Under **THE COMMISSIONS OF INQUIRY ACT 1908**

In the matter of the **CANTERBURY EARTHQUAKES ROYAL COMMISSION OF INQUIRY INTO THE COLLAPSE OF THE CTV BUILDING**

CLOSING SUBMISSIONS
FOR ALAN REAY CONSULTANTS LIMITED AND DR REAY

Dated 3 September 2012

BUDDLEFINDLAY
Barristers and Solicitors
Christchurch

Solicitor Acting: **Willie Palmer / Kelly Paterson**
Email: kelly.paterson@buddlefindlay.com
Tel 64-3-379 1747 Fax 64-3-379 5659 PO Box 322 DX WP20307 Christchurch 8140

Counsel Acting: **H B Rennie QC**
Harbour Chambers Tel 64-4-4992684 Fax 64-4-4992705 PO Box 10242 Wellington

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MAY IT PLEASE THE COMMISSIONERS:

Background

1. Alan Reay Consultants Limited ("**ARCL**") and Dr Alan Reay as affected parties have both taken an active role in this hearing. Extensive expert and other evidence has been adduced to assist in the Commission's investigation of the catastrophic and tragic collapse of the CTV Building.

2. ARCL was incorporated in 1988. Dr Reay was founding director. Dr Reay remains on the board of ARCL and in practice as a structural engineer. Prior to ARCL Dr Reay practiced as Alan M Reay, Consulting Engineer ("**ARCE**").

3. In 1986, ARCE undertook the structural design for an office building project at 249 Madras Street, which ultimately became the CTV Building. A senior engineer, (ARCE's sole Registered Engineer employee, and an Associate from June 1986), Mr David Harding was responsible for the work. Construction was completed in 1987 but the building remained unoccupied for several years, with the developer going into receivership.

4. With the exception of a period in 1990-1991 when remedial works were designed to address a structural weakness that had then been identified, Dr Reay, ARCE and ARCL had nothing further to do with the CTV Building prior to its collapse on 22 February 2011.

5. As stated in opening, Dr Reay and those who work with him were shocked and distressed to learn of the building's collapse. Dr Reay has expressed his sincere condolences to the families and friends of those who lost their lives. Dr Reay has apologised to the families and friends of the victims that the CTV Building failed to meet his personal standards. While there are no words that can adequately address the grief of those who lost a loved one, Dr Reay has sought that the Royal Commission and his and ARCL's involvement in it, will help provide answers to what occurred.

Canterbury Earthquakes Royal Commission

6. Commissions of Inquiry and Royal Commissions are empowered by the Commissions of Inquiry Act 1908 ("**Act**"). Royal Commissions are created by Warrant; statutory Commissions by Order in Council. Royal

Commissions often, as here, are established for major accidents or disasters.[1]

7. Important principles about Commissions of Inquiry, supplementing the Act, have been developed by the Courts. We have been guided by such decisions in these submissions.

8. The Canterbury Earthquakes Royal Commission was established by the Governor General on behalf of the Sovereign by Terms of Reference dated 11 April 2011 ("**TOR**").

9. The TOR identify the CTV Building as one of four buildings specifically to be the subject of the Commission's investigations, together with a representative sample of other buildings. Hearings in respect of the other buildings have already been held and reports issued.

10. Relevant to the CTV hearing, the TOR require the Commission to inquire into and report on:

    (a) why the CTV Building failed severely;

    (b) why the failure of the CTV Building caused extensive injury and death;

    (c) why the CTV Building differed from other buildings in the extent to which:

        (i) it failed as a result of the Canterbury earthquakes; and

        (ii) its failure caused injury and death;

    (d) the nature of the land associated with the CTV Building and how it was affected by the Canterbury earthquakes;

    (e) whether there were particular features of the CTV Building (or a pattern of features) that contributed to whether the building failed, including (but not limited to):

        (i) the age of the building;

        (ii) the location of the building;

        (iii) the design, construction, and maintenance of the building; and

[1] Department of Internal Affairs; *Setting up and Running Commissions of Inquiry*, 2001, page 16, for example: Strongman Mine Disaster, Cave Creek

(iv) the design and availability of safety features such as escape routes; and

(f) whether the CTV Building (as originally designed and constructed and, if applicable, as altered and maintained) complied with earthquake-risk and other legal and best-practice requirements (if any) that were current:

(i) when it was designed and constructed; and

(ii) on or before 4 September 2010;

(g) whether, on or before 4 September 2010, the CTV Building had been identified as "earthquake-prone" or was the subject of required or voluntary measures (for example, alterations or strengthening) to make the buildings less susceptible to earthquake risk, and the compliance or standards they had achieved[2]; and

(h) the nature and effectiveness of any assessment of the CTV Building, and of any remedial work carried out on it, after the 4 September 2010 earthquake, or after the 26 December 2010 (or Boxing Day) aftershock, but before the 22 February 2011 aftershock.

11. It is submitted that in summary three key areas require consideration in the case of the CTV Building:

(a) The cause or causes of the collapse;

(b) Whether the design and/or construction of the CTV Building contributed to the collapse; and

(c) The nature and effectiveness of the inspections of the CTV Building post the 4 September and Boxing Day 2010 earthquakes.

Context

12. In considering the evidence received by the Commission four important factors need to be borne in mind at all times:

(a) Exceptional earthquakes: The earthquakes which were the primary cause of all the matters for investigation were of such a nature, location, force, and direction as to impose demands on Christchurch

[2] It is common ground that the CTV Building was not earthquake prone and therefore this point is not considered further.

buildings which had not been foreseen, nor specifically designed to meet. While a structural designer is expected to provide for all situations (whether anticipated or not) the 4 September 2010 earthquake was unprecedented in its effects and location proximate to Christchurch (arising from the previously unknown Greendale Fault); the 22 February earthquake arose from a different fault, in a different direction, and with a proximity to the centre of Christchurch which gave it devastating effect. None of the engineers who had any connection with the CTV Building at any time in its pre-22 February history anticipated this.

(b) 1986 knowledge: Between 1986 and 22 February 2011, there have been major advances in knowledge of earthquakes, in structural design, in building materials, and in knowledge of their characteristics, and especially in computer modelling and verification of structural design. Each event which occurred is to be assessed in terms of the knowledge at the time. In the case of the 1986 events the TOR make this expressly clear.

(c) Information gaps: Despite all efforts, the information available to the Commission is incomplete. The gaps that exist are not to be filled by guesswork or by speculation. Their proper relevance is in the way in which they affect the ability of the Commission to achieve a level of confidence about each finding that it is to make.

