A2. Was the company multi-disciplinary at the time or just structural?	Mostly structural.
A3. What locations did the company typically do work in?	Had 5 offices - Auckland, New Plymouth, Wellington, Christchurch and Sydney and did work in all these areas as well as most areas of New Zealand.
A4. What type of structural work was the company typically carrying out during this time?	Mostly architect-led jobs, emphasis on CBD multi-storey commercial. Not much industrial, government or institutional. Architect/Developer focused.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Limited Liability.
A6. What was your position/role?	Managing Director.
A7. What was your experience at the time (years, type of work, etc)?	Graduated from University of Canterbury in 1963, completed a Masters at UC Berkeley in 1970, had approx 20 years' experience in the mid-1980s.
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	Knew Mr Harding but not well. Had cordial relations with [redacted]. Dr Reay and [redacted] have been competitors for more than 30 years.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system? Generally, was there pressure from the architects and/or developers?	In Christchurch the sites dictated that you had eccentric buildings because you always had a boundary wall at the back and on the side if you had close neighbouring buildings and then open faces on the other sides to let light in. The core would be placed at the back, close to the boundary walls, and then had good office space around the open sides. [redacted] recalled examples of where the site dictated the need for big walls but they would get creative with seismically separating them to reduce eccentricity in some cases, but generally not possible in Christchurch.

B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	No (not separating out), got results from ETABS and allocated according to that. Recognised that the secondary structure had to go with the flow, or along for the ride, and gave them detailing to be adequate. A rule-of-thumb was to double-up the spacing of transverse reinforcement in the top and bottom quarter.
B3. How was the primary seismic system typically analysed? Please include response on: Which computer programs were used? Were secondary elements included in the analysis of the primary seismic force-resisting system? Was an equivalent static method used or modal response spectrum, or both? Was accidental eccentricity considered? If Yes, in what way? What allowance was made for cracking in the primary system?	They were using SAP and ETABS and were modelling everything in the same model. They were carrying out equivalent static and modal response spectrum methods, no time-history until much later. Considered accidental eccentricity by following the code, +/- 0.1b methods. Recalls using cracked sections in some analyses but cannot recall specific values.
B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	Were not following the codes, were following their instincts and putting in ductile detailing because the secondary frames had to go with the flow.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	Already built into analysis model, cannot recall specific values of cracking.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	No comment given.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	Generally did the same the whole way through, tried not to change too many times.
B8. What was your inclination to choose non-seismic or limited ductile detailing?	Had frames that were principally gravity but knew they were going along for the ride so gave them some ductility.
B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	No comment given.

B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	Modelled the diaphragms to get the loads, and to provide understanding of the load paths. De-coupled the shearwalls in some cases if required (more common in Auckland).
B11. How far beyond the end of the wall would you take the connections?	Were putting in collectors, even in the early-1970s, load paths were always thought about.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE? What other reference documents were you or your designers using or aware of?	Knew about and aware of them, but relied more on going to seminars, visiting UC Berkeley every now and then and very in touch with the University of Canterbury and also industry, not as much on reading papers.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	No significant change from mid-1980s.
C2. How far beyond the end of the wall would you take the connections?	See C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	See C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project (i.e. is consideration made of their experience, etc)?	Always had a Director in charge of each project. The Director would make all the key calls. On a decent sized project they would have a Senior Engineer below the Director leading a design team that consisted of 1 or 2 designers and the drafties.
D2. What checking procedures and/or supervision did you employ?	Supervision from the Director as above. [redacted] started out by checking all calculations but this became less as time went on, checked drawings for every job and wrote the specification.
D3. How would your checking and/or supervision procedures change depending on the experience of the person carrying out the work?	See D1.
D4. What reliance would you have placed on the Council Consent review process?	None. [redacted] had often been in the situation where the Council checkers had no idea what they were doing, but mentioned that you cannot expect the Council to be up with the latest innovations anyway.

D5. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	Westminster System, if you take the money you take the can (responsibility)
D6. Did you review the drawings before they were issued and who was responsible for signing them?	Always signed by the Director in charge. [redacted] drawings from the mid-1980s have a space for 'Drawn', 'Traced' and 'Approved' and then a revision history table.

E.4 [redacted]

Date/Location of Interview: 17 April 2014, [redacted].

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	a) Approx 35 b) 3 admin, 15 or 16 each of engineers and technicians
A2. Was the company multi-disciplinary at the time or just structural?	Multi-disciplinary, approx half structural.
A3. What locations did the company typically do work in?	Mostly Christchurch for structural work, did the odd structural project outside the region (e.g. PSIS in Wellington and Palmerston North projects).
A4. What type of structural work was the company typically carrying out during this time?	Very broad; housing, industrial, education, commercial of all types including multi-storey commercial.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Limited Liability.
A6. What was your position/role?	Managing Director and headed Structural department.
A7. What was your experience at the time (years, type of work, etc)?	Finished university in 1966, worked in local firms then completed a Masters at Imperial College for a couple of years before returning. Worked as a designer the whole time.
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	Knew Mr Harding's name, never met him. Knows Dr Reay through foundation/scholarship activities and was asked by Dr Reay's solicitor to provide a submission during the CERC Inquiry.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system? Generally, was there pressure from the architects and/or developers?	Pushed very hard for regularity. Can't imagine designing one that was highly irregular. Most of his buildings were frame-shear wall interaction, sometimes with core offset but used a balancing shear wall on the opposite side. Tried to keep the COR within +/- 0.1b from the COM.
B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Recognised the two different systems and that the primary system imposed deformations on the secondary system that produced actions.

B3. How was the primary seismic system typically analysed? Please include response on: Which computer programs were used? Were secondary elements included in the analysis of the primary seismic force-resisting system? Was an equivalent static method used or modal response spectrum, or both? Was accidental eccentricity considered? If Yes, in what way? What allowance was made for cracking in the primary system?	Initially (pre-ETABS) modelled just the primary system and imposed the amplified deformations on the secondary system by hand, but had low confidence in this method. Got ETABS at some point in the 80s, but was not at the forefront of ETABS/software use. Once they had ETABS, modelled the secondary system as well, had more confidence in the actions induced in the secondary system. Made frames quite stiff to get the benefit of them, especially the kick at the top for frame-wall interaction. Accidental eccentricity considered using usual code methods. Modal always used once ETABS available. Cracking considered, recalled values such as 50% in hinge zones and reduced I in wall hinge zones.
B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	Seldom if ever applied as they were treating them as seismic, not as purely gravity frames.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	Including secondary system in model, could not recall values of cracking used for secondary framing members.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	No comment given.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	Answered above in B3.
B8. What was your inclination to choose non-seismic or limited ductile detailing?	Generally, for large buildings normally designed as fully ductile, small buildings as limited ductile and if there was a "lot of structure" checked if there was sufficient strength to respond elastically and designed accordingly. In general, if it was a regular building with good confidence then they would look for savings. But, if irregular or had low confidence then they were more cautious with detailing.
B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	No comment given.

B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	Follow your loads. Took diaphragm shear and followed it into the walls. Bars at right angles to wall or if more complicated used diagonals.
B11. How far beyond the end of the wall would you take the connections?	Used strut-and-tie models for in-plane shear, develop strut from far side. Tried to develop a tie to pick up enough slab.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE? What other reference documents were you or your designers using or aware of?	Frequently. Depended on them. Read and used all of the appropriate papers ("Green Books"). Green Books were more useful than the codes.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	Noted that there appeared to be significantly less development in diaphragm design approaches than other primary seismic elements. Modelling/analysis pretty simplistic. Did not see a great deal of change through this period.
C2. How far beyond the end of the wall would you take the connections?	See C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	See C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project (i.e. is consideration made of their experience, etc)?	When graduates came, started them on simple structures such as stairs and factories then moved them on to more complicated structures. In high-rise, every one that [redacted] was in charge of he did the preliminary design. They had a Director in charge of every project, then generally a Senior and a Junior (depending on size of project). 2 Structural Directors ([redacted] was one). [redacted] was very involved in his projects. There was a Director in charge of every job but their involvement would vary with the scale of the job.

D2. What checking procedures and/or supervision did you employ?	They had engineer and drafting checks, and the project had to be signed off by someone other than the designer. They did implement a formal QA system at some point but probably not until the 90s. In the 80s the system was more informal, but there was always a lot of interaction between [redacted] and the designer. Bigger emphasis on drafting checks than engineering checks. The Director in charge looked at critical elements to see if it generally looked right.
D3. How would your checking and/or supervision procedures change depending on the experience of the person carrying out the work?	As above.
D4. What reliance would you have placed on the Council Consent review process?	None, because he was not impressed with the competency of the people there doing the checking.
D5. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	Buck stopped with [redacted] on his projects, irrespective of the way the project developed.
D6. Did you review the drawings before they were issued and who was responsible for signing them?	Chief draftsman signed off the drafting. Did not have a process for signing off 'For Construction'. Believes practice in South Island was to prepare more detailed drawings than elsewhere. [redacted] drawings had boxes for 'Designed', 'Drawn' and 'Checked'.

E.5 [redacted]

Date/Location of Interview: 4 April 2014, Video-link.

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	a) Approximately 200 staff across six offices - predominantly in upper North Island locations. b) 30No structural comprising 20No engineers plus 10No technical/draught support.
A2. Was the company multi-disciplinary at the time or just structural?	Multi-discipline/architectural, civil/structural/geotechnical engineering, planning (large), surveying. Structural capability in three offices. Bridging capability in Hamilton via Specialist Group.
A3. What locations did the company typically do work in?	Auckland, Hamilton, Rotorua, Tauranga, Whangarei Structural only in Auckland, Hamilton, Rotorua.
A4. What type of structural work was the company typically carrying out during this time?	Majority light commercial/institutional/and light industrial across all offices - civil infrastructure for some major projects (e.g. Taranaki Petrochem) and for Refinery-Auckland pipeline (e.g. Wiri Terminal). Structural review work for Auckland City and Manukau City Councils. Auckland/Had additional capability in special structures, proprietary system support, R & D, etc. Primarily acting as principal consultant with very small amount of building work for [redacted]
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)?	Limited liability, previously Partnership (1970) Approximately 15No Directors/Principals (3No structural)
A6. What was your position/role?	Principal Structural Engineering/Auckland office (100No total, including 15No structural staff)
A7. What was your experience at the time (years, type of work, etc)?	Graduated 1971 (Auckland) Overseas 1972 - 1975 (including post-grad) [redacted] 1976 - 1994 Inaugural Chair SESOC 1988 Structural work generally as Q4 above
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	Harding - nil Reay - met during early 1990s when involved with NZS 3101 and precast wall panel reviews via BRANZ, but only once or twice. Can't say I know him

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system?	1.1 Generally push was for a "rational structural system" that was logical and easy to analyse. 1.2 Most (Auckland/Hamilton) building structure solutions relied on limited ductile seismic resisting system. 1.3 In-house architects were "bullied" to achieve the above, but sometimes we (structural) came in too late to influence the outcome - i.e. "Can you get compliance based on this architectural concept that I have already sold my client on"
B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Response to Q2-12 in this section is based on small number of building projects of the same scale at CTV Building, mostly locations in Auckland (Zone C), Hamilton, and/or Rotorua (Zone B). 2.1 Probably treating everything as components of primary system (although might have treated columns as gravity props with secondary system if primary seismic force resisting systems were shear walls). 2.2 Primarily based on stiffness contribution (walls stiffest) and sensitivity of answers to variation in Ie (e.g. columns, beams, cracked first and shed load back to walls).
B3. How was the primary seismic system typically analysed? Please include response on: 3.1 Which computer programs were used? 3.2 Were secondary elements included in the analysis of the primary seismic force-resisting system? 3.3 Was an equivalent static method used or modal response spectrum, or both? 3.4 Was accidental eccentricity considered? If Yes, in what way? 3.5 What allowance was made for cracking in the primary system?	Reference here is to major - i.e. 6-storey - office building with basement located in an upper North Island centre which I am trying to obtain full design documentation for - hence answers provided should be treated as provisional and subject to check. 3.1 2-D frame analysis package linking shear walls to frames on each primary frame line with wall loads allocated in plan via MUTO, etc. 3.2 Gravity frame treated as secondary, but detailed for limited ductile or fully ductile under "amplified deformation" conditions (typically SM = 4 threshold/check). 3.3 Spatial arrangement of shear walls deliberately chosen to allow seismic analysis to be undertaken using Equivalent. Static Force analysis method generally following NZS 4203:1184 cl 3.4.7. 3.4 By use of < = ±0.1B as per NZS 4203:1984 cl 3.4.7.2. 3.5 Standard allowance - say 75% × Ig* uniform on all shear walls, but solely for drift calculations only. NB : Estimate only based on NZSEE 1980 Section B/Paulay & Williams/check required.

B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	4.1 There was general understanding of strength/ductile detailing trade off within NZS 3101:1982 Concrete Structures Standard cl 3.5.14.3, but perhaps some uncertainty as to what the amplified deformation, viz vD, actually corresponded to. NB Reasonable interpretation is that it is the actual NZS 4203:1984 derived deformation without the K/SM factor reduction used in NZS 4203:1984 cl 3.8.1. 4.2 In subject building, primary structure was ductile cantilevered shear walls, and secondary structure was the gravity frame. 4.3 Choice regarding type of detailing driven mainly by: a) need to keep section dimensions at minimum, e.g. gravity frame loads; and b) need to limit deformation induced moments (M*) and shear (V*) (particularly shear) both of which push detailing choice towards limited ductile/fully ductile detailing. 4.4 Refer attached notes indicating inferred "interpretation" of secondary elements design rules to NZS 3101:1982 cl 3.5.14.3 prepared by this respondent to assist recall for this assignment. NB: Do not have comparative document prepared at the time, but believe there might well have been one in. [redacted] office specifically prepared by me to create a unified approach to this type of evaluation.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	5.1 Given that the gravity frame comprised the secondary structure beams, columns which were subjected to "amplified deformation", objective was to minimise amount of sway-induced load attracting to it. 5.2 Consequently, a generous (but conservative) approach to use of cracked column, beam I g, particularly where ductile/limited ductile hinge behaviour expected on in the beams and/or columns. 5.3 Generally derate factors applied to Ig would be axial load dependent and follow 1980 NZSEE/ Section B/Paulay and Williams guidelines.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	6.1 Generally 2D frame modelling would have been used without e.g. rigid end block components. 6.2 Notional scaling adjustments would have been made for shear span between hinges, e.g. sway-induced shear assessments.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	7.1 Not known. 7.2 Believe general approach with uniform frame geometry would have been to detail, e.g. transverse steel in columns as uniformly/ consistently as possible. 7.3 Some "banding" of solutions within 7.2 possible, depending on column location within the building footprint.

B8. What was your inclination to choose non-seismic or limited ductile detailing?	8.1 Given Auckland/Hamilton/Rotorua location of most projects, limited ductile detailing to Section 14 of NZS 3101:1982 was favoured "as a package" for the primary (and secondary) system, notwithstanding that in some form of buildings the nature and extent of post-yield behaviour might have been unclear. NB: Primary reference to NZSEE 1980 guidelines/ Section C (Lou Robinson paper), and Peter Leslie's paper in the NZCS Red Book 1983. 8.2 For larger buildings, with significant gravity (and seismic load) demands, as for sample building in the upper North Island referred to earlier, the tendency was: a) away from non-seismic (elastic) detailing; and b) towards fully ductile or limited ductile detailing. 8.3 Trend might be towards non-seismic detailing in e.g. oversized/low aspect ratio reinforced concrete walls with low axial loads, but this would not extend to smaller sized elements in lateral force resisting system, e.g. columns, blade walls, etc, where detailing would be limited ductile to Section 14 at least.
B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	9.1 Acknowledged that column transverse/confining reinforcement is axial load dependent, and some concern at the time over the interpretation/ implications of NZS 3101:1982 cl 6.4.7.1(b). NB: Acknowledged that confinement rules in this clause derived from ACI-318-71 and were aimed at maintaining axial load capacity after spalling of cover concrete rather than being rules for ensuring rotational ductility in the potential hinge zones above/below beams in the column section being considered. 9.2 As we ([redacted]/Structural) then saw it, designers had significant amount of discretion in choosing amount/spacing, etc, of transverse/confining steel subject to minimum levels of flexural and shear reinforcement required by the secondary frame analysis being provided in the secondary system, particularly through potential column hinge zones.
B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	10.1 Designer would identify early on which diaphragms were Category 1 and 2, particularly those acting as transfer diaphragms. Special function of Category 2/transfer diaphragm and reactant forces required to be resisted by it, particularly in ground level slabs above basement. 10.2 Reference made to NZSEE 1980 Section D/Diaphragms - Kolston & Buchanan to assist interpretation of NZS 4203:1984 cl 3.4.9. 10.3 Typically diaphragms comprised precast slab panels supported on grid of beams which framed into shear walls. NB: These beams formed part of the diaphragm tie – i.e. drag bar – system.

[redacted]

B11. How far beyond the end of the wall would you take the connections?	11.1 Refer NZSEE 1980/Section D - Diaphragm - Kolston & Buchanan - p165/Figure 3. 11.2 Cannot be sure - i.e. need to look at specific job for which we do not have the documentation - but believe approach would have been to identify load paths and then ensure they were well anchored. 11.3 Given the limited ductile design philosophy adopted, the upper bound threshold for inertial accelerations would be based on upper bound elastic (SM = 4) response rather than seeking to cover the "pushover" mechanism associated with the overstrength case.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol.13, No. 2, June 1980 Bulletin of the NZSEE	12.1 We [redacted] found achieving consistent interpretations within the parts/portions provisions of NZS 4203:1984 cl 3.4.9.2 (including Table 9) to be difficult. 12.2 Consequently the guidance contained in NZSEE 1980/Section D - Diaphragms (Kolston & Buchanan) was used as a primary source to clarify the various uncertainties within Table 9. 12.3 In summary, to my recall, most designers (whilst understanding the structural engineering concepts involved) found the number crunching around diaphragms as per Table 9 difficult.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	1.1 By early 1990s (say 1994) limit state loading Standard NZS 4203:1992 and material Standard NZS 3404:1992 (Steel) had been issued, and from July 1992 the regulatory regime encompassed requirements of Building Act 1991 (BA91). 1.2 In terms of technical knowledge underpinning structural engineering practise, there was perhaps an "improved understanding" - i.e. more technical conferences had been attended by practitioners to enable more consistent interpretations to be developed. 1.3 Otherwise response is as for Q10 above.
C2. How far beyond the end of the wall would you take the connections?	2.1 Refer background comment in response to Q1 - i.e. items 1.1 and 1.2 above. 2.2 Otherwise response is as for Q11 above.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	3.1 Refer background comment in response to Q1. 3.2 Otherwise response is as for Q12 above.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project?	1.1 For [redacted] organisation framework refer response to earlier questions, e.g. AQ1 through Q4. 1.2 Structural Principal agreed fees budget, received briefing, and developed primary structural concepts for use on the project. 1.3 Senior (or intermediate) engineer allocated to project to lead the detailed analysis/design work, referring back to Principal as required for technical guidance/instruction as work progressed. 1.4 Principal would "remain connected" to the project for reasons discussed in item 4.
D2. What checking procedures and/or supervision did you employ?	2.1 Structural Principal would undertake high level check on implementation of solutions for primary structure, plus general review of calculations, etc, for sufficiency, given that these would be ultimately going to regulator for structural check/sign off as part of building permit approval. 2.2 For major building, e.g. sample 6-storey upper North Island centre project, we would have another senior engineer review job in more detail than above, but it would not have been equivalent of current "design review" (PS2) check operating currently.
D3. What reliance would you have placed on the Council Consent review process?	3.1 In Auckland, a 6-storey building submitted for building permit would have been reviewed internally by Council's structural engineering team or would have been put out to another structural consulting firm for an independent check. 3.2 For the upper North Island centre project, the review was undertaken by local Council staff who had a reputation for rigor and who we found very thorough in terms of e.g. working through overstrength checks in ductile shear walls. 3.3 Reality at the time was that Council checking was seen as part of the "getting the project through", and the interaction between [redacted]/Council staff was constructive and deserving of respect (between peers) rather than being combative.
D4. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	4.1 As an ACENZ member firm (with significant liability exposure and therefore commensurate PI cover), there was expectation that the Structural Principal would provide the necessary technical overview and staff management on behalf of the firm to ensure that a technically-competent end product was achieved. 4.2 This did not mean that mistakes/errors did not occur, but it did mean that ultimately the firm (and the Structural Engineering Principal involved) was responsible to Directorate for the sufficiency of the end product and, if mistakes were identified along the way, then to correct them.

	4.3 This "self-preservation" requirement forced the Principal Structural Engineer to properly staff the project and, where shortfalls became apparent on review, then to ensure additional or alternative resources were provided to assist that end objective. 4.4 Overall, in [redacted]/other firms of this type, the "buck did stop" with the top man - i.e. the Principal/Structural Engineering - and through him to the firm's Directorate as a whole.

Part E – Limitation/Exclusions

Question	Response
E1. Are there any limitations or restrictions on your responses to questions A, B, C, or D?	1.1 Responses provided to Beca are generic and based on personal recall of representative projects undertaken by [redacted] rather than specific on the record "Case Study" documentation as such. 1.2 Responses provided to Beca are given in good faith in order to indicate the "practise of the day" standards which applied to the period under investigation. 1.3 Information provided is provisional and may not be relied upon by third parties or be used for any other purpose. We reserve the right to modify or amend the responses given at any time in the future should a need arise, and at our sole discretion

E.6 [redacted]

Date/Location of Interview: 8 May 2014, [redacted].

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	a) 20 b) 6 engineers, 4 technicians and the rest surveyors and admin, etc.
A2. Was the company multi-disciplinary at the time or just structural?	Structural, civil and surveying.
A3. What locations did the company typically do work in?	Wellington and central New Zealand.
A4. What type of structural work was the company typically carrying out during this time?	Residential, reservoirs, grandstands, industrial, halls. Specifically not multi-storey commercial until late 1990s. Did not typically work for developers.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Limited Liability.
A6. What was your position/role?	Director of company with a minimum of 3 Directors.
A7. What was your experience at the time (years, type of work, etc)?	Graduated in 1968, approximately 15 years by mid-1980s.
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	None with Mr Harding. Was involved in [redacted] as a Claims Convenor in the early-1990s when the retrofit works to install drag bars in the CTV building were carried out. Role was to approve expenditure up to insurance excess for Alan Reay Consultants Limited's claim.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system? Generally, was there pressure from the architects and/or developers?	Structure-dependent, low-rise or smaller projects wouldn't push very hard but considered it was important and did push for more significant structures. There was pressure from architects.
B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Weren't doing a lot of work on relevant projects during this time but recognised that you might design the seismic system for the loads but that the secondary structure had to trail behind or go along for the ride.

<table>
<tr><td>B3. How was the primary seismic system typically analysed? Please include response on:
Which computer programs were used?
Were secondary elements included in the analysis of the primary seismic force-resisting system?
Was an equivalent static method used or modal response spectrum, or both?
Was accidental eccentricity considered? If Yes, in what way?
What allowance was made for cracking in the primary system?</td><td rowspan="5">Minimal involvement with structures over 2 storeys in height. No comments given.</td></tr>
<tr><td>B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?</td></tr>
<tr><td>B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?</td></tr>
<tr><td>B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?</td></tr>
<tr><td>B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?</td></tr>
<tr><td>B8. What was your inclination to choose non-seismic or limited ductile detailing?</td><td>Would avoid using non-ductile detailing and generally provided more confinement than required by the code.</td></tr>
<tr><td>B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?</td><td>Did not use an increased ϕ to 0.9 but was aware of the issue.</td></tr>
<tr><td>B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?</td><td>Generally, took a pragmatic approach to diaphragms starting with consideration of the plan shape and location of openings, not using the minimum topping thickness as that were commonly used at the time. Stayed away from Hollowcore and used Stahlton or metal deck insitu floors.
Analysis generally to code and industry accepted technical papers.</td></tr>
</table>

B11. How far beyond the end of the wall would you take the connections?	Mild steel reinforcement provided to 600mm into the diaphragm.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE? What other reference documents were you or your designers using or aware of?	Yes, made use of the Bulletin papers generally. Said that designers had to in those days because the codes were thinner. Also mentioned Park and Paulay.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	Generally felt there was a deterioration rather than an improvement through the period 1985 to 1992. Felt that the industry slipped during the boom of the mid to late 1980s where details were driven by ease of construction rather than good engineering detailing.
C2. How far beyond the end of the wall would you take the connections?	See C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	See C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project (i.e. is consideration made of their experience, etc)?	Capability-based. Feels that the concept of responsibility was much stronger in the 80s and a Director couldn't be seen to take on a job that they weren't experienced in or allocate an unsuitably experienced staff member to a particular job. A Director (had 3 structural Directors in mid-1980s) was responsible for each project to deal with the client and sign Producer Statements.
D2. What checking procedures and/or supervision did you employ?	Depending on the type of project with the more complex ones having regular discussions during the design but the simpler ones less so. Due to the nature of hand-produced and traced drawings in those days it was much easier to keep an eye on the drawings as they were being produced. Tended to review the drawings first to gain an appreciation of how the structure works and, if required, would look at the calculations to drill down on specific details. Mentioned carrying out day-to-day review of outputs.
D3. How would your checking and/or supervision procedures change depending on the experience of the person carrying out the work?	Would also depend on the complexity of the job. Amount of checking would depend on the experience of the team.

[redacted]

D4. What reliance would you have placed on the Council Consent review process?	None. "If you are serious about achieving quality work you don't rely on the Council".
D5. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	The buck stops with the Director. Experience was that this was the practice in the industry at the time. Responsibility for projects sat with a Director.
D6. Did you review the drawings before they were issued and who was responsible for signing them?	Directors didn't sign the drawings but were always required to sign any Design Certificates that were issued.

[redacted]

E.7 [redacted]

Date/Location of Interview: 17 April 2014, [redacted].

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	a) 7 b) 3 engineers, 3 technicians and 1 admin
A2. Was the company multi-disciplinary at the time or just structural?	Mainly structural (around 80%), rest Civil. Civil work included subdivisions, water supply, did own soil testing.
A3. What locations did the company typically do work in?	South Island, mostly Canterbury, some in Palmerston North and Wellington.
A4. What type of structural work was the company typically carrying out during this time?	Commercial and Industrial. Have designed fewer than 5 multi-storey commercial buildings in total (i.e. greater than 2 storeys). Mostly low-rise, 2-storey, sometimes 3 and very rarely a 5-storey plus.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Partnership, 2 partners.
A6. What was your position/role?	Managing Director.
A7. What was your experience at the time (years, type of work, etc)?	Initially completed an NZCE before spending 2 years at the University of Canterbury to complete bachelor's degree. Started work after university in 1971. Did mainly industrial work in the early years, designed a multi-storey building at [redacted] in 1970s and a multi-storey carpark in the 80s.
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	Worked with Mr Harding for 2 years while at [redacted] in the 1970s, saw him at social events. Saw Dr Reay at Structural Group meetings, did not know well. Technically he was a competitor but didn't see it like that.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system? Generally, was there pressure from the architects and/or developers?	Found it was hard to push for it as developers and architects had already come up with concepts and once they were well down the track they would bring in the structural engineer, then want engineer's price once the concept was fixed in their minds. Preferred designing regular, sensible buildings though, and even walked away from the odd job because of the nature of the concept.

B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Did not generally deal with the type of structure that warranted it.
B3. How was the primary seismic system typically analysed? Please include response on: Which computer programs were used? Were secondary elements included in the analysis of the primary seismic force-resisting system? Was an equivalent static method used or modal response spectrum, or both? Was accidental eccentricity considered? If Yes, in what way? What allowance was made for cracking in the primary system?	Modelled the primary system on a 2D Microstran program plus hand calculations, more hand calculations than what we would do today. Lumped everything into the same analysis. Generally not doing modal analyses. Accidental eccentricity was considered, recalls +/- 0.1b methods. Cracking was just a concept in the 1970s and adopted it when it became part of the Standard.
B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	Usually limited ductile for the majority of the structures (low-rise) they were dealing with. Recalls analysing for elastic loads and checking what the displacements and actions would be for ductilities of 2 or 3.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	No comment given.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	Mentioned they were likely to be using them when Microstran had the capability.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	More likely to make decisions for the whole building. Did not look at member ductility demands, relied on what the code said for level of global structural ductility in terms of what was required for members.
B8. What was your inclination to choose non-seismic or limited ductile detailing?	Non-ductile if the building could be considered as "elastic".

[redacted]

B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	No comment given.
B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	Treated as a beam, spanning between walls. Recalled strut and tie methods. Recalls using parts and portions. They were usually dealing with regular buildings, low-rise, didn't go near irregular layouts and didn't know where to start with respect to analysis, especially did not deal with transfer diaphragms.
B11. How far beyond the end of the wall would you take the connections?	Recalls designing connections for shear friction. Have seen examples of reinforced concrete block wall buildings by others without much attachment to floors.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE? What other reference documents were you or your designers using or aware of?	Used the Bulletins. Read the diaphragm paper but recognised it wasn't considering buildings that are irregular in plan. When a new code came out they would buy it and would take it home to read fully over the course of a few evenings to come up to speed. Mentioned that it would take their firm a couple of years to get completely used to a new code.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	Summarised that he didn't think it was significantly different.
C2. How far beyond the end of the wall would you take the connections?	See C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	See C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project (i.e. is consideration made of their experience, etc)?	Allocated on the basis of experience, didn't give a job to someone unless they knew how to do it or he was prepared to show them how. If it was a big or important job, would try to give it to someone who had the appropriate experience.
D2. What checking procedures and/or supervision did you employ?	Jobs allocated to one of the Partners. One Partner would check the other Partner's jobs by taking the drawings home to check in the evenings. By mid-80s they had brought in an Engineering Check and Drafting Check. Recognised that we all make mistakes.

D3. How would your checking and/or supervision procedures change depending on the experience of the person carrying out the work?	The Partner checking the other Partner's work would be looking for big errors, not the small things or details.
D4. What reliance would you have placed on the Council Consent review process?	None. The Council didn't tend to pick up gross errors so he had little confidence in their checking process. Feels they tended to look for small things that didn't matter as much and often missed big picture things.
D5. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	Tried to keep ahead of staff in terms of technical ability, e.g. [redacted] went back to university last year to do a Displacement-based Design paper to get up to speed with it since his graduates had been taught it at uni. He wanted to lead them technically. Buck stopped with Partners, he looked at why he was checking things and said he felt the weight of responsibility that no one gets hurt due to design carried out in his name.
D6. Did you review the drawings before they were issued and who was responsible for signing them?	Yes, took the drawings home and reviewed them looking for big-issue errors.

[redacted]

E.8 [redacted]

Date/Location of Interview: 7 May 2014, By Phone.

Part A – Preliminary Questions

Question	Response
A1. Size of company (approximately is sufficient): a) Total number of employees b) Breakdown by number of engineers, draughties and other	a) 9 b) 4 engineers, 4 technicians and 1 admin.
A2. Was the company multi-disciplinary at the time or just structural?	Mostly structural (small amounts of civil and fluid dynamics).
A3. What locations did the company typically do work in?	Otago, Southland and a bit in Auckland. Specifically not Wellington or Christchurch for commercial reasons.
A4. What type of structural work was the company typically carrying out during this time?	Mostly buildings. Other work included aerials, walls and freight containers.
A5. What type of company was it (e.g. Limited Liability, Partnership, Sole Practitioner, etc)	Limited Liability.
A6. What was your position/role?	Director. [redacted] had three Directors in the mid-1980s.
A7. What was your experience at the time (years, type of work, etc)?	Established [redacted] in 1968, so roughly 15 to 20 years. Committee member for the development of NZS 3101:1982 and the Bulletin Study Groups.
A8. What, if any, is your relationship to either Dr Alan Reay or Mr David Harding?	None with Mr Harding. Knows Dr Reay and had some involvement surrounding CTV including being asked to help defend Dr Reay's position (which never eventuated) and producing a paper summarising code requirements and deficiencies for the CERC.

Part B – Technical Questions, Mid-1980s

Question	Response
B1. How hard did you push at the concept stage for regularity in the seismic structural system? Generally, was there pressure from the architects and/or developers?	Didn't push very hard for low-rise buildings as liked the challenge of designing irregular buildings. If taller, say 10 stories, would insist on regularity more.
B2. Were you allocating elements (walls, beams and columns) to either the primary or secondary system with respect to the seismic force-resisting system (refer Clause 3.5.14.1 and Commentary of NZS 3101:1982)? - Yes or No. If Yes, how was the allocation decision made?	Yes. [redacted] mentioned that this was in the code, in part due to his instigation. Particularly the clauses dealing with imposing deformations on Group 2 Secondary Structural Elements due to concerns over other designers ignoring them, i.e. not contributing significantly to the lateral strength of the building but then not checking them for displacements under lateral loading.

[redacted]

B3. How was the primary seismic system typically analysed? Please include response on: Which computer programs were used? Were secondary elements included in the analysis of the primary seismic force-resisting system? Was an equivalent static method used or modal response spectrum, or both? Was accidental eccentricity considered? If Yes, in what way? What allowance was made for cracking in the primary system?	No decent commercially available computer programs in the early 1980s. [redacted] were using their own in-house programs in the 1980s, and particularly not ETABS as recognised problems (i.e. rigid diaphragms). Sometimes included the secondary elements but not always mostly due to RAM/memory constraints. Both, but only used spectral modal analysis as a guide and felt that the 4203 methods were flawed. Stuck to equivalent static method as it was easier for Capacity Design. Yes, but not by code methods. [redacted] would prefer to use 'base input' and let rotational resistance properties of the structural system determine the effects. [redacted] felt this was more reliable than the code methods. As per code, recalls Ig for columns and 0.5Ig for beams or primary system.
B4. How was the choice of which level of detailing to provide for secondary elements made, i.e. either non-seismic, limited ductile or ductile seismic (refer Clause 3.5.14.3 and Commentary of NZS 3101:1982)?	Would use non-seismic provisions if the structure could be shown to remain elastic at (4/SM)Δ instead of the code-implied (K/SM)Δ. [redacted] would need to be sufficiently convinced that the secondary elements were protected by the primary system before using less detailing. If the secondary elements didn't remain elastic, then they would step through the limited and fully-ductile provisions as appropriate.
B5. If imposing displacements on either a subassembly or 2D frame model of the secondary system, what allowance was made for cracking for beams and columns?	On the same basis as the primary structure because if were going through the same level of deformation then should use same level of cracking.
B6. If imposing displacements on either a subassembly or 2D frame model of the secondary system, were rigid end blocks incorporated and for what percentage of length of joint?	Used Muto analysis by hand which determined the length of end-block based on relative clear and centreline span lengths of columns and beams. This would often lead to no end blocks.
B7. If imposing displacements on either subassembly or 2D frame model of the secondary system and only isolated elements were found to require the more stringent requirements, were only these detailed for increased levels or was a blanket decision made for entire frame or all similar?	Took a pragmatic approach and detailed similar elements in a similar way irrespective of the analysis results.
B8. What was your inclination to choose non-seismic or limited ductile detailing?	Would do so if elements remained elastic at (4/SM)Δ.
B9. How was the choice of strength reduction factor ϕ for columns made (refer Clauses 4.3.1.2 and 6.4.7.1 of NZS 3101:1982)?	If fully-confined, used 0.9. If elements were shown to remain elastic then used 0.7 to 0.9.
B10. How would you analyse the actions in the connections to the diaphragms for design (refer Clause 3.4.9 of NZS 4203:1984)?	Based on capacity actions of the primary seismic system, followed up with a check using Parts and Portions and used the larger of the two, contrary to what the code suggested (which was to take the smaller). In determining the capacity actions, would typically take into account the overstrength but not the dynamic magnification.

[redacted]

B11. How far beyond the end of the wall would you take the connections?	Would have dragged through far enough so enough mesh in the slab was included to take over. Would not rely on Hi-bond for strength contribution.
B12. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE? What other reference documents were you or your designers using or aware of?	Yes, helped write it and was one of the lecturers touring the country to introduce it.

Part C – Technical Questions, Early 1990s

Question	Response
C1. How would you analyse the actions in the connections to the diaphragms for design?	No significant changes until introduction of NZS 4203:1992.
C2. How far beyond the end of the wall would you take the connections?	As for C1.
C3. Were you aware of and using the thinking outlined in the paper "Diaphragms in Seismic Resistant Buildings" in Vol. 13, No. 2, June 1980 Bulletin of the NZSEE	As for C1.