(d) Inquisitorial approach: The essential purpose of a Commission is to obtain information, and accordingly its function and mode of operation are essentially inquisitorial and informal, as distinct from the adversarial and formalised procedures appropriate to a Court or judicial tribunal.[3] The Privy Council has emphasised the distinction between litigation and Inquiries:[4]

> An investigative inquiry into facts by a tribunal of inquiry is in marked contrast to ordinary civil litigation the conduct of which constitutes the regular task of High Court Judges in which their experience of the methodology of decision-making on factual matters has been gained. Where facts are in dispute in civil litigation conducted under the common law system of procedure, the Judge has to decide where, on the balance of probabilities, he thinks that the truth lies as between the evidence which the parties to the litigation have thought it to be in their respective interests to adduce before him. He has no right to travel outside

[3] *Re Royal Commission on Thomas Case* - [1980] 1 NZLR 602 page 626 (FC)
[4] *Re Erebus Royal Commission; Air New Zealand Ltd v Mahon* - [1983] NZLR 662 at page 666 (PC)

> that evidence on an independent search on his own part for the truth; and if the parties' evidence is so inconclusive as to leave him uncertain where the balance between the conflicting probabilities lies, he must decide the case by applying the rules as to the onus of proof in civil litigation. In an investigative inquiry, on the other hand, into a disaster or accident of which the Commissioner who conducts it is required, as the Judge was in the instant case, to inquire into and to report upon "the cause or causes of the crash", it is inevitable, particularly if there are neither survivors or eye-witnesses of the crash, that the emergence of facts, and the realisation of what part, if any, they played in causing the disaster and of their relative importance, should be more elusive and less orderly, as one unanticipated piece of evidence suggests to the Commissioner, or to particular parties represented at the inquiry, some new line of investigation that it may be worthwhile to explore; whether, in the result, the exploration when pursued leads only to a dead end or, as occurred in one particular instance in the present case, it leads to the discovery of other facts which throw a fresh light on what actually happened and why it happened.

13. In an Inquiry there is not a "burden of proof" on any party and while the standard of proof is a civil standard (since the essential purpose is to obtain information, not prosecute a charge); it is for the Commission to determine the level of confidence it must hold about facts when making a factual finding.

14. Great care is needed at all times to ensure that the reliability of each matter presented in evidence is weighed and assessed. This is especially so when much of the evidence relates to a time more than twenty-five years ago, when some witnesses are not available and some documents no longer exist or cannot be found. In particular, evidence from a witness who has no recollection but asserts that something "must have" happened is of little or no value unless there is other evidence tending to corroborate it. Hearsay evidence is similarly of very little probative value. Where an orally reported memory is contradicted by a contemporary written record, the latter will normally prevail.

15. In summary, it is emphasised that the 1986 events and design work are to be assessed against the knowledge, information available and the practices adopted at the time of the design, and not with perfect hindsight vision. It is wrong to apply a hindsight judgment to the events leading to the 22 February 2011 earthquake. That earthquake was unprecedented and unexpected in New Zealand in terms of its size and force. As a result, territorial authorities around the country are assessing thousands of buildings within their respective territories, leading to buildings being taken out of use and/or subject to upgrade requirements. Christchurch did not have the benefit of such a warning.

16. A Commission of Inquiry does not determine liability. In this instance, the TOR state that expressly.[5] The Oxford English Dictionary (2009) gives as the primary meaning of liability "*the condition of being liable or answerable by law or equity*". Accordingly findings of fact do not extend to include findings as to the legal consequences of those facts.

ARCL's approach to the issues

17. Dr Reay and ARCL have sought to identify the truth as to what caused the collapse of the CTV Building.

18. The response of Dr Reay and ARCL to the building's collapse has been to investigate and understand what happened, regardless of the outcome. Despite initial rebuff from the Department of Building and Housing ("**DBH**"), they have remained committed to this approach throughout.

19. Dr Reay and ARCL have dedicated substantial internal and external resources to this. A comprehensive response to the draft DBH Building Collapse Report was made. A comprehensive concrete testing programme in the USA was undertaken. World recognised experts in several fields have been engaged. The equivalent of one full time in-house engineer was committed to provide technical and general assistance to this work.

20. Every effort has been made to complete that work to a high forensic standard, independently, and with no pre-determined outcome. A particular focus has been to put the issues in the hands of independent experts and be guided by them.

Limitations on scope

21. It is unfortunate that the task for the Royal Commission investigating the causes of the collapse of the CTV Building is more difficult than it might have been.

22. The task was always going to be a difficult one. The passage of time since the building was designed and constructed, the almost total collapse of the structure, the associated fire and somewhat inconsistent eyewitness accounts compound to make the task independently difficult. However, a number of events make the task even more difficult.

[5] Terms of Reference, *Exclusions from inquiry and scope of recommendations, (a)*

23. One of these, it is submitted, is approach ultimately taken in the Building Collapse Report prepared by Dr Hyland and Mr Smith for the DBH ("**DBH report**").[6] The DBH report was released well prior to the commencement of this hearing. It went beyond gathering facts, and identifying any immediate responses needed from Government, to assert a definitive statement on the causes of the collapse of the building. It has, however, been shown to be inadequate and incorrect in material respects, and many of its conclusions as to causes require review. Some were contradicted by its own Expert Panel members. The outcome has been that the responses to the DBH report have involved great use of valuable resources that could better have been employed.

24. The Royal Commission has stated, before and at this hearing, that its inquiry is not limited by and quite separate to the DBH inquiry.[7] Nonetheless, considering and dealing with key shortcomings in the DBH report has been a significant obstacle to affected parties, and particularly ARCL and its experts.

25. There have been other obstacles. There was a regrettable failure to preserve evidence and the site as a whole. Whilst Messrs Frost and Heywood did an admirable job in difficult circumstances, much of their good work was undone in subsequent stages, particularly by the removal of debris from the site and the destruction of the north core tower. As a result, it will never be known whether, for example, there was differential subsidence of the foundations; or what occurred at the lower part of the south shear wall. Professor Shepherd detailed how a proper forensic investigation is to be undertaken. The CTV Building investigation is under-informed in important respects. The suggestion of Counsel Assisting that guidelines for best practice structural failure investigations would be of assistance in New Zealand[8] is endorsed.

26. The role of Counsel Assisting has been substantial and has gone beyond the scope of that role as recorded in the Department of Internal Affairs' publication *Setting up and Running Commissions of Inquiry*:[9] The impact of this, as an obstacle to making accurate factual findings, is discussed later.

[6] Building Collapse Investigation Report: BUI.MAD249.0189
[7] TRANS.20120709.1317
[8] TRANS.20120827.CS.10, paragraph 30
[9] Department of Internal Affairs; *Setting up and Running Commissions of Inquiry*, 2001, page 40:
Counsel Assisting are practising lawyers who:
- *advise the Commission on how to interpret its Warrant*
- *liaise with Counsel for parties on matters of procedure*

27. The transcript of evidence from the hearing spans over 4,200 pages. Much evidence has been presented about what people think they would have done, much less about what they are certain was done, and even less corroborated by other evidence. This is understandable given the length of time since the key events and the fact that memories dim. A key task for the Commission, in considering each issue in the TOR, is to identify these different types of evidence and utilise the most reliable evidence in each particular issue.

Credibility

28. As a result of the passage of time since the key events, incomplete records and the serious implications of the events, even though various parties have made legitimate attempts to assist the Commission with their recollection of the events all evidence relating to the period 1986 to 1991 must be taken with a degree of caution.