Part D – Practice Questions

Question	Response
D1. How would you allocate staff to a particular project (i.e. is consideration made of their experience, etc)?	Allocated based on availability and interest. Every job was appointed to a Team Captain which was generally a senior staff member, either a draftie or engineer. The Team Captain was responsible for dealing with client, writing the specification, etc. Directors only would be able to sign the Design Certificates.
D2. What checking procedures and/or supervision did you employ?	Team Captain would check the drawings in detail. [redacted] would do separate calculations to check aspects. The Team Captain would fill out a Design Features Report and that would be checked for general consistencies with the drawings.
D3. How would your checking and/or supervision procedures change depending on the experience of the person carrying out the work?	More involved for more complex jobs generally.
D4. What reliance would you have placed on the Council Consent review process?	No comment given.

[redacted]

D5. How did you see your responsibilities as partner/director/owner/senior regarding the technical output of your firm? In other words, does the buck stop with the Director?	"If you don't have a reputation for quality, you don't have a firm at all" - so if you don't produce quality work you quickly get into trouble. As a Director, [redacted] would never blame staff for any product that has come out of the office, so the buck stops with the Directors when things go wrong.
D6. Did you review the drawings before they were issued and who was responsible for signing them?	Yes, [redacted] reviewed them. [redacted] had boxes for 'Designed', 'Drawn', 'Traced' and 'Checked' but not "For Approval" or "For Construction".

[redacted]

Appendix F

Beca Compliance Assessment

Sections of this document have been redacted to protect the privacy of individuals.

Appendix F Beca Compliance Assessment

F.1 General

The following appendix presents a summary of our assessment of the compliance of the 1986 design of the CTV building with the relevant codes of the day. The scope of this review is generally limited to the seismic design analysis process and the resulting compliance of the diaphragms and primary gravity structure under seismic loading. The detailing of specific elements such as the shear walls and compliance of the structure for gravity-only load cases has not been included.

The applicable standards of the time were the loadings standard NZS 4203:1984, and the concrete structures standard NZS 3101:1982. Both were in general use in New Zealand at the time.

The '1986 design' of the building refers to two components; the available structural calculations by Mr Harding showing some aspects of the design process, and the permitted structural drawings showing the output of this process. Compliance or not is ultimately defined by the product shown on the drawings. The calculations help to highlight any errors or omissions in the design process that may have led to any identified non-compliances.

F.2 Primary Seismic System

In order to assess both the seismic design analysis process and the resulting compliance of the diaphragms and primary gravity structure, we have carried out our own analysis of the primary seismic structure.

F.2.1 Modelling Approach

Our analysis has followed the requirements of the relevant standards of the time and used the configuration of the building as shown on the structural drawings. We used assumptions and analysis techniques as close as close as possible to what was available and common at the time, and what is understood to have been used by Mr Harding.

Our modelling of the primary seismic system was therefore as follows:

- Built a three-dimensional ETABS (v9.7.3) model of the primary seismic structure only, i.e. the North Wall Complex and the South Shear Wall, and not the beams and columns of the primary gravity structure (Figure F1)
- Modelled the structure above ground only and assumed the shear walls are fixed at ground level, i.e. neglecting foundation rotations and soil-structure interaction
- Modelled the North Wall Complex as a series of interconnected 'area' elements
- Modelled the South Shear Wall as a series of interconnected 'frame' elements
- Calculated the seismic weight, centre of mass and rotational inertia of each floor level and lumped the properties at a node representing the centre of mass.
- Offset the lumped mass or point of application of horizontal force by a distance of 0.1b as appropriate to include the effects of accidental eccentricity
- Applied rigid diaphragms at each level.

These assumptions were carried through for calculation of the period and base shear and also for calculation of the deformations of the primary seismic structure to be imposed on the primary gravity structure.

We have used the floor level naming convention from the original design, i.e. Level 1 as ground. Refer to Figure F1 for the location of our origin, i.e. southwest corner with positive 'x' towards the east and positive 'y' towards the north.

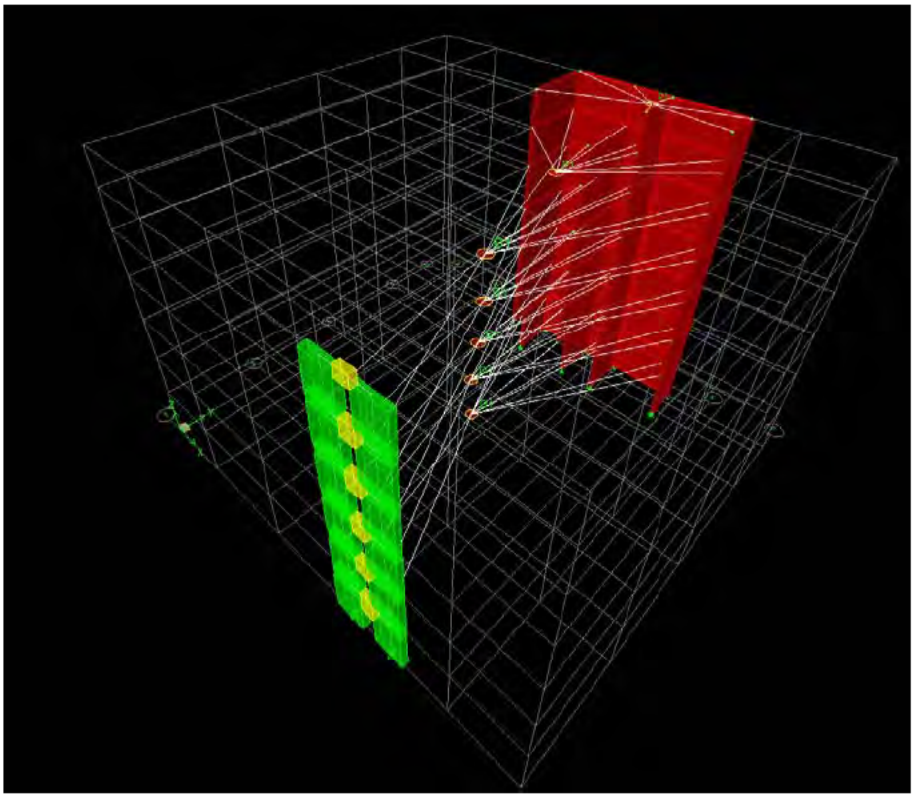


Figure F1: Beca ETABS model of the primary seismic structure

F.2.2 Modelling Inputs

F.2.2.1 Stiffness Properties

We used the following to represent the stiffness properties of all reinforced concrete elements of the primary seismic structure:

Table F1: Primary Seismic Structure Stiffness Properties

Property	Value
Concrete strength (MPa)	25
Elastic modulus (MPa)	23500
NWC wall elements effective stiffness (I_e/I_g)	0.6
SSW coupling beams effective stiffness (I_e/I_g)	0.4
SSW piers effective stiffness (I_e/I_g)	0.6
SSW frame joints rigid zone factor	1.0
SSW frame joints end length offsets	Full joint length

F.2.2.2 Floor Level Seismic Properties

We used the following seismic properties of each floor level:

Table F2: Floor Level Seismic Properties

Level	Height (m)	Weight (kN)	COMx (m)	COMy (m)	Rot. Inertia, I (kN.m2)
8	21.66	419	17.9	25.2	8,000
7	19.26	966	16.5	18.3	127,000
6	16.66	5,837	16.5	12.5	880,000
5	13.42	5,926	16.5	12.6	897,000
4	10.18	6,032	16.3	12.6	920,000
3	6.94	6,137	15.9	12.6	959,000
2	3.70	6,221	15.9	12.8	1,002,000
Total		**31,538**			

In comparison, the value reported in the 1986 calculations for total seismic weight was 33,014kN. There is less than 5% difference between the values.

F.2.3 Natural Periods of Vibration

The period of vibration of the first translational mode in each direction was obtained from ETABS using two-dimensional analyses. The values obtained and a comparison with those reported in the 1986 calculations is as follows:

Table F3: Natural Periods of Vibration

Direction	Beca Value	ARCE Value
North-South Period (s)	0.8	1.06
East-West Period (s)	0.3	1.06

F.2.4 Seismic Coefficient and Base Shears

With reference to Section 3.4.2 of NZS 4203:1984, the design seismic coefficient and base shear for the building in each direction using the periods obtained above was calculated as follows:

$$C_d = CRSM$$
$$V = C_d W_t$$

C **– Basic Seismic Coefficient**
Zone B
Flexible Soils
C = 0.115 for North-South direction (T = 0.8s)
C = 0.125 for East-West direction (T = 0.3s)

R **– Risk Factor**
R = 1.0 for both directions (Category 4 – normal occupancy)

S **– Structural Type Factor**
S = 1.0 for both directions (For overall building analysis take as 1.0. Sub-assembly design may use different)

M – Structural Material Factor
M = 0.8 for both directions (Reinforced non-prestressed concrete)

C_d – Design Seismic Coefficient
C_d = 0.09 for North-South direction
C_d = 0.10 for East-West direction

V – Base Shear
V = 0.09*31538 = 2840kN for North-South direction
V = 0.1*31538 = 3150kN for East-West direction

F.2.4.1 Comparison of Base Shear Values

The following table provides a comparison between our base shear values and those reported in the original calculations used to calculate the deflections of the primary seismic structure:

Table F4: Base Shear Comparison

Property	Beca Value	ARCE Value	Ratio (ARCE/Beca)
North-South Base Shear (kN)	2840	2350	.83
East-West Base Shear (kN)	3150	2350	.75

F.2.4.2 Distribution of Base Shear

With reference to Section 3.4.6 of NZS 4203:1984, 90% of the calculated base shear was distributed up the height of the building in an inverted triangle fashion with 10% concentrated at the top storey (Figure F2).

We have considered Level 6 to be the uppermost significant seismic weight, so have applied the 10% to this level. In comparison, the approach taken in the 1986 calculations was to apply the additional 10% at an elevated level representing the roof. This is expected to have only a minor effect on the results.

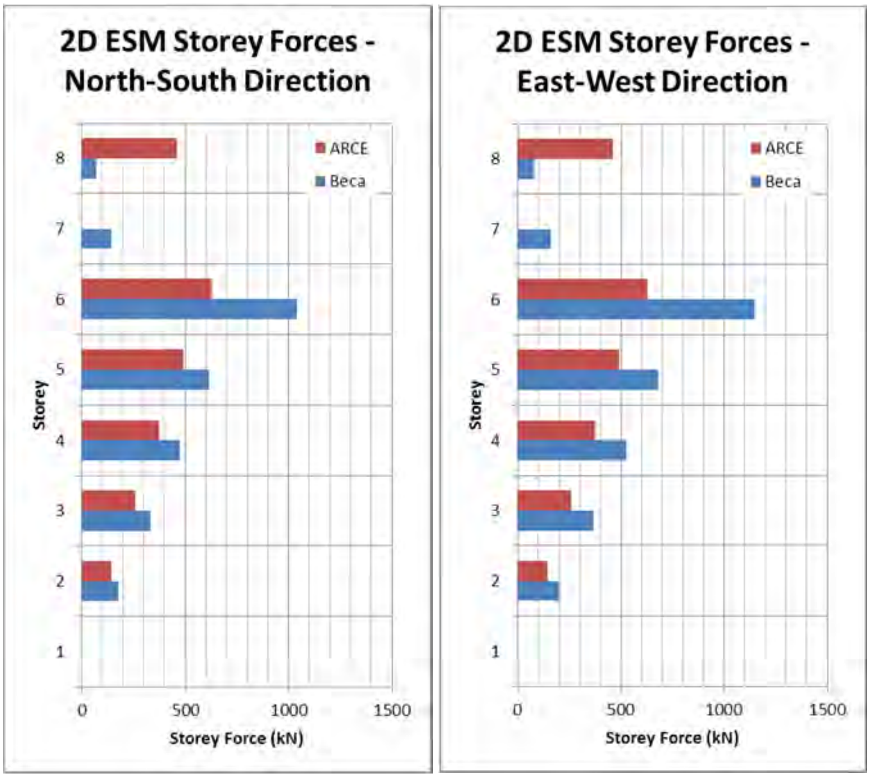


Figure F2: Distribution of base shear comparison

F.2.5 Building Eccentricity and Required Method of Analysis

With reference to Section 3.4.7.1(b) of NZS 4203:1984, the minimum requirement for analysis of a four-plus storey building that is reasonably regular but with a high degree of eccentricity is the static method of clause 3.4.7.2.

Our interpretation of Section C3.4.7.1 is that ‘regularity’ refers to the plan shape of the floor slabs and ‘eccentricity’ to the degree of symmetry of the lateral load-resisting system. With this interpretation, the CTV building would be considered to fit into the category referred to above (rectangular floor plates and eccentric for loading in the east-west direction). Therefore, as a minimum, only the static method is required.

The static method requires the storey forces to be applied in turn at each of two points distant 0.1b from the centre of mass in a three-dimensional analysis to account for accidental eccentricity. This is the method of analysis we have used to calculate the building deflections for imposing on the primary gravity system.

F.2.6 Building Deflections

With reference to Section 3.8.1 of NZS 4203:1984, the computed deformations of the primary seismic structure are calculated using the base shear calculated earlier, amplified by the factor K/SM, where K is taken as 2 for the equivalent static method and 2.2 for the spectral modal method of analysis.

Since we only calculated building deflections using the equivalent static method, we used the following factor to amplify deformations:

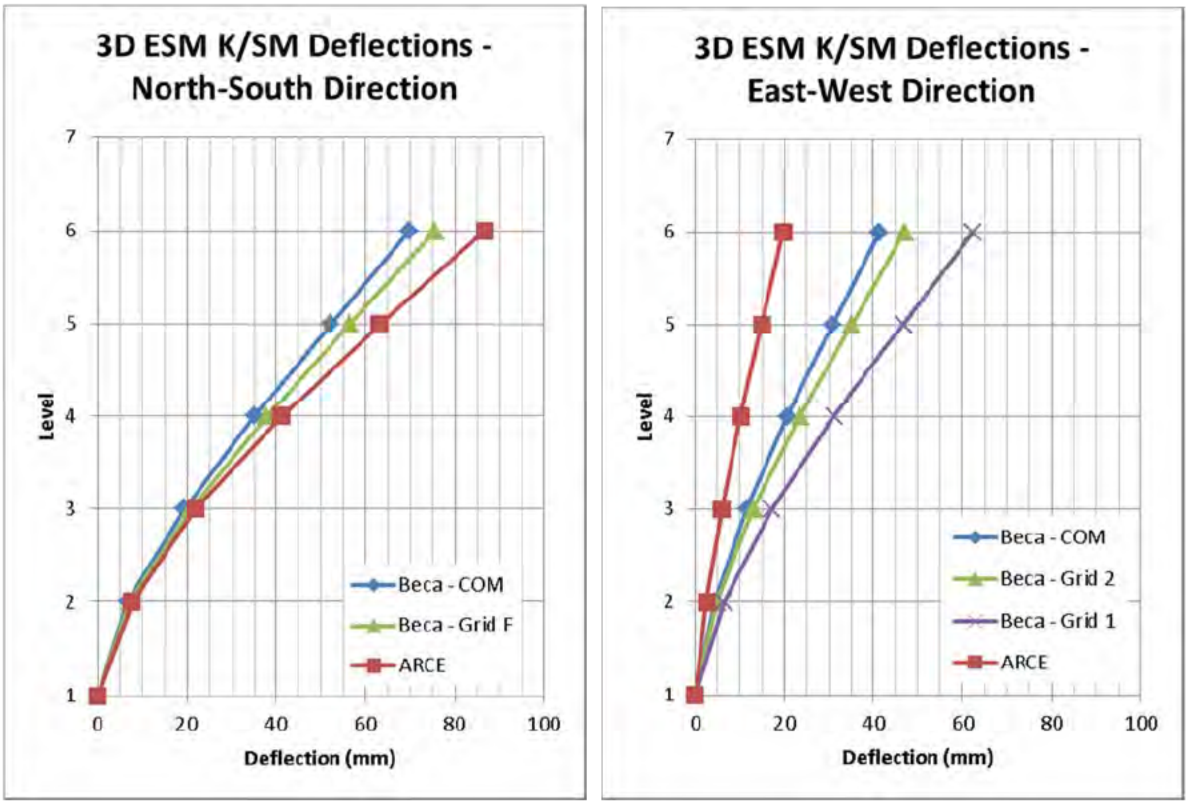
$K/SM = 2/(1.0*0.8) = 2.5$

We calculated building deflections at the centre of mass and grid line F for the north-south direction of loading, and the centre of mass and grid lines 1 and 2 for the east-west direction of loading. For grid line F, the storey forces were applied 3m to the east of the centre of mass. For grid lines 1 and 2, the storey forces were applied 2.3m to the south of the centre of mass (the least favourable direction in each case).

The following deflections were obtained on each grid line considered. Also shown are the relevant values reported in the 1986 calculations. It is not known where on the building in plan the deflections are being reported in the 1986 calculations (i.e. COM or some other location). The deflections are illustrated graphically in Figure F3. Only deflections for the levels with significant seismic weight (Levels 2 to 6) are reported.

Table F5: Scaled building deflections from equivalent static method

Level	Beca N/S COM (mm)	Beca N/S Grid F (mm)	ARCE N/S (mm)	Beca E/W COM (mm)	Beca E/W Grid 2 (mm)	Beca E/W Grid 1 (mm)	ARCE E/W (mm)
6	70	76	87	41	47	62	20
5	52	56	63	31	35	47	15
4	35	38	41	21	24	31	10
3	19	21	22	12	13	17	6
2	7	7	8	4	5	6	2
1	0	0	0	0	0	0	0



a) North-South direction

b) East-West direction

Figure F3: Comparison of scaled equivalent static deflections

With reference to Figure F3a, the ARCE deflections are slightly larger than the Beca deflections. This suggests a more flexible ARCE model that is also reflected in the longer natural period in this direction (ARCE 1.06s versus Beca 0.8s).

With reference to Figure F3b, the Beca deflections are much larger than the ARCE deflections (between two to three times larger depending where the ARCE deflections are reported). This suggests the Beca model is more flexible in this direction. However, this conflicts with the natural periods obtained in this direction (Beca 0.3s versus ARCE 1.06s).

F.2.7 Diaphragm Connections Compliance Check

The reinforced concrete floor slabs act as diaphragms to distribute horizontal seismic loads between the North Wall Complex and the South Shear Wall. In addition to the slabs themselves being required to have adequate capacity, their connections to the primary seismic load resisting elements must also have a viable load path with adequate capacity.

In this section we calculate the demand forces that the connections were required to be designed for, make comparison with the values used in the 1986 calculations, and assess the compliance of the configuration shown on the drawings.

F.2.7.1 Demand Forces

Compliance with the relevant codes with respect to the forces used to design the connections of the diaphragms to the shear walls is considered to be use of the smaller of the those calculated using the Parts and Portions section of NZS 4203:1984, and the maximum forces that can be resisted by the vertical primary seismic load resisting system (the shear walls). For the CTV building, the Parts and Portions values are smaller, and therefore the minimum requirement.

With reference to Section 3.4.9 of NZS 4203:1984, the diaphragm connection force demands using the Parts and Portions were calculated as follows:

At a given level,

$F_p = C_p W_p$

W_p = weight of part, taken to be seismic floor weight from Table F2

$C_p = \alpha K_x Z R C_{pmax}$

α

$\alpha = h_{cq}/h_n$

$h_{cq} = (\Sigma W_x h_x)/W_t$ = 331,466kNm/31,538kN = 10.5m

h_n = 21.66m

α = 10.5m/21.66m = 0.5

K_x

$K_x = h_x/h_{cq}$ but not less than 1 for multi-storey buildings

Z

Z = 5/6 (seismic zone B)

R

R = 1.0 (Category 4 – normal occupancy)

C_{pmax}

C_{pmax} = 0.3 (floors acting as diaphragms in multi-storey building with S_p=1.0)

The values obtained at each level are shown in Table F6 below.

Table F6: Parts and Portions values from NZS 4203:1984

Level	K_x	C_p	W_p (kN)	F_p (kN)
6	1.6	0.20	5837	1170
5	1.3	0.16	5926	950
4	1.0	0.13	6032	780
3	1.0	0.13	6137	800
2	1.0	0.13	6221	810

The values above represent the demand for a whole level and should be allocated to connections based on the relative contribution of the individual vertical primary seismic load resisting element being considered.

In the north-south direction, 100% of the lateral load is assumed to be resisted by the North Wall Complex. In the east-west direction, including the effects of accidental eccentricity, 60% of the lateral load is carried by both the North Wall Complex and the South Shear Wall.

Based on these allocations, the connection demand forces at each level are shown in Table F7 including a comparison with the values used in the original 1986 calculations.

Table F7: Connection force demand values comparison

Level	Beca NWC N/S (kN)	ARCE NWC N/S (kN)	Beca NWC E/W (kN)	ARCE NWC E/W (kN)	Beca SSW E/W (kN)	ARCE SSW E/W (kN)
6	1170	-	700	300	700	300
5	950	-	570	300	570	300
4	780	-	470	300	470	300
3	800	-	480	300	480	300
2	810	-	490	300	490	300

F.2.7.2 Connection Compliance

We calculated the capacity of the details shown on the drawings for the various diaphragm connections and compared these to the demands shown in Table F7 to assess compliance.

North-South Direction

The only significant connection between the floor slabs and the North Wall Complex was to walls C and C-D through starter bars between the slabs and the walls (Figure F4).

[redacted]

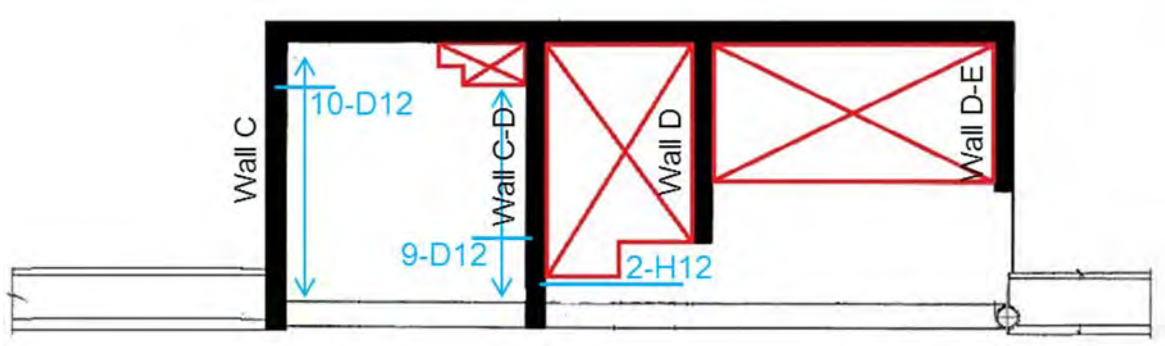


Figure F4: Details of diaphragm connection to North Wall Complex

Assuming the best case scenario where the shear capacity provided by these connections can fully develop into the floor slabs, the capacity of the connection assessed using shear friction principles from 7.3.11 of NZS 3101:1982 is as follows:

$$\phi V = \phi A_{vf}\mu f_y = 0.85*[(19*113\text{mm}^2)*1.4*275\text{MPa} + (2*113\text{mm}^2)*1.4*380\text{MPa}] = 800\text{kN}$$

The capacity of the connection as shown on the drawings (800kN) is less than the required demands at levels 2, 5 and 6 (810kN, 950kN and 1170kN) rendering it non-compliant. Even if it had the required capacity in shear friction, it may not have been able to be developed sufficiently into the floor slabs. It is also poor detailing to connect only two flanges of the four to the diaphragm (at Wall C and C-D, but not at Wall D and D-E).

East-West Direction

a. North Wall Complex

The diaphragm connection to the North Wall Complex for loads in the east-west direction relies on the area of floor slab enclosed by grids 4 and 5 and walls C and C-D, known as the toilet slab. In order to sufficiently develop diaphragm forces between the North Wall Complex and the floor slabs, the following aspects, at least, must have adequate capacity (Figure F5):

1. Shear friction at interface between toilet slab and grid 5 wall
2. Shear in the toilet slab itself
3. Moment in the toilet slab near grid 4
4. Shear in the floor slab outside grid 4
5. Development of the moment generated by the lever arm between grids 4 and 5 into the floor slab.

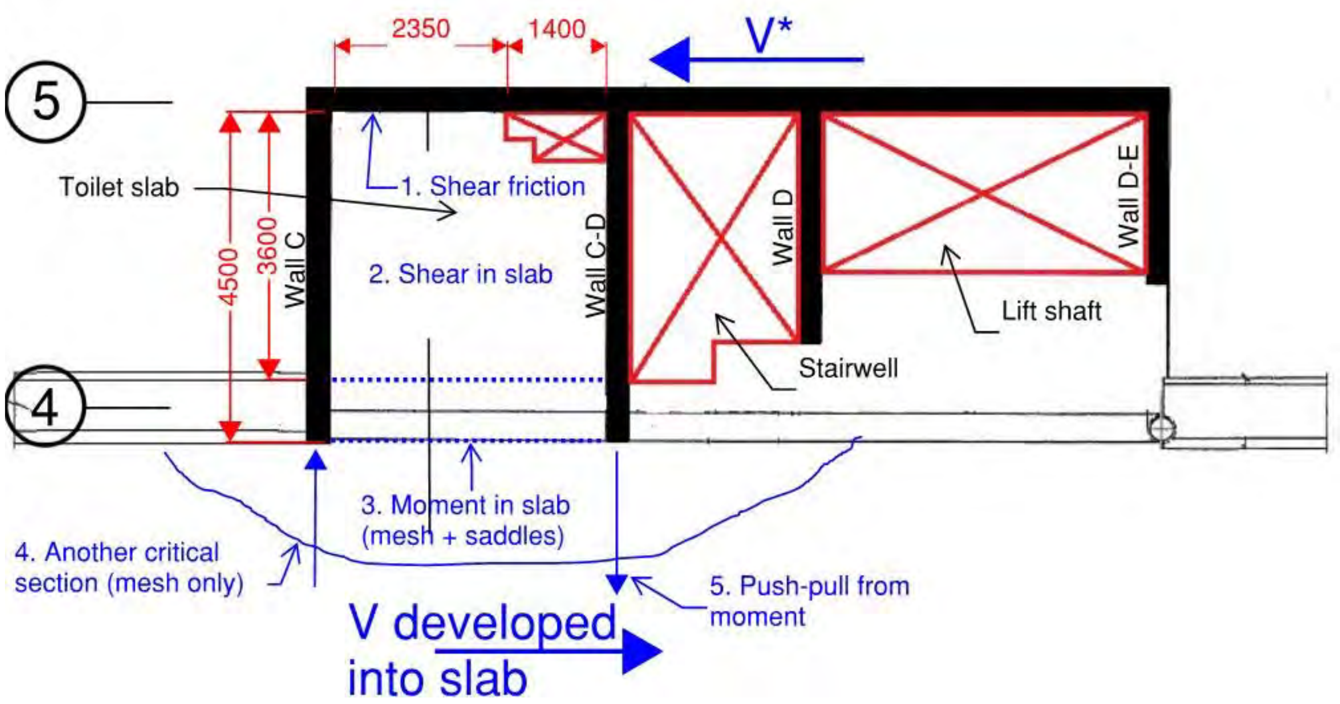


Figure F5: Diaphragm connection to North Wall Complex for east-west loading

We calculated the capacity of the first three and compared against the demands from Table F7.

1. Shear friction at interface between toilet slab and grid 5 wall
The drawings show eight D12 bars and two layers of 664 mesh crossing the interface. 664 mesh consists of 6mm diameter wires (f_y = 485MPa) at 150mm centres in both directions. However, the mesh is shown to be embedded only approximately 150mm into the wall. Given the spacing of wires in the mesh was 150mm, there was likely to be only one parallel-to-joint wire embedded in the wall and therefore only half the commonly accepted development length (two parallel wires) required.

Therefore, we assumed only one layer of mesh is effective in shear friction as opposed to two.
Clause 7.3.11.6 of NZS 3101:1982 also states the design yield strength of shear friction reinforcement shall not exceed 415MPa.

$\phi V = \phi A_{vf} \mu f_y = 0.85*[(8*113\text{mm}^2)*1.4*275\text{MPa} + (16*28\text{mm}^2)*1.4*415\text{MPa}] = 520\text{kN}$

The capacity of the connection based on shear friction as shown on the drawings (520kN) is less than the required demands at levels 5 and 6 (570kN and 700kN) rendering it non-compliant.

2. Shear in the toilet slab
Considered as a 3.75m deep member, 150mm thick. Clause 7.3.6.1 of NZS 3101:1982 also states the design yield strength of shear reinforcement shall not exceed 415MPa.

$\phi V = \phi(V_c+V_s) = 0.85*[0.2\sqrt{25\text{MPa}}*150\text{mm}*0.9*3750\text{mm} + 28\text{mm}^2*415\text{MPa}*0.9*3750\text{mm}/150\text{mm}] = 650\text{kN}$

The capacity of the connection based on the shear capacity of the toilet slab as shown on the drawings (650kN) is less than the required demand at level 6 (700kN) rendering it non-compliant.

[redacted]

3. Moment in the toilet slab near grid 4
Consider as a 3.75m deep member, 150mm thick with a lever arm of 4.5m to the point of application of shear. The section has one layer of 664 mesh and 19 H12 saddle bars evenly distributed.

From a sectional analysis program:

ϕM = 1800kNm

For a 4.5m lever arm this corresponds to a shear force of:

ϕV = 1800kNm/4.5m = 400kN

The capacity of the connection based on the moment capacity of the toilet slab as shown on the drawings (400kN) is less than the required demand at all levels (470kN to 700kN) rendering it non-compliant.

b. South Shear Wall

The diaphragm connection to the South Shear Wall for loads in the east-west direction relies on the 5m length of direct connection and the grid 1 beams either side acting as drag beams. In order to sufficiently develop diaphragm forces between the South Shear Wall and the floor slabs, the following aspects, at least, must have adequate capacity (Figure F6):

1. Shear friction at the grid 1 interface

2. Drag beam connections

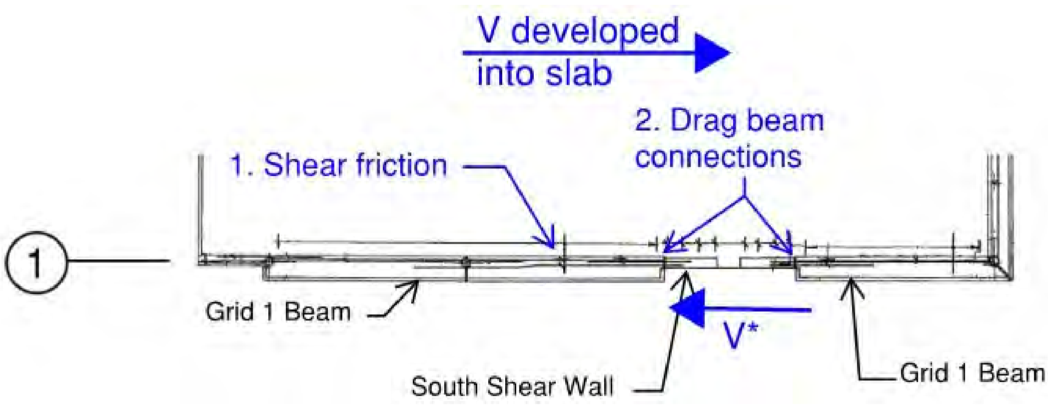


Figure F6: Diaphragm connection to South Shear Wall for east-west loading

We calculated the capacity of these aspects and compared against the demands from Table F7.

1. Shear friction at the grid 1 interface
The drawings show a total of 35 H12 starter bars along grid 1 at each level (8 directly into shear wall, 12 on eastern side and 15 on western side).

$\phi V = \phi A_{vf} \mu f_y$ = 0.85*[(35*113mm^2)*1.4*380MPa] = 1790kN

Capacity of the direct connection to the South Shear Wall only (8 H12 bars) in shear friction:

$\phi V = \phi A_{vf} \mu f_y$ = 0.85*[(8*113mm^2)*1.4*380MPa] = 410kN

Since the capacity of the direct connection to the shear wall only (410kN) is less than the demands at all levels (470kN to 700kN), some drag action is required of the grid 1 beams for the connection as a whole to be compliant.

2. Drag beam connections
To be compliant with the maximum demand of 700kN at level 6, the grid 1 beams must contribute a further 290kN of capacity acting as drag beams. The connections of the beams into the South Shear Wall consisted of six H24 bars (four top, two bottom) with straight embedment into the 400mm thick wall piers. There is no embedment length shown on the drawings for the top bars, but the bottom bars are shown to have 700mm embedment. This is an insufficient development length of a H24 bar using the provisions of NZS 3101:1982.

However, for a given direction of loading (i.e. east or west), one grid 1 beam acts in tension and the other in compression in drag action. In compression the beams can easily transmit the required 290kN. This is still the case at level 2 taking into account the reduction in starter bars that occurred during the installation of a stair penetration just east of the South Shear Wall in 2000.

Therefore, the diaphragm connection to the South Shear Wall, while not well detailed for forces in excess of the code demands, was compliant.

F.3 Primary Gravity System

The primary gravity system consisted of the various beams, columns, and their connections. These elements were not required to contribute to the lateral strength of the building, but did form the large majority of the primary gravity system and were required to sustain the seismic displacements while performing their gravity load function. NZS 3101:1982 allowed them to be considered as ‘Group 2 secondary structural elements’. There were three levels of ductile detailing available for these elements; the non-seismic (non-ductile) provisions, the limited ductile provisions and the seismic provisions (increasing level of ductility required/ provided). NZS 3101 provided a method for determining which provisions, as a minimum, must be used. In general terms, the method involved imposing deformations (relating to the deformations of the primary seismic structure, i.e. Table F5 above) on models of the primary gravity structure and assessing the extent of yielding in the members.

The detailing of the beams and columns as shown on the drawings appears generally in accordance with the use of the non-seismic provisions. For this to be valid under the code, the members must remain elastic when models of the frames are subjected to the imposed deformations. In addition, the beam-column joints required consideration using a rational method of analysis to determine the reinforcement to be provided in them.

The 1986 calculations do not contain any evidence that these methods or suitable equivalents were carried out to justify the use of the non-seismic provisions, and therefore the detailing provided. Since Mr Harding used ETABS for the analysis of the primary seismic structure, it was also available for him to carry out analyses on the primary gravity structure.

We have carried out this method and present a summary of our inputs below. We have only presented information for the frames on grids F, 2 and 1 as these are the frames with the largest displacement demands for the two orthogonal directions of loading.

F.3.1 Modelling Approach

Our analysis has followed the relevant standards of the time and used the configuration of the building as shown on the structural drawings. We have also used assumptions and analysis techniques as close as possible to what was available and common at the time.

[redacted]

Our modelling of the primary gravity system under imposed deformations was as follows:

- Built separate two-dimensional ETABS (v9.7.3) models of the frames on grids F, 2 and 1
- Modelled the structure above ground only and assumed the columns are fixed at ground level
- Modelled the beams and columns as a series of interconnected frame elements, taking into account the effect of cracking and joint flexibility using common assumptions used in the mid-1980s
- Assigned rigid diaphragms at each level and applied appropriate gravity loads to be considered in combination with the prescribed lateral displacements of each level (Table F5)
- Ran elastic analyses
- Obtained the bending moment and shear force demand diagrams
- Calculated the dependable bending moment and shear force strengths of the members and compared these to the demands to identify those areas that do not remain elastic.

F.3.2 Modelling Inputs

F.3.2.1 Stiffness and Material Properties

We used the following to represent the stiffness properties of the reinforced concrete elements of the primary gravity structure:

Table F8: Primary Gravity Structure Stiffness Properties

Property	Value
Concrete strength (MPa)	35 (Columns L1 to L2) 30 (Columns L2 to L3) 25 (Columns above L3 and beams)
Elastic modulus (MPa)	27,800 (35MPa concrete) 25,700 (30MPa concrete) 23,500 (25MPa concrete)
Columns effective stiffness (I_e/I_g)	1.0
Beams effective stiffness (I_e/I_g)	0.5
Beam-column joints rigid zone factor	0.5
Beam-column joints end length offsets	Full joint length
'H' reinforcement yield strength (MPa)	380
Other reinforcement yield strength (MPa)	275

F.3.2.2 Column Gravity Axial Loads

NZS 4203:1984 specified the loading combinations to be considered. The relevant combinations for earthquake loading were:

$U = 0.9D + E$ – reduced dead and earthquake

$U = 1.0D + 1.3L_R + E$ – dead, reduced live and earthquake

We have calculated the axial load demands on the columns on grids F, 2 and 1 for the two gravity load combination cases using a tributary area approach (Table F9).