29. That said, it is submitted that the evidence of some witnesses had a greater air of credibility than others. Mr Harding, in particular, displayed a tendency to remember events only when helpful to his position. During questioning from his counsel in relation to his first brief of evidence, Mr Harding for the first time claimed that whenever he had used the phrases *"it was considered"*, *"it was agreed"* or *"it was determined"*, on each occasion it was Dr Reay's consideration, decision or determination, as if he could recall each occasion clearly.[10] This can only be seen as an opportunistic shift of position from what he first described.

30. In a number of respects his memory was found to be seriously wanting. An important example was the sequence of jobs carried out by Mr Harding. He was, initially, confident that he had completed the design for the Westpark Tower project after the CTV Building design.[11] But later he had to concede that his memory had failed him as the Westpark Tower job was carried out before the CTV job.[12] Despite his unequivocal position on certain events during his examination in chief, he later admitted:

• *ensure all the relevant evidence is brought before the Commission*
• *ensure hearings are conducted in a fair and balanced manner, and*
• *advise the Commission on legal issues throughout the inquiry*

[10] TRANS.20120730.27, lines 3 to 5, 10, 19 to 21; TRANS.20120730.33, lines 20 to 21
[11] TRANS.20120730.85, lines 9 to 15, 22
[12] TRANS.20120731.71, lines 3 to 6

> It's just too long ago. I, I can't, I can't describe how it all worked from a day to day basis.[13]

This is, it is submitted, a more realistic admission.

31. In terms of the structural draughtsmen, there was a tendency on the part of some to seek to avoid any evidence of their extensive involvement in the job. Mr Horn, in particular, was difficult to believe on this issue. His evidence is inconsistent with the contemporaneous timesheets. He (and others) attempted to suggest that the timesheets must be incorrect. However, such a conclusion is contradicted by the documents and other evidence:

(a) The timesheets are a contemporary record of the work carried out at the time, and the times recorded (and the time of the related work) corresponds with the records of the buildings. If they were incorrect then many clients of ARCE's from the time were incorrectly billed, and did not dispute it – a proposition so unrealistic that it must be rejected.

(b) Mr Horn was the most experienced structural draughtsman in the office at the time and it is logical that the CTV job would have been his primary responsibility. Mr Horn had also carried out the majority of the drafting work on the Landsborough House job and was the obvious candidate to have responsibility for draughting the CTV Building. In early correspondence with the Royal Commission, Mr Harding said that the CTV drawings had been prepared by structural draughtsmen specifically recruited from Holmes (being Holmes, Wood, Poole and Johnstone Limited (later Holmes Consulting Group Limited) ("**Holmes**")).[14] This could only have been Mr Horn.

(c) After the documents had been traced, it became almost impossible to ascertain who had carried out the drafting work. This was accepted by Mr Harding.[15]

(d) While Mr Horn may have attempted to avoid the argument in (c) above by suggesting that the drawings did not seem to be in his style, regardless of tracing considerations, Mr Fairmaid later clarified that

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[13] TRANS.20120730.99, lines 1 to 2
[14] BUI.MAD249.0041.RED.1
[15] TRANS.20120730.138, lines 2 to 6

many aspects of the final drawings were *"house style"* and could have been adjusted by tracers in the tracing process.[16]

(e) Both Mr Strachan and Mr Fairmaid accepted that the timesheets were likely to be accurate although they were not able to specifically recall the extent of their involvement.

32. On this basis, contrary to what Counsel Assisting seems to suggest, Mr Horn cannot have been responsible for the foundation design only. Mr Fairmaid's evidence is that the foundation design would not have taken more than 20 to 30 hours of Mr Horn's 141 recorded hours.[17] The timesheets, it is submitted, should be accepted as an accurate contemporaneous record.

33. It is regrettable that Mr Harding and other former employees like Mr Horn have mischaracterised the events in material respects with both direct and indirect criticisms of Dr Reay as a consequence. To correct the record, so that the Commission has accurate facts before it, it has been necessary to identify the credibility gaps and state the correct position.

34. It is proper to also examine the credibility of Dr Reay, but the only challenge came from Counsel Assisting who seemed to confuse their opinion on his demeanour and his approach to his evidence with issues of credibility. When he first gave evidence he visibly struggled to focus and respond – an understandable reaction to finally being able to give evidence. Unlike other witnesses who would say what they “must have” or thought they would have done, when he did not remember, he said so.

35. There has been judicial recognition of his character and abilities on other occasions. Mr Banks' departure from the firm led to litigation and the decision of the High Court (French J) held:

> [7] The business was founded by its namesake, the second defendant, Dr Alan Reay. Dr Reay is one of New Zealand's foremost structural engineers and lead consultants. The evidence established that he is an engineer of exceptional ability whose work has been acclaimed not only in New Zealand but also overseas.[18]

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[16] TRANS.20120815.107, line 25 to TRANS.20120815.108, line 23
[17] TRANS.20120815.89, lines 2 to 6, BUI.MAD249.0463A.1
[18] *Duncan & Ors v Alan Reay Consultants Limited* HC, Christchurch, 1 December 2008, CIV 2006-409-251

36. Earlier, in the report of the Commission of Inquiry into the collapse of a viewing platform at Cave Creek near Punakaiki on the West Coast[19] the Commissioner, Judge Noble, said:

> Dr Reay has high academic qualifications, is a learned theoretician with very sound practical skill and is conservative and careful in his approach. Very substantial weight can be attached to his evidence, which was of great assistance. In cross-examination he demonstrated all the hallmarks of the expert witness, giving careful consideration to questions, providing balanced answers and being prepared to acknowledge that another expert might hold a different opinion.

37. Dr Reay was readily prepared to concede that he may be wrong on matters of factual recall but stood firm where he had the basis to do so.[20] Dr Reay precisely stated what he could and could not remember, and rarely expressed an assumption of what would have happened.

38. Mr Banks, like Dr Reay, put aside past differences to give evidence of similar quality. The visible surprise of Counsel Assisting that Mr Banks had not discussed his evidence with ARCL or its lawyers demonstrates a failure to recognise the professionalism with which each responded. Likewise, Dr Reay (admittedly on advice) did not read the expert evidence in advance of giving his own. Counsel Assisting's claim that Dr Reay's knowledge of Mr Strachan's evidence contradicts this is wrong – Mr Strachan was a factual witness. Dr Reay's knowledge of Mr Latham's evidence reflected the fact that that related to an ARCL project, not to evidence from external experts.

Documentation

39. All parties had incomplete files relating to this building:

    (a) Alun Wilkie had barely any file:

    > I confirm that I have no files at all relating to the design and construction of the building. I do still retain the original architectural plans.[21]
    > ...my files ... have long since been destroyed.[22]

    (b) David Falloon did not know the location of his files:

    > I no longer have my records relating to this matter because after the time of the 22 Feb 2011 earthquake my office was in St Elmo Court building and this has since been demolished due to earthquake damage.[23]

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[19] 1995, Department of Internal Affairs
[20] TRANS.20120801.106
[21] TRANS.20120730.3, line 18
[22] TRANS.20120730.6, line 2
[23] TRANS.20120725.80, line 3

Then, under cross-examination it was shown that in fact his office had shifted from the St Elmo Court building prior to the 22 February 2011 earthquake:

> Q: Does it remain possible therefore that you may in fact still have the files for this piece of work?
> A. That could be possible but I doubt it, I would - if required I could check it.[24]

Counsel Assisting never explained this and, as far as counsel is aware, never followed up on those files (which might well have contained information from Council records not now located by the Council).

(c) The Council records were also incomplete. A memorandum dated 24 July 2012 by Dr Reay and ARCL annexes a schedule of key documents related to the CTV Building and identifies whether those documents had been produced from the files of ARCL and/or the Council. As shown by that schedule, ARCL retained some documents not held by the Council and vice versa. The Council's 1980’s use of microfiche, often disposing of the originals, compounded the problem. It appears to have also affected building inspections at the time.

40. Dr Reay's evidence was that ARCL places emphasis on retaining drawings, calculations and geotechnical reports for significant projects.[25] The documents on ARCL's file for this building were consistent with that policy. No ARCL information relevant to the CTV Building investigation was lost or disposed of after the 22 February 2011 earthquake. All records previously stored on a disc were transferred to a hard drive and all material on the hard drive was made available to the Commission,[26] and (contrary to the unfounded claims of Counsel Assisting) have proved to be the major part of the records that have survived.

History of the building

41. The CTV job came to Williams Construction Canterbury Limited ("**Williams**") as a result of a meeting between Mr Neil Blair of Prime West Corporation Limited ("**Prime West**") and Mr Brooks of Williams.[27] Williams submitted a design build proposal to Prime West which was accepted. Mr

[24] TRANS.20120725.89, line 22
[25] TRANS.20120731.99, line 30 ff
[26] TRANS.20120731.101; see also TRANS.20120801.69
[27] TRANS.20120808.4, lines 9 to 11

Brooks had a clear view on how the building should look, including the lift shaft at the back of the building to allow for maximum rentable space and he had drawn up a preliminary sketch. Tony Scott of Williams was involved as a quantity surveyor to prepare a cost estimate.

42. Counsel Assisting wrongly claim that Dr Reay said in a written brief that he believed David Harding had brought the CTV contract to the firm but said in cross-examination that he no longer held that view. That is not correct. At paragraph 23 of his first brief of evidence Dr Reay said that he was unable to recall how the CTV Building job came to ARCE.[28]

43. Mr Brooks' drawing was given to Alun Wilkie to draw up architectural plans. Mr Brooks was familiar with Alun Wilkie's work from working with him at Industrial Holdings. Mr Wilkie was asked to base the design of the building on the Contours Building which had recently been constructed on Durham Street. Mr Brooks had an objective for the building to be as efficient as possible and provide the maximum rentable space.[29] There was an objective for low-cost basic office space.[30] Features from the Contours Building to be retained included pre-cast spandrel panels and circular columns.

44. Mr Brooks had previously worked with Dr Reay who was the lead consultant on the Aged Persons Welfare Building on Cashel Street which Williams had built.[31] Mr Scott and Williams decided to engage ARCE.[32]

45. Preliminary work was done by Mr Wilkie and ARCE on a no-job, no-fee basis. ARCE was engaged on a design-build basis meaning its sole responsibility was to prepare the structural drawings for the building and undertake some construction observation. Mr Scott's evidence was that ARCE's fee was just under $50,000,[33] but no other witness was able to confirm this. There seems to be a common view that it would have been a fixed price job. ARCE was not responsible for engaging other consultants and therefore was not "lead consultant" on the project.

46. ARCE was a relatively new entrant into this area of structural work. Previously, ARCE's practice had focused mainly on Dr Reay's work in the concrete tilt slab form of construction that Dr Reay and ARCL have become

[28] WIT.REAY.0001.5
[29] TRANS.20120808.4, lines 27 to 29
[30] TRANS.20120808.6, line 30
[31] TRANS.20120808.12, lines 23 to 25
[32] TRANS.20120808.101, lines 1 to 6
[33] TRANS.20120808.113, line 22

renowned for. Other work included developing building systems for Fletcher Brownbuilt, and smaller scale buildings. However, ARCE was facing an increasing number of enquiries from clients looking to design efficient multi-storey office buildings and began to expand into this area. To advance this development in the mid-1980s:

(a) John Henry was recruited as an experienced and senior engineer, intended to advance to Associate. He then did the structural design of several buildings. He worked independently, did not seek assistance from Dr Reay and there are no significant issues known with any of his buildings. Mr Henry joined ARCE in 1984 and left in September 1985, before the CTV Building came to ARCE.

(b) Draughtspeople were recruited with the relevant experience, most notably Terry Horn from Holmes for his experience with high-rise buildings.

47. Mr Henry then left. Mr Harding was known to be looking for a position with this type of work. He wanted to leave Waimairi District Council where he had been for 4 ½ years as leader of the civil engineering team. This followed over 7 years in structural engineering, first at Hardie & Anderson (1973 – 1977), becoming a Registered Engineer in 1976, and at ARCE (1978 – 1980). Dr Reay became aware of Mr Harding's availability and offered him a position at ARCE. Mr Harding agreed to return to ARCE to pursue his interest in multi-storey new buildings and to gain more structural engineering experience. Mr Harding, at this time, had engineering experience several years greater than Mr Henry's.

Gravity / shear wall approach

48. Through the hearing, the CTV Building has been described in various ways, including *"non-descript"*,[34] *"innovative"*,[35] *"revolutionary"*,[36] *"quite simple and straightforward"*.[37] In many cases it was not remembered at all by persons who worked on it, such as Mr Horn and Mr Fairmaid.

49. From the outset, the CTV Building was designed as a gravity/shear wall building. Dr Reay gave Mr Harding the calculations and file for the Landsborough House building which had been designed on this basis by Mr

[34] TRANS.20120815.85, line 24
[35] TRANS.20120723.115, line 23
[36] TRANS.20120723.23, line 6
[37] BUI.MAD249.0005.11

Henry while at ARCE and which were intended to guide Mr Harding through the design of the CTV Building.

50. There is some contention about whether the south shear wall was always a feature of the design. Mr Harding's evidence was that it was added after he did an initial ETABS run which suggested that the building did not work without it and so it was added.[38] Dr Reay's firm recall was that the south shear wall was always on the drawings from the inception of the design.[39] This position was supported by Mr Scott[40] and possibly Mr Wilkie.[41] Certainly Mr Wilkie's drawings showed the wall and these drawings were lodged with the Council some weeks prior to the structural drawings, which followed later.[42]

51. The adoption of a gravity/shear wall approach per se is not to be criticised. Mr Henry unhesitatingly agreed that a building of this type could be constructed using this method[43] and he noted that by then he had been involved in the design of several such buildings himself.[44] These included Landsborough House, Bradley Nuttall, and the Aged People's Welfare Building. Mr Henry's evidence reflects the fact that whereas most multi-storey buildings prior to the 1980s were mostly moment frames, after this time shear-wall stabilised gravity frame systems had become relatively commonplace, at least in Christchurch.