Table F9: Column gravity axial load demands

Column	Level	Axial Load Demand (kN) 0.9*D*	Axial Load Demand (kN) 1.0D + 1.3 L_R
C1/C17	5-6	240	320
	4-5	430	560
	3-4	620	790
	2-3	800	1020
	1-2	990	1260
C2	5-6	230	340
	4-5	440	620
	3-4	640	890
	2-3	850	1170
	1-2	1060	1440
C3	5-6	150	230
	4-5	280	420
	3-4	420	600
	2-3	550	780
	1-2	690	960
C4	5-6	40	60
	4-5	70	120
	3-4	110	180
	2-3	150	230
	1-2	190	290
C5/C11	5-6	220	300
	4-5	430	570
	3-4	630	820
	2-3	840	1080
	1-2	1040	1340
C6/C9	5-6	220	360
	4-5	420	650
	3-4	620	930
	2-3	820	1210
	1-2	1020	1490
C7/C8	5-6	260	430
	4-5	500	770
	3-4	740	1110
	2-3	990	1450
	1-2	1230	1790
C10	5-6	100	170
	4-5	180	300
	3-4	280	440
	2-3	380	570
	1-2	470	710

F.3.2.3 Beam Gravity Line Loads

We have calculated the line load demands on the beams on grids F, 2 and 1 for the two gravity load combination cases using a tributary width/area approach (Table F10).

Table F10: Beam gravity line load demands

Beam	Level	Line Load Demand (kN/m) 0.9D	Line Load Demand (kN/m) 1.0D + 1.3L_R
Grid F	All	11	12
Grid 2	All	33	54
Grid 1	All	26	38

Note: The beams on grid F are assumed to carry no weight from the floors since they span parallel to the Hi-bond permanent formwork.

F.3.3 Frame Member Flexural Capacities

Using the material properties shown above in Table F8, the column axial load demands shown above in Table F9, and the details as shown on the structural drawings, we calculated the dependable moment capacities of the column and beam potential yielding regions.

To calculate dependable capacities in flexure, we used a strength reduction factor, ϕ, equal to 0.9 (NZS 3101:1982 clause 4.3.1.2a).

Our calculated capacity values are shown below in Table F11 for columns, and Table F12 for beams.

Table F11: Column dependable flexural capacities

Column	Level	ϕM (kNm) 0.9D	ϕM (kNm) 1.0D + 1.3L_R
C1/C17	5-6	110	110
	4-5	120	130
	3-4	140	140
	2-3	150	160
	1-2	170	180
C2	5-6	110	120
	4-5	120	130
	3-4	140	140
	2-3	160	160
	1-2	170	180
C3	5-6	100	110
	4-5	110	120
	3-4	120	130
	2-3	140	150
	1-2	150	170

C4	5-6	60	60
	4-5	60	60
	3-4	60	70
	2-3	70	70
	1-2	70	80
C5/C11	5-6	110	110
	4-5	120	130
	3-4	140	140
	2-3	160	160
	1-2	170	180
C6/C9	5-6	110	120
	4-5	120	130
	3-4	140	140
	2-3	150	160
	1-2	170	180
C7/C8	5-6	110	120
	4-5	130	130
	3-4	140	140
	2-3	160	160
	1-2	180	180
C10	5-6	60	70
	4-5	70	80
	3-4	80	90
	2-3	90	100
	1-2	100	120

[redacted]

Table F12: Beam dependable flexural capacities

Grid	Location	Level	Beam Size, *d* x *b* (mm)	Reinforcement, top/bottom	ϕM top/bottom (kNm)
Grid F	1R	All	550 x 960	2-H24/2-H24	150/150
	2L/2R	All	550 x 960	4-H24 + 664 mesh/2-H24	320/150
	3L/3R	All	550 x 960	4-H24 + 664 mesh/2-H24	320/150
	4L	All	550 x 960	2-H24/2-H24	150/150
Grid 2	AR	All	350 x 400	2-H28/2-H28	110/110
	BL/BR	All	550 x 400	4-H28 + 664 mesh/2-H28	400/190
	CL/CR	All	550 x 400	4-H28 + 664 mesh/2-H28	400/190
	DL/DR	All	550 x 400	4-H28 + 664 mesh/2-H28	400/190
	EL/ER	All	550 x 400	4-H28 + 664 mesh/2-H28	400/190
	FL	All	350 x 400	2-H28/2-H28	110/110
Grid 1	AR	All	550 x 400	2-H24/2-H24	150/150
	BL	All	550 x 400	3-H24 + 664 mesh/2-H24	230/150
	BR	All	550 x 960	3-H24 + 664 mesh/2-H24	250/150
	CL/CR	All	550 x 960	4-H24 + 664 mesh/2-H24	320/150
	SSW	All	550 x 960	4-H24/2-H24	290/150
	FL	All	550 x 960	2-H24/2-H24	150/150

Notes:

- 'Grid F, 1R' refers to the beam section on grid F, just to the right of grid 1 (i.e. at column face). Convention is looking north or looking west.
- Beam flexural capacities have been calculated assuming all longitudinal reinforcement is fully developed at the column face.

F.3.4 Beam-Column Joint Compliance Check

In addition to assessing the potential yielding regions of the beams and columns for yielding under the prescribed imposed deformations, and designing the transverse reinforcement accordingly, the engineer was required to design the transverse reinforcement to be provided in the beam-column joints.

The 1986 calculations for the CTV building do not contain any checks or specific design of the beam-column joints. The transverse joint reinforcement indicated on the drawings is the same as that in the columns, i.e. an R6 spiral at 250mm pitch.

As for the beams and columns, the beam-column joints section (Section 9) of the concrete structures standard contained three levels of ductile detailing to be provided; the non-seismic provisions, the limited ductile provisions and the seismic provisions (increasing level of ductility required/provided).

Whereas for the potential yielding regions of the beams and columns it was well-defined how to determine which level to use for design, for the beam-column joints it was not as clear. This is particularly so for the case where the beams and columns meeting at a joint do not yield adjacent to the joint under design seismic loads/deformations.

We have taken the minimum level of compliance for a joint for this case to be the following:

1. Obtain the demands from an elastic analysis as described in F2.1 and F2.2 above.
2. Use a rational method of analysis to convert these to joint demands.

[redacted]

3. Determine the joint reinforcement to be provided from the non-seismic beam-column joint provisions (Section 9.4 of NZS 3101:1982).

We have carried out the above on a typical interior joint at level 2 on grid 2 to assess the level of compliance of the R6 spiral at 250mm pitch provided in the design:

1. Frame demands from elastic analysis
Frame member demands near interior joint on grid 2 at level 2 from 2D elastic analysis (Figure F7), with modelling assumptions and imposed deformations as per Table F8 and Table F5 respectively.

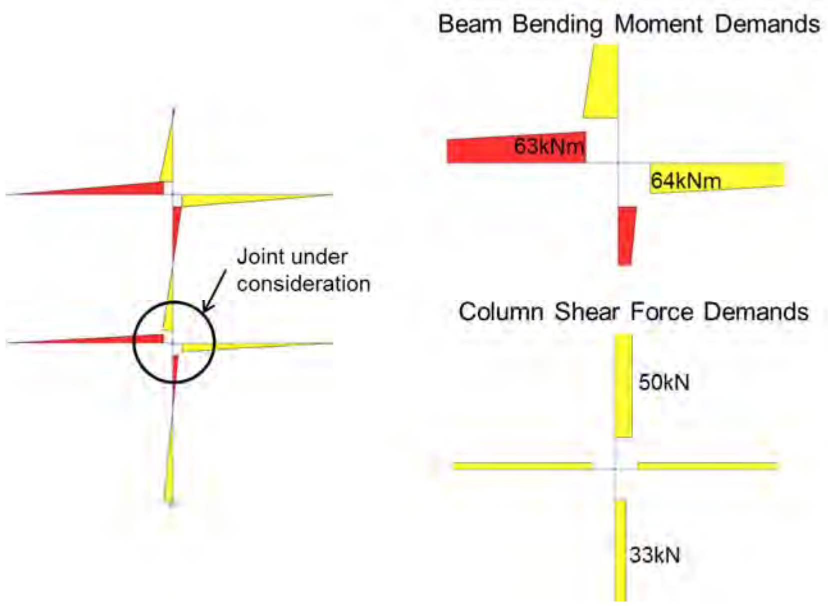
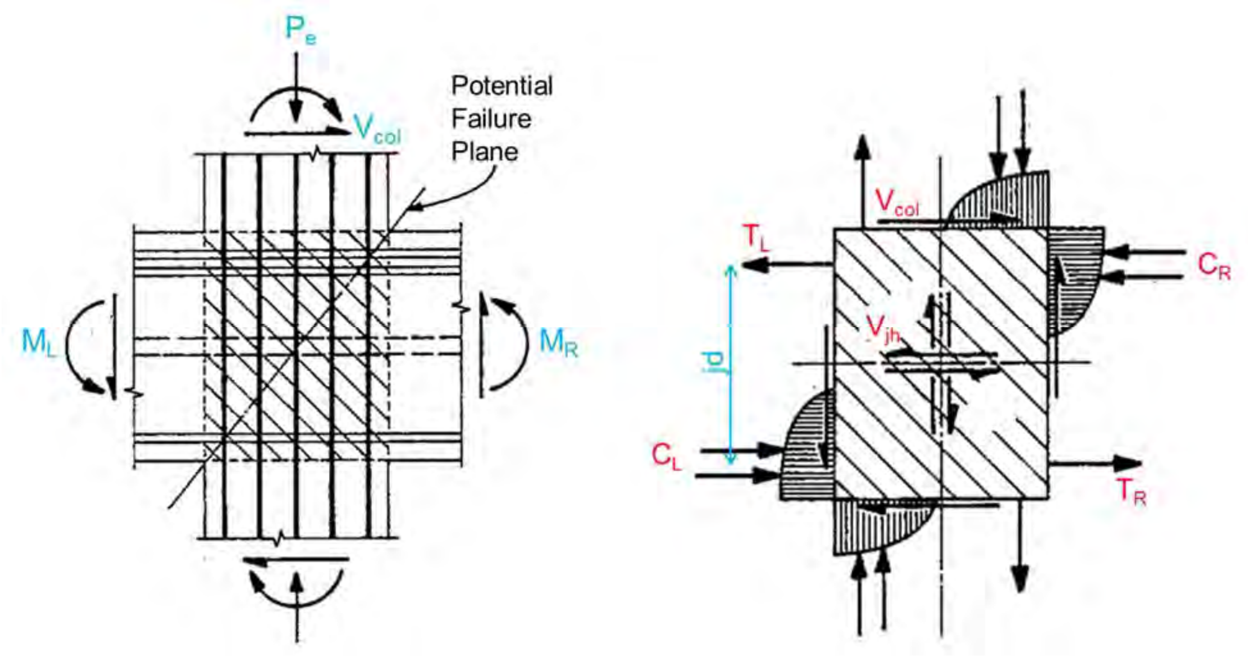


Figure F7: Frame member demands at level 2 internal joint on grid 2

[redacted]

2. Joint demands

The important parameter used to design the horizontal joint shear reinforcement is V_{jh}, the total horizontal shear force across the joint. V_{jh} should be evaluated from a rational analysis taking into account the effect of all forces acting on the joint (Figure F8).



a) External actions

b) Internal forces

Figure F8: External actions and internal force diagrams for interior beam-column joint

From horizontal equilibrium for top half of joint:

$$V_{jh} = T_L + C_R - V_{col}$$

From beam section equilibrium:

$$C_R = T_R$$

Therefore:

$$V_{jh} = T_L + T_R - V_{col}$$

$$T_i = M_i / jd$$

$$jd = 0.45\text{m}$$

M_L = 63kNm, T_L = 140kN

M_R = 64kNm, T_R = 142kN

V_{col} = 50kN

V_{jh} = 140kN + 142kN – 50kN = **232kN**

3. Determine horizontal joint shear reinforcement to be provided

$$V_{\mathrm{sh}} = \frac{V_{\mathrm{jh}}}{\emptyset} - V_{\mathrm{ch}}$$

$$V_{\mathrm{ch}} = 0.5V_{\mathrm{jh}}\left(1 + \frac{C_{\mathrm{j}}P_{\mathrm{u}}}{0.4A_{\mathrm{g}}f_c'}\right)$$

ϕ = 0.85 (forces not derived from overstrength member actions)

C_j = 1.0 (for one-way joint)

P_u = 990kN (smaller value from Table F9 for C7/C8 level 2-3, seismic-induced axial loads ignored as equal beam spans either side)

A_g = 125664mm^2

f'_c = 25MPa (assumed joint concrete same as slab concrete)

$V_{ch} = 0.5 \times 232000\left(1 + \frac{1.0\times990000}{0.4\times125664\times25}\right)$ = **207kN**

$V_{sh} = \frac{232}{0.85} - 207$ = **66kN**

Required area of horizontal joint shear reinforcement:

$$A_{jh} = \frac{V_{sh}}{f_y} = \frac{66000N}{275MPa} = 240mm^2$$

A_{jh} is the effective total area of horizontal reinforcement that crosses the critical diagonal plane, determined according to the orientation of the individual tie legs with respect to this failure plane. It is calculated as follows:

$$A_{jh} = \frac{\pi}{2} \times \frac{h_e}{s} \times A_{te}$$

An R6 spiral at 250mm pitch (as specified) provides the following effective total area of horizontal reinforcement:

$$A_{\mathrm{jhpro}} = \frac{\pi}{2} \times \frac{450mm}{250mm} \times 28.3mm^2 = \boldsymbol{80mm^2}$$

The provided amount (80mm^2) is significantly less than the required amount (240mm^2). Therefore, the beam-column joint transverse reinforcement is non-compliant. This calculation was carried out for a level 2 interior joint on grid 2. However, it can be shown that the interior joints on grid 2 at all other levels are also non-compliant.

The minimum spacing of joint transverse reinforcement for the interior joint on grid 2 at level 2 to be compliant is:

$$s = \frac{\pi}{2} \times \frac{h_{\mathrm{e}}}{A_{\mathrm{jh}}} \times A_{\mathrm{te}} = \frac{\pi}{2} \times \frac{450mm}{240mm^2} \times 28.3mm^2 = \boldsymbol{85mm}$$

[redacted]

Appendix G

1980s Christchurch Buildings Review

Appendix G Comparison of Relevant Details in a set of 1980s Christchurch Shear Wall Buildings

Address	Design Date	Design Company	Engineer Initials	Storeys	Typical floor plan area (m^2)	Relative Feb 2011 EQ PGA (CTV = 1.0)	Foundations	Typical gravity column size	Bottom floor gravity column longitudinal steel	Typical gravity column PPHZ transverse steel	Typical joint transverse steel
249 Madras Street (CTV Building)	1986	Alan M. Reay Consulting Engineer	DH	5	700	1	Ground beams and raft	400mm dia	6-H20	R6 spiral @ 250 pitch	R6 spiral @ 250
9 Cathedral Square	1989	[redacted]	RBR	3	1300	1	Basement raft	450mm dia	6-H28	H10 spiral @ 50 pitch	H10 spiral @ 50
32 Oxford Terrace	1986	[redacted]	RBR	4	800	0.9	Ground beams and raft	500 x 400mm	8-D20	R10 stirrups @ 300	R10 stirrups @ 100
115 Kilmore Street	1986	[redacted]	ADW	4	800	1	Ground beams	400mm dia	12-Y20	R10 spiral @ 100 pitch	R10 beam stirrups
79 Cambridge Terrace	1985	[redacted]	-	6	600	0.9	Ground beams and piles	400 x 400mm	4-H28 + 4H20	R10 stirrups @ 150	4 sets R10 in 550 deep joint
287 Durham Street (Landsborough House)	1985	[redacted]	JH	7	600	1	Ground beams and piles	400 x 400mm	4-H28 + 4H20	R10 stirrups @ 150	4 sets R10 in 550 deep joint
								550mm dia	4-H24 + 4H20	R10 spiral @ 150 pitch	4 sets R10 in 550 deep joint
227 Cambridge Terrace	1984	[redacted]	-	6	700	1	Ground beams	800 x 400mm	12-H20	R10 stirrup pairs @ 40 (base) and 80 generally	8 pairs R12 links in 600 deep joint
								500 x 500mm	4H20 + 4H16	R10 stirrup pairs @ 50 (base) and 100 generally	3 pairs R12 links in 600 deep joint

Notes:

1. Number of storeys is defined as the number of reinforced concrete floor slabs above ground.
2. Relative Feb 2011 EQ PGA determined from a high-level assessment of information contained in Canterbury Geotechnical Database. Assessment carried out by Beca.

[redacted]

Appendix H

Nonlinear Time-History Analyses

Appendix H Nonlinear Time-History Analyses

H.1 Introduction

Non-linear time history analyses (NLTHAs) were undertaken jointly by Beca and [redacted] to analyse the performance of the CTV building structure under shaking recorded at other sites in Christchurch at the time of the earthquake, but considered to be representative of what the CTV site could have experienced.

These additional analyses were to supplement the previous NLTHAs carried out by Compusoft Engineering Limited under the instruction of the Canterbury Earthquake Royal Commission (CERC) and the former Department of Building and Housing (DBH)'s Technical Investigation.

The purpose for undertaking the additional NLTHAs was:

- To investigate the overall performance of the building and in particular the demands on the primary gravity structure during the earthquake(s)
- To refine and remove some uncertainties identified in the previous NLTHA and investigations
- To assess and inform on the role that different aspects of the design played in the pancaking collapse
- To inform on Beca's interpretation of building response and the cause of pancaking collapse

Fundamentally, we sought to test the following hypothesis:

If a model of the building representing a code compliant building did not reach a state where pancaking collapse was predicted when it is subjected to the most appropriate record for the site available from the 22 February 2011 earthquake (the Christchurch Cathedral College record), but the as designed model did reach this state for the same input, the design omissions are a substantial cause of the pancaking collapse.

Pancaking collapse in this context is complete loss of gravity support in the primary gravity structure as opposed to a severely deformed structure but one where the floor slabs are still held apart.

A key difference between these analyses and those carried out previously was the post-processing of the results to determine the collapse potential of the gravity system.

A number of aspects of the analyses, modelling assumptions failure criteria and consequences of design aspects and non-compliances are discussed in Sections H2 to H4 and then the conclusions are brought together in Section H5.

H.2 Revised Beca Analyses

In this section we summarise the NLTHAs that were completed for the CTV building as part of the initial investigations into the collapse and describe the changes that we have instigated to the computer model of the building and to the input motions as part of our investigation.

H.2.1 Previous Non-Linear Time History Analyses

H.2.1.1 DBH Technical Investigation – Hyland/Smith Report

Dr Clark Hyland and Mr Ashley Smith undertook the technical investigation of the collapse of the CTV building for DBH. Compusoft assisted with the computer analyses and Tonkin & Taylor provided geotechnical engineering inputs. The non-linear analysis assumptions and results are summarised in Compusoft's CTV Building Non-Linear Seismic Analysis Report (Ref: 11033-00) dated 9 February 2012, prepared for Hyland and Smith. A summary of the Hyland/Smith report can be found in Section 5.2 of Vol 6 of the CERC report.

H.2.1.2 CERC Technical Investigation – Expert Panels and further Compusoft NLTHA report

Under the instruction of the CERC, Compusoft undertook further analyses using a modification of the SAP2000 computer model of the CTV building developed for the DBH and Hyland/Smith investigation. These analyses were to provide support for the CERC's non-linear time history analysis (NTHA) expert panel deliberations and recommendations.

The major changes to the model from that used in the DBH analyses were:

- Non-linearity of the beam column joint was modelled using stiffness/strength degrading hinges
- Plastic hinges at the top and bottom of each column were modelled to incorporate axial load-moment interaction, ie the moment capacity of the columns was linked to the axial load in the columns to more correctly represent the actual situation
- The possibility of non-linear behaviour of the floor diaphragm adjacent to the north and south walls, and beam lines 2 and 3 was modelled
- The Rayleigh damping coefficients were slightly adjusted so that appropriate levels of damping were assigned to the vertical vibration of the floors
- The ground motions recorded from the CBGS and CCCC sites were run sequentially, for both the September and February events, to investigate whether possible damage from the first event influenced the results of analyses during the February earthquake

The assumptions and results from the CERC NLTHAs are summarised in Compusoft's CTV Building Non-Linear Time History Analysis Report (Ref: 12026-00) dated 31 August 2012. Further discussion and debate on the second NLTHA report is given in the *Joint Report in relation to Interpretation of Second Compusoft NTHA* prepared by the NITHA Expert Panel instigated by CERC.

H.2.2 Revised Beca Models

The same analysis software used by Compusoft for the technical investigations instigated by DBH and CERC was used for the additional analyses.

Two nonlinear models were created:

- ***Beca "as-designed" model:*** A model to represent the as-designed building prior to 4 September 2010. This is referred to in this report as the as-designed model
- ***Beca "compliant" model:*** A model to represent the building without the identified design non-compliances. This is referred to in this report as the compliant model.

Sub-models were also created to test the sensitivity of the modelling parameters. The modifications made for these are described below.

The NLTHAs did not model the construction deficiencies.

H.2.3 Changes Made to NLTHA Models

The following changes were made to the CTV SAP2000 model used by the CERC investigation to create the as-designed model for our investigations:

- **Column non-linearity and axial load collapse modelling:** The uncertainty in the modelling of the column potential plastic hinge region in the earlier analyses was removed by assuming a "fully confined column core" region. This was achieved by modifying the concrete material model used for the concrete core region. The concrete model adopted for the core region does not have a strength degradation branch (i.e. elastic-perfectly plastic behaviour was modelled without limits on the deformation capacity).

- **Soil-foundation stiffness** was updated based on further geotechnical interpretation of the existing geotechnical data by Beca.
- The effect of **vertical acceleration ground motions** was de-coupled in the result interpretation by running additional parallel analyses with and without vertical acceleration inputs.
- The **effects of ground motion variability** were briefly tested by running input ground motions from two ground recording stations (REHS and CCCC). These analyses were used to provide trends and an indication of the differences in response. It is acknowledged that neither of these ground motions are necessarily the motions that may have been experienced at the site. However, the CCCC record is considered to be the most representative for this site from those obtained from the 22 Feb 2011 earthquake.

No change was made to the beam-column joint non-linear model and the diaphragm non-linear modelling.

To create the compliant model, the following changes were made to our as-designed model:

- **Compliant beam-column joints:** Compliant beam-column joints were assessed by calculation to remain elastic under the expected actions and, therefore, were modelled to remain elastic with beam effective. Note: The beam-column joint springs in the E-W direction were modelled as elastic elements which by definition do not crack or degrade. As the joints remain elastic for loading in the N-S direction, no modelling change was made for this direction.
- **Compliant floor diaphragm connections:** The connections between each of the northern core walls and the floor diaphragm in the north-south direction were modelled as elastic perfectly plastic with strength determined from our compliant building checks.
- **Southern shear wall (SSW):** This wall was modelled to be approximately 60% stronger in terms of its lateral load resistance, to correct for the non-conservative original design lateral load levels in the E-W direction. The additional capacity was modelled as additional longitudinal reinforcing steel and coupling beam reinforcing steel, while maintaining the initial stiffness of the SSW.
- **Northern core walls (NWC)** – Additional longitudinal reinforcing was added to the walls on line C/D and D, resulting in a stronger and stiffer NWC to correct for the non-conservative original design lateral load levels in the N-S direction.

H.2.4 Rationale for the changes made

H.2.4.1 Column non-linearity and axial load collapse modelling

The changes made to the column non-linearity modelling are to simplify the non-linear analysis and to allow the axial failure of the columns to be investigated using a collapse assessment procedure (by post-processing the NLTHA results) which is more informative than the previous approaches used for the DBH and CERC NLTHAs.

In the earlier analyses for DBH and CERC, non-ductile column failure was determined to be a primary collapse initiator, based on the predicted individual column strain demands. The use of column strain demands in this manner has the potential to overestimate the possibility of building collapse as reaching the flexural capacity in individual columns is not necessarily indicative of loss of axial support and, therefore, building partial or pancaking collapse. There are limitations in the current NLTHA tools to capture the various column and column/joint "collapse modes" (axial-shear, squashing, buckling, sidesway, etc), but other more refined assessment tools are available that provide a better prediction of "collapse" initiation in non-ductile columns (ASCE-41 2013, NIST 2013, Elwood and Moehle (2013)). The peer review of the CERC analyses by William Holmes noted the similar weakness of the previous NLTHAs in this aspect (Holmes, (2012)).

We believe the column strain is representative of the column flexural behaviour, but not necessarily adequate to represent the axial-shear-flexural and, more importantly, axial collapse (loss of gravity support) behaviour. We therefore assessed the “column axial load collapse” potential using post-processing of the NLTHA predicted drifts as described in Section H4.

H.2.4.2 Soil-Foundation Modelling

Previous NLTHAs were based on geotechnical advice from Tonkin and Taylor. Beca geotechnical engineers reviewed Tonkin and Taylor’s inputs and provided further modelling recommendations. Revised soil spring stiffnesses were recommended including a reduced stiffness to recognise expected degradation during the earthquake shaking.

H.2.4.3 Vertical acceleration ground motion

We have observed from the previous Compusoft analyses, and also from our analyses that the vertical acceleration maxima components for the 22 February 2011 earthquake are generally out of phase in the strong ground motions records (i.e. the peak values of vertical and horizontal ground accelerations are separated by approximately 1.5 seconds (refer Figures H9 and H10).

Therefore, we elected to de-couple the vertical acceleration effects in the result interpretation by running additional parallel analyses with and without vertical acceleration inputs. The effects of vertical acceleration on the collapse propensity were assessed by comparing the results from these analyses. This is discussed in Section H3.4

H.2.5 List of time-history analysis runs completed

The following additional non-linear time history analysis runs were completed using the revised models.

- As-designed model
  - CCCC ground motion without vertical acceleration
  - CCCC ground motion with vertical acceleration
  - REHS ground motion without vertical acceleration
  - REHS ground motion with vertical acceleration

- Compliant model -
  - CCCC ground motion without vertical acceleration
  - CCCC ground motion with vertical acceleration
  - REHS ground motion without vertical acceleration
  - REHS ground motion with vertical acceleration

- Compliant B model to investigate the effect of drag bar failure (refer to Section H3.6)
  - CCCC ground motion with vertical acceleration.

Only a selection of results from these analyses are presented in the following sections. However, the discussion on the implications of the results of the NLTHAs include our assessment of the results from all analyses that we have completed.

[redacted]

H.3 Non Linear Time History Analysis Results

H.3.1 Reference Plan and Labels

The discussion in the following sections of this report is with reference to the labels and plan shown in Figure H1. In accordance with the convention used elsewhere in this report the levels are labelled consecutively from L1 to L7 with L1 being the floor at ground level and L7 at roof level.

Interstorey drift in this report is taken as the difference in lateral deflection between two adjacent levels divided by the difference in height of these levels.

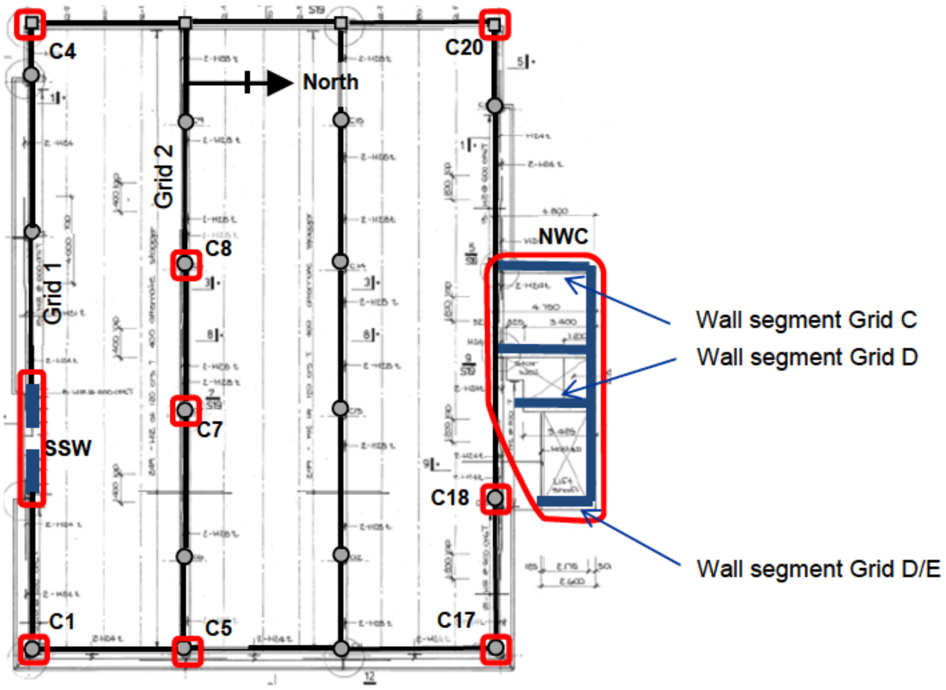


Figure H1: Reference plan and labelling of critical columns and walls

H.3.2 As-designed model response for CCCC record

The sequence of significant movements for the building predicted by the NLTHAs for our as-designed model using the CCCC record is shown in Figures H2 and H3. Note: the horizontal building movements have been exaggerated for viewing purposes and are not representative of the scale experienced by the actual building during the 22 February 2011 earthquake.

The analyses indicated that the building initially underwent a couple of cycles predominantly in the north-south direction, and then towards the north-west corner. The inter-storey drift along the perimeter elevations exceeded 0.5% to 1.0%, suggesting some of the glass panels would break and perimeter columns would form top and bottom flexural hinges. The modelled retrofit drag bars exceeded their axial tension capacity at approximately 2-3 seconds into the analysis (between steps 1 and 2 in Figure H2 and as shown in Figure H3). Joint shear cracks were predicted at selected interior and exterior beam-column joints.

Approximately 4.5 to 5 seconds into the earthquake analysis (step 2 in Figure H2 and as shown on Figure H3), the building moved significantly to the east, pivoting about the NWC. This resulted in very high inter-storey drift demands along the southern elevations (Grid Lines 1 and 2), with maximum inter-storey drifts in the order of 2.5 to 3.5% drift. This movement was sufficient to cause significant yield at the base and

[redacted]

in the coupling beams of the SSW and in the column regions just below the soffit of the internal beam on Grid 2 at Level 2 (L2).

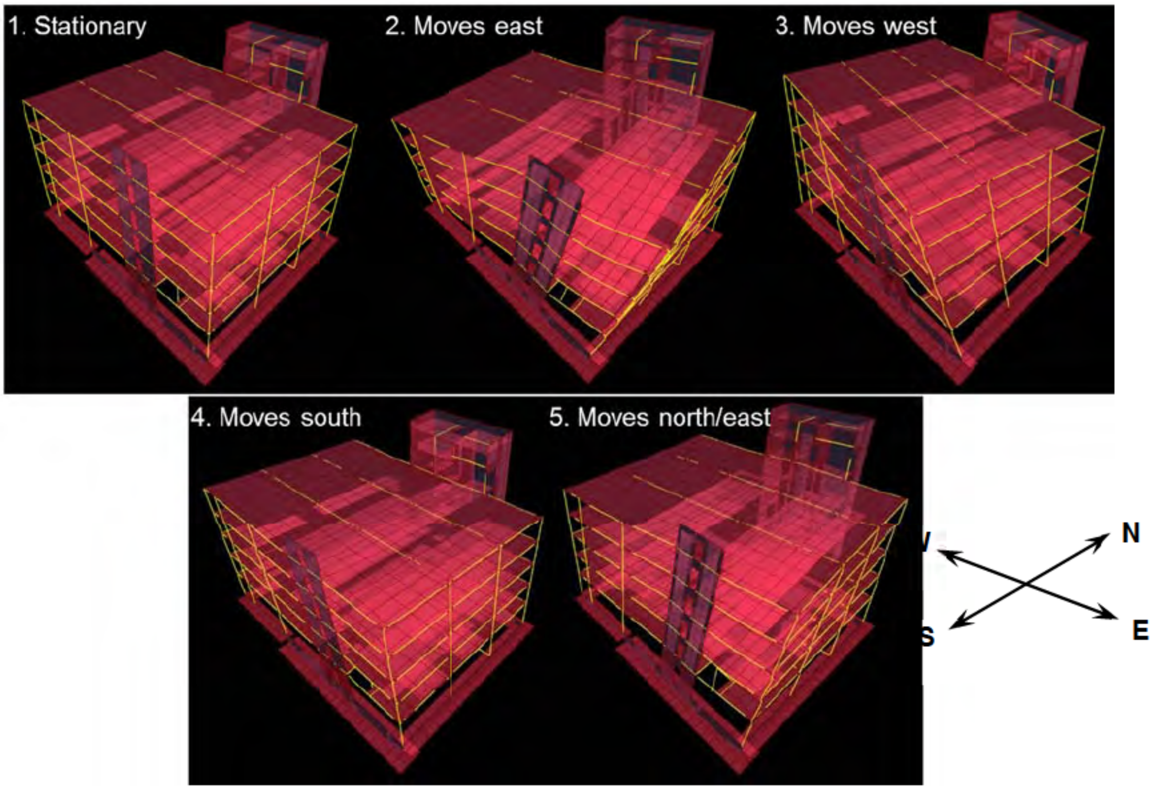


Figure H2: Building movement sequence from the NLTHA using the CCCC record for the 22 February 2011 earthquake (Source: [redacted] Note the deformations are shown to an exaggerated scale.

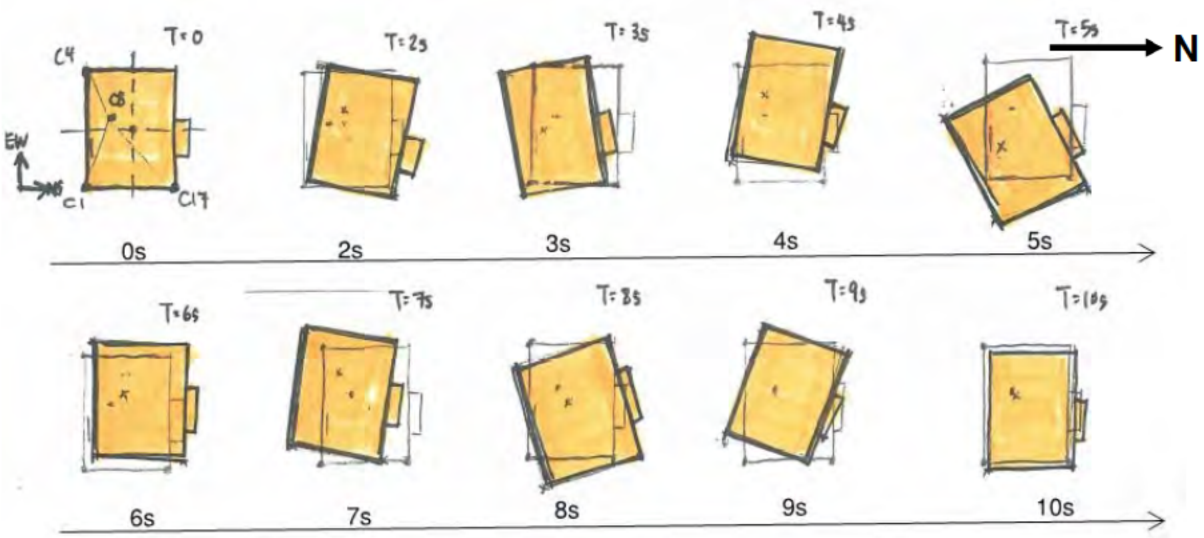


Figure H3: Building movement sequence from the NLTHA using the CCCC record for the 22 February 2011 earthquake. Note the deformations are shown to an exaggerated scale.

Beyond 5 sec (Figure H3), the building swayed towards north-west and then south-east for a few cycles, with predominant movement in the east-west direction.

Significant twisting about the NWC is evident, showing that the torsional eccentricity of the CTV building resulted in torsional response and an amplification of north-south response from east-west shaking. This is also evident from the building movement trends shown in Figure H4.

Table H1 presents the drift response for Column C8 for both our compliant and as-designed models. Column C8 is considered to be a critical column due to the high gravity load taken by it. The horizontal inter-storey drift demands for Column C8 observed for the as-designed model exceeded 2% to 2.5% in the east-west direction and 1.5% in the north-south direction.

Figure H4 shows the time-history of the Column C8 inter-storey drift response. Although very similar over the height of the building, the drift response is typically higher in the lower floors in the east-west direction and higher in the upper floors in the north-south direction.

Table H1: Inter-storey drift response of Column C8 with CCCC 22 February 2011 records without vertical acceleration

	X - north south direction					Y - east west direction				
Storey	**L1-L2**	**L2-L3**	**L3-L4**	**L4-L5**	**L5-L6**	**L1-L2**	**L2-L3**	**L3-L4**	**L4-L5**	**L5-L6**
As-designed model	1.38	1.49	1.55	1.57	1.56	2.58	2.43	2.37	2.33	2.29
Compliant model	1.14	1.16	1.20	1.22	1.21	1.87	1.94	1.94	1.89	1.79
% Difference	**-17.3**	**-22.2**	**-22.5**	**-22.7**	**-22.4**	**-27.6**	**-20.0**	**-18.2**	**-19.0**	**-21.6**

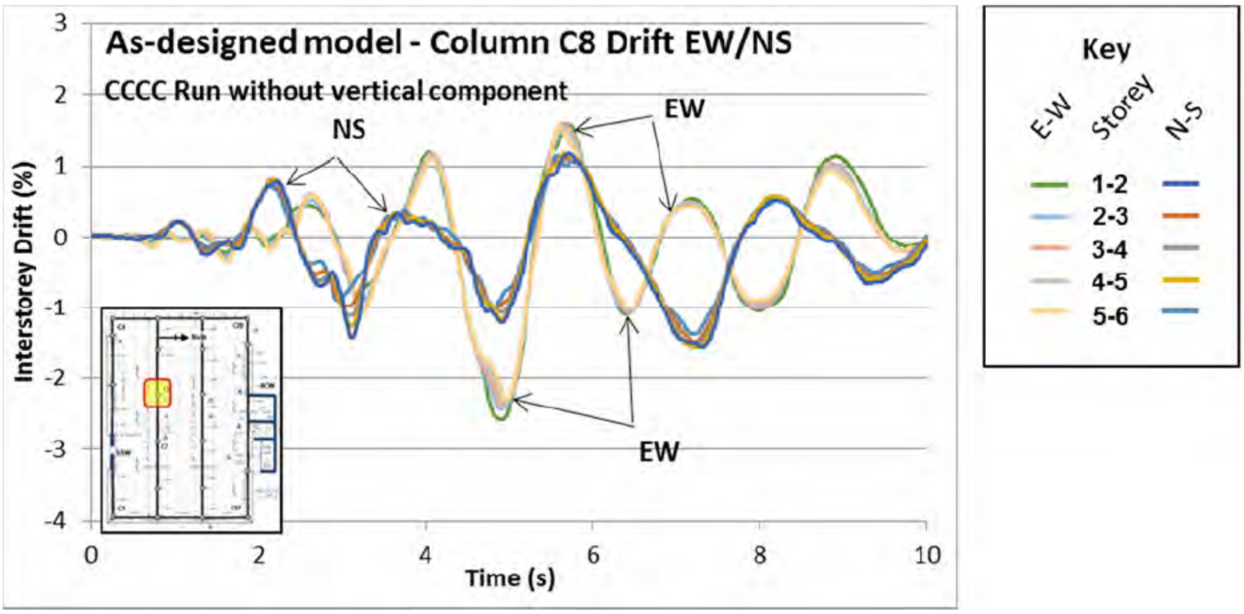


Figure H4: Inter-storey drift response of Column C8 for As-designed model for CCCC 22 February 2011 record without vertical acceleration component. Positive drift/displacement is movement to the west and north respectively.

Figures H5, H6 and H7 show the time-history plots of the inter-storey drift response at Column C17 (north-east corner), column C1 (south-east corner) and column C4 (south-west corner) respectively.

These plots indicate that the inter-storey drifts were generally reasonably consistent over the height of the building and were a maximum on the south perimeter.

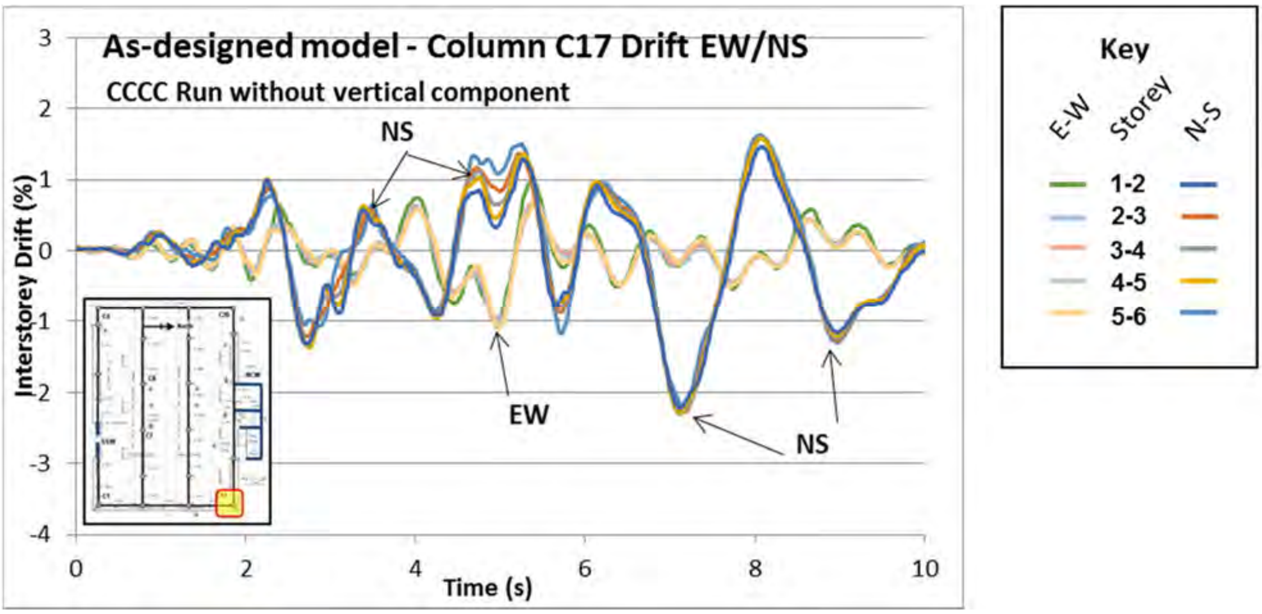


Figure H5: Inter-storey drift response of Column C17 for As-designed model for CCCC 22 February 2011 record without vertical acceleration component. Positive drift/displacement is movement to the west and north respectively.

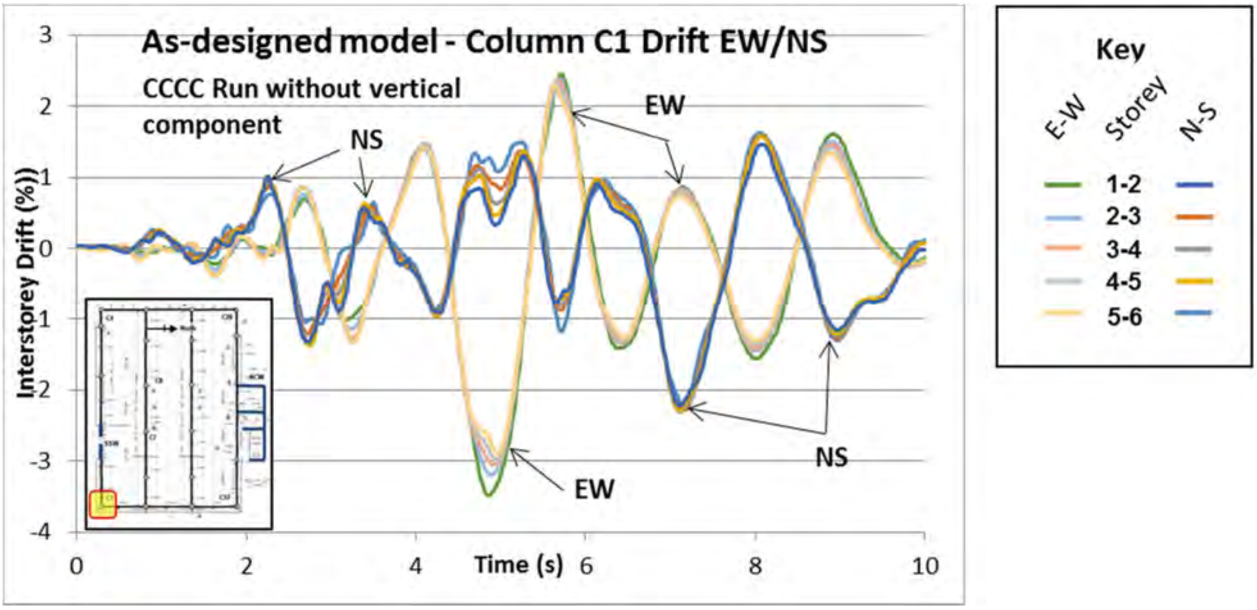


Figure H6: Inter-storey drift response of Column C1 for As-designed model for CCCC 22 February 2011 record without vertical acceleration component. Positive drift/displacement is movement to the west and north respectively.

An interpretation of these analysis results indicated that the primary gravity structure, being the internal and external frames and columns and the floor slabs, had to sustain significant horizontal deformations in both the north-south and east-west directions.

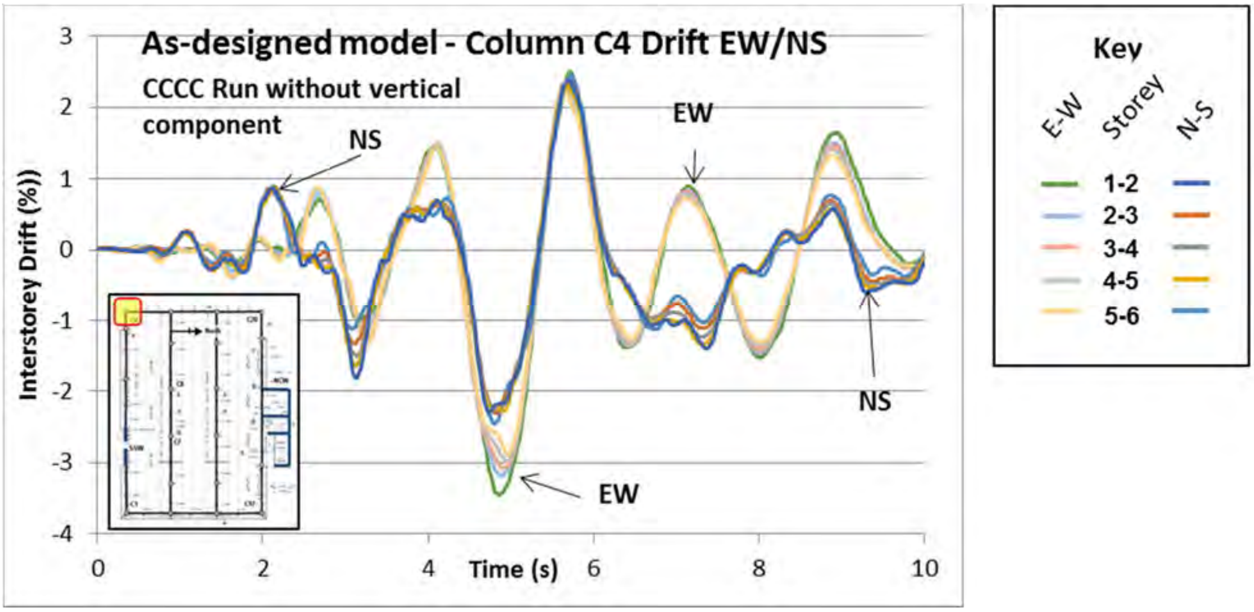


Figure H7: Inter-storey drift response of Column C4 for As-designed model for CCCC 22 February 2011 record without vertical acceleration component. Positive drift/displacement is movement to the west and north respectively.

H.3.3 Compliant model response for CCCC record

The general response and trend of the results using the compliant model are similar to those for the as-designed model as the overall response is still predominantly governed by the response of the walls.

The stronger NWC and SSW for this model mean the lateral displacement responses were lower than for the as-designed model. The horizontal inter-storey drift demands observed in the compliant model are approximately 1.9-2.0% in the east-west direction and 1.2% in the north-south direction.

As shown in Figure H8, the significant swing towards the East around 4.5s was lower than for the as-designed model by around 20-28% at Column C8. This however increases the reversing response towards the west at approximately 5.5s. The overall north-south and east-west responses were more regular over the positive and negative cycles, suggesting less one-sided twisting movement than for the as-designed model. These changes in response are significant when considering the demands on the gravity columns.

The reduction in the inter-storey drift demands between the as-designed and compliant model means the columns and beam-columns within the gravity frames sustained a lower demand than for the as-designed model. Furthermore, as the beam-column joints are expected to remain elastic in the compliant model, the susceptibility of collapse due to the axial-load carrying capacity degradation of the beam-column joints is removed. This is a significant difference from the as-designed model.

The ductility demands for the SSW are higher for the compliant model, as the stronger SSW yields first and the east/west spike in displacement demand observed in the results is well in excess of the yield displacement of the stiffened wall. This does not have significant consequences as the ductile demand on the SSW of the compliant model is well within its capacity.

[redacted]

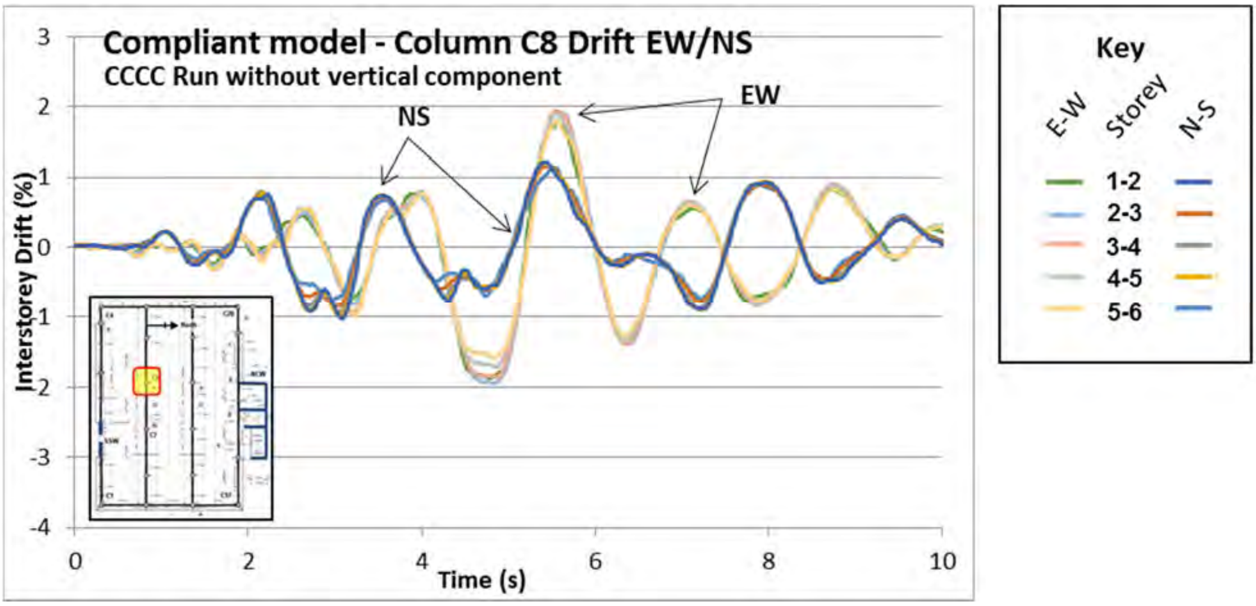


Figure H8: Inter-storey drift response of Column C8 for Compliant model for CCCC 22 February 2011 record without vertical acceleration component. Positive drift/displacement is movement towards the west and north respectively.

H.3.4 Influence of Vertical Acceleration

The vertical component of earthquake ground motion is associated with the arrival of vertically propagating P-waves, while the horizontal component is associated with S-waves. As the wavelength of P-waves is shorter than that of S-waves, the vertical component of ground motion has much higher frequency content than the horizontal component. Generally the P-wave and the vertical peak ground accelerations occur earlier than the S-wave and horizontal peak ground accelerations for sites greater than 5km from the source (Collier and Elnashai, 2001). This was observed for the CCCC record for the 22 February 2011 earthquake as shown in Figure H9, where the peak horizontal accelerations in the CCCC record occurred 1 to 2 seconds after the peak vertical acceleration.

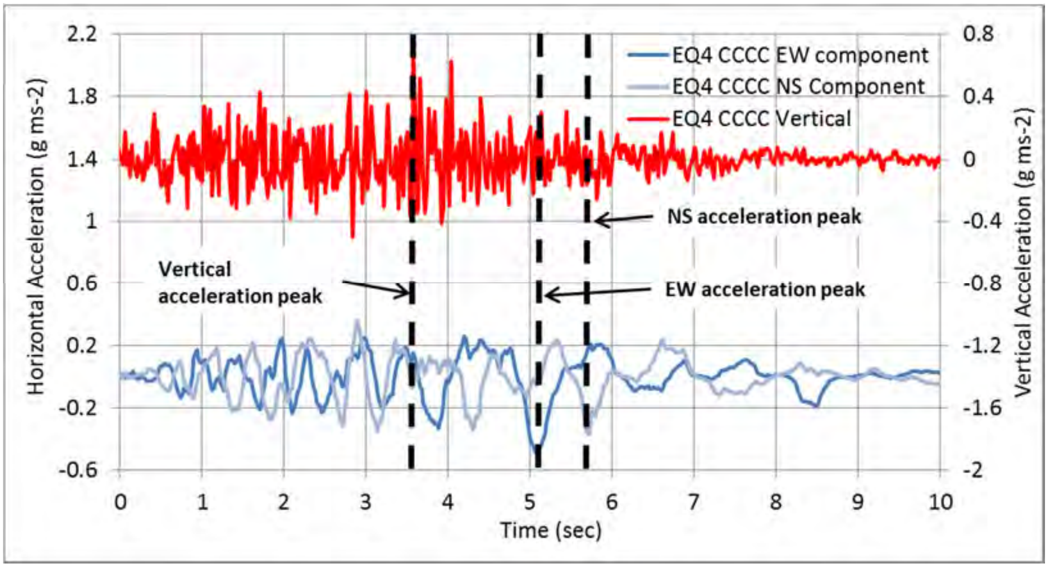
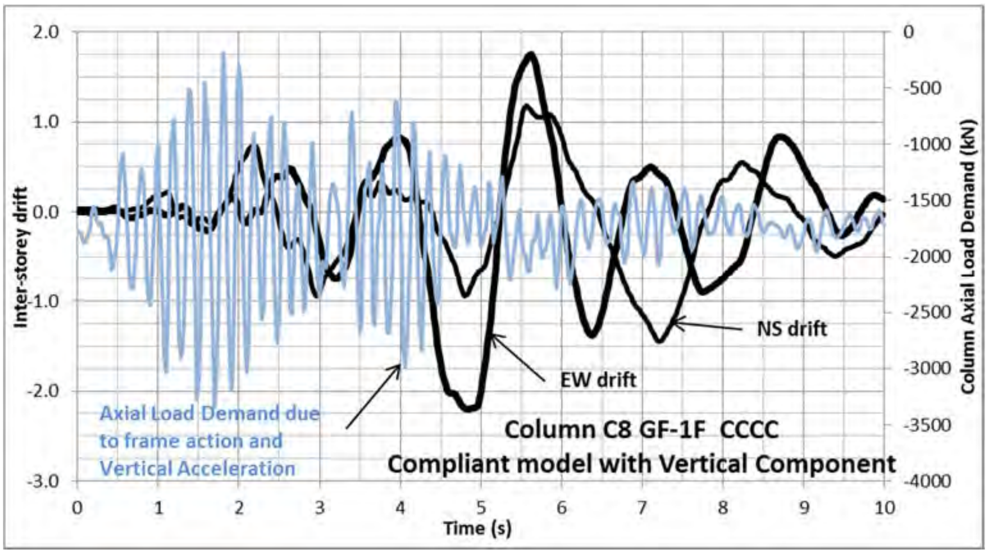
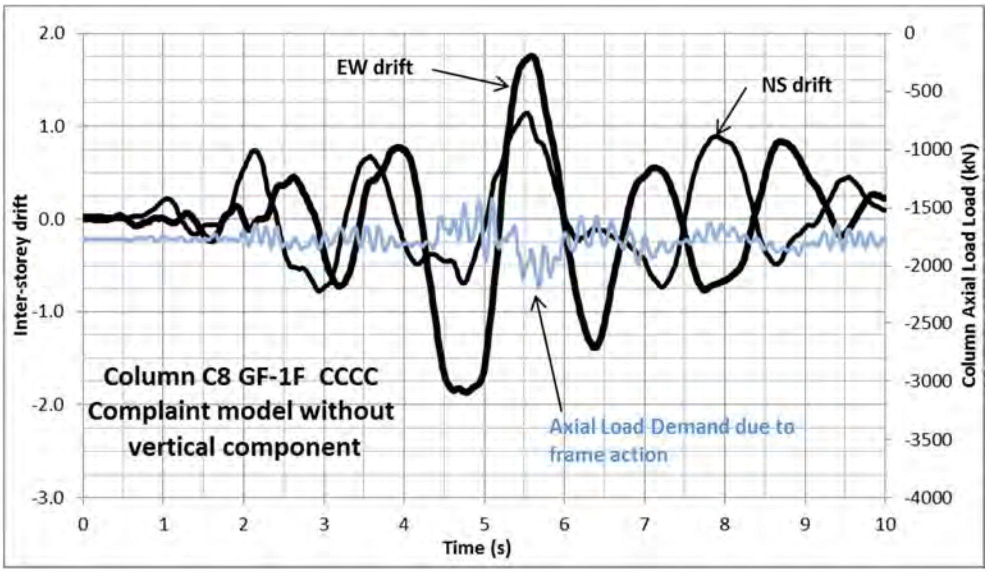


Figure H9: 22 February 2011 CCCC Strong Ground Motion Records – Acceleration Time History

Axial load and the lateral inter-storey drifts for Column C8 for the compliant model are shown in Figure H10 for the CCCC record, with and without vertical acceleration. The difference in the column axial load response between the two plots is indicative of the axial load demand arising from the vertical acceleration component. It can be observed that the high frequency vertical acceleration demand generally occurred early in the record (<4s) and had significantly reduced by the time the building was subjected to the main horizontal pulses which resulted in the large swing to the east at approximately 4.5s.



a) With vertical acceleration



b) Without vertical acceleration input

Figure H10: Time history response of Vertical Acceleration/Axial Load Demand and inter-storey drift for Column C8 for CCCC 22 February 2011 record

The interaction between the flexural frame actions resulting from horizontal drift and the axial load capacity were investigated using post processing analyses that are reported in Section H4 below. These analyses indicated that while vertical accelerations could result in column axial load demands that were higher in the initial stages of the earthquake, the increase in axial load due to vertical accelerations was not of sufficient magnitude in conjunction with the simultaneous flexural demands to have been a significant contributor to the CTV building collapse.

H.3.5 Influence of Foundation-Soil Flexibility

The soil-foundation stiffness was updated based on further geotechnical interpretation of the existing geotechnical data for both the compliant and as-designed models. The soil stiffness used for these models was generally lower than for the earlier Compusoft CERC/DBH models.

The overall effect of the foundation-soil flexibility on the displacement demands on the internal frames is expected to be the combined effect of the reduction in lateral inertial forces (due to an increase in the fundamental building period) resulting in lower lateral displacements, offset by an increase in lateral displacements due to increased rotation at the base of the walls due to the more flexible foundation soil.

By comparing the overall inter-storey drift response of the as-designed model and the earlier CERC/DBH analysis, it can be concluded that the foundation-soil flexibility is unlikely to have significantly changed the overall response of the CTV building. It is apparent that although the inter-storey drift responses vary between the different analyses, the overall trends and relative contributions of the NWC, SSW and internal frames to the lateral load resistance are similar.

H.3.6 Influence of Drag Bars Retrofit

H.3.6.1 Changes made for the sub-model

To determine the extent to which the drag bars at the NWC could affect the overall behaviour of the building, one additional sub model (compliant model B") was created and analysed. This analysis specifically focused on how the failure of the drag bars would influence the inter-storey drift responses and therefore the demands on the gravity frames on an otherwise compliant model. It also provided an indication of how likely it would be that the drag bars exceeded their tensile capacity.

The following changes were made to the compliant model for the compliant model B:

- The drag bars and the whole connection between the floor slabs and the NWC were modelled in the same way they were modelled for the as-designed model.
- The tensile strength for drag bars matched Clark Hyland's data in Table 8 of Compusoft rev0 31 Aug 2012 NLTHA report. The drag bars were modelled to disconnect for horizontal support if their axial capacity was exceeded, but vertical support was maintained along that line of support.

H.3.6.2 Results

It can be observed from Figure H11 that the analysis results for the model with compliant drag bars (black line plot in Figure H11) has a higher inter-storey drift demand in the east-west direction at the critical column C8 than for the compliant B model.. The response in the north-south direction is also affected by the presence of the compliant drag bars.

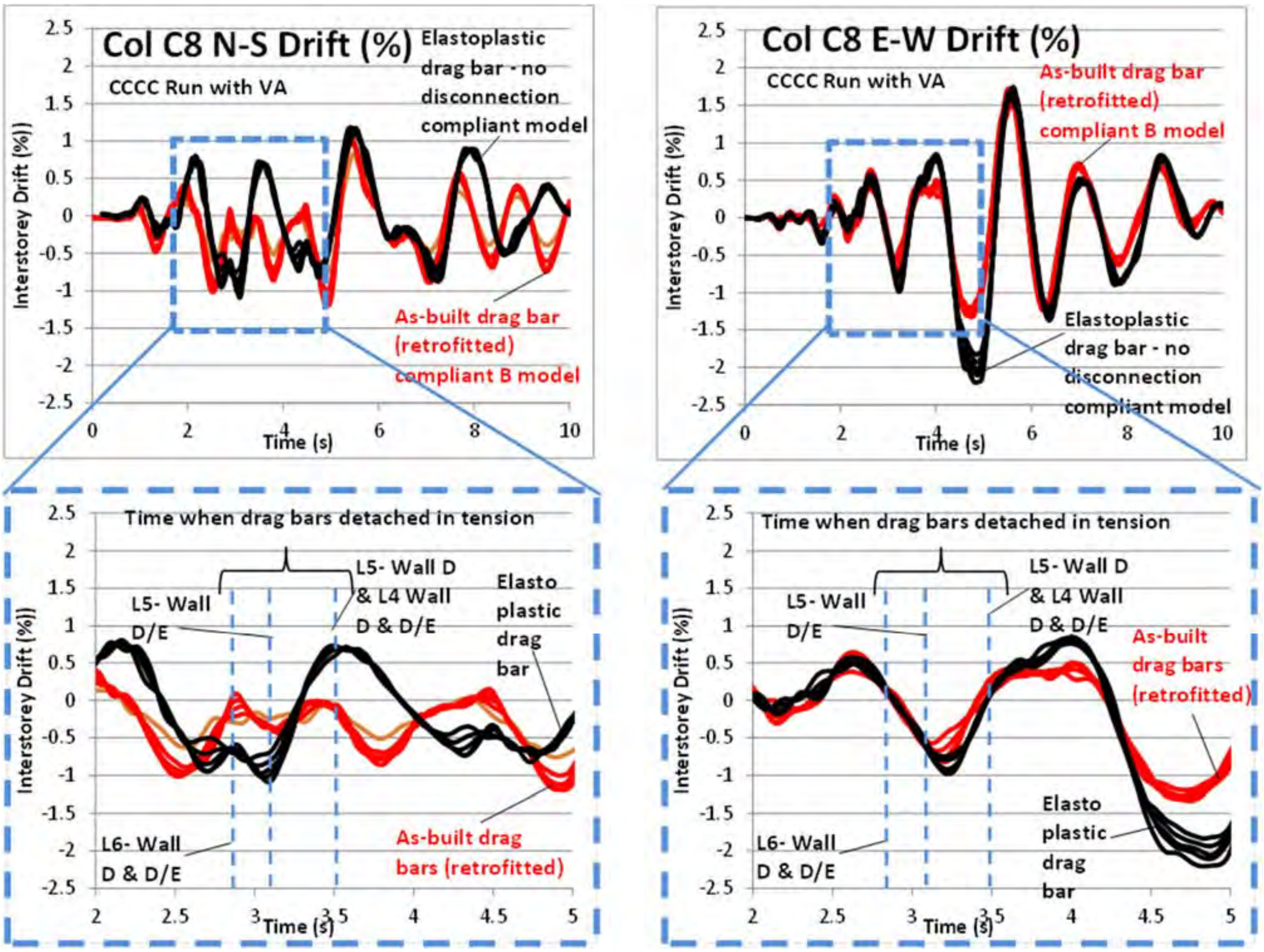
This result may be counter-intuitive as one may expect the presence of infinitely strong drag bars would reduce the inter-storey drift response and improve the overall building performance.

Our explanation for the observed results is as follows:

- For the compliant model, the torsion force couple within the NWC is mobilised between Grid C and D/E and the NWC acts effectively as a core wall with walls on Grid Lines C and D/E acting as "flanges" to the NWC. This increases the east-west translational stiffness and torsional stiffness of the NWC. The higher east-west translational stiffness in turn accentuates the plan stiffness eccentricity, shifting the centre of rigidity towards the north. This increased plan eccentricity induces a more torsional response and amplifies the displacement demand along the southern elevations (Grid Lines 1 and 2) under an east-west direction shaking.

- Conversely in the compliant model B, the drag bars are effectively detached circa 3 to 3.5s into the analysis. The loss of the force-couple and axial connection to the slab reduces the effective lever arm of the NWC in the east-west direction. The torsion force couple is, therefore, limited to Grid Lines C and C/D. This reduces the east-west translational stiffness and torsional stiffness of the NWC. This significantly reduces the east-west translational stiffness which in turn reduces the plan irregularity and displacement demand along the southern elevations (Grid Lines 1 and 2) under the east-west direction shaking. The reduced torsional stiffness has less influence than the reduced plan eccentricity.

Notwithstanding these differences, the results indicate that for both the compliant and compliant B models the interstorey drifts are below those that would result in an axial failure in the columns. We therefore conclude that the deficiencies identified in the retrofitted drag bars as installed, on their own (i.e. in an otherwise compliant building), were not a substantial cause of the collapse.



a) N-S response

b) E-W response

Figure H11: Influence of drag bar retrofit - Inter-storey drift response of Column C8 for CCCC 22 February 2011 record without vertical acceleration. Two models used are compliant model with infinitely strong drag bars (black lines) and compliant B model with as-built retrofitted drag bars that can detach in tension.

[redacted]

H.4 Assessment of Potential for Collapse

Building collapses in earthquakes can generally be categorised into two different modes (NIST, 2013):

- **Lateral dynamic instability collapse**: The building collapses sideways due to dynamic instability resulting from excessive lateral deformation and lateral strength degradation during the earthquake shaking. Collapse occurs when the structure does not have sufficient lateral capacity to provide restoring actions in cyclic earthquake shaking. This failure mode is generally simulated in NLTHAs.
- **Gravity/axial load collapse**: The primary gravity system fails, causing the building to collapse without excessive lateral sway. Once axial failure occurs in one or more components, vertical loads due to gravity and seismic inertia try to redistribute to adjacent support components. Progressive or wholesale collapse is likely to occur if there is insufficient ability to redistribute the gravity loads. This type of failure cannot generally be simulated in non-linear structural analyses

The NLTHA can account for the potential for lateral dynamic instability with explicit modelling of the force-displacement cyclic behaviour of the primary seismic load resisting systems (reinforced concrete core walls, frames, and foundations). Collapse from lateral dynamic instability would be indicated by excessive lateral displacement (>5-10% inter-storey drift), which is not observed in the CTV NLTHAs.

Prediction of axial failure of the columns and/or joints was not attempted in our NLTHAs. Yielding of the steel in the columns was modelled but the ability of the concrete to continue to carry vertical load was not.

To investigate the potential for gravity/axial load collapse, we completed the following post processing comparisons for the NLTHA results for the primary gravity structure:

- The column bending moments against the expected flexural capacity of the columns based on the unidirectional axial load moment interaction relationship for the column. Exceeding this relationship does not in itself predict failure but indicates that the moment capacity has reached a limit for the particular applied axial load, ie rotation at the strain capacity of the column core concrete is occurring, and the column is likely to be deteriorating due to spalling of the cover concrete.
- The shear forces generated in the columns against the expected shear capacity of the columns.
- The joint rotations in joints with low levels of confinement reinforcement against criteria that were considered indicative of diagonal joint cracking, joint degradation and severe joint damage.
- The column inter-storey drifts against failure (collapse) criteria developed by Elwood et al (2005) and also with similar criteria from ASCE 41 (2006). These criteria are a function of the axial load in the columns which varies during the analyses.

The potentially critical columns were identified from an assessment of all of the results to be Columns C8 and C7. An assessment of the column and joint results for Column C8 is presented in the following sub-sections. The results for Column C7 were similar.

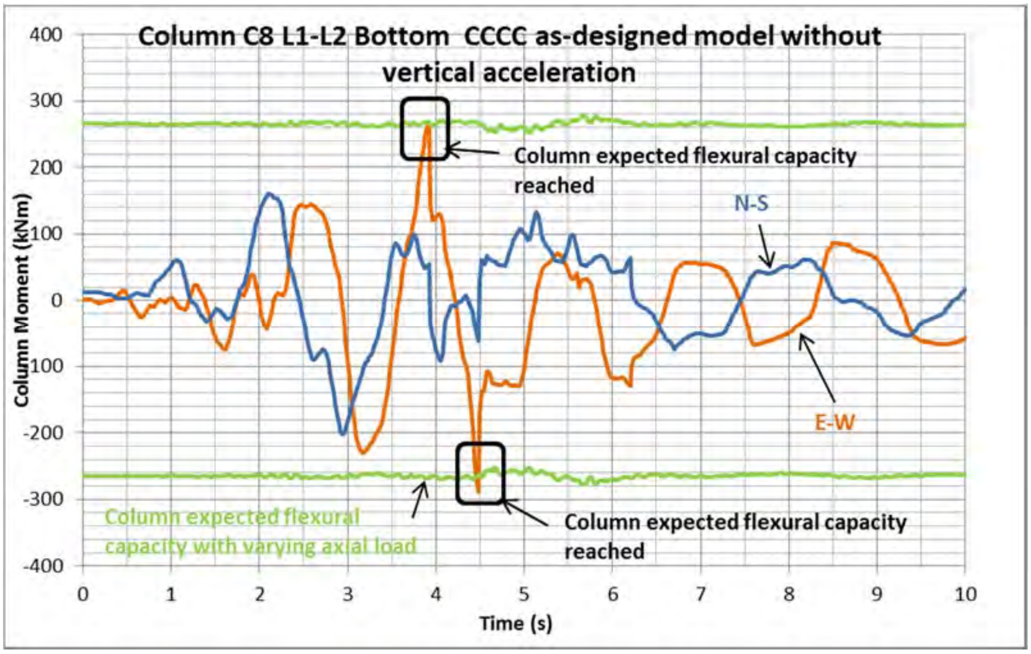
H.4.1 Column flexural capacity versus flexural demands

The time history plots of the column moment responses for the top (at the soffit of the L1 beam) and bottom (immediately above the floor slab) of the ground floor C8 column are shown in Figures H12 and H13 for the as-designed and the compliant models respectively. Also shown on these figures are the flexural (moment) capacities of the columns determined for the applied axial load at any point of time.

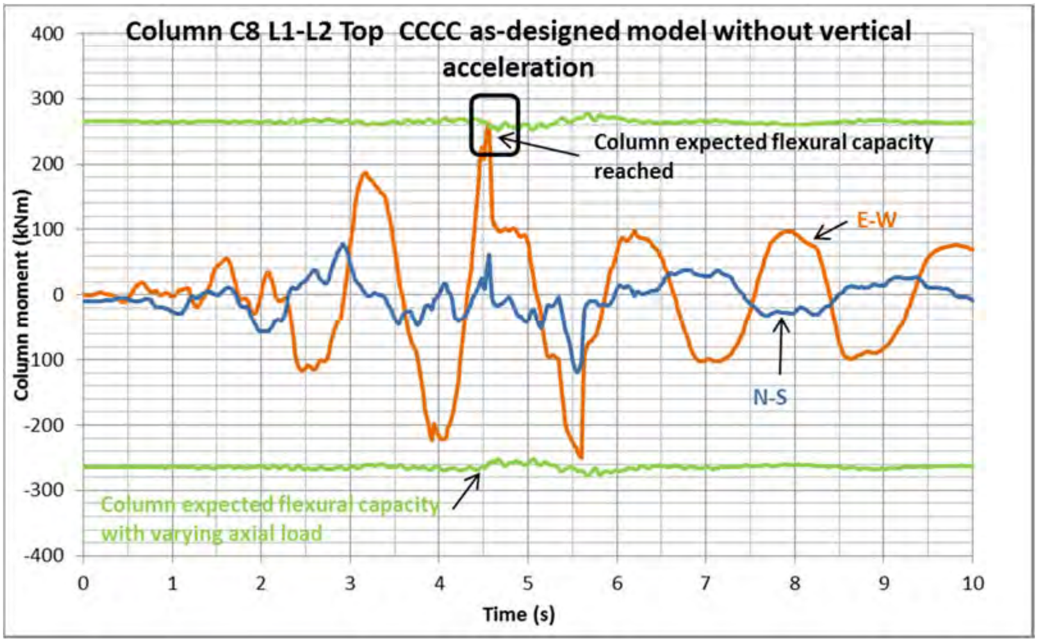
For the as-designed model, the moment at the base of this column exceeded its flexural capacity (with predicted concrete edge crushing and associated spalling of cover concrete) circa 3.9s (sway to the west) and at 4.4s (sway towards the east). The ground floor column exceeded its flexural capacity at the top circa 4.6s (sway towards the west) and at 5.6s (sway towards the east). This indicates that the column edge fibres would have exceeded the unconfined concrete crushing strain and concrete spalling would be expected circa 4.5s to 6s. As the column had high axial load, the flexural mechanism would have been compression controlled where concrete crushing and spalling occurs at or before full yielding of the column longitudinal reinforcement. Therefore the column end regions experiencing flexural yielding would have limited ductile capability and would be expected to rapidly lose lateral strength and stiffness once the flexural capacity is reached.