52. Mr Henry also gave evidence that he discussed the Landsborough House structure with Professor Paulay and there was no suggestion that the Professor raised any concern with the proposed gravity/shear wall basis for the design.[45] Mr Henry's evidence was that aside from commenting on the eccentricity of the building and a possible loss in stiffness (which issues Mr Henry stated he had addressed through the ETABS analysis), "*Professor Paulay did not raise any such fundamental issues with regard to Landsborough House*."[46]

[38] TRANS.20120730.118, lines 31 to 32
[39] TRANS.20120801.47, lines 18 to 19; TRANS.20120801.48, lines 28 to 29
[40] TRANS.20120808.106, lines 21 to 28
[41] TRANS.20120815.68, lines 24 to 31; see also TRANS.20120815.74, although Mr Wilkie could not be certain whether the south wall which he believes was always part of the design was a shear wall from the outset.
[42] BUI.MAD249.0284.43 to 51, BUI.MAD249.0141.8
[43] TRANS.20120802.69, lines 4 to 17
[44] TRANS.20120801.122, 124
[45] TRANS.20120801.130 and 132
[46] TRANS.20120801.132, lines 13 to 22

53. Mr Hare described the approach as one he was familiar with and which did not give rise to any concerns in principle on his part.[47] In their 1990 report (discussed below), Holmes referred to the gravity structure as "sound".[48]

54. There is no suggestion from any engineer that the building could not be designed this way. Mr Tapper's notes[49] were about implementation, not whether the design principle could be executed.

Allocation of task to Mr Harding

55. As noted, the task of carrying out the structural design of the CTV Building was allocated to Mr Harding. Dr Reay was involved at a preliminary stage and introduced Mr Harding to the Williams team.[50] Since rejoining ARCE, Mr Harding had already completed the structural design of the Westpark Towers, including a new ETABS analysis which had been started by Mr Henry before he left, and designed a four-storey medical accommodation building.[51] Shortly before he became an Associate the CTV Building job was assigned to him as his project.[52]

56. Mr Harding commenced work on the CTV Building in March 1986. Mr Harding at this particular time had the following skills and expertise:

    (a) Including as noted in his evidence and paragraph 47 above, Mr Harding was an honours graduate from Canterbury University's School of Engineering (May 1973)[53] and a fully qualified Registered Engineer (May 1976) under the Engineers Registration Act 1924. By 1986, Mr Harding was 35 years old, had 7 years experience as a structural engineer at Hardie & Anderson and ARCE from 1973 to 1980, followed by 4 1/2 years as leader of the Waimairi District Council civil engineering team. By 1986, Mr Harding also had 10 years post-registration experience and had applied for and been accepted as a member of New Zealand Institution of Engineers (subsequently becoming a Chartered Professional Engineer under the registration system implemented in 2002).

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[47] TRANS.20120816.96, lines 10 to 22
[48] BUI.MAD249.0130.8
[49] BUI.MAD249.0141.14
[50] TRANS.20120801.43, lines 15 to 24, TRANS.20120808.102, lines 22 to 24
[51] BUI.MAD249.0562.1 (note that Mr Harding started work on the medical accommodation building before officially joining ARCE)
[52] See BUI.MAD249.0562.1 for a list of relevant jobs worked on by Mr Harding prior to and after the CTV job
[53] TRANS.20120730.21, lines 29 to 30

(b) He was engaged at a senior level.[54] It was intended that he would become an Associate of ARCE, a proposition which Mr Harding saw as attractive.[55] He in fact became an Associate while the CTV Building design was underway.[56] Dr Reay confirmed in his evidence that Mr Harding's role was a senior one that he wanted, considered himself qualified for and was entitled to.[57]

(c) The job that Mr Harding took on when he rejoined ARCE was one that he aspired to. He wanted to have the contact with architects, builders and the like.[58] The CTV job, and the associated responsibility was exactly the sort of job that Mr Harding also aspired to. He accepted that this job was a "challenge" that he "wanted to take on"[59] and said that Dr Reay was giving him "the opportunity to do one".[60] Mr Harding was as confident in himself to do this job as Dr Reay was in him.

(d) Mr Harding clearly believed he could do this work. He took the job on and with one exception did not seek assistance. During cross-examination on being taken through each key element of the building, Mr Harding asserted his competence at the time to undertake each key task.[61] He confidently stated that the elements of the structural design were all matters within his skills and expertise noting "there was nothing new".[62] With the exception of the south shear wall, Mr Harding was unable to recall any other occasion where he went to Dr Reay to raise issues or concerns or needed to.[63] Dr Reay was available to Mr Harding if there were specific issues that he wanted to raise.[64] Mr Harding said that he had a high level of confidence that if he followed the Landsborough House work then he could design a good building.[65] Mr Harding had access to the full file for the Landsborough House building. He could also have inspected that building if he wished to do so.

[54] See for example BUI.MAD249.0466.5 where Mr Harding is referred to as a "Senior Engineer"
[55] TRANS.20120731.40, lines 6 to 9
[56] TRANS.20120730.94, lines 12-19; TRANS.20120730.95, lines 12 to 15
[57] TRANS.20120801.3, lines 24 to 28
[58] TRANS.20120730.99, lines 12 to 24
[59] TRANS.20120731.46
[60] TRANS.20120730.81, lines 20 to 21
[61] TRANS.20120730.119 to TRANS.20120730.125
[62] TRANS.20120730.125, lines 17 to 19
[63] TRANS.20120730.130, lines 18 to 30; TRANS.20120730.132; TRANS.20120731.46, lines 7 to 18
[64] TRANS.20120731.48, lines 9 to 18
[65] TRANS.20120731.45, lines 3 to 10

(e) Mr Harding said he was not calling out for supervision or review.[66] Dr Reay confirmed that Mr Harding never communicated any lack of competence to do the job.[67] If he had, the job would not have been taken on.[68]

(f) Mr Harding had taken over and completed the Westpark Tower job with no known issues. Mr Harding was initially under the impression that the Westpark Tower design was completed after the CTV job. He said in his evidence that having run the ETABS analysis for the CTV job he felt that he was in a position to do ETABS on the Westpark job[69] and that he was not suggesting he lacked the necessary skills for the Westpark job.[70] Mr Harding later accepted, having reflected on the documentation, that the calculations for Westpark Tower, including the ETABS work, were done prior to the CTV Building.[71] The inescapable conclusion is that Mr Harding, having completed the ETABS analysis on the Westpark Tower, believed he was in a position to proceed with the CTV Building analysis. He did not make any suggestion that he did not have the necessary skills for the Westpark Tower job.[72] While the analysis on the Westpark Tower building had been started by Mr Henry, Mr Harding redid the ETABS calculations from scratch.[73] Ultimately, the load path and beam column joint defects that have been identified with Mr Harding's design are not related to the ETABS analysis.

(g) Mr Harding had previous experience with the concrete code from his earlier time at ARCE and, in the civil engineering context, at the Waimairi District Council. The work included structural engineering such as bridge analysis and reinforced concrete construction for swimming pools. Equally, his work at Hardie & Anderson would have involved use of the concrete code. Mr Harding accepted that during his earlier time with ARCE, the firm was engaged in the design of concrete structures using the current concrete and loadings codes.[74] Dr Reay's evidence, which has not been challenged, was that at the

[66] TRANS.20120731.83, lines 23 to 27
[67] TRANS.20120815.25, lines 32 to 34
[68] TRANS.0807.97, lines 19-21
[69] TRANS.20120730.104, lines 4 to 5
[70] TRANS.20120730.106, lines 1 to 3
[71] TRANS.20120731.71, lines 3 to 6
[72] TRANS.20120730.106, lines 1 to 4
[73] TRANS.20120801.4, lines 3 to 5; TRANS.20120801.28, lines 21 to 33
[74] TRANS.20120730.90, lines 1 to 3

time he prepared the CTV Building design, Mr Harding was more familiar with the concrete code than Dr Reay was.[75]

(h) Mr Harding had attended key seminars before joining ARCE and in July 1986 he attended an intensive 3 day seminar on "*Design of Concrete Structures*", at a time prior to his signing off on the CTV plans.[76] Mr Harding acknowledged his attendance at the "*Seismic Design of Ductile Moment Resisting Reinforced Concrete Frames*" seminar in May 1979 and a later (May 1982) seminar in geo-mechanics in-situ testing.[77] Mr Harding acknowledged that any issues arising from the 3 day July 1986 seminar presented by Professors Park and Paulay could have been taken into account in the CTV Building design, and that after going to the seminar he was fully informed on the construction issues raised in the papers presented.[78] [79] Mr Harding also recalled that he had attended (with Dr Reay) another seminar relating to eccentrically braced frames.[80]

(i) Mr Harding was able to satisfy Mr Tapper that the concerns raised in his letter of 27 August 1986[81] had been satisfied. Mr Tapper signed the structural approval box on the permit form.[82] Mr Harding apparently attended a meeting with Mr Tapper in relation to the design during the permit stage as he recorded in a later letter.[83] Mr Harding's calculations and plans met Mr Bluck's due diligence standard.

(j) Despite suggesting otherwise in his evidence, it is clear that Mr Harding dealt directly with Williams throughout the design/build project. He fully met their expectations as an engineer. Mr Harding was described as the "principal engineer" for the building by Mr

[75] TRANS.20120731.105, line 26 to TRANS.20120731.106, line 7; TRANS.20120815.58, lines 16 to 34
[76] TRANS.20120730.108, lines 12 to 30
[77] TRANS.20120730.90; TRANS.20120730.93
[78] TRANS.20120730.109, lines 8 to 15 and 22 to 29
[79] The 3 day seminar in 1986 preceded the main structural design of the CTV building by Mr Harding. The written material (BUI.MAD249.0519) included a comprehensive section on beam column joints (BUI.MAD249.0519.73). If Mr Harding had any uncertainty about that aspect of structural design, the papers, the seminar, and access to the presenters should have resolved that. Other guidance was in line with his approach on other aspects. For example the papers refer to "assigning" the lateral load to the shear walls (BUI.MAD249.0519.98 and .131) and records at BUI.MAD249.0519.131 that "*Design procedures were outlined for reinforced concrete buildings in which earthquake resistance is provided entirely by ductile structural walls*." The submission of Counsel Assisting that the gravity frames still contributed to the seismic resistance and should not have been "assumed" not to form part of the lateral load resisting system is thus contradicted. A further statement at BUI.MAD249.0519.99 that "*When resistance close to their ideal strength is required to be developed, most structural walls will be stiff enough, even after extensive flexural and shear cracking, to provide full protection against damage to non-structural contents of the building*" is not specific to gravity frames but is consistent with the assumption in the 1980's that shear wall buildings were stiff enough to protect the gravity load system.
[80] TRANS.20120730.106, lines 16 to 20
[81] BUI.MAD249.0141.14
[82] BUI.MAD249.0141.8
[83] BUI.MAD249.0152.3

Brooks. He was involved from the outset.[84] Similarly, Mr Scott said that he liaised with Mr Harding right from the stage of preliminary structural details.[85] The evidence of those involved in the construction was that Mr Harding was tasked with inspecting and approving all concrete pours, all reinforcing steel in position prior to pouring, inspecting concrete in the columns after the form work had been stripped and verifying and approving concrete dockets.[86] Mr Harding was said to be there "on a regular basis".[87] Mr Scott described Mr Harding as the engineer that Williams principally dealt with during the course of the CTV project. Mr Scott said:

> He attended all the design meetings at Williams' office and signed all letters, document transfer forms and sketch plans on behalf of Alan Reay Consultants [sic]. I considered David Harding to be the principal structural designer of the CTV building. The early A4 structural sketches were done in his handwriting and he signed off all the correspondence as David Harding, Registered Engineer.[88]

Similarly, Mr Harding personally approved and initialled every drawing, satisfying Mr Tapper's requirement for the designer to sign the drawings.[89]

57. Dr Reay has made it clear throughout his evidence that if there is found to be an issue with the structural design of the CTV Building that caused the collapse, he accepts responsibility for such issue or issues, as principal of the firm.[90] Dr Reay made this statement on the first occasion he gave evidence at the hearing. It is therefore unfair of Counsel Assisting to submit that Dr Reay "finally" publically acknowledged that his firm was responsible.[91] He has never suggested otherwise. To the extent that it relates to liability, this issue should not have been raised by Counsel Assisting.

Overview and summary – Mr Harding

58. The evidence therefore is that Mr Harding was a fully qualified, experienced and very competent engineer. He was 35 years old at the time, with 13 years' post-graduate experience and 10 years' post-registration experience.

[84] TRANS.20120808.14
[85] TRANS.20120808.49, lines 25 to 27
[86] TRANS.20120808.60, lines 7 to 9; TRANS.20120808.63, lines 31 to 33
[87] TRANS.20120808.72, lines 27 to 28; TRANS.20120808.88, lines 21 to 29
[88] TRANS.20120808.91, line 29 to TRANS.20120808.92, line 24
[89] BUI.MAD249.0284
[90] TRANS.20120712.132, line 9, TRANS.20120807.94, lines 4 to 5, TRANS.20120815.21, lines 29 to 31. This repeated acknowledgment of responsibility effectively puts paid to the suggestion of Mr Harding's counsel that Dr Reay has "distanced himself" from Mr Harding TRANS.20120815.31, lines 3 to 6
[91] TRANS20120827.CS.6, paragraph 12

The New Zealand Institution of Engineers accorded him the title of 'Registered Engineer' which acknowledged his knowledge and expertise. He had the knowledge and experience, and also the professional requirement to work within his ability and knowledge. He would know the steps he needed to take if he was not doing so: he was either to cease work or obtain the necessary knowledge to complete the work. Mr Harding's claim now that he was not sufficiently experienced is not credible and, on the evidence, not one he would have made in 1986.

59. Clause 6 of the New Zealand Institution of Engineers Code of Ethics (February 1986) provided that a member "shall not misrepresent his competence nor, without disclosing its limits, undertake work beyond it." If Mr Harding knew the CTV Building job was outside the level of his expertise, it was incumbent on him to say so.