Similar behaviour can be observed from the results using the compliant model although the potential for exceeding the flexural capacity of the column is lower.

As expected the moment capacity does not vary significantly with time as the axial load for these columns does not vary significantly in the absence of vertical accelerations.

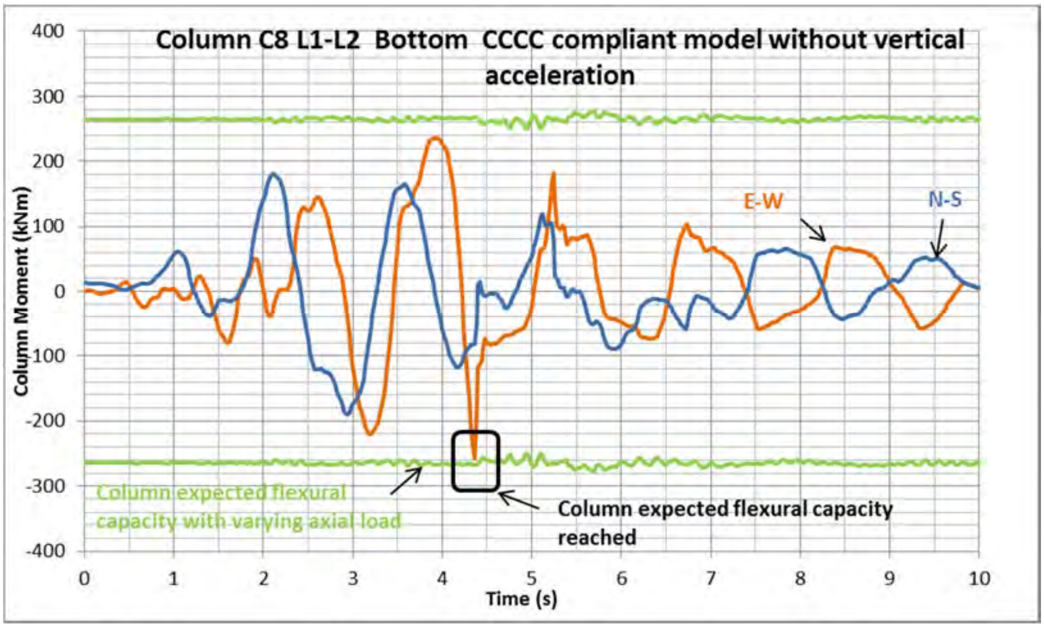


a) Bottom of Ground Floor

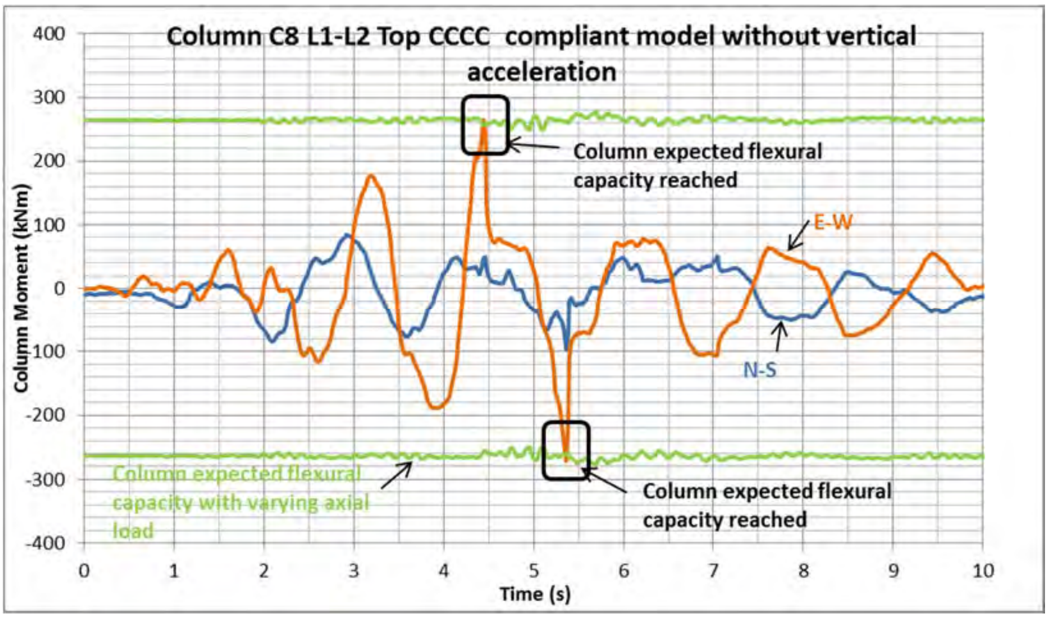


b) Top of Ground Floor

Figure H12: Column C8 end moment assessment for As-designed Model under CCCC 22 Feb 2011 shaking without vertical acceleration component



a) Bottom of Ground Floor



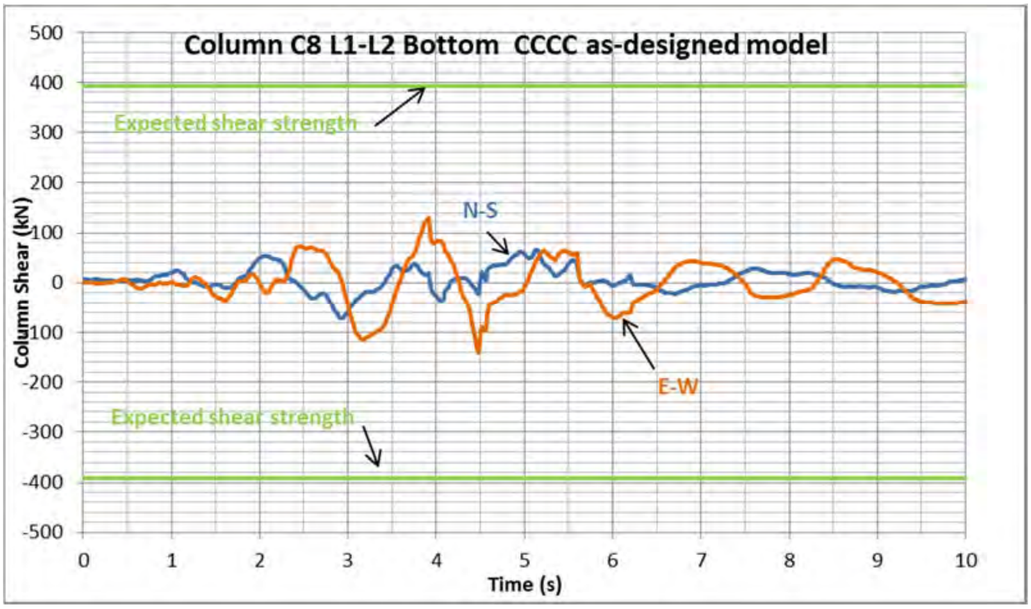
b) Top of Ground Floor

Figure H13: Column C8 end moment assessment for Compliant Model under CCCC 22 Feb 2011 shaking without vertical acceleration component

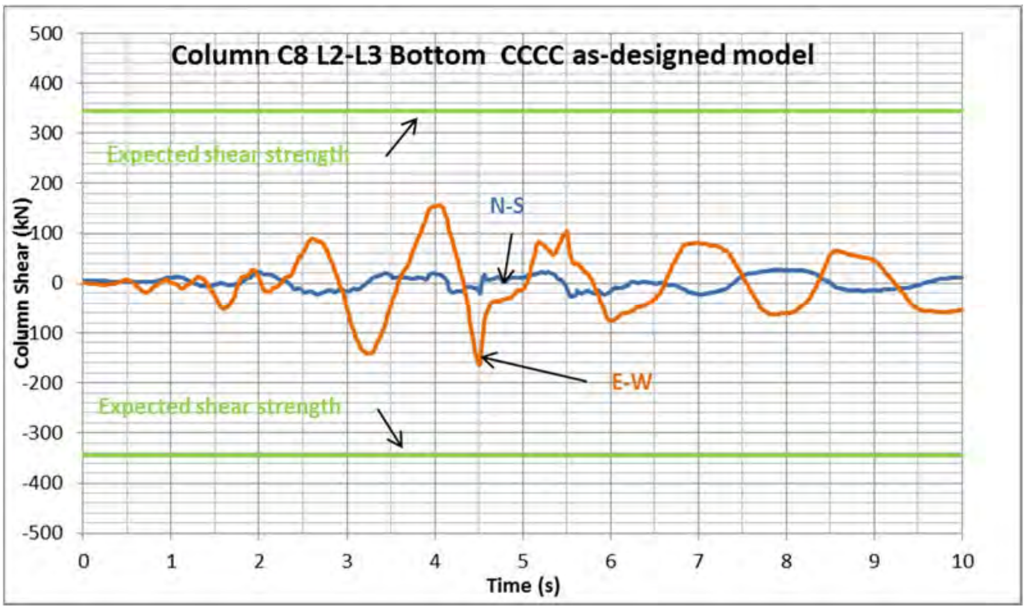
[redacted]

H.4.2 Column shear strength versus shear demands

In general, the internal columns are expected to have flexurally-dominated behaviour with flexural hinging of the column ends expected to occur prior to shear cracking at the column mid-height. This is confirmed by the NLTHA results where flexural yielding at the top and/or bottom of the column generally limited the shear forces generated in the columns. Figure H14 shows the variation with time of the shear force actions in Column C8 under CCCC 22 February 2011 shaking without the vertical acceleration component. It is apparent that these shear forces are well within the shear strength of this column.



a) Bottom of Ground Floor



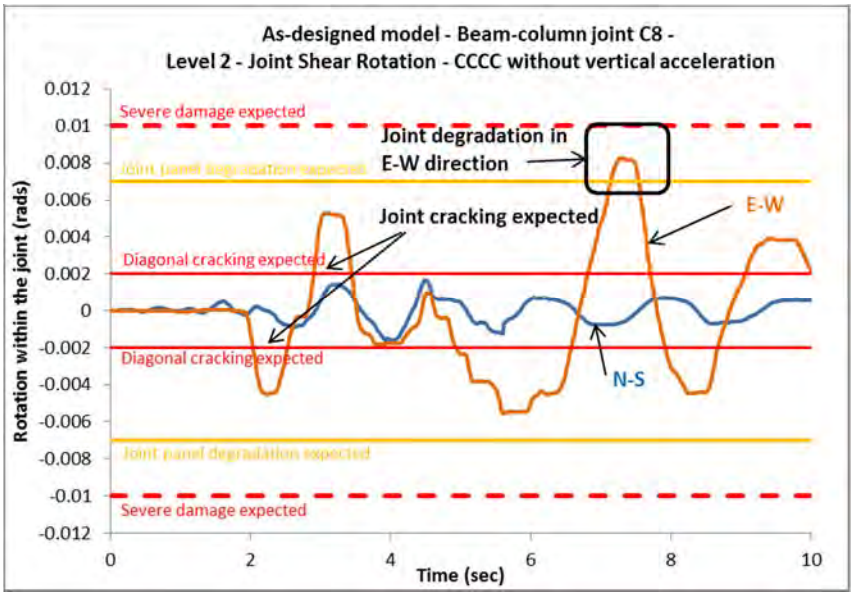
b) Bottom of 1st Floor to 2nd Floor

Figure H14: Column C8 end shear assessment for As-designed Model under CCCC 22 Feb 2011 shaking without vertical acceleration component

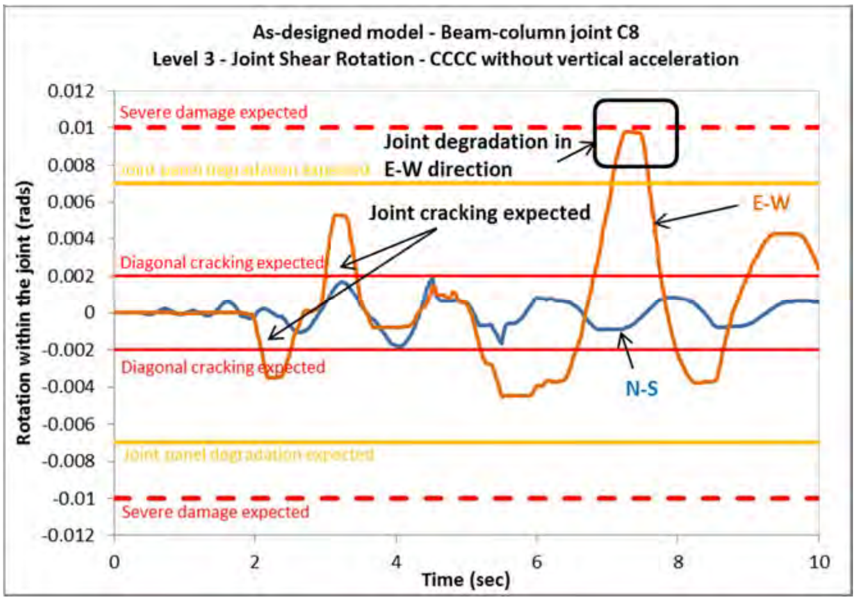
H.4.3 Interior joint shear capacity versus joint shear demands

Previous analyses by Compusoft for CERC and DBH using the CCCC 22 February 2011 record indicated that a number of beam-column joints would respond in a non-linear manner. Refer to Section 11.4 and Figures 75-83 of the Compusoft NLTHA Report rev July 2012. Appendix C of the Compusoft report also provides a summary of the post-processing assessment method used for investigation of the beam-column joints.

Figure H15 presents an assessment of the damage expected in the beam-column joints at Column C8 for the as-designed model. The joint shear rotation response from our NLTHA is compared against the various available empirical joint shear deformation performance levels.



a) Beam-column joint C8 Level 1



b) Beam-column joint C8 Level 2

Figure H15: Joint shear rotation capacity assessment for As-designed Model under CCCC 22 February 2011 shaking without vertical acceleration component

It can be observed from Figure H15 that the interior joints are predicted to sustain in-plane joint shear cracking at early stages of the shaking (circa 2s to 3s). In subsequent shaking the shear strain within the joint is generally below that considered to represent ‘spalling’ (~ 0.008 radians). Significant damage to the joint panel zones is predicted circa 7s indicative of the beam-column joints losing both their shear and axial capacities as the joint panel regions spall and crush under the combined shear and axial compression demand.

The compliant model’s joints were modelled to remain elastic in line with our calculations and therefore minimal rotation within the joint is predicted or expected for these runs. This assumption was confirmed to be valid in the physical testing of the compliant specimen.

H.4.4 Investigation of loss of column and beam-column joint axial gravity support

The time history plots of inter-storey drifts for Column C8 for the as-designed and compliant models and the CCCC record are shown in Figures H16 and H17 respectively. Also shown on these figures are the drifts at which column axial load capacity is predicted to be exceeded based on the work of Elwood and Moehle (2005) and also the criteria presented in ASCE 41-06.

Figure H18 shows the inter-storey drift time history plots for the as-designed model for Column C8 together with the drift at which the beam-column joint axial load capacity is expected to have been exceeded based on the work of Hassan and Moehle (2013).

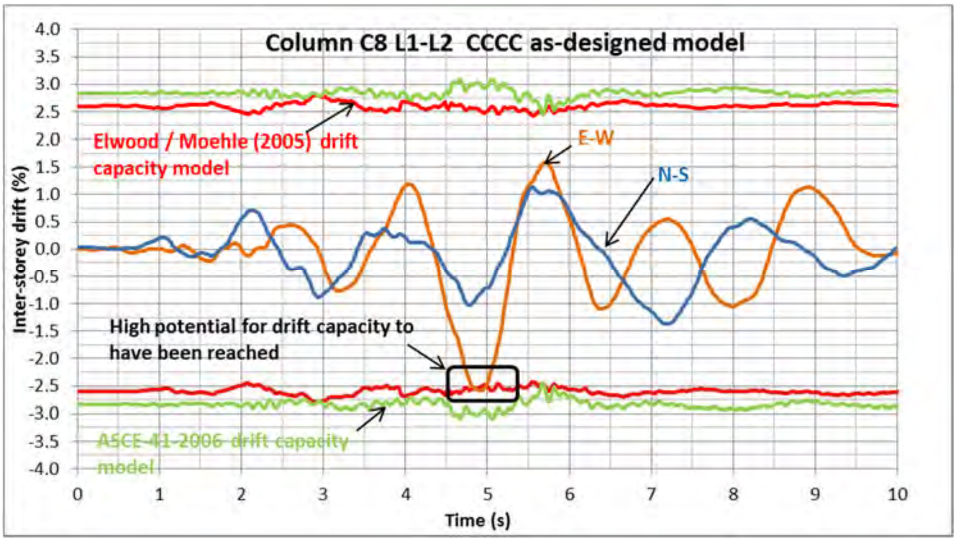
Figures H16 and H17 suggest that Column C8 for the as-designed model would have exceeded its drift capacity during the earthquake (circa 4.9s) but that this is unlikely for the compliant model.

Similarly Figure H18 indicates that the beam-column joint at location C8 on L2 for the as-designed model was susceptible to failure at approximately the same time (circa 4.9s) as for the column.

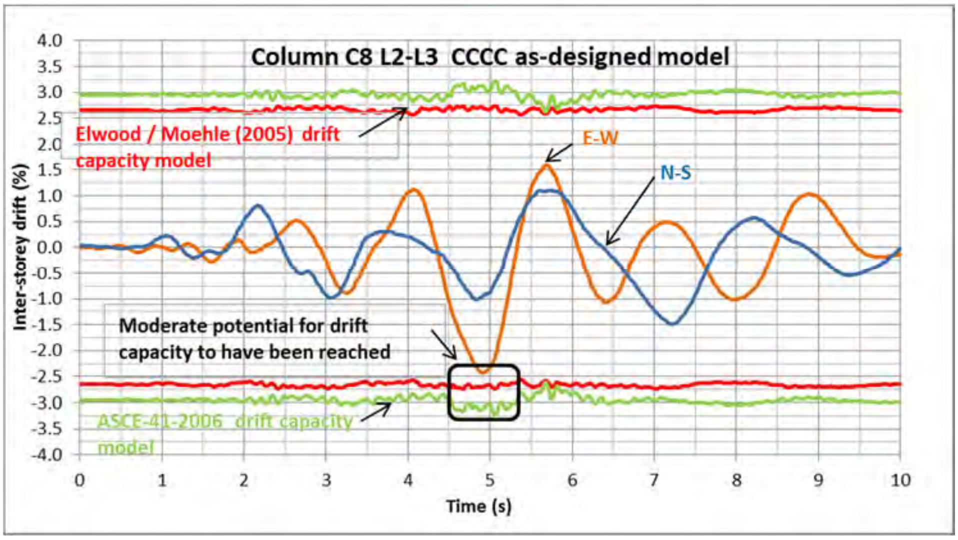
The joint axial load capacity assessment is very sensitive to the adequacy of the longitudinal beam bars to contribute to the shear-friction component of the residual axial load capacity, and this depends on the anchorage of the beam bars which were recognised to be problematic in the CTV building. The drift capacities are plotted for 50% and 100% beam longitudinal bars contribution in Figure H18, as a sensitivity check.

We note that the assessment procedure for column and joint axial load failure as used here is semi-empirical. The dataset is very limited and contains test specimens with configurations and detailing that are very different from the CTV building. Therefore, while considered the best currently available, this assessment of collapse potential should be taken as being indicative only. The physical testing completed as part of this investigation (reported on in Appendix K) was set up in part to provide further physical evidence on these matters from test specimens directly comparable to the CTV building.

Joints in the compliant model with code required levels of confinement and shear capacity are expected to remain essentially elastic (and have been modelled as such) and therefore they are not expected to be susceptible to axial load failure.

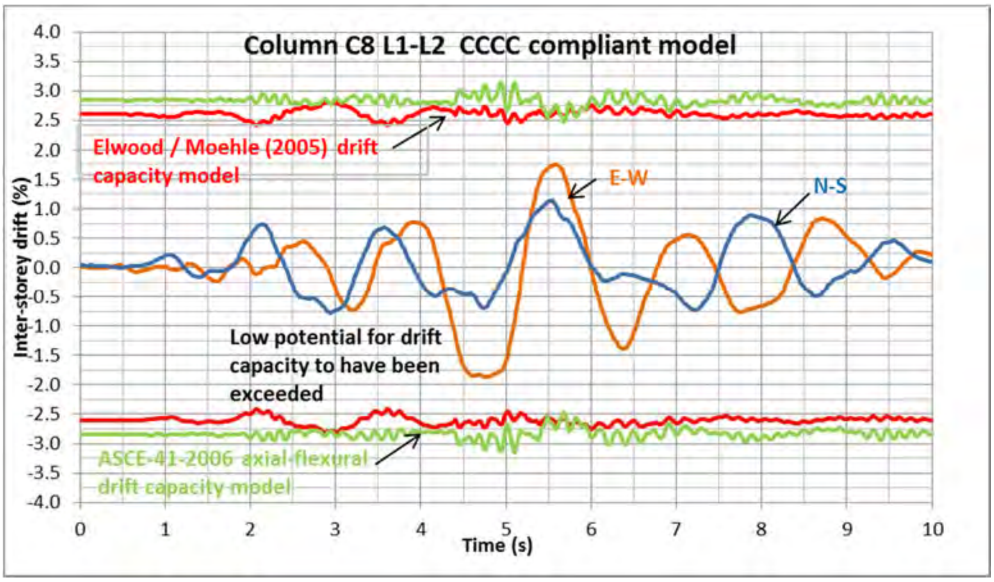


a)

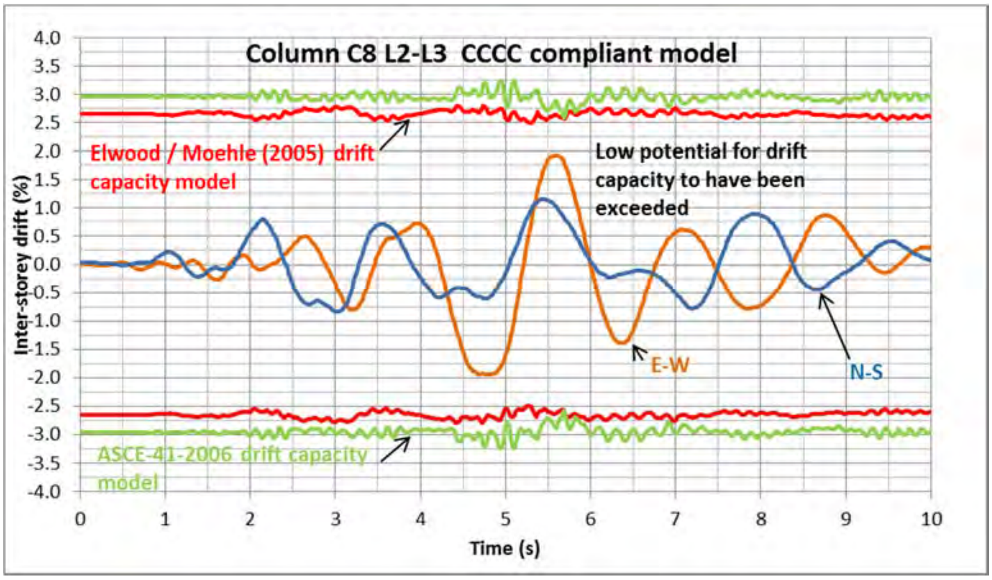


b)

Figure H16: Column C8 axial drift capacity assessment for As-designed Model under CCCC 22 February 2011 shaking without vertical accelerations



a)



b)

Figure H17: Column C8 axial drift capacity assessment for Compliant Model under CCCC 22 February 2011 shaking without vertical accelerations

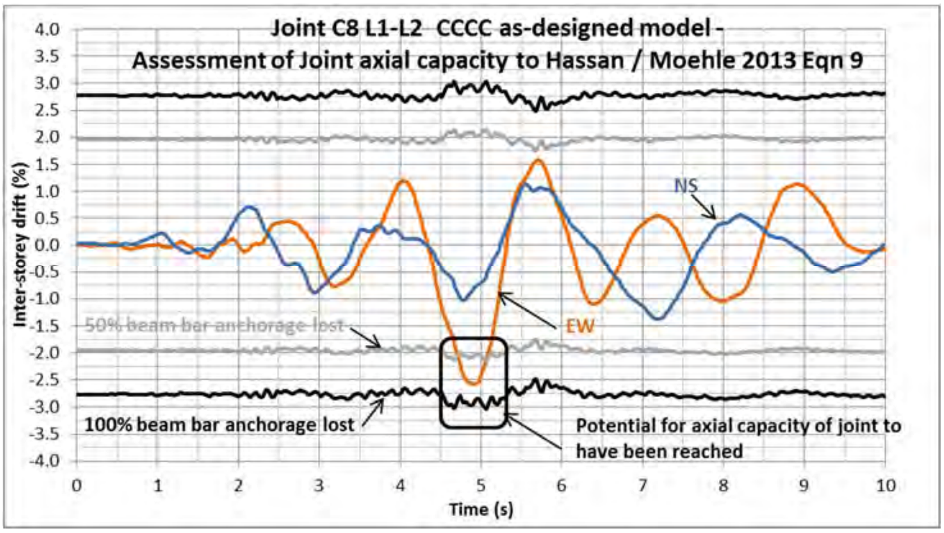


Figure H18: Beam-column joint at C8 on L1 axial drift capacity assessment for As-designed Model under CCCC 22 February 2011 shaking without vertical accelerations

H.5 Conclusions from NLTHAs

H.5.1 As-designed model response

The overall response of the building as predicted by the NLTHA results are consistent with eye-witness accounts and the post-event forensic observations.

Our interpretation of the NLTHA results for the as-designed model suggests that Columns C8 and/or C7 and the beam-column joints at the interior location on Grid Line C8 and/or C7, at lower levels in the building cracked very early in the earthquake (2 to 3 seconds into the analysis). The columns cracked flexurally while the joints experienced diagonal shear cracking.

The analyses indicate that at approximately 4.5 to 5.5 seconds, Columns C8 and/or C7 had reached their flexural capacity (at the column ends) and the C8 and/or C7 beam-column joints at L2 had sustained significant cracking. The inter-storey drifts in the ground floor interior columns at this time (2.1% to 2.6% depending on the analysis) were sufficient to lead to axial failure in these columns and/or beam column joints.

Even if the beam-column joint and column ends survived the significant east-west shake at 4.5 to 5 seconds, the subsequent strong sway cycles in the north-west to south-east direction (circa 7 sec) were sufficient to lead to joint panel degradation and axial failure of the joint.

The loss of axial capacity of either Column C8 or C7, or the interior beam column joints, at the lower levels would lead to progressive and pancaking collapse as observed on 22 February 2011.

Our review of the NLTHA results indicated that other columns and/or beam-column joints were not as susceptible to failure as those discussed above. Notwithstanding that other columns and joints were subjected to higher inter-storey drifts, the axial loads were not sufficient to indicate a high probability of axial load failure.

Column shears obtained from the NLTHAs were found to be within the calculated expected capacities.

H.5.2 Compliant model response

The principle difference in the response of the compliant and as-designed models was the level of inter-storey drifts, particularly at the interior columns on Grid 2 (ie C7 and C8 locations). These drifts were in the order of 1.9% to 2% or 17 to 28% less than for the as-designed model. The lower drifts arise from a number of factors including:

- Stronger SSW (for east west response)
- Stronger NWC (for east-west and north-south response)
- Stiffer and stronger beam-column joints.

As a result of the lower drifts, axial column failure was not indicated even though the interior columns reached their flexural capacity.

Our interpretation of the NLTHA results from the compliant model is that the compliant building would have survived the 22 February 2011 shaking.

H.5.3 Testing of the hypothesis that design omissions were a substantial cause of pancaking collapse

The NLTHAs predicted that a compliant building would not have reached a state of pancaking collapse during the 22 February 2011 earthquake, whereas the as-designed building would have. Therefore, in regard to the hypothesis that we sought to test by completing the NLTHAs, we have concluded that the design omissions were a substantial cause of the pancaking collapse.

Two aspects were found to be critical in the prediction of pancaking collapse for the as-designed building. They were:

- the level of inter-storey-drift (which was up to 28% higher than for the compliant building) that placed higher deformation demands on the heavily axially loaded interior columns, and
- the level of detailing in the interior beam-column joints.

The as-designed internal beam-column joints, without the compliant confinement reinforcement, were very susceptible to strength and stiffness degradation resulting in axial load failure of the connected internal columns under this level of drift.

H.5.4 Postulated collapse sequence

The following is our interpretation of the collapse sequence on 22 February 2011 based on the results from the NLTHAs:

1. The building initially underwent a couple of cycles predominantly in the north-south direction, and then towards toe north-west corner. The inter-storey drift along the perimeter elevations exceeded 0.5% to 1.0%, suggesting some of the glass panels would break and the perimeter columns would form top and bottom flexural hinges (cracking and yielding).

2. The retrofit drag bars exceeded their axial tension capacity at approximately 2-3 seconds into the earthquake. Diagonal shear cracks formed at selected interior and exterior beam-column joints. Around 3 to 4 seconds into the earthquake, some cover concrete crushing and spalling occurred at critical column end regions due to flexural hinging degradation of the lightly confined column sections.

3. The relative difference in stiffness between the NWC and the SSW resulted in rotation in plan about the NWC. The significant twisting about the NWC is evidence that the torsional eccentricity of the CTV

[redacted]

building resulted in torsional response and an amplification of north-south response from east-west shaking. This is also evident from the sequence of building movement shown in Figure H3.

4. Approximately 4.5 to 5 seconds into the earthquake analysis, the building moved significantly to the east, pivoting about the NWC (as shown by the second visual in Figure H2 and T = 5s sketch in Figure H3). This resulted in very high east-west inter-storey drift demand along the southern grid lines (Grid Lines 1 and 2), with maximum east-west inter-storey drifts in the order of 2.5 to 3.5% on Grid 1 and 2.2 to 2.6% on Grid 2. This movement was sufficient to cause significant yield at the base and in the coupling beams of the SSW. The yielding of the SSW reduced the torsional resistance of the building in the east-west direction. Axial load failure of the interior columns C7 and/or C8 between Ground Level and Level 1and beam-column joints at Level 1 occurred.

5. Beyond 5sec, the building swayed towards north-west and then south-east for a few cycles, with predominant movement in the east-west direction. Even if the beam-column joint and column ends survived the significant east-west shake at 4.5 to 5 seconds, the subsequent strong sway cycles in the north-west to south-east direction would have been sufficient to lead to axial load failure.

6. The loss of gravity carrying capacity in either column C8 or C7, or both, at the lower levels of the building would lead to the progressive and pancaking collapse as observed.

H.5.5 Other conclusions

From the various NLTHAS we completed, we have also concluded that:

- The primary seismic structure (NWC and SSW) would have survived the shaking notwithstanding the demands were greater than the required capacity in the original design. The under capacity however, resulted in larger deformations and therefore larger demands on the primary gravity frames (columns, beams). The increase in lateral drift response between the as-designed and the compliant models is in the order of 17% to 28%, insufficient to significantly change the mode of failure.
- The effect of the retrofitted drag bar connections to the NWC alone was found to be insufficient to have been a significant contributor to the CTV building collapse.
- Vertical accelerations were not a significant contributor to the collapse.

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H.6 References

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Hassan, W.M., Moehle J.P. (2013) Quantification of residual axial capacity of beam-column joints in existing concrete buildings under seismic load reversals. Proc. Of the 4th ECCOMAS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering (COMPDYN 2013), Kos Island, Greece 12-14 June 2013.

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NIST (2013) “Review of Past Performance and Further Development of Modelling Techniques for Collapse Assessment of Existing Reinforced Concrete Buildings. NIST-Report NIST GCR 14-917-28”. National Institute of Standards and Technology (NIST), Gaithersburg, Maryland. 201 pages.

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Appendix I

Earthquake Characteristics

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Appendix I Earthquake Characteristics

I.1 Introduction

This appendix summarises the characteristics of the three significant earthquakes that are relevant to the investigation of the CTV building collapse: the Darfield earthquake (4 September 2010), the Boxing Day earthquake (26 December 2010) and the Christchurch earthquake (22 February 2011). The material presented has been obtained from various sources including the CERC report and Beca processing of the strong motion records available from GNS.

Prior to the Darfield earthquake, strong-motion ground recorders had been installed by GNS at various locations around Christchurch city. These instruments record the ground motion at the site where they are located. Four of the closest recorders to the CTV building were Christchurch Resthaven (REHS), Christchurch Botanic Gardens (CBGS), Christchurch Hospital (CHHC) and Christchurch Cathedral College (CCCC). The locations of each of these recorders, together with the location of the CTV building, are shown in Figure I1.

While the shaking experienced at the CTV building site in any of these earthquakes cannot be known with certainty, our assessment of the underlying ground conditions indicates that the ground motions recorded at the CCCC recording station are likely to be the most representative of those that are likely to have occurred. The CCCC site was also the closest recording site to the CTV building.

The records obtained from the REHS recording site are more severe than was recorded at the other stations but the geotechnical and geological conditions at this site were not representative of those at the CTV building.

The strong motion records obtained from the CCCC recorder for each earthquake are provided in Figures I2, I5 and I9.

The intensity of shaking can be represented as the peak elastic response of a single degree of freedom oscillator. The result of plotting the peak response for single degree of freedom oscillators of various flexibilities (structural periods) is a response spectrum. The peak response can be expressed in terms of the maximum spectral acceleration and spectral displacement reached. Linear elastic response spectra derived for a particular earthquake can be used to estimate the peak elastic response of the first mode of vibration of a multi degree of freedom structure, such as the CTV building, in that earthquake.

Plots of the elastic response spectra for each earthquake are provided below in the form of plots of spectral acceleration and spectral displacement. These are referred to as acceleration-displacement response spectra or ADRS. Refer Figures I3, I6 and I10 for the ADRS from the horizontal motions and Figures I4, I7 and I11 for the ADRS from the vertical motions. The spectra are plotted for an equivalent viscous damping of 5% and include the results from each of the four recording sites. The plots for each of the horizontal and vertical motions for each earthquake are shown at the same scale to allow comparison of the relative intensity of shaking.

The further out the plots are on a radial line from the bottom left hand corner of the plots of spectral acceleration against spectral displacement, the higher the intensity of shaking that was recorded. It is apparent that the relative intensity of shaking from each recording station varies for different choices of radial line. The radial lines also represent different natural (first mode) periods of vibration.

Our analyses indicate that the initial natural periods of vibration (horizontal) for the CTV building were approximately 0.8 and 0.3 seconds for the north-south and east-west directions respectively, assuming a

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fixed base for the building. These periods should be taken as indicative only of the actual periods of vibration of the building. The effective period will increase as a building sustains structural damage. Our analyses indicate that the effective period of the CTV building increased to between 1 and 2 seconds. Radial lines for these periods of vibration are shown in the spectral response plots.

I.2 Darfield Earthquake (4 September 2010)

Figure I1 shows the location of each recording stations relative to the CTV building and arrows indicating the predominant direction of ground acceleration recorded during the Darfield earthquake at each. It can be seen that the predominant direction of shaking at all four recording stations was approximately north-south.

Figure I1: Location of four strong-motion recorders relative to the CTV building and the predominant direction of ground accelerations recorded during the Darfield earthquake (Source: CERC Report 2012)

Figure I2 shows the strong motion record obtained from the CCCC recorder for the Darfield earthquake. These are plots of the horizontal ground motion.

Figure I3 shows the horizontal elastic response spectral acceleration versus spectral displacement response spectra plots for the ground motions recorded at the four CBD strong-motion stations during the Darfield earthquake.

Vertical acceleration spectra for the motions recorded at each of the recording stations are shown in Figure I4.