60. While he had limited experience in designing multi-storey buildings at the time he came to the CTV job, that is to a large extent irrelevant to the faults that have been identified in the building design. Mr Harding had considerable experience using the relevant codes (NZS4203:1984 and NZS3101:1982). He believed he had a good understanding of the key elements of design under those codes including load paths and ductility. From his time (approximately 4 years) at Hardie & Anderson and previously at ARCE, Mr Harding had experience as a structural engineer in designing both new and existing buildings to code compliance. He worked on the four-storey medical accommodation building before joining ARCE. This was followed by having responsibility for testing a major fibreglass structure and reporting on it to the Christchurch Drainage Board Chief Engineer. He then played a large part in the design of the 9-storey Westpark Tower job, also having responsibility for inspections during the construction phase. This work was all carried out to the complete satisfaction of Dr Reay and the clients.

61. Dr Reay's assessment, given in evidence, was that he was confident in Mr Harding's experience and perceived competence for the job.[92] Dr Reay relied in part on the fact that an important aspect of gaining registration under the Engineers Registration Act is that an engineer knows what he or she does not know and knows how to go and find it out and deal with it.[93] Dr Reay believed Mr Harding was not overconfident and had a good

[92] For example, see TRANS.20120801.3, lines 5 to 11
[93] TRANS.20120801.26, lines 8 to 14

understanding as a structural engineer of the design of structures.[94] Dr Reay also said that he considered Mr Harding to be "*a very competent structural engineer who understood his limitations and knew how to work through them*".[95]

62. Mr Scott also observed that he found Mr Harding to be both confident and competent.[96]

63. Likewise, Mr Harding seems to have been perceived by experienced draughtspeople in the ARCE office to have been competent and appropriately allocated the CTV job. Mr Horn described Mr Harding as a *"conservative engineer"* who *"seemed to produce the right numbers"*.[97] Mr Horn accepted that if he felt the engineer he was working with was not competent he would be pushing back and raising questions and he had no recollection of having to push back to Mr Harding.[98] None of the structural draftsmen, some of considerable experience, raised any concern about the design.

64. Peter Nichols described Mr Harding in the following way:

> I did not know David Harding well, but I do recall him being less dogmatic than Alan Reay, although still assertive. I regarded him as a very competent engineer whose design work I considered to be characterised with elegant simplicity, practicality and economic construction.[99]

65. Mr Harding himself considered he had the necessary skills for the CTV job. He wanted to do it and accepted that he had the necessary skills for each key aspect of the structural design.[100] It is submitted that if Mr Harding was in a position where he felt a review of his work was crucial he would have ensured that review occurred.

A single mysterious error

66. There is therefore a striking contrast, and a mystery, in respect of Mr Harding's CTV design work. No issue whatsoever has been identified with any work Mr Harding did in any part of his career as an engineer, before the CTV work, or in the 26 years that have followed. When he was at ARCE before and after the CTV job his work was respected and from June 1986

[94] TRANS.20120801.26, lines 15 to 20
[95] TRANS.20120801.27, lines 14 to 30
[96] TRANS.20120808.105, line 4
[97] TRANS.20120806.12
[98] TRANS.20120806.13, lines 1 to 8
[99] TRANS.20120806.66, lines 21 to 25
[100] TRANS.20120730.125, lines 17 to 19

he worked as an Associate and therefore, from that point forward, as a principal. Even now, under the intense scrutiny of this Inquiry, no issues have been found with any other job Mr Harding worked on. Mr Banks gave evidence that he had been involved in construction monitoring on the Heatherlea Apartments, which Mr Harding had designed.[101] Mr Banks referred to this work in the context of expressing his surprise at the error made by Mr Harding on the CTV job.[102]

> A...It led me to conclude that it might've been, well most likely it was an isolated issue.
> Q. I see.
> A. There is a further issue that's come to light actually just in my reading of the evidence yesterday –
> Q. Yes?
> A. – and that is I refer to my inspection of the Heatherlea Apartments project. I believe that was a project of seven or eight storeys, that was designed by Mr Harding I think as well.
> Q. And did you look at that at the time that you were thinking about these issues?
> A. No, no I didn't but just in reading 'cos my recollection is one of the first things I did when I started work was –
> ... my thought process at the time would have been that I had recently inspected another building of significance that Mr Harding had designed.
> Q. And this is Heatherlea is it?
> A. Heatherlea Apartments.

67. Clearly Mr Banks was fully satisfied with the quality and competence of Mr Harding's work. He had no cause for wider concern with Mr Harding's work, and he was surprised by the issue he had identified with the CTV Building.

68. In the context of Mr Harding's experience and level of seniority there was no reason for Dr Reay to review his colleague's work, unless specifically approached by Mr Harding with a request for such input. Mr Harding was a near equal to Dr Reay, he was not a junior or an inexperienced engineer. At the time he designed the CTV Building Mr Harding was fully entitled to practice on his own account doing the work that he then did. Indeed during this period he became an Associate at ARCE (and effective principal). He has remained a principal in an engineering firm until today, in good standing.

69. The mystery therefore is – how or why did Mr Harding, contrary to his known ability and expertise, make basic errors in this one project? After 4200 pages of evidence we do not know.

[101] TRANS.20120816.148, lines 22 to 23, TRANS.20120817.47, lines 29 to 31
[102] TRANS.20120817.47 to TRANS.20120817.48

Supervision of work

70. A number of witnesses at the hearing were asked to comment on the issue of supervision of structural engineers within an engineering practice. Those asked to comment included Mr Jury,[103] Mr Falloon,[104] Mr Henry,[105] Dr O'Leary,[106] Mr John O'Loughlin.[107] However, none was in a position to comment on ARCE in the mid-1980s era, or on any firm of similar size. There was no evidence establishing that an Associate in a firm working as a principal as Mr Harding was by June 1986 would have been supervised. To the contrary in the larger firms persons at that level would have been providing the supervision.

71. ARCE was, at the time, a small practice. Dr Reay and Mr Harding were the only structural engineers. In contrast:

   (a) Mr Jury: Mr Jury's entire career has been with Beca Carter Hollings & Ferner Limited ("**Beca**").[108] Beca is one of New Zealand's largest engineering firms and has been for decades.

   (b) Mr Falloon: Falloon & Wilson had 4-5 engineers at the time it did the CTV job.[109] The supervision he spoke of related to a graduate engineer.[110] Mr Harding was considerably more experienced than this.

   (c) Mr Henry: All of the firms in respect of which he spoke of supervision were larger than ARCE and he also talked about supervision in times much more recent that the mid 1980s.

      (i) 1972-1975 (and 1978-1979 while studying) - Griffith Moffat & Partners: 30 staff, 3 engineers;

      (ii) 1980 – 1984 Holmes: 20 staff, 2 partners, 4 associates, 3-4 engineers

      (iii) 1985 Dick Cusiel (Lovell-Smith Cusiel): Mr Henry was assisting Mr Cusiel with the analysis of one building, he was not the design engineer.