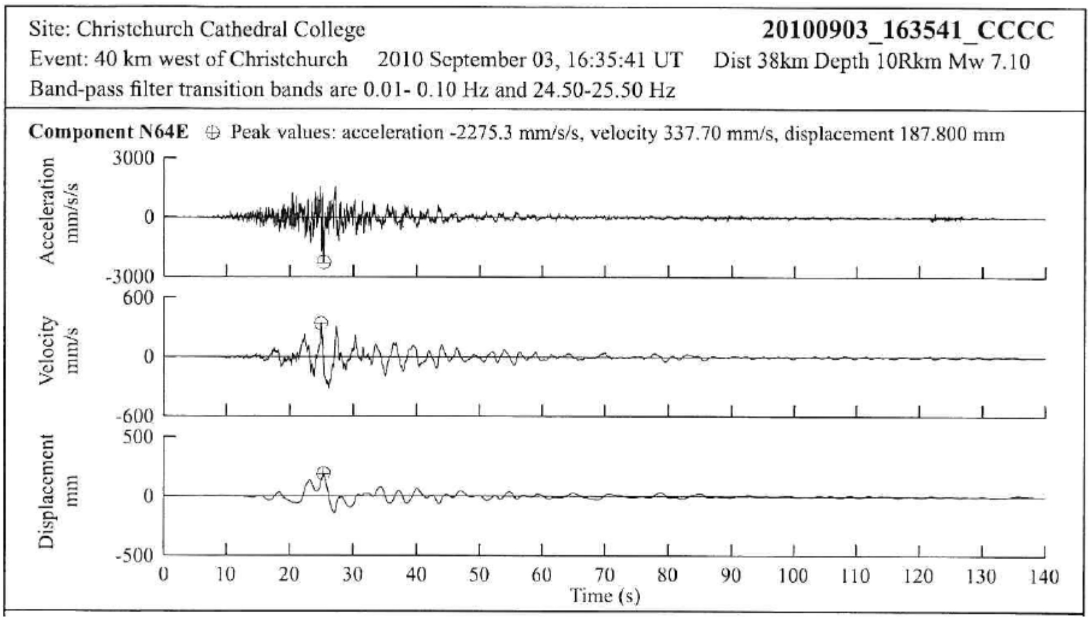

Figure I2: Strong ground-motion recording from CCCC site in the Darfield earthquake for N64E component (Source: GeoNet)

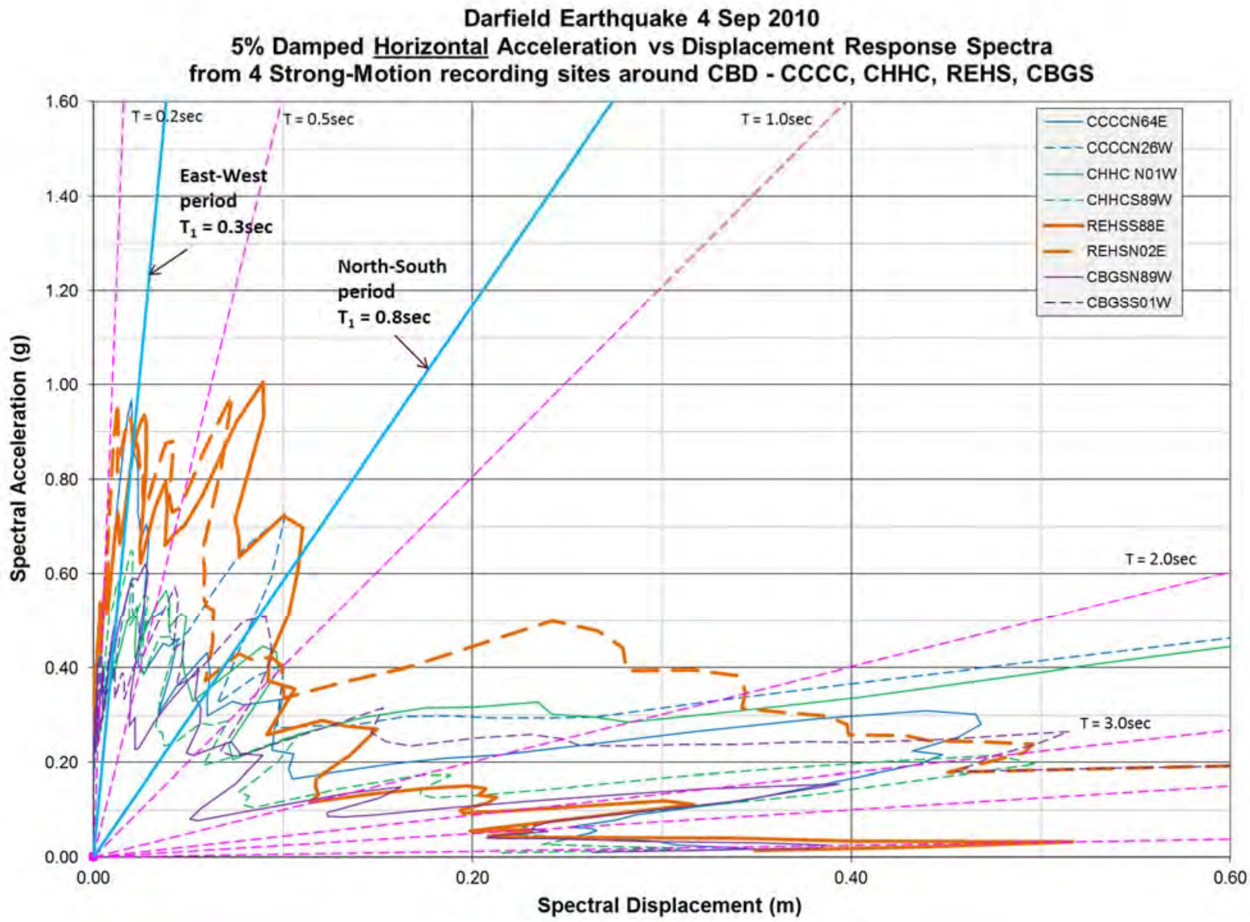

Figure I3: Spectral acceleration versus spectral displacement response spectra plots for recordings at four strong-motion stations (both horizontal components) during the Darfield earthquake

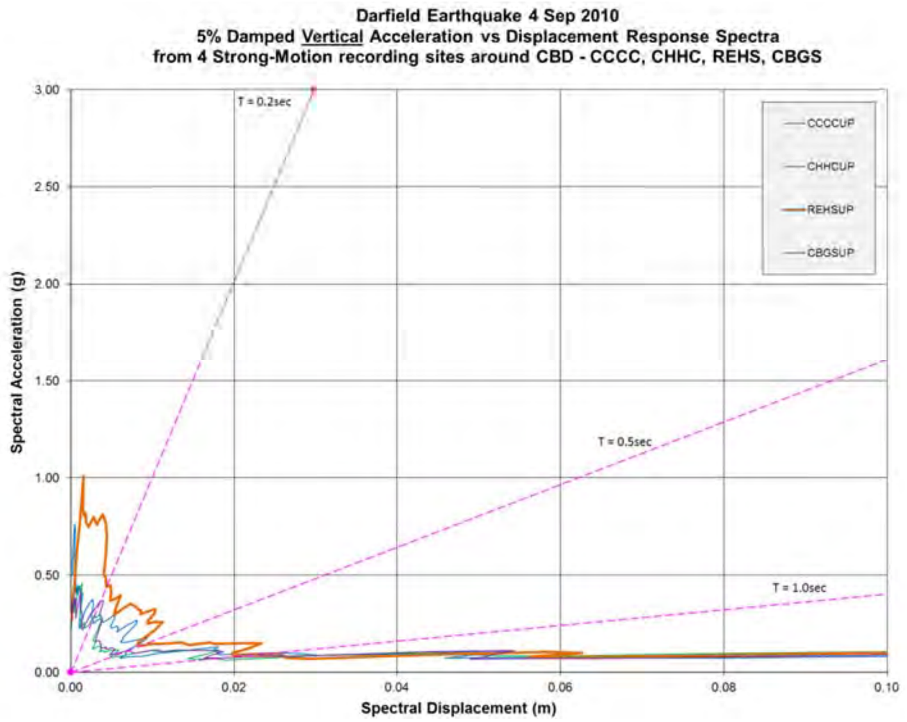

Figure I4: Spectral vertical acceleration versus spectral vertical displacement response spectra plots for recordings at four strong-motion stations during the Darfield earthquake

I.3 Boxing Day Earthquake (26 December 2010)

Figure I5 shows the strong motion plots from the CCCC recording station for the Boxing Day earthquake.

Figure I6 shows the horizontal elastic response spectral acceleration versus spectral displacement response spectra plots for the ground motions recorded at the four CBD strong-motion stations during the Boxing Day earthquake.

Figure I7 shows the vertical elastic response spectra from this earthquake.

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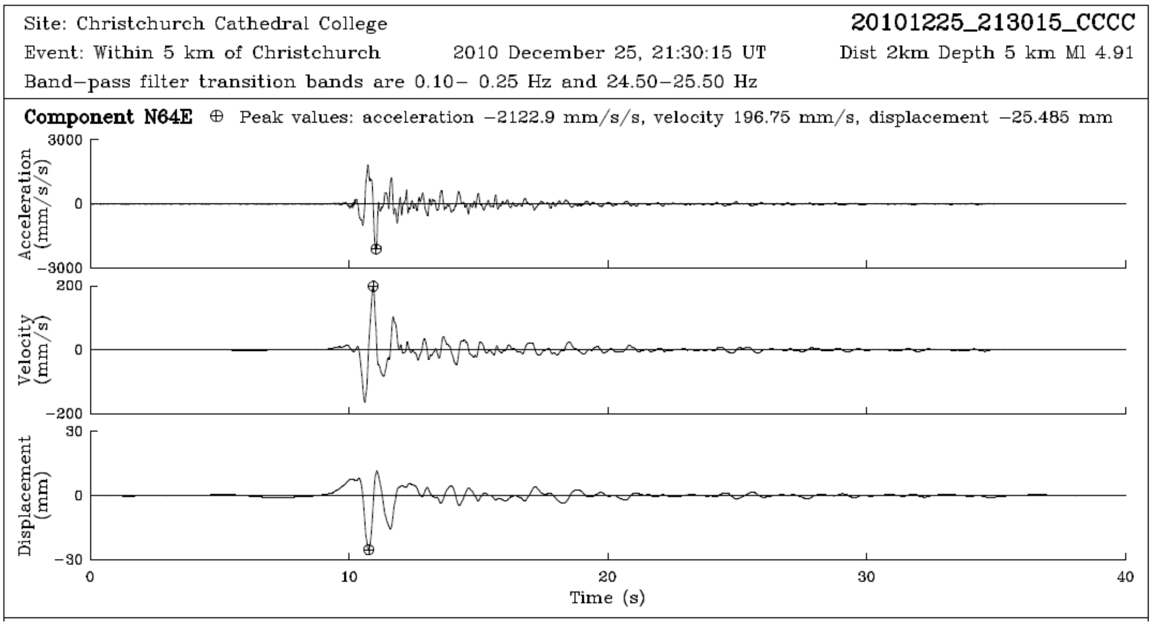

Figure I5: Strong ground-motion recording from CCCC site in the Boxing Day earthquake for N64E component (Source: GeoNet)

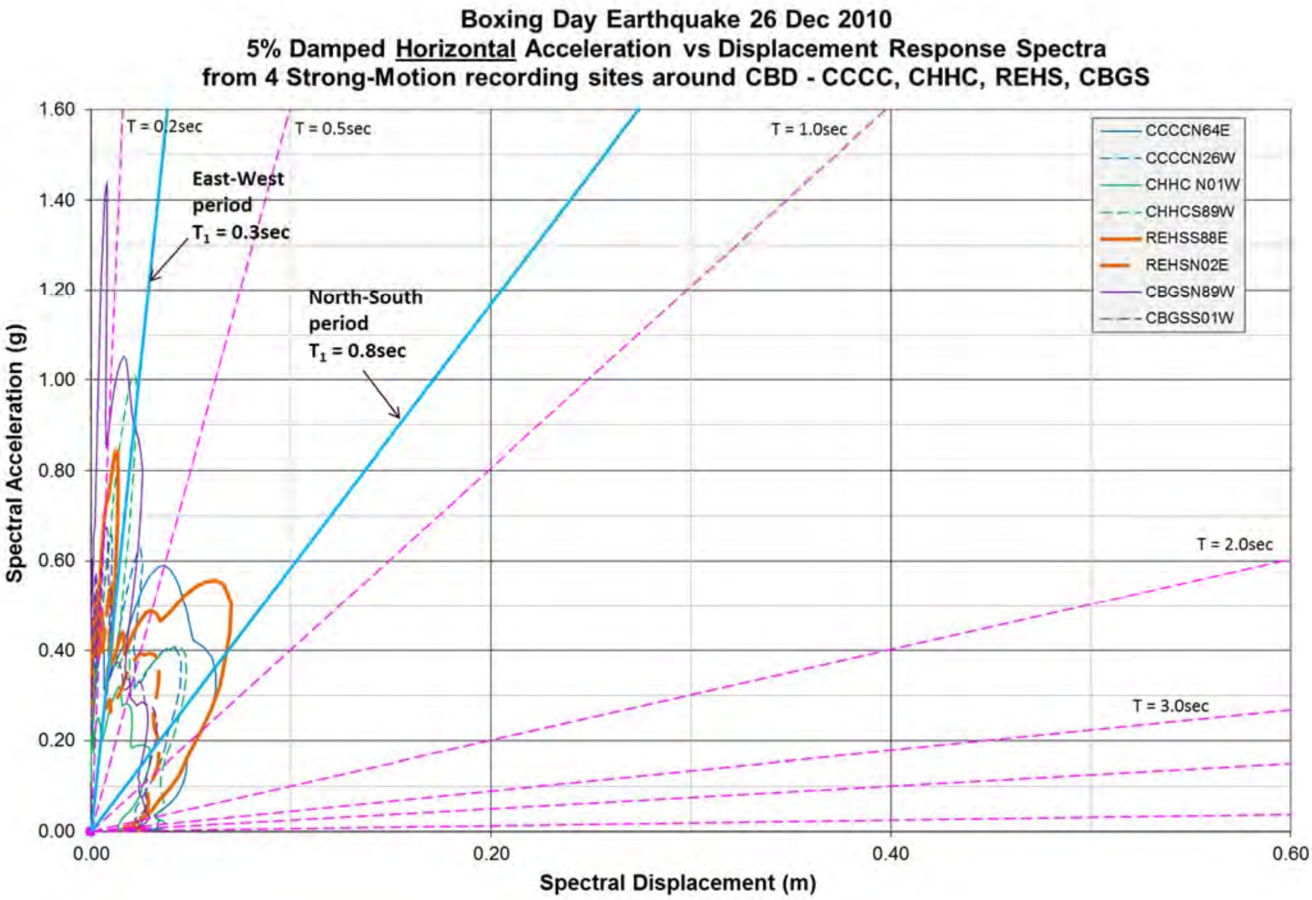

Figure I6: Spectral acceleration versus spectral displacement response spectra plots for recordings at four strong-motion stations (both horizontal components) during the Boxing Day earthquake

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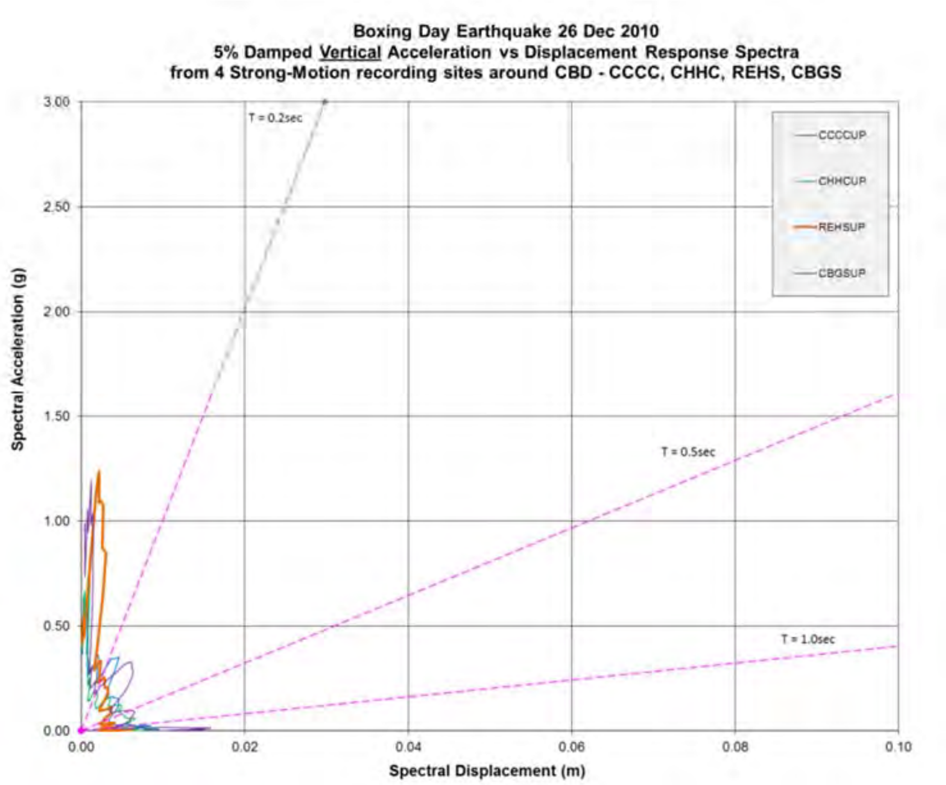

Figure I7: Spectral vertical acceleration versus spectral vertical displacement response spectra plots for recordings at four strong-motion stations during the Boxing Day earthquake

I.4 Christchurch Earthquake (22 February 2011)

Figure I8 shows the location of each recording stations relative to the CTV building and arrows indicating the predominant direction of ground acceleration recorded during the Christchurch earthquake at each. It can be seen that the predominant direction of shaking at all four recording stations was approximately east-west.

Figure I9 shows the strong motion plots from the CCCC recording station for the Christchurch earthquake.

Figure I10 shows the horizontal elastic response spectral acceleration versus spectral displacement response spectra plots for the ground motions recorded at the four CBD strong-motion stations during the Christchurch earthquake.

Figure I11 shows the vertical elastic response spectra from this earthquake.

Figure I8: Location of four strong-motion recorders relative to the CTV building and the predominant direction of ground accelerations recorded during the Christchurch earthquake (Source: CERC Report 2012)

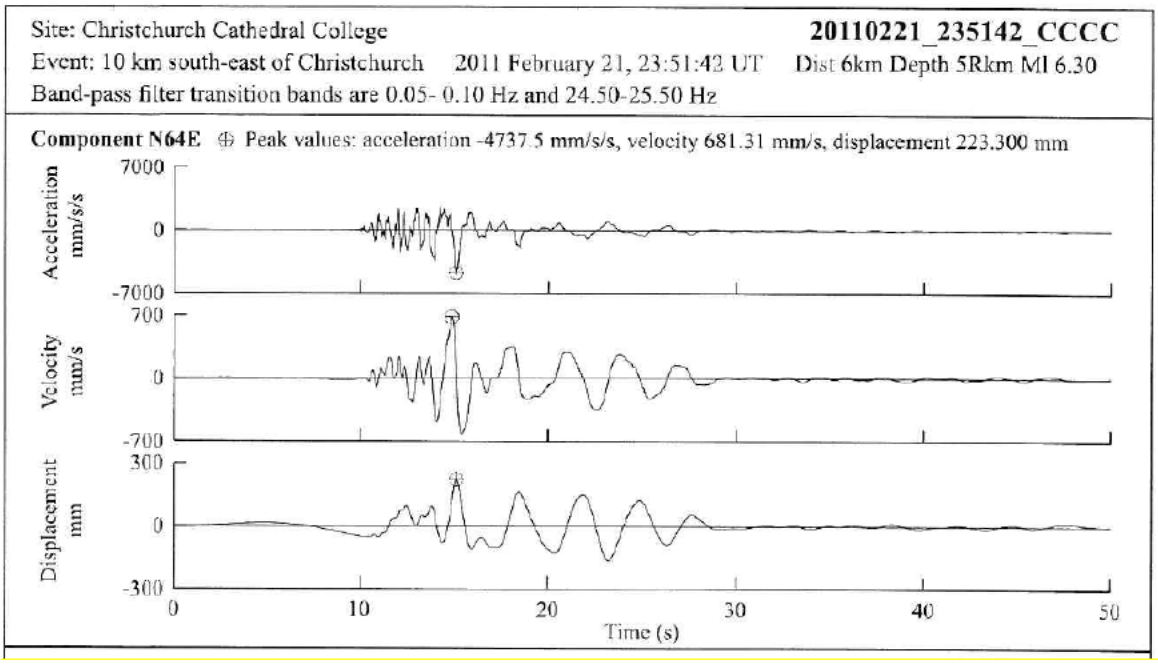

Figure I9: Strong ground-motion recording from CCCC site in the Christchurch earthquake for N64E component (Source: GeoNet)

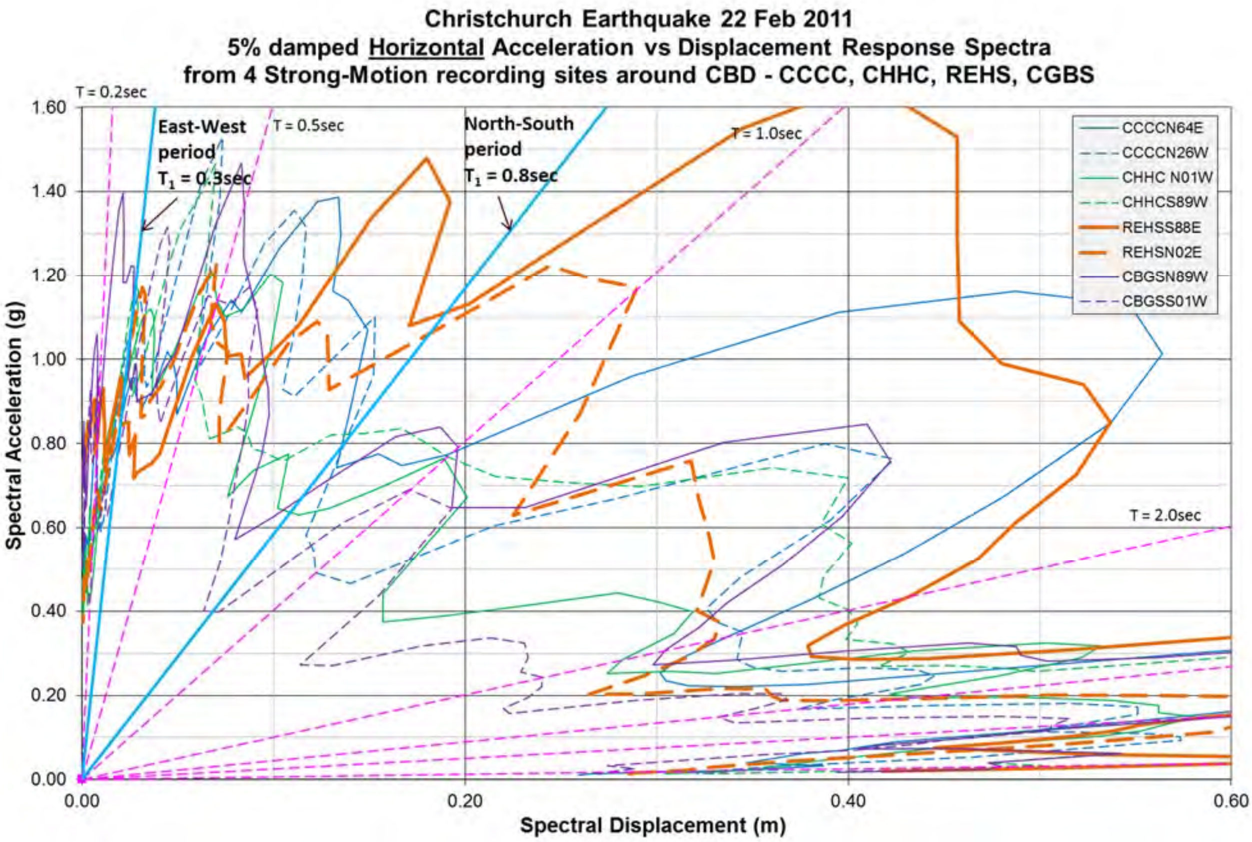

Figure I10: Spectral acceleration versus spectral displacement response spectra plots for recordings at four strong-motion stations (both horizontal components) during the Christchurch earthquake

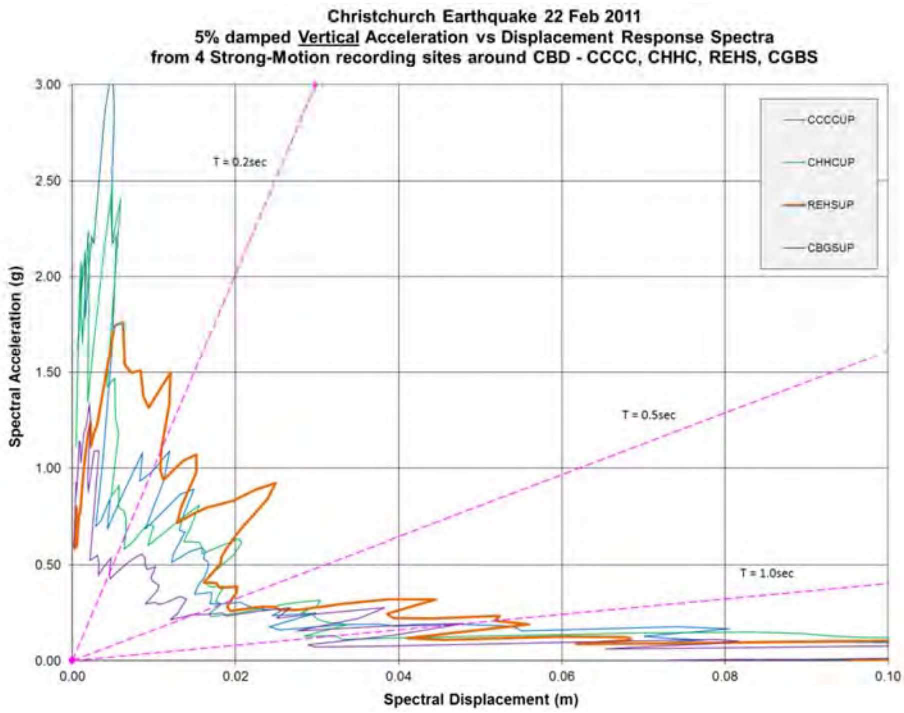

Figure I11: Spectral vertical acceleration versus spectral vertical displacement response spectra plots for recordings at four strong-motion stations during the Christchurch earthquake

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Appendix J

CTV Site and Recording Stations Ground Conditions

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Appendix J CTV Site and Recording Stations Ground Conditions

J.1 General

When considering the earthquake shaking experienced in the 22 February 2011 earthquake, there was a suite of four strong-motion recording stations in the vicinity of the Christchurch CBD available; REHS, CCCC, CHHC and CBGS.

The shaking recorded at the four stations varies considerably in both the time-history recordings and the associated response spectra.

When carrying out the NLTHAss or comparisons with the shaking experienced, it is useful to consider which of the four recording stations is the most representative of the CTV site. This appendix presents our selection of the most appropriate record to use to represent the likely shaking at the CTV site during the February earthquake.

J.2 Information Reviewed

We reviewed the following information in order to make our selection:

- Google Earth to obtain distances (Figure J1)
- Christchurch Central City Geological Report by Tonkin and Taylor, Version 1.1, December 2011 which describes ground profiles at the four CBD recording sites
- Conditional PGA's during 22 February 2011 Earthquake (after Bradley et al 2012) retrieved from the Canterbury Geotechnical Database (Figure J2)
- Plan showing surficial evidence of liquefaction in the CBD presented in Liquefaction Impact on Critical Infrastructure in Christchurch, Bray et al, March 2013
- CPT profiles close to CTV, CCCC and CHHC sites obtained from the Canterbury Geotechnical Database
- Design and assessment reports for the CTV building by Soils and Foundations and Tonkin and Taylor.

Figure J1: Distances between recording stations and CTV building (Source: Google Earth)

J.3 Interpretation and Selection

Our assessment of each of the four sites is as follows:

- REHS – discard because of thick organic soils at the surface which are believed to have led to greater observed ground motions at the site. This view is supported by the T&T 2011 Geological Report, and by calculated Arias Intensities which are the highest of the four CBD sites by a clear margin. The Arias Intensity is a measure of the strength and destructiveness of an earthquake motion determined by integrating the square of the ground acceleration over the length of the record.
- CBGS – the most distant from the CTV site, site liquefaction and the resulting modification of ground motion is least likely, and less likely than at the CTV site.
- CHHC – has a greater thickness of liquefiable material than the CTV site, and sits in an area of greater mapped surface expression of liquefaction. The PGA is markedly lower than the value interpreted by Bradley for the CTV site.
- CCCC – less likelihood of liquefaction damage at the surface but greater liquefiable layer thickness than the CTV site, similar conditional PGA, same mapped (no to low) surface expression of liquefaction.

Comparison between parameters from CPT profiles for CTV, CCCC and CHHC shows similarities and dissimilarities, so cannot be used to choose a record.

J.4 Conclusion

Adopt the CCCC record for this work. It is the closest of the four instruments, the recorded PGA is close to that estimated for the CTV site (Canterbury Geotechnical Database), and the evidence of surface liquefaction evidence is similar (none to minimal).

It is important to also note the recommendation of the Tonkin and Taylor Geological Report (section 6.2) that the most relevant station should be selected based on site conditions, rather than using some interpolation between two or more stations.

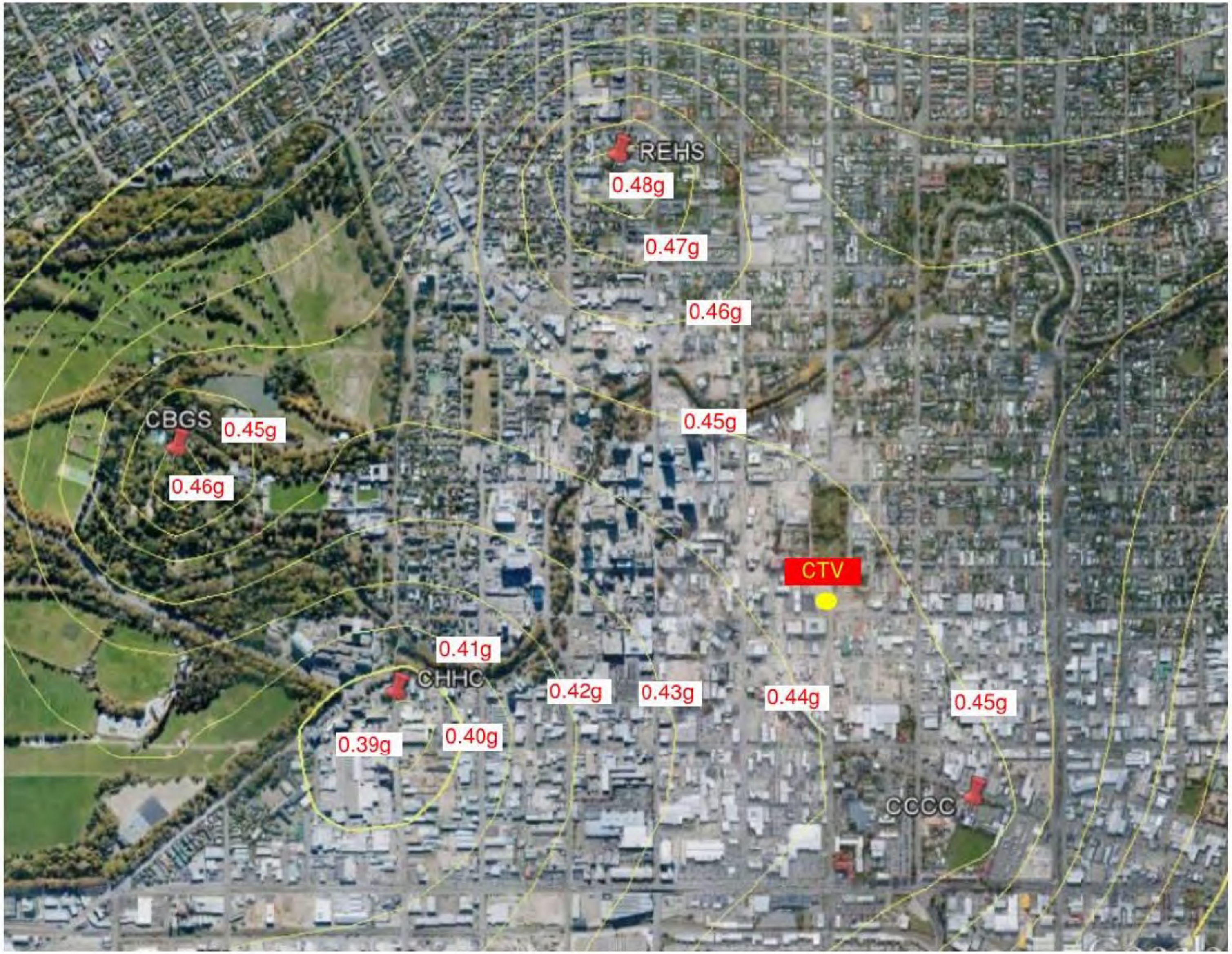

Figure J2: Conditional PGAs (after Bradley et al 2012)

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Appendix K

Physical Testing

Appendix K Physical Testing

K.1 Background

Due to a lack of testing of similar beam and column configurations internationally, and to assist in understanding of the failure sequence and mechanism, further investigation of this region was proposed in the form of physical testing.

A programme was developed and implemented to experimentally test full-scale replicas of a section of the interior frame at the University of Auckland (UoA). Three different scenarios were tested; 'As-designed', 'Compliant design/construction errors', and 'Code-Compliant'.

The following appendix outlines the key aspects of the testing from the perspective of Beca. UoA were commissioned by the Police and have produced a separate testing report titled *Reinforced Concrete Beam-Column-Joint Testing*. While UoA carried out the testing in collaboration with Beca, Beca's appendix and UoA's testing report are independent. As a result, the two documents may use different terminology and may have a different perspective.

K.2 Methodology and Approach

The following section outlines the general approach followed, a description of the three specimens, and an overview of important aspects of the testing such as setup and loading protocol.

K.2.1 Approach

The generalised approach to carrying out the proposed physical testing programme was:

1. 'Stage 1' which involved detailed design of the testing programme.
2. 'Stage 2' which involved the construction of the specimens, and the experimental testing and associated interpretation of results.

K.2.2 Methodology and objective

The objective of the testing programme was to further understand the potential for axial collapse of the identified critical region of the building, that being the region comprising the ground floor column and beam-column joint at the first floor above ground on Grid 2 at either grid C (column C8) or grid D (column C7), as shown in Figure K1. These are the most highly axially loaded columns under gravity loads, and the part of the building identified by analysis as the most susceptible to loss of axial load carrying capacity in an earthquake.

The naming convention used throughout this appendix is that shown on Figure K1, with ground referred to as Level 1, and the first level above ground Level 2. This has been chosen for consistency with both our main report, and the original design documentation.

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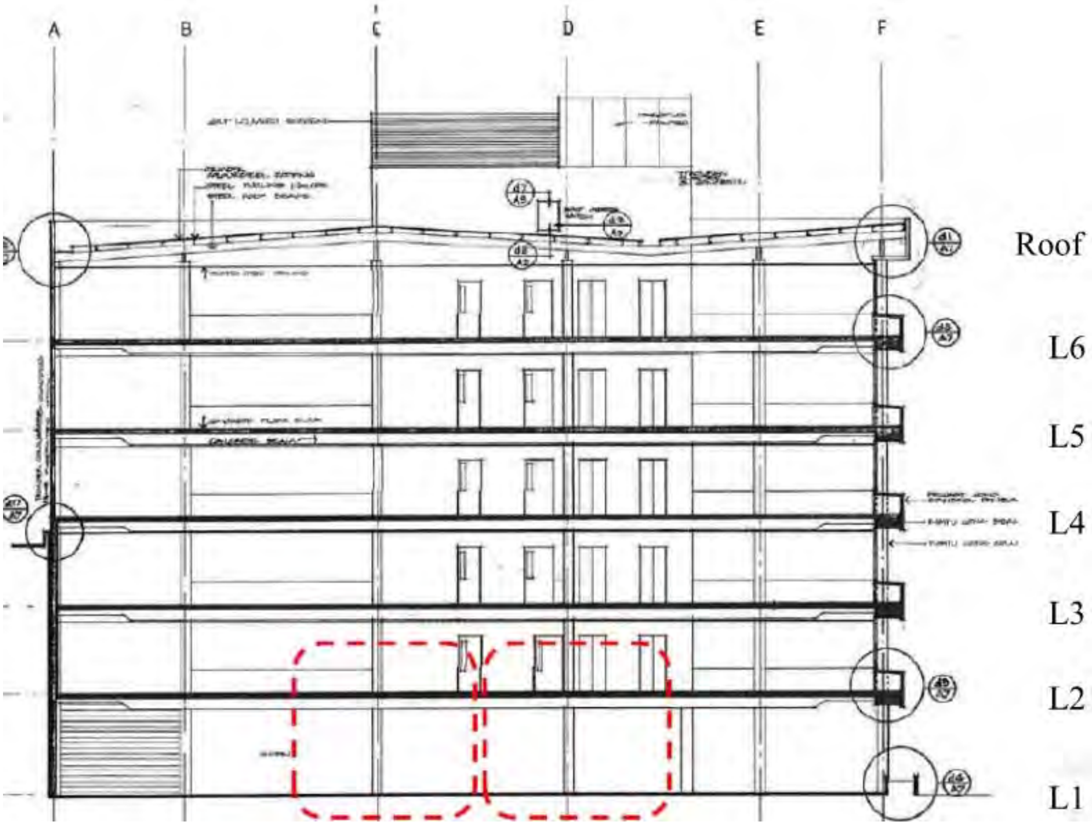

Figure K1: Identified region of the building to be investigated

By testing different scenarios, the relative contributions of the design and construction deficiencies to the collapse could be investigated. To that end, three specimens were designed, constructed and tested:

- 'As-designed' specimen – this specimen was designed to represent the region under investigation, including the known design deficiencies, in the absence of any known construction deficiencies. The major design deficiencies modelled were the non-compliant beam-column joint horizontal reinforcement, and the effect of the under-designed shear walls (included in the applied deformation). Throughout this appendix, this specimen and its test will be referred to as As-designed.
- 'Compliant design/construction errors' specimen – this specimen was designed to represent the region under investigation, including the known construction deficiencies, in the absence of any known design deficiencies, i.e. a building that was otherwise compliant to the codes of the day (including compliant shear walls). The known construction deficiencies modelled were the smooth construction joints on the ends of the precast beam portions, and no beam-column joint horizontal reinforcement. Since this test models the scenario where the known construction deficiencies are included, but not the design deficiencies, it is similar to but not strictly an as-built condition.
- 'Code-compliant' specimen – this specimen was designed to represent the region under investigation, in the absence of both the known design and construction deficiencies, i.e. a building that was compliant to the codes of the day. Throughout this appendix, this specimen and its test will be referred to as Code-compliant.

The general aim was to model the critical region to as near as reasonably practical (or known) to the in-service condition of the building, to simulate the estimated effects on this region that were or would have been experienced in the 22 February 2011 earthquake.

Any physical testing programme of this type includes compromises, as it is impossible to rebuild a specimen that exactly models the geometry, details, material properties and earthquake loading that the original building possessed and experienced on 22 February 2011. Trade-offs must be made, and to that end it was aimed to minimise these compromises while maximising the relevancy of the test.

The following is an overview of the important aspects of the construction of each specimen, the test setup, and loading protocol to achieve this aim.

K.2.3 Sub-assembly

Full-scale cruciform-shaped subassemblies modelling the region comprising column from ground (Level 1) to halfway up the second storey (Level 2.5), and the beams out to half-span were chosen (Figure K2).

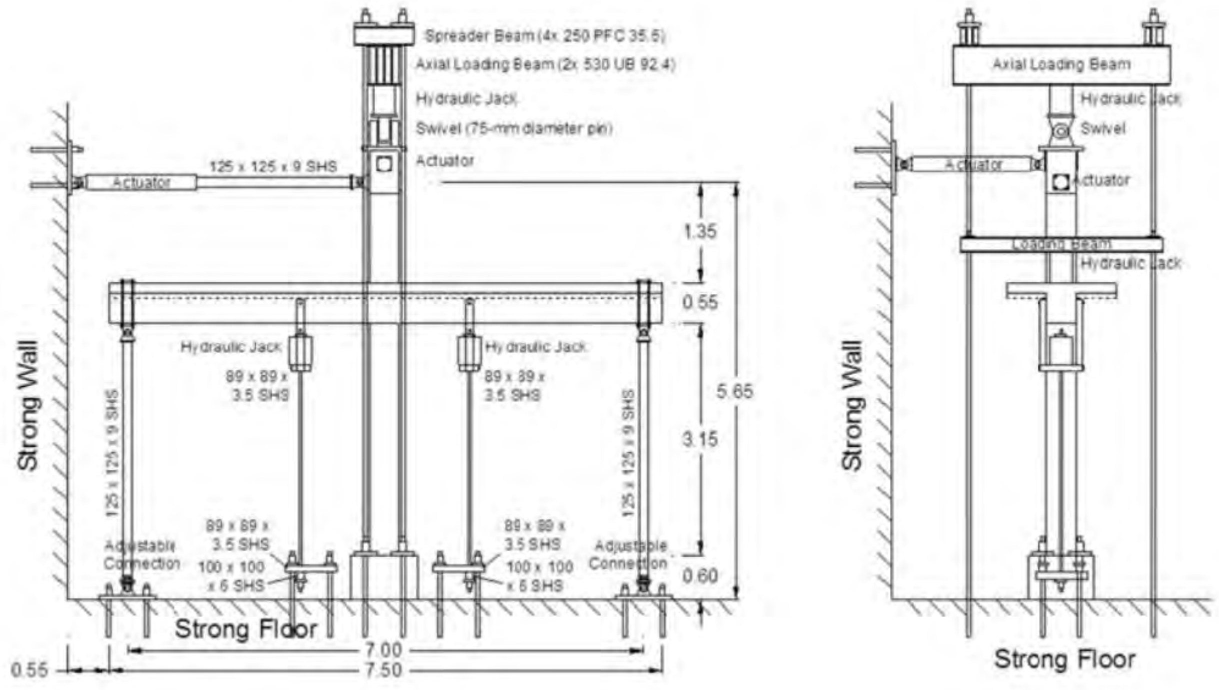

Figure K2: Indicative test setup

Loads were designed to be applied as follows:

- Gravity – in addition to the self-weight of the members, vertical point loads on top of the column and at the estimated point of inflection (under gravity loading only) in the beams to simulate in-service dead and imposed loads from the surrounding tributary structural system. These loads were applied through tensioned vertical rods reacting against the strong floor below and controlled by a hand-operated hydraulic jack system.
- Earthquake – imposed horizontal displacements by way of horizontal point loads in the north-south and east-west directions at the estimated point of inflection (under seismic loading only) in the column at Level 2.5, with vertical restraint at the beam ends. These loads were applied through two independent actuators reacting against the perpendicular strong walls.

The column was modelled as fully fixed (translation and rotation) at ground by way of a foundation block tensioned to the strong floor. This was deemed to be applicable due to the large foundation pads under these columns in the actual building coupled with their high axial load, and as informed by non-linear time history analyses that included the effect of soil stiffness showing minimal rotation at ground. In addition, any rotation of the column base at ground was removed in determining the target inter-storey drifts as outlined in section K.2.7 below.

The slab was included to a width of 1.5m to simulate the effective width of a T-beam where the slab is built integrally with the beam. Due to the discontinuity of the Hi-bond decking where it meets a beam, and the long span of the flooring relative to its depth, the contribution of the slab to joint resistance and stiffness under north-south loading was deemed insignificant, and therefore not modelling the slab its full 7m span either side of the Grid 2 beam was considered a reasonable compromise.

K.2.4 Details

In general, the construction details were designed to match the details shown on the original 1986 structural drawings, with the exception of those aspects that were varied between the three specimens (e.g. horizontal beam-column joint reinforcement and smoothness of curved beam-end construction joints). Below is a selection of important details from the structural drawings produced for the testing (Figures K3 to K6).

Other notable aspects of the detailing chosen for the specimens:

- The beams of the original building contained longitudinal reinforcement of 28mm diameter. This is no longer a standard size produced in New Zealand, and as a result a special run was required.
- It was determined that the deformation pattern used for deformed reinforcement in the mid-1980s is ostensibly the same as is used today.
- Reinforcement bend diameters, hook extensions etc. were as per NZS 3101:1982.

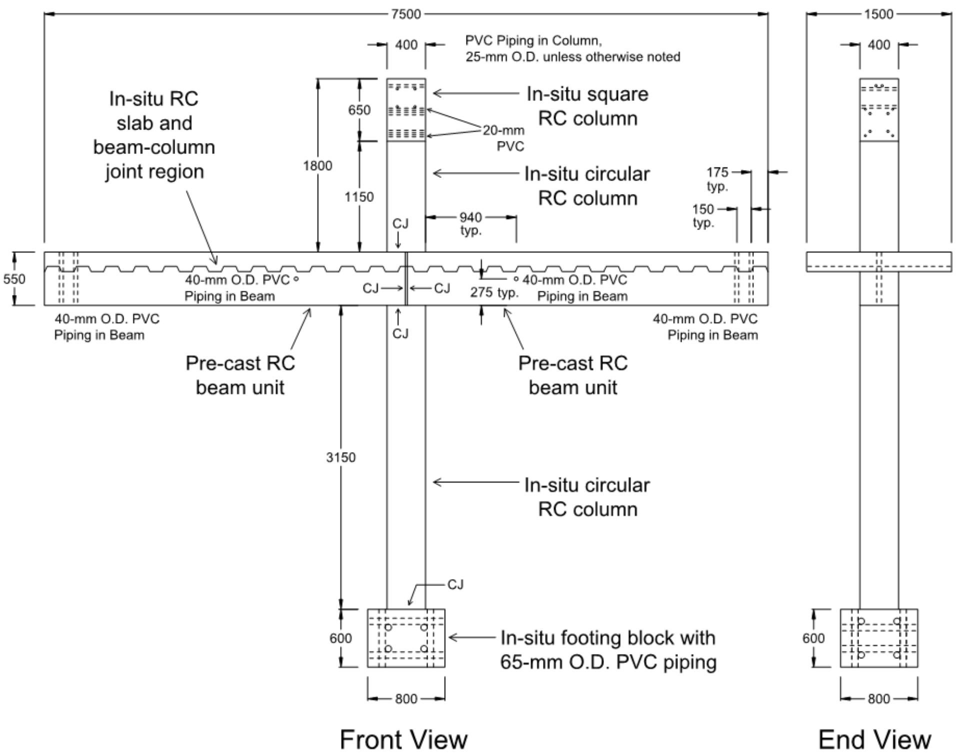

Figure K3: Specimen elevations showing overall geometry

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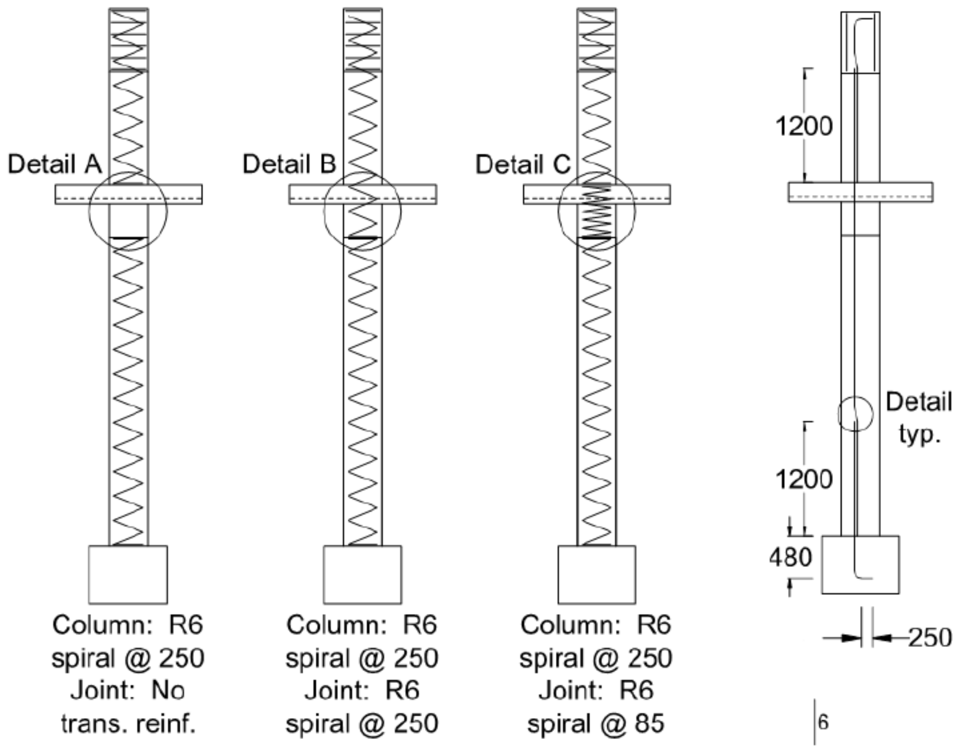

Figure K4: Column reinforcement details

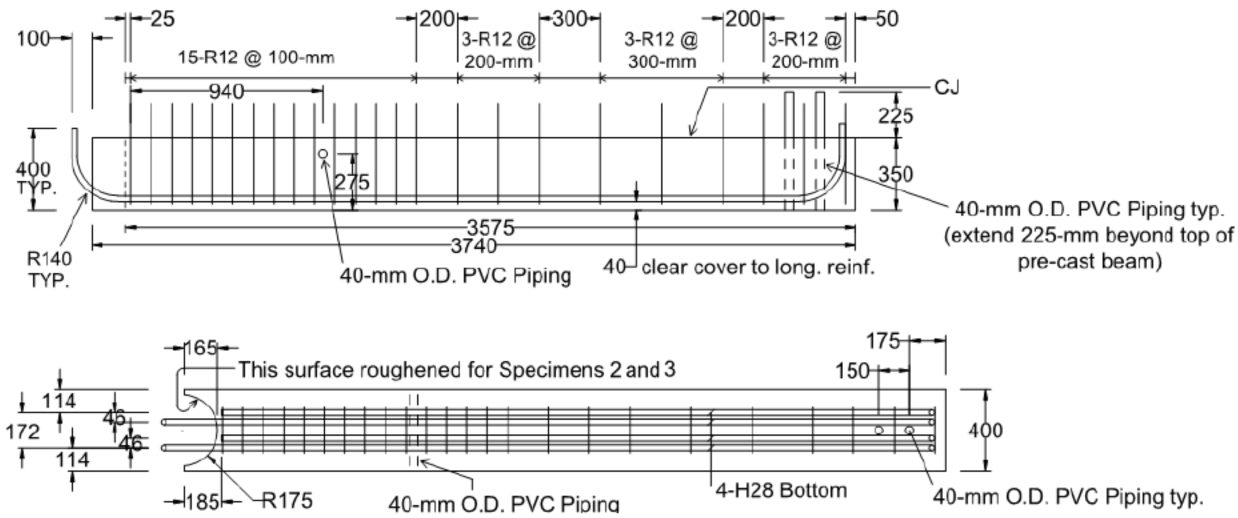

Figure K5: Beam reinforcement details

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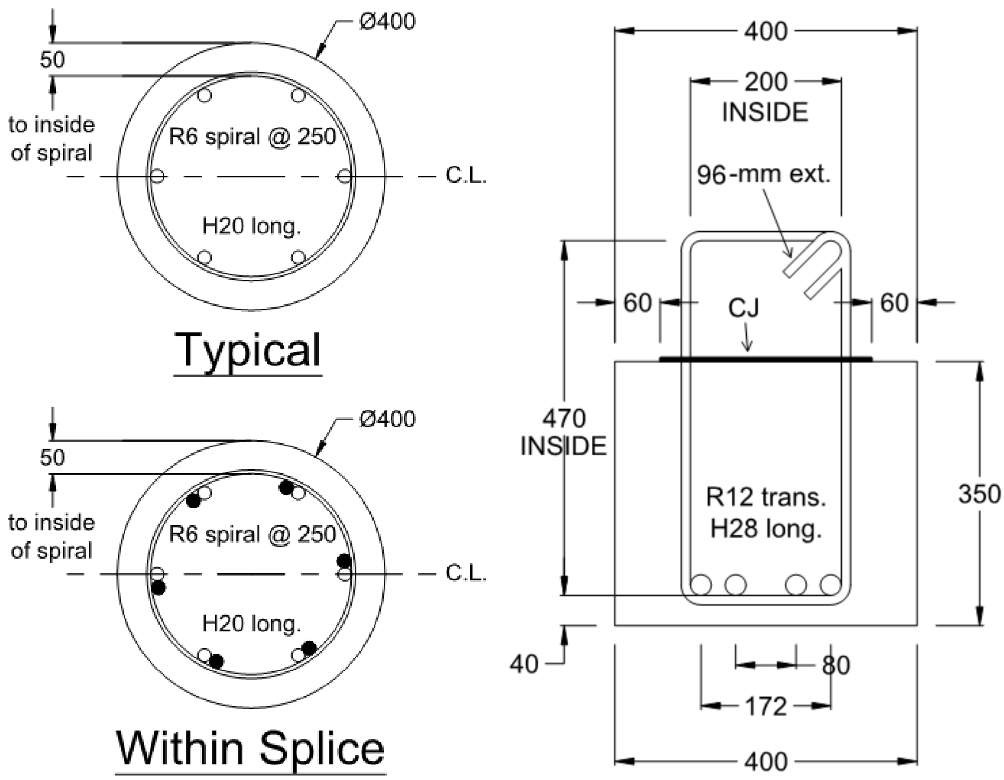


Figure K6: Typical column and beam sections

K.2.5 Construction Sequence

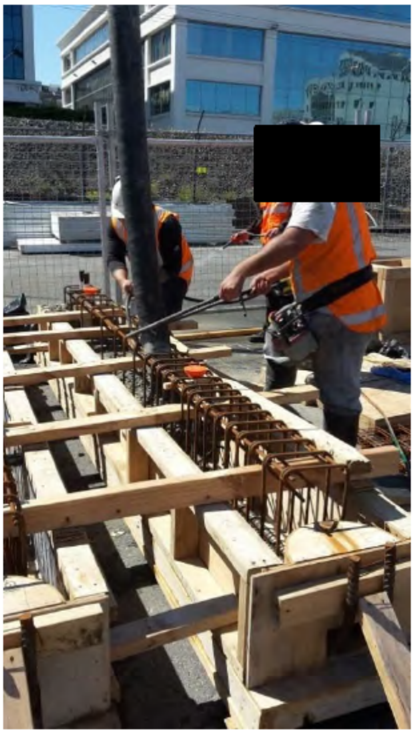
It is believed that the general construction sequence of the CTV building in relation to this region of the building was as follows:

1. Formwork erection, placement of reinforcement cage and in-situ pouring of foundation pad concrete with column vertical reinforcement starters protruding above ground floor slab
2. Formwork erection, placement of reinforcement cage and in-situ pouring of column concrete from ground floor slab to underside of Level 2 beams with column vertical reinforcement starters protruding above future Level 2 slab level
3. Precast beams:
   a. Adjacent or off-site production of precast beams including formwork, placement of reinforcement cage and pouring of concrete
   b. Placement of precast beam sections to sit on column section previously poured and adequately cured
4. Placement of Hi-bond decking on beams with 60mm seating
5. Placement of beam-column joint transverse reinforcement, beam top longitudinal reinforcement and slab reinforcement
6. In-situ pouring of Level 2 slab and beam-column joint concrete
7. Repeat steps 2 to 6 for floors above

It was attempted to mimic this sequence in the construction of the test specimens, including separate in-situ concrete pours for the foundation block, lower column, slab and joint, and upper column. The lower portion of the beams was also precast at ground level and craned in as per the actual building. See photos below showing key stages of the construction of the test specimens as they relate to the steps above.

Step 1 – Pouring of foundation block concrete

Step 2 – Pouring of lower column concrete



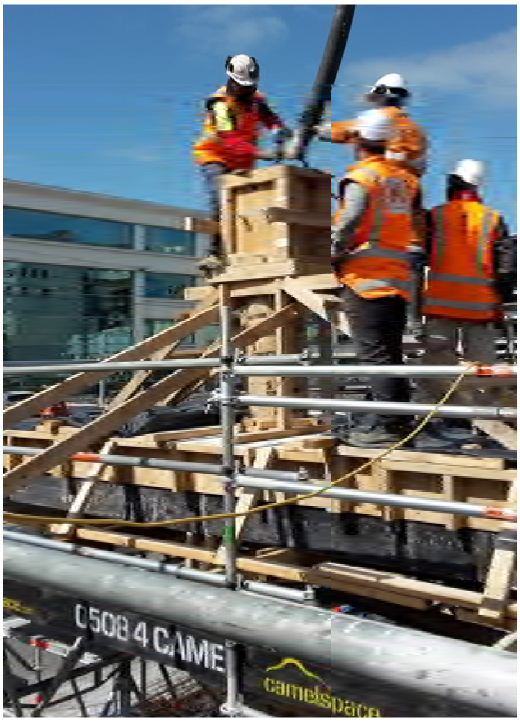
Step 3a – Production of precast beams

Step 3b – Placement of precast beams on column

Step 4 – Placement of Hi-bond decking on beams

Step 5 – Placement of beam-column joint, beam top longitudinal and slab reinforcement

Step 6 – Pouring of slab and joint concrete



Step 7 – Pouring of upper column concrete

Completion – cure and strip formwork, prop beams, move into lab

K.2.6 Material Properties

The general philosophy was to attempt to match as closely as possible in the test specimens the material properties that would have been present in the building on 22 February 2011.

The region being investigated contains both concrete and reinforcing steel, each with varying grades to be considered, as well as other less significant materials used for the floor mesh and Hi-bond decking.

It is impossible to know exactly what the actual properties of the materials used in the prototype region were, particularly for the concrete. Also, different grades of reinforcement are commonly available today (G300E and G500E) compared to when the building was built in the mid-1980s (G275 and G380). Therefore, decisions had to be made that aimed to minimise the compromises while still maintaining the feasibility of the process.

The decisions made regarding material properties are outlined below.

K.2.6.1 Reinforcing steel

The following decisions were made regarding grades of reinforcing steel:

- For reinforcement that was specified in the original design as being Grade 275 (plain round bar used for beam stirrups and column spiral), Grade 300E reinforcement was used. The justification for this was that the probable yield strength of G300E reinforcement is close to that of the historic G275 (within 5%).
- For reinforcement that was specified in the original design as being Grade 380 (deformed bar used as longitudinal reinforcement in the columns and beams), Grade 500E reinforcement was used. The justification for this was that while G500E reinforcement has a probable yield strength approximately 20% greater than that of the historic G380, having stronger reinforcement in these regions was expected to have a negligible effect on subassembly performance during testing (i.e. same hierarchy of beam to column flexural strength at the Level 2 joint, and negligible effect on moment-curvature behaviour of potential column hinge region since by calculation compression was expected to control). In addition, it was deemed to be more important to maintain the same diameter of longitudinal bar used in the beams and particularly the columns as specified, as the post-yield resistance of the subassembly was expected to be more likely determined by geometrically-affected mechanisms such as buckling.

K.2.6.2 Concrete

The region under investigation contained three different specified concrete strengths; 25MPa specified for the precast beams and floor slabs/beam-column joint, 30MPa specified for the upper column, and 35MPa specified for the lower column.

Typically, it is considered that the actual compressive strength of concrete in existing structures is likely to exceed the specified value (NZSEE). The NZSEE recommends, in the absence of specific information, taking a value 1.5 times the nominal compressive strength.

As part of producing the Hyland report, core samples were taken from remains of the actual building and tested to determine properties of the concrete. The Hyland report concluded that the columns were likely to have compressive strengths less than the specified values. However, this finding was contended by experts including Dr James Mackechnie and the Cement and Concrete Association New Zealand (CCANZ), and the CERC found that “there is no reliable evidence to suggest the concrete was understrength in any columns”.

Furthermore, due to factors such as the type of cement used in the mid-1980s and the slenderness of the columns, both Mackechnie and CCANZ conclude that the strength of the concrete in the columns of the actual building in 2011 was not likely to be significantly greater than the specified strengths.

Based on these findings, the approach taken was to attempt to achieve concrete strengths in the specimens that were at least equal to the applicable specified 28 day value (25, 30 or 35MPa), and not significantly higher. Furthermore, it was aimed to minimise the difference in concrete strengths between specimens on their respective testing days.

In light of this, the following measures were taken to attempt to maximise similarity between the concrete in the test specimens and that of the real building on 22 February 2011:

- Used aggregate specially sourced and transported from Christchurch to better match aggregate likely to have been used in the actual building (more rounded greywacke gravels rather than more angular Auckland basalt).
- Used no special admixtures or retarders to better match the mix design that is likely to have been used in the actual building.
- Used mix designs corresponding to specified 28 day strengths 5MPa lower than the actual specified values (e.g. 20MPa mix for 25MPa specified). These mixes have target strengths approximately equal to the specified value (e.g. 24.5MPa for 20MPa mix), and mean tested strengths slightly higher again (e.g. 27MPa for 20MPa mix). This approach was taken to ensure that the concrete in the specimens was likely to be at least that specified, but not significantly higher.
- Used concrete from the same batch/truck in each applicable part of the three specimens (e.g. same batch/truck for all three lower columns) in order to minimise variability between the three specimens.
- Tested the first specimen more than three months after the last concrete pour so as to minimise variability of concrete strength between the three specimens that were tested several weeks apart.
- For each concrete pour (precast beams, lower column, joint/slab, and upper column), a large number of cylinders were taken to be tested in order to assess the concrete compressive strength at different ages (7 days, 28 days and the day of each specimen test), and as affected by two different curing methods (‘ambient’ versus ‘ideal’). This comprehensive cylinder testing programme was undertaken to ensure a high degree of confidence that the concrete in the specimens had a low variability across the different specimens, and that the concrete was of an adequate strength.
- Took core samples from the tested specimens when practical (i.e. were uncracked), to be tested to obtain an indication of in-situ concrete compressive strengths.

Figure K7 presents a summary of the cylinder test results showing the cylinder strengths obtained over time. Each data point is an average of between two to four individual cylinder tests. The figure combines tests relating to four different pours in chronological order; precast beams, lower column, joint/slab, and upper column. It also has lower bound curves (lighter colours) for the cylinders cured in ambient conditions, and upper bound curves (darker colours) for cylinders cured in ideal conditions (fogroom). The figure also contains horizontal lines corresponding to the target strengths for each.

The results show with good consistency the strength gain that would be expected between 7 and 28 days before a flattening off of the strength gain from 28 days on. In all cases, beyond approximately 21 days the ideally cured cylinders have reached or exceeded their applicable target strength. By the three testing days (117 days and older), the target strength lies roughly halfway between the upper and lower bound curves. For the specimens, it is expected that the curing conditions vary within the cross sections of the members, with the cover concrete being closer to ambient and the core concrete closer to ideal.

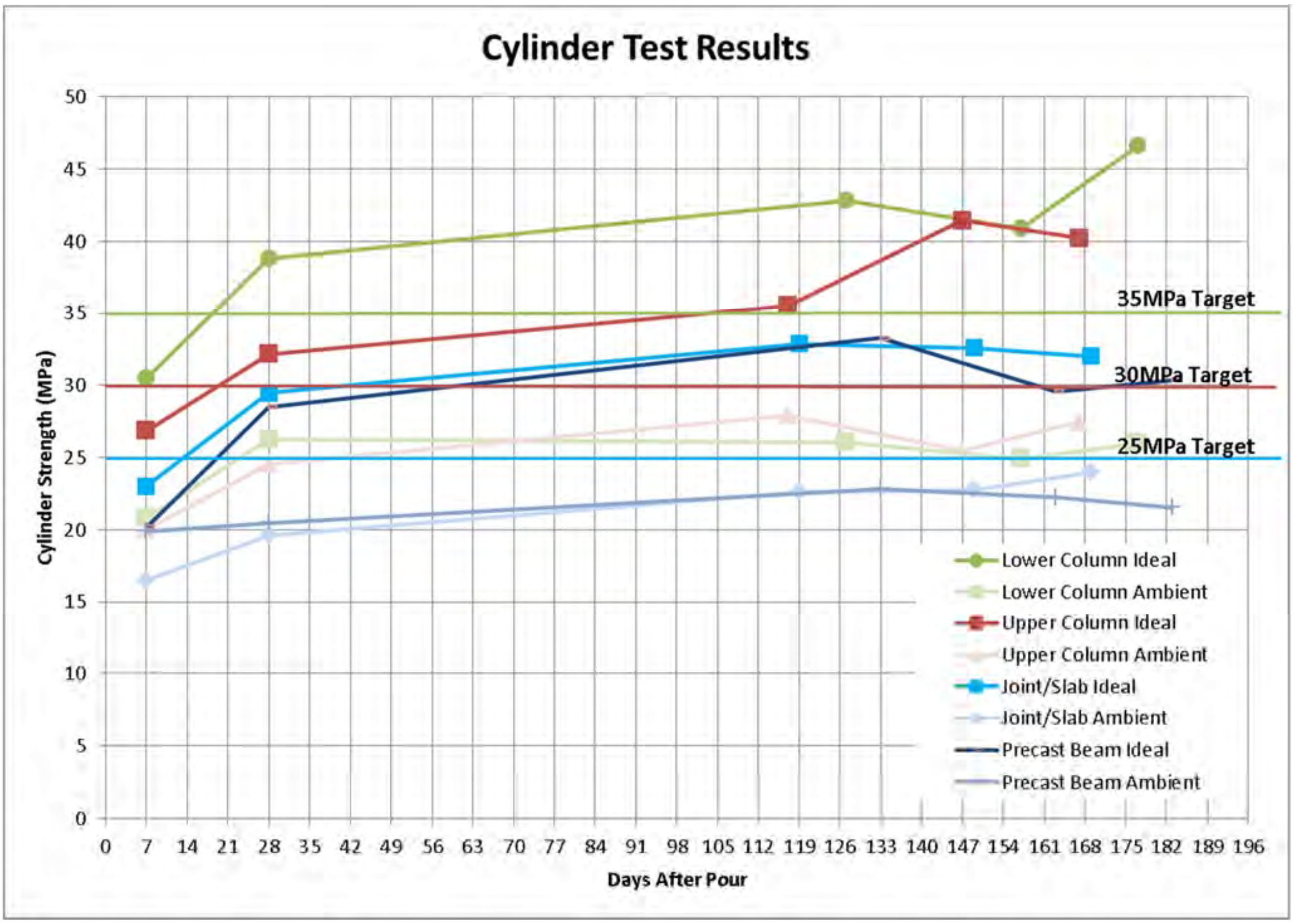


Figure K7: Comparison of concrete cylinder test results

Following testing, core samples were taken from the middle region of the lower column of the As-designed specimen. The lower column was chosen as the damage in all three specimens was most pronounced in this area. The target strength for the concrete in the lower column was 35MPa. Only the As-designed specimen had a length of visibly uncracked column that was suitable for taking cores. Seven cores were taken from this specimen, consisting of three cored horizontally, and four cored vertically. Two of the vertically cored cylinders were discounted as they were significantly shorter than the other five cylinders. The average compressive strength of the five suitable cylinders was 34MPa. The average compressive strength of the three horizontally cored cylinders was 35MPa, and of the two vertically cored cylinders was 31MPa. It was observed that the compressive strength was highest for two cores near the middle of the uncracked length (40.5 and 37.4MPa), and lower for the three either side (28.3, 29.8 and 32.3MPa). This suggests that the strength of the cored cylinders was affected by cracking in the specimen due to the testing.

Based on the results, and recognising that pre-pour cylinder tests are only an indexing of concrete strengths rather than a direct correlation with what is in a real structure, it is concluded that the concrete in the specimens is likely to have met the objectives stated above.

K.2.6.3 Other

There has been negligible change in the material properties of Hi-bond metal tray decking between the mid-1980s and today, so no special considerations were required for this aspect.

Regarding the properties of the mesh used in the floor slabs, hard drawn steel wire fabric can be and was sourced today that has similar properties to that used in the mid-1980s.

K.2.7 Loading Protocol

The following section outlines the gravity and earthquake loads that were applied to each specimen and an overview of how they were determined.

K.2.7.1 Gravity Loads

Gravity loads applied to the specimen were applied in three locations; two at the location of the inflection point of the beams under gravity loading, and one at the top of the column, as shown in Figure K8 below. These loads are in addition to the self-weight of the specimen members, and targeting axial loads of 1310kN in the column above Level 2, and 1630kN in the column above Level 1 (ground).

The applied loads were calculated based on the tributary structural system surrounding the specimen in the actual building, and intended to represent a best estimate of what was present on 22 February 2011. This included an allowance of 0.5kPa for superimposed dead load on all floors corresponding to the expected load due to fixed finishes/services/partitions, and 1.0kPa for imposed load on all floors corresponding to the expected load due to unfixed furnishes and live loading (the building was occupied at the time).

All specimens had the same target applied gravity loads.

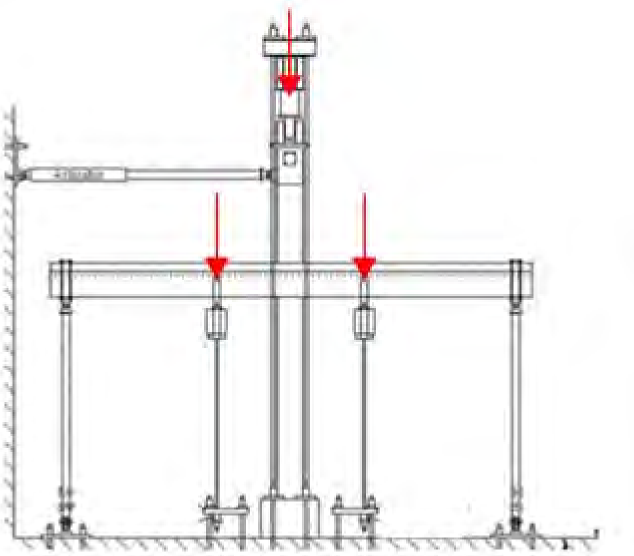


Figure K8: Gravity loading applied for all three specimens

K.2.7.2 Earthquake Loads

Once a specimen had the gravity loads applied as above, lateral earthquake loads were then applied.

The earthquake loading protocol was defined by displacements at the Level 2 slab, as informed by nonlinear time-history analyses (NLTHA) previously carried out on analytical models of the building using 22 February 2011 earthquake ground motions.

In order to achieve these targeted displacements at Level 2, loads were applied through two perpendicular actuators at the top of the specimen, at the approximate location of the point of inflection in the second storey column under earthquake loading in the east-west direction.

One actuator was able to apply push-pull actions to the specimen for loading in-plane (east-west direction) from a frame perspective, and the other for loading out-of-plane (north-south) from a frame perspective. The

[redacted]

actuators were unable to be operated simultaneously, but a displacement could be applied in one direction and held while the actuator was operated in the perpendicular direction.

The loading rate was quasi-static, being much slower than real-time earthquake motions.

The general sequence of loading for each specimen with reference to Figure K9 was as follows:

1. Cyclic in-plane only (east-west) displacements following a stylised protocol derived from the results of a selected NLTHA simulating the 22 February 2011 earthquake matching the test sample.
2. If the specimen survived (1) without loss of axial load carrying capacity, one cycle of out-of-plane only (north-south) loading was applied to the peak north-south displacements in each direction as derived from the same NLTHA.
3. If the specimen survived (2) without loss of axial load carrying capacity, two sets of biaxial displacements were applied to the worst-case coincident peaks in each of northwest, southeast, northeast, and southwest directions as derived from the same NLTHA.
4. If the specimen survived (3) without loss of axial load carrying capacity, a monotonic in-plane only (east-west) push or pull was applied until the specimen loses axial load carrying capacity.

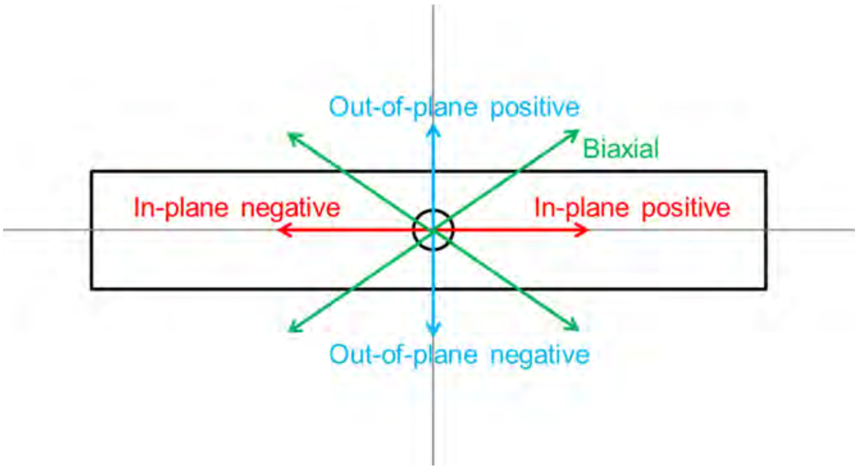


Figure K9: Loading naming convention

The exact loading protocol applied in each step above was not the same for each specimen, and varied depending on the individual objective of each test. These objectives, the source of the loading protocols, and the reasons for their application are outlined for each specimen below.

However, in all cases, the source NLTHA from which the above loading protocols were derived were based on the following:

- 22 February 2011 earthquake only (i.e. not including any previous earthquakes such as 4 September 2010) – it was not believed that any previous earthquakes would have damaged or degraded the region under investigation sufficiently to compromise the test. The validity of this conclusion was investigated during the testing, the results of which are covered later in this appendix.
- No vertical acceleration effects – it was deemed that the modelling of such effects contains too much uncertainty in regards to the actual axial load in the column at any point in time.
- Earthquake motions as recorded at the Christchurch Cathedral College (CCCC) strong motion recording site – the recordings at this site are deemed to be the most representative of the CTV site (see Appendix J – CTV Site and Recording Stations Ground Conditions).

As-designed specimen

The objective of this test was to determine if the building was likely to have collapsed in the 22 February 2011 earthquake due solely to the known design deficiencies. If the specimen loses axial load carrying capacity in loading steps 1, 2 or 3, the conclusion can be made that, due to progressive collapse, the building was likely to have collapsed in the earthquake.

Since this scenario represents the building including all known design deficiencies, but without any known construction deficiencies, a target protocol and displacements in steps 1, 2 and 3 above were derived from a NLTHA of a computer model that included the understrength shear walls – termed the ‘As-designed Model’.

The loading protocol for step 1 of the sequence is shown in Figure K10. This shows the intended target lateral displacement of Level 2 relative to the top of the foundation block (at Level 1).

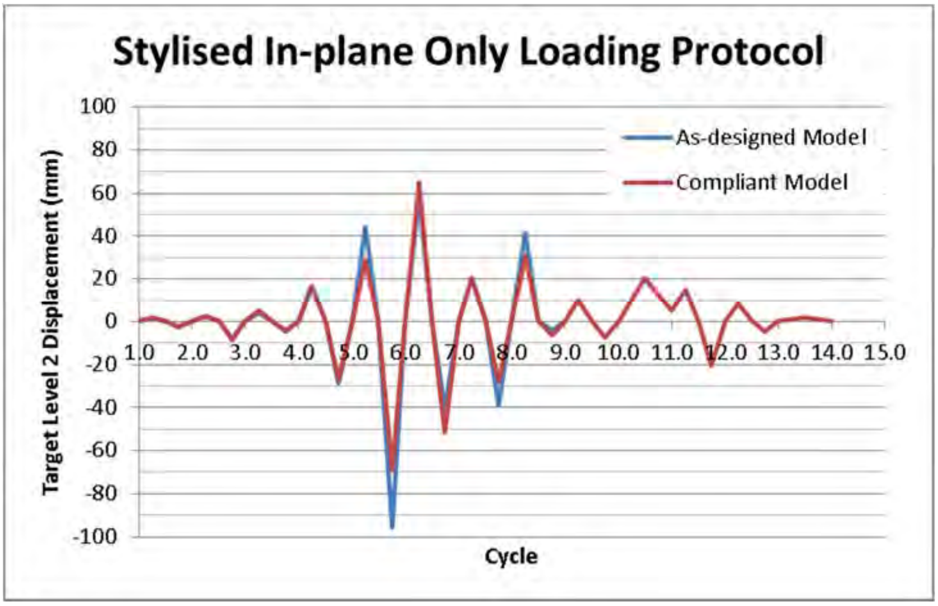


Figure K10: Stylised in-plane loading protocol used for step 1 of the sequence

‘Compliant design/construction errors’ specimen

The objective of this test was to determine if the building was likely to have collapsed in the 22 February 2011 earthquake due solely to the known construction deficiencies. If the specimen loses axial load carrying capacity in loading steps 1, 2 or 3, the conclusion can be made that, due to progressive collapse, the building was likely to have collapsed in the earthquake.

Since this scenario represents the building including all known construction deficiencies, but without any known design deficiencies, a target protocol and displacements in steps 1, 2 and 3 above were derived from a NLTHA of a computer model that included compliant (stronger) shear walls – termed the ‘Compliant Model’.

The loading protocol for step 1 is also shown in Figure K10. The Compliant Model protocol is similar to the As-designed Model protocol, but has a lower peak displacement (69mm versus 95mm).

The target displacements for step 2 were:

- Maximum positive out-of-plane displacement = +42mm
- Maximum negative out-of-plane displacement = -29mm

The target displacements for step 3 were:

- Worst-case positive biaxial displacements = +65mm in-plane and +42mm out-of-plane
- Worst-case negative biaxial displacements = -69mm in-plane and -26mm out-of-plane

Code-compliant specimen

The objective of this test was to determine if the building was likely to have collapsed in the 22 February 2011 earthquake even if it was fully compliant to the codes of the day. If the specimen loses axial load carrying capacity in loading steps 1, 2 or 3, the conclusion can be made that, due to progressive collapse, the building was likely to have collapsed in the earthquake.

Since this scenario represents the building in the absence of all known construction and design deficiencies, a target protocol and displacements in steps 1, 2 and 3 above were derived from a NLTHA of a the Compliant Model – i.e. the same as reported above for the 'Compliant design/construction errors' specimen.

K.3 Results

The following section presents the results of the three specimen tests. Interpretation of the results is provided in the following section.

K.3.1 As-designed specimen

The As-designed specimen lost axial load carrying capacity suddenly towards the end of cycle 5 in step 1 of the loading sequence, at a Level 2 displacement of approximately 95mm. The test stopped at this point, and the remainder of step 1, and steps 2 to 4 were unable to be carried out.

At the point of loss of axial load carrying capacity, the damage was most pronounced towards the bottom of the lower column, just above ground level (Figure K11). The damage in this location can be characterised as consisting of a steep diagonal failure plane (Figure K12a) with significant vertical and lateral displacement. The vertical drop along this failure plane led to the specimen's beams coming to rest on the 'catch-frames', which prevented any further downwards movement. There was some spalling of cover concrete at the top of the lower column just below the beam soffit (Figure K12b). There was also damage to the visible exterior of the beam-column joint region (Figure K13). It should be noted that the visible exterior of the joint zone shows any cracks manifesting in the unreinforced 'wings' of the ends of the precast beams – these cracks do not necessarily match what has manifested in the more-important in-situ core of the joint.

Figure K11: Overview of As-designed specimen following failure at approximately 95mm in-plane Level 2 displacement

a) Bottom of lower column

b) Top of lower column

Figure K12: Damage to As-designed specimen following failure at approximately 95mm in-plane Level 2 displacement

Figure K13: Joint exterior damage following failure of As-designed specimen at approximately 95mm in-plane Level 2 displacement

Figure K14 is a full force-displacement history of the test, where applied force is the load in the in-plane actuator at Level 2.5, and displacement is that measured at Level 2.

The plot shows the excursions out to the displacement peaks corresponding to the cycles before failure. The specimen degraded in lateral stiffness from about 15mm displacement from vertical, with peak applied lateral force (140kN) occurring at a displacement of approximately 60mm. Applied lateral force began to reduce as lateral displacement increased beyond 60mm, at an ever increasing rate. At approximately 95mm in-plane displacement, the specimen lost axial (and lateral) load carrying capacity.

A measured displacement of 95mm corresponds to an inter-storey drift of 2.6%, not including foundation rotation. The peak in-plane only displacement demand in step 1 was 95mm.

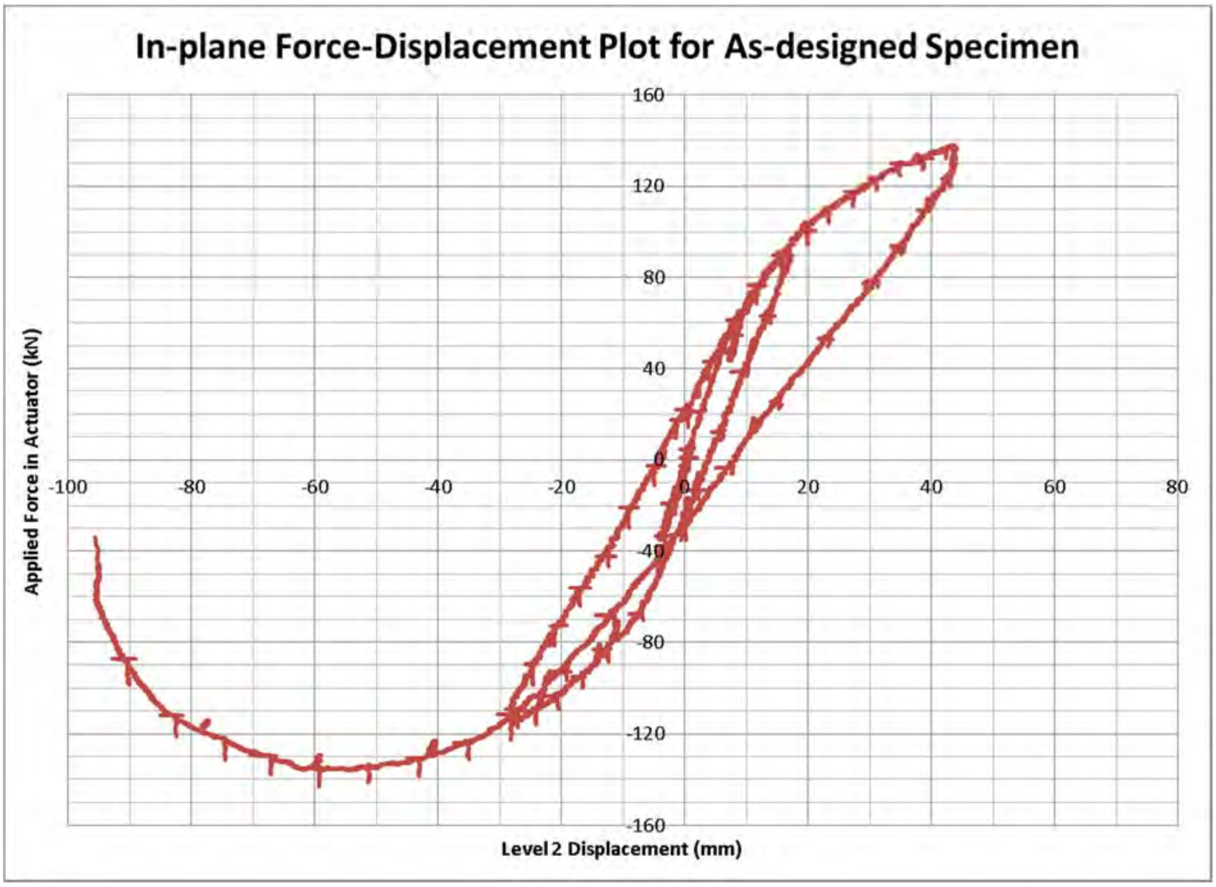


Figure K14: Force-displacement plot for As-designed specimen

K.3.2 'Compliant design/construction errors' specimen

The 'Compliant design/construction errors' specimen maintained axial load carrying capacity throughout steps 1 to 3 of the loading sequence. By the end of step 3, the specimen was exhibiting damage as follows:

- Near the bottom of the lower column – significant cracking and the onset of cover spalling (Figure K15, note: cracks artificially enhanced by marking)
- Near the top of the lower column - significant cracking and the onset of cover spalling (Figure K16, note: cracks artificially enhanced by marking)
- Over the extent of the beam-column joint – significant cracking to the visible exterior (Figure K16, note: cracks artificially enhanced by marking). Again, it should be noted that the visible exterior of the joint zone shows any cracks manifesting in the unreinforced 'wings' of the ends of the precast beams – these cracks do not necessarily match what has manifested in the more-important in-situ core of the joint.

Figure K15: ‘Compliant design/construction errors’ specimen bottom of lower column damage at end of step 3 (max applied in-plane drift of 69mm)

Figure K16: ‘Compliant design/construction errors’ specimen top of lower column/joint damage at end of step 3 (max applied in-plane drift of 69mm)

Step 4 of the loading sequence was then commenced, and the specimen lost axial load carrying capacity suddenly at a Level 2 in-plane displacement of approximately 75mm.

At the point of loss of axial load carrying capacity, the damage was similar to the As-designed specimen, being most pronounced at the bottom of the lower column, with some spalling of cover concrete at the top of the lower column and cracking to the visible exterior of the beam-column joint region (Figure K17).

The damage at the bottom of the lower column was again characterised by a steep diagonal failure plane (Figure K18), similar to the As-designed specimen.

Figure K17: Overview of 'Compliant design/construction errors' specimen following failure at approximately 75mm in-plane Level 2 displacement

[redacted]

Figure K18: 'Compliant design/construction errors' specimen bottom of lower column damage following failure at approximately 75mm in-plane Level 2 displacement

Figure K19 is a full force-displacement history of the test, where applied force is the load in the in-plane actuator at Level 2.5, and displacement is that measured at Level 2.

The plot shows the excursions out to the displacement peaks corresponding to the cycles before failure. The 'Compliant design/construction errors' specimen exhibited similar hysteretic behaviour to the As-designed specimen with increasing rate of softening of the specimen up to the peak applied lateral force (150-160kN) occurring at a displacement of approximately 60mm. Also, with each significant cycle, the specimen became softer as observed by the progressive flattening of the pre-peak line.

Again, applied lateral force began to reduce as lateral displacement increased beyond 60mm. During step 4, the specimen lost axial load carrying capacity at a lateral displacement of approximately 75mm, which was the greatest in-plane displacement the specimen was subjected to during the test.

A measured displacement of 75mm corresponds to an inter-storey drift of 2.2%, not including foundation rotation.

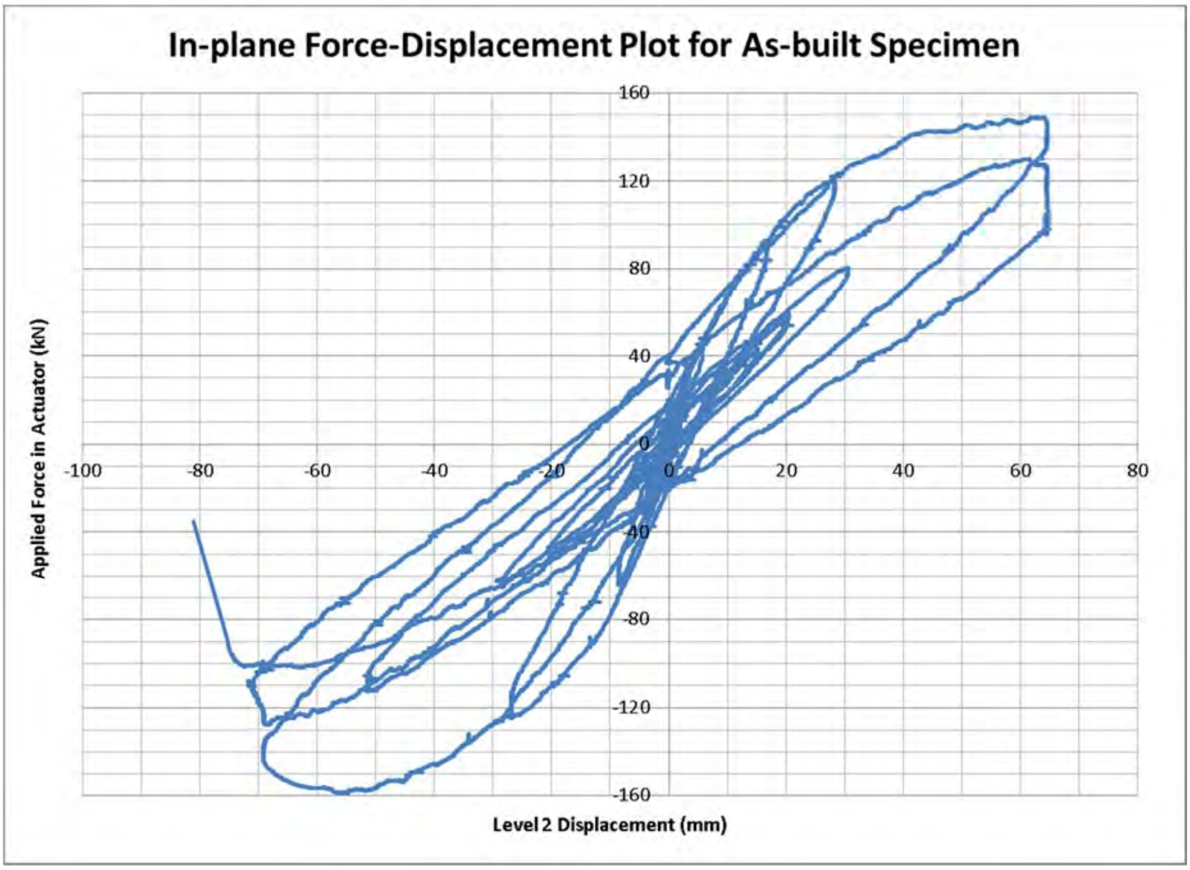


Figure K19: Force-displacement plot for ‘Compliant design/construction errors’ specimen

Another aspect of interest is the degree of damage visible after the specimen had been subjected to drifts approximately representative of the maximum experienced in the September 2010 earthquake. Compusoft’s 2012 report titled *CTV Building Non-Linear Time History Analysis (NLTHA) Report,* prepared for CERC, presents results of NLTHA’s carried out on a model that attempted to represent the in-situ building as closely as possible, subjected to earthquake records from the September 2010 ‘Darfield’ earthquake.

Figure G.5 of the Compusoft report presents east-west inter-storey displacement histories for the frame on Grid 1 in the Darfield event, and shows a maximum inter-storey displacement for any level of approximately 20mm. The region modelled in our physical testing programme is on Grid 2, which is expected to have slightly less drift than Grid 1.

Nevertheless, with reference to Figure K10, we present below photos of the ‘Compliant design/construction errors’ specimen at the end of cycle 4, having been through an excursion to approximately 20mm in one direction, and 30mm in the other (more severe than was likely to have been experienced in the September 2010 earthquake) under the ‘Compliant Model’ regime. Figure K20 shows the top of lower column and joint region, and Figure K21 the bottom of lower column region (note, the dots in Figure K21 are paint marks used in conjunction with high-definition photogrammetry to determine crack patterns). These photos show that there is no visible damage, which suggests this region of the building following the September 2010 earthquake was likely to have also shown no visible signs of damage.

[redacted]

Figure K20: ‘Compliant design/construction errors’ specimen joint region at the end of cycle 4 after a maximum inter-storey displacement of 30mm

Figure K21: ‘Compliant design/construction errors’ specimen bottom of lower column at the end of cycle 4 after a maximum inter-storey displacement of 30mm

K.3.3 Code-compliant specimen

The Code-compliant specimen maintained axial load carrying capacity throughout steps 1 to 3 of the loading sequence. By the end of step 3, the specimen was exhibiting minimal damage, with some minor hairline cracking visible at the top and bottom of the lower column and to the visible exterior of the joint (Figure K22, note: cracks artificially enhanced by marking). Again, it should be noted that the visible exterior of the joint zone shows any cracks manifesting in the unreinforced 'wings' of the ends of the precast beams – these cracks do not necessarily match what has manifested in the more-important in-situ core of the joint.

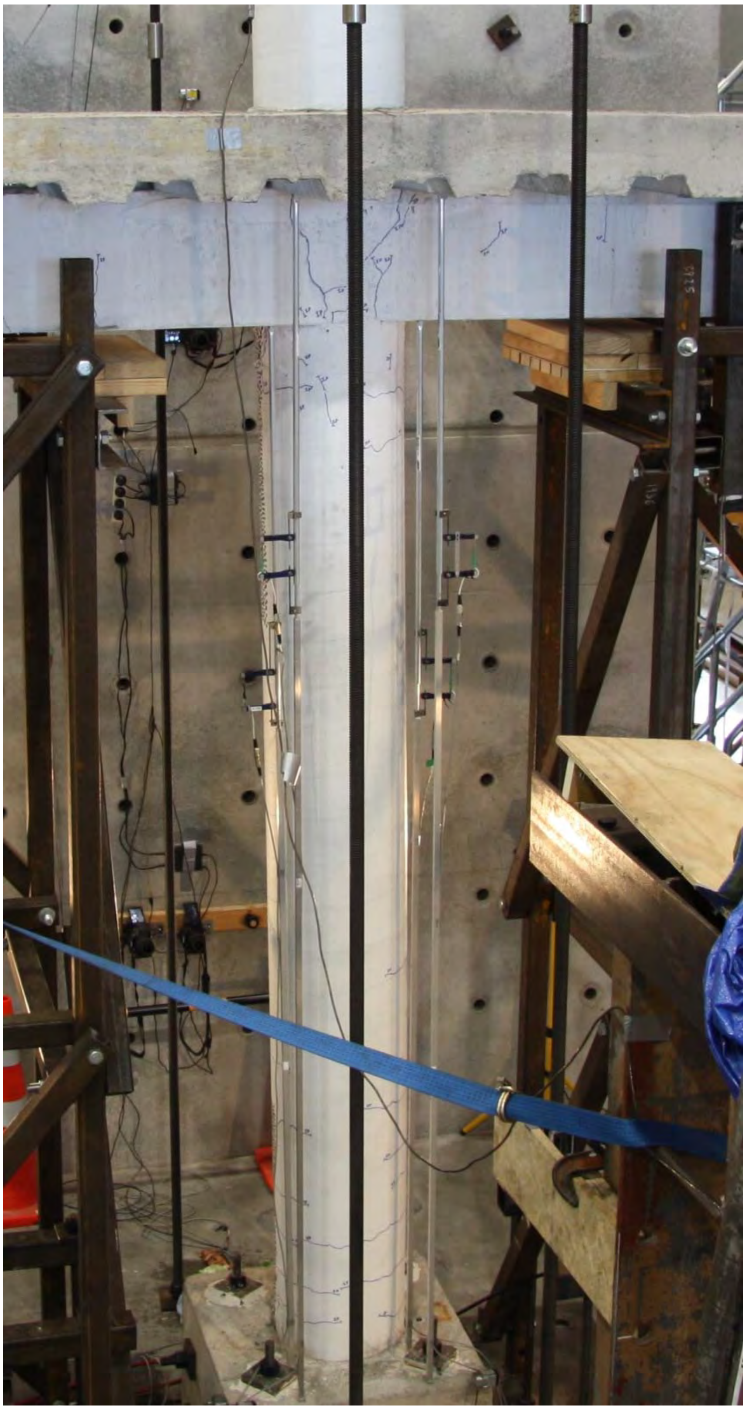


Figure K22: Code-compliant specimen at the end of step 3 of the loading sequence (max applied in-plane drift of 69mm)

Step 4 of the loading sequence was then commenced, and the specimen lost axial load carrying capacity at a Level 2 in-plane displacement of approximately 150mm.

At the point of loss of axial load carrying capacity, the damage was markedly different to the other two specimens, being more evenly distributed between the top and bottom regions of the lower column, with a slight bias towards more damage at the top (Figure K23).

The damage at the top of the lower column had more of an hourglass shape than the failure regions of the other two specimens (Figure K24). However, steep diagonal planes are still visible and it was along one of these that the specimen failed, losing its axial load carrying capacity.

Figure K23: Overview of Code-compliant specimen following push to failure at an in-plane displacement of 150mm

Figure K24: Top of lower column of Code-compliant specimen following push to failure at an in-plane displacement of 150mm

Figure K25 is a full force-displacement history of the test, where applied force is the load in the in-plane actuator at Level 2.5, and displacement is that measured at Level 2.

The plot shows the excursions out to the displacement peaks corresponding to the cycles before failure. The Code-compliant specimen exhibited similar hysteretic behaviour to the other specimens with increasing rate of softening of the specimen up to the peak applied lateral force (140-150kN) occurring at a displacement of approximately 60mm. Also, with each significant cycle, the specimen became softer as observed by the progressive flattening of the pre-peak line.

Again, applied lateral force resistance to reduce as lateral displacement increased beyond 60mm. During step 4, the specimen lost axial load carrying capacity at a lateral displacement of approximately 150mm, which was the greatest in-plane displacement the specimen had seen during the test.

A measured displacement of 150mm corresponds to an inter-storey drift of 4.2%, not including foundation rotation.

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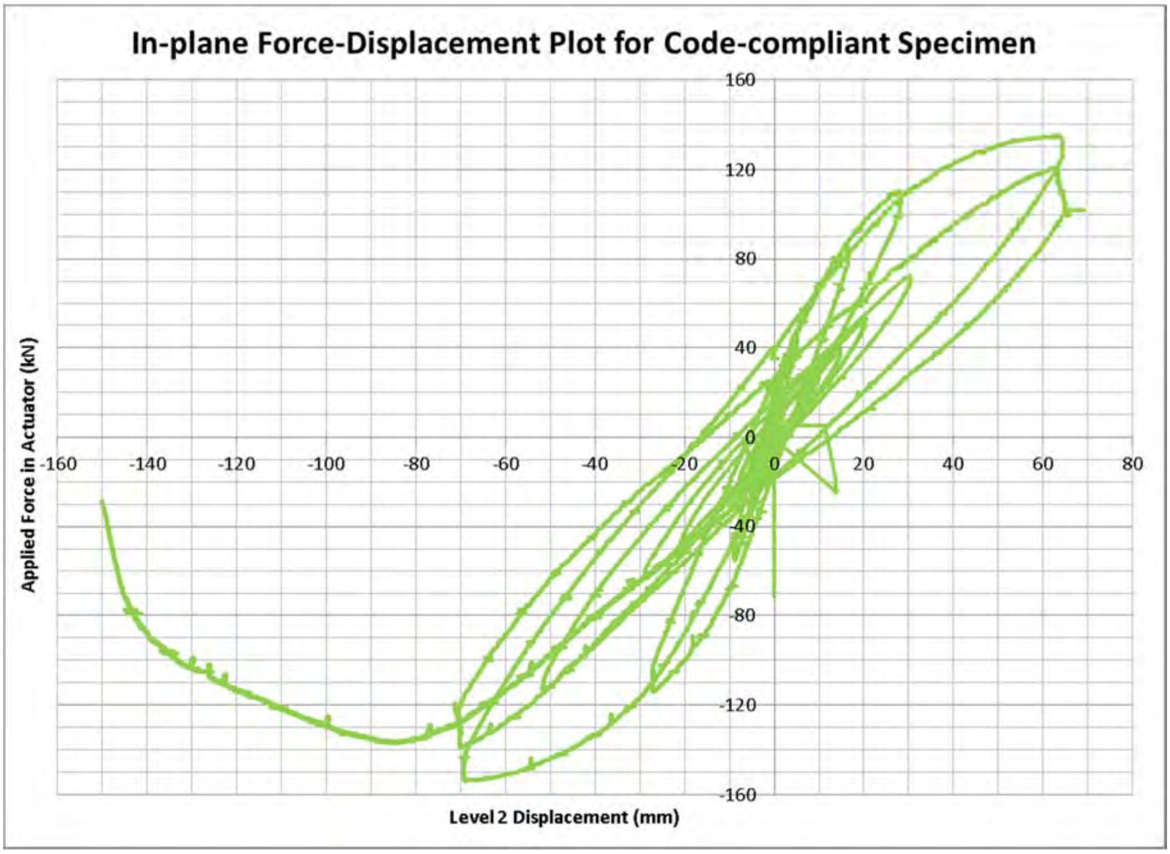

Figure K25: Force-displacement plot for Code-compliant specimen

Figure K26 presents the in-plane force-displacement plots from each of the three tests on the same plot for comparison. The figure shows that the hysteretic behaviour of all three specimens is comparable, with similar slopes in the early cycles and similar levels of softening in later cycles. The figure also illustrates the similar level of peak applied lateral force in both directions, and for all three specimens (varying between 135 to 160kN). In addition, the figure shows the relatively superior performance of the Code-compliant specimen in terms of displacement capability compared to the As-designed and 'Compliant design/construction errors' specimens.

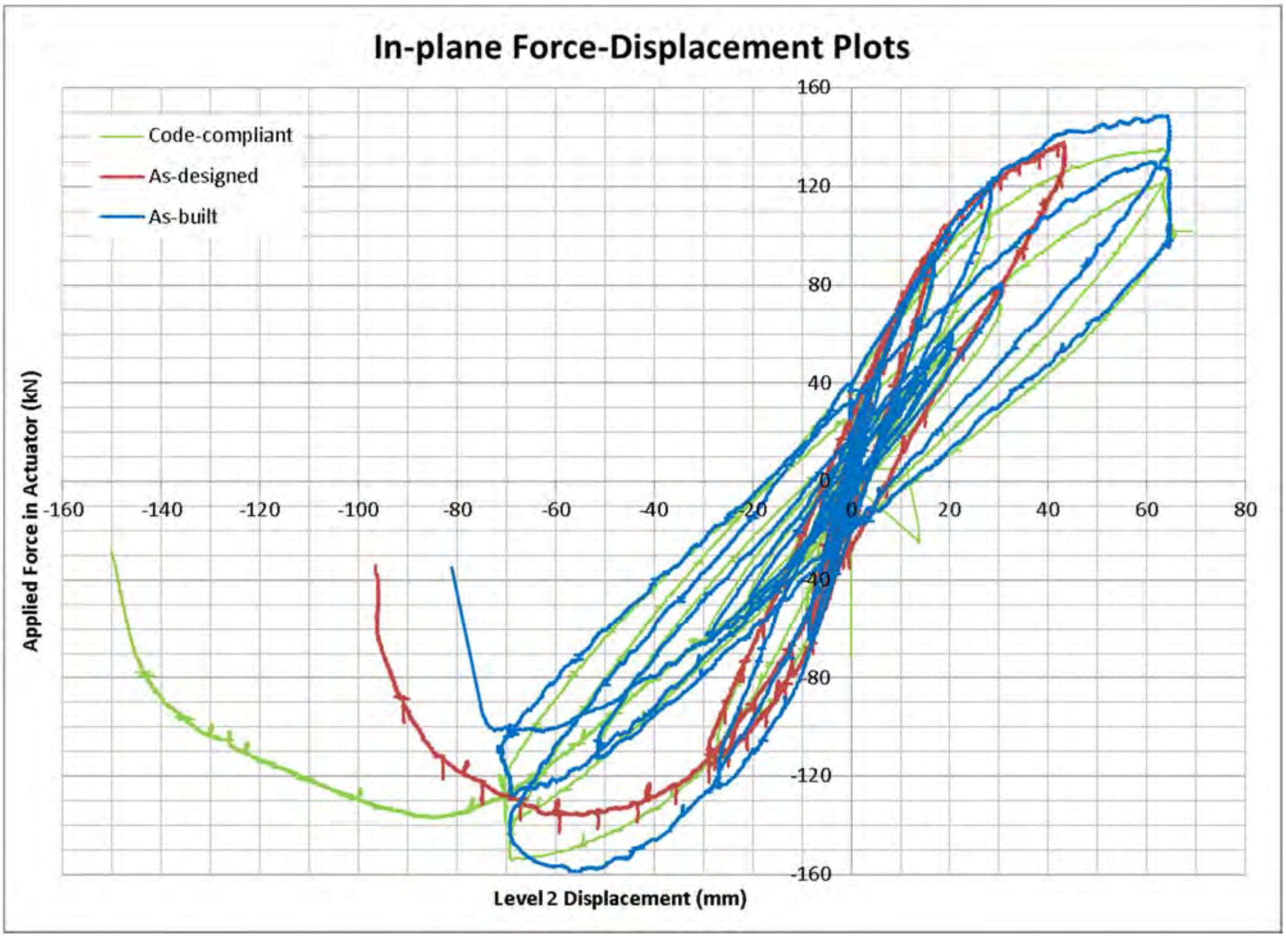

Figure K26: Comparison of force-displacement plots for the three specimens

K.4 Interpretation

The following section presents interpretations of the results of each test, and the implications of the results on our conclusions on cause of collapse involving the relative contributions of previously identified factors such as design and construction deficiencies.

K.4.1 As-designed

As outlined in Section K.3, the As-designed specimen lost axial load carrying capacity during step 1 of the loading sequence at an inter-storey drift in the ground floor of 2.6% (not including foundation rotation).

The specimen lost axial load carrying capacity during the largest cycle of step 1 of the loading sequence. Step 1 of the loading sequence represents a less severe loading scenario than is likely to have been experienced by the region in the 22 February 2011 earthquake since the loading was in-plane only, and did not contain the concurrent out-of-plane component that our analyses have indicated occurred during the earthquake, and that would have been represented by step 3 of the loading sequence (i.e. three-dimensional effects).

The testing indicates that it is likely that the region of the building under investigation would have lost axial load carrying capacity in the 22 February 2011 earthquake if it was constructed as per the design documentation. Our progressive collapse analyses have indicated that if this region lost axial load carrying capacity, it is likely that, due to redistribution, the primary gravity structure would progressively lose axial load carrying capacity leading to pancaking collapse.

This result is consistent with our conclusion that the design deficiencies were a substantial and operating cause of the collapse, and therefore the resulting deaths.

K.4.2 'Compliant design/construction errors'

As outlined in Section K.3, the 'Compliant design/construction errors' specimen lost axial load carrying capacity during step 4 of the loading sequence, at an inter-storey drift in the ground floor of 2.2% (not including foundation rotation). This level of drift is similar to that at which the As-designed specimen lost axial load carrying capacity (2.6%).

The 'Compliant design/construction errors' specimen was subjected to a less severe loading regime in step 1 than the As-designed specimen (due to being considered as part of an otherwise compliant building including stronger shear walls), which it survived without loss of axial load carrying capacity. As a result, it was subjected to more cycles than the As-designed specimen before failure, leading to more degradation in the yielding areas (such as the bottom of the lower column and the joint). This could be an explanation for the slightly lower drift capacity compared to the As-designed specimen (2.2% versus 2.6%).

In addition, the combined effect of steps 1 to 3 of the loading sequence represents a more severe loading scenario than is likely to have been experienced by the region in the 22 February 2011 earthquake (i.e. the testing involved more cycles).

Since the specimen did not lose axial load carrying capacity during steps 1 to 3 of the loading sequence, the testing indicates that it is likely that the region of the building under investigation would not have lost axial load carrying capacity in the 22 February 2011 earthquake if it was contained within an otherwise compliant building.

This result is consistent with our conclusion that the construction errors on their own were not a substantial and operating cause of the collapse, and therefore the resulting deaths.

K.4.3 Code-compliant

As outlined in Section K.3, the Code-compliant specimen lost axial load carrying capacity during step 4 of the loading sequence, at an inter-storey drift in the ground floor of 4.2% (not including foundation rotation).

Since the specimen did not lose axial load carrying capacity during steps 1 to 3 of the loading sequence, the testing indicates that it is likely that the region of the building under investigation would not have lost axial load carrying capacity in the 22 February 2011 earthquake if it was contained within a compliant building. Furthermore, due to the large drift capacity observed during testing, it is also likely that the region of the building under investigation would not have lost axial load carrying capacity in the 22 February 2011 earthquake even if the surrounding building was non-compliant (i.e. contained the as-designed shear walls which were weaker than the code required).

This result is consistent with our conclusion that the design deficiencies were a substantial and operating cause of the deaths, because if the building was compliant (even potentially just in the beam-column joints) it was likely to have survived the 22 February 2011 earthquake.

K.4.4 Summary

The main aim of the testing programme was to assess the effect that the amount of beam-column joint horizontal reinforcement (the 'independent variable') has on the lateral displacement capacity (while maintaining axial load carrying capacity) of the subassembly under cyclic loading (the 'dependent variable').

[redacted]

It is recognised that there is uncertainty in the results of any physical testing programme, especially if only one of each specimen is being tested. In that respect, it cannot be completely discounted that the performance of one or more of the specimens was significantly affected by an unexpected phenomena, as opposed to being directly attributable to the variable being investigated (the amount of beam-column joint horizontal reinforcement).

Nevertheless, considering the measures taken to achieve consistency in the control variables (e.g. construction, material properties, gravity loading etc.) of each test, we believe the test results support other conclusions we have reached regarding the cause of collapse of the building.

Overall, we consider that the Code-compliant specimen's performance relative to the tested performance of the other two specimens indicates the potential (and perhaps significant) beneficial effect on the lateral displacement capacity of an increased amount of beam-column joint horizontal reinforcement.

The maximum drift that was sustained in the Code-compliant specimen test was well above that which was predicted to have occurred in the CTV building during the 22 February 2011 earthquake, whereas the level of drift at which the As-designed and 'Compliant design/construction errors' specimens failed was within that predicted to have occurred. This provides further support that it was the lack of compliant joint reinforcement that made the building vulnerable to 22 February 2011 shaking.

The relative performance of the As-designed and 'Compliant design/construction errors' specimens supports the view that the construction deficiencies (lack of roughening of beam end construction joints and absence of any joint reinforcement) had little effect on the expected drift capacity. This supports a conclusion that the construction deficiencies (on their own) were not significant contributors to the collapse.