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[103] TRANS.20120710.109, lines 8 to 29
[104] TRANS.20120725.92
[105] TRANS.20120801.113 to .119
[106] TRANS.20120813.72, 74
[107] TRANS.20120814.102
[108] TRANS.20120710.109, line 18
[109] TRANS.20120725.87
[110] TRANS.20120725.92

(iv) 1986: Holmes. As above (if not larger).

(v) 1996-2002: MWH: Large, corporate firm.

(vi) 2003: Elliot Sinclair: 60 staff, 7 engineers, 4 draughtsmen.

(d) Dr O'Leary: In 1986, Dr O'Leary was at Morrison Cooper Limited. The firm was 100 strong, with 15-20 structural engineers.[111]

(e) Mr O'Loughlin (J): O'Loughlin Taylor Spence Limited (sold in 2012): Always had three senior, experienced engineers. When a junior engineer was employed, they would watch carefully what he was doing. The firm never had an engineer of equivalent experience to Mr Harding.[112]

72. Dr Reay's evidence was that he relied on the Council review as a check of the work from his office. This applied as much to his work or Mr Henry's work as to Mr Harding's. It is submitted that is entirely reasonable in the permitting processes at that time. In addition, the way in which building permits were then given led to such an approach. If a design certificate was called for by Council, the standard wording of that certificate was that the design, in the opinion of the certifying engineer, complied with the codes.[113] Whether it complied with the by-laws was solely a matter for Council determination.

73. An engineer's ACENZ certificate constitutes the engineer's opinion that the design complies with New Zealand standards as specified. If the Council bylaws require additional design criteria then this would normally be a matter for the Council itself to assess. It is understood that as a matter of practice the Council did not require any certification other than to the relevant New Zealand standards.

74. On the evidence it is probable that no design certificate was ever called for. Indeed, the evidence was that such a certificate would not have been called for if the Council found the calculations were in order. At the time, design certificates were sought when the Council did not look at calculations or wanted the engineer to take direct responsibility.[114] That was the clear thrust of Mr McCarthy's evidence when he said that a design certificate was

[111] TRANS.20120813.72, 74
[112] TRANS.20120814.114
[113] BUI.CAS056.0001.15
[114] TRANS.20120801.32, lines 17 to 20; TRANS.20120801.33, lines 10 to 11 (no design certificate for CTV); TRANS.20120801.103, lines 8 to 14; TRANS.20120806.64, lines 15 to 22

the alternative to provision of the drawings and calculations.[115] Mr O'Loughlin's evidence was consistent.[116]

75. The evidence about a number of engineers who worked in a very small (one or at most two engineer) practice did not show a practice of internal peer review. Engineers who had worked in that situation for a time included Mr Falloon,[117] Mr Henry,[118] Mr Cusiel,[119] Mr Harding,[120] Mr Banks,[121] Mr Tyndall[122] and Dr Reay. A sole-practice engineer would and did rely on structural designs presented to a territorial authority for permit or consent. This is consistent with ARCE in 1986.

76. Counsel Assisting have submitted that the CTV job should not have been taken on at all because Dr Reay had insufficient experience and competence in the design of complex multi-level structures.[123] Dr Reay had done such work but was not currently engaged in it. Examples given in evidence included Ibis House and the Kamahi Building.[124] Mr Henry did such work when at ARCE without requiring any input from Dr Reay. It is only now known that Mr Harding's experience and competence is not seen as equivalent to that of Mr Henry. The evidence is that Dr Reay and Mr Harding each believed the opposite at the time.

Construction

77. Construction of the CTV Building proceeded largely without incident. Although there was the disruption of the Union Construction Limited takeover, it appears that the transition from Williams to Union Construction Limited was reasonably seamless insofar as the CTV Building was concerned. However, the disruption caused by the jostling for positions may well have impacted on the quality control at the site. Mr Harding made various site observations.[125] The Council also inspected the construction site from time-to-time.[126] Mr Harding was perceived by all those involved in the construction to be competent. His work was not queried or challenged and all those who gave evidence regarding the

115 TRANS.20120807.17
116 TRANS.20120814.100, line 10
117 TRANS.20120725.84 – in-house for Industrial Holdings
118 TRANS.20120801.114 – own firm between time at Holmes and Christchurch City Council
119 TRANS.20120801.114, 117 – referred to by Mr Henry
120 TRANS.20120731.98 – practised on own account since leaving ARCE
121 TRANS.20120816.148 – Cambridge Consulting Ltd, between time at Holmes and ARCE
122 TRANS.20120626.107 – Tyndall & Associates – practiced in own account for 43 years
123 TRANS.20120827.CS.7, paragraph 15
124 At the time he designed Ibis House, Dr Reay was 32 years old and had 3 years post-registration experience
125 Harding, paragraphs 9 to 32, TRANS.20120808.60, lines 7 to 13; TRANS.20120808.66, lines 20 to 28; TRANS.20120808.68, lines 7 to 9; TRANS.20120808.72, lines 26 to 28; TRANS.20120808.88, lines 20 to 27; TRANS.20120808.113, line 25 to TRANS.20120808.114, line 7; TRANS.20120808.144, lines 29 to 34
126 BUI.MAD249.0117

construction phase seem to have acknowledged that he was the structural engineer for the project and not expressed any concern.

78. The extent to which there may have been any flaws in the construction of the CTV Building is difficult to assess fully in light of the complete collapse and inadequate forensic operation.

79. Mr Brooks identified a construction issue. He referred to the importance of the foundation steel connecting to the column steel and, in turn, to the beam steel and shear wall and so on. Mr Brooks believes the connection has been interrupted in the CTV Building at the point where the beam connects to the shear wall. He notes the absence of a horizontal H24 rod through the semicircular end of each beam, tied into place, which should have been inserted prior to the concrete pour. Mr Brooks observes:

> The insertion of this item would have provided continuity to the steel components and would have gone some way towards frustrating any forceful attempt to collapse the structure, if not to prevent it.[127]

80. Mr Brooks produced a diagram illustrating his concern in this respect.[128]

81. A second issue was also identified with the beam to wall connection. The structural drawings required the bottom bars of the beams to be bent up within the thickness of the shear wall.[129] However the bars were instead bent up at the face of the shear wall.

82. Mr Brooks' evidence on this issue was that the bars could not have been bent that way on site.[130] It therefore seems more likely to have been a defect in the pre-cast beam as supplied. Regardless, it should not have been installed in that form and this represents another construction fault, with the result that the connection was considerably less robust than it should have been. This issue was also identified in the DBH report and is supported by photographs.[131]

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[127] TRANS.20120808.35, lines 15 to 18
[128] BUI.MAD249.0423B
[129] See details 5 and 9 on S19 BUI.MAD249.0284.20
[130] TRANS.20120808.38, line 16 to TRANS.20120808.40, line 17
[131] BUI.MAD249.0189.110, figure 44 at BUI.MAD249.0189.109

Figure 44, BUI.MAD249.0189.109

83. The failure to roughen the ends of pre-cast beams where they connected with in-situ concrete is a further issue. This detail was sufficiently shown in the drawings and specification and the practice was also a standard construction practice which Williams should have followed. Other potential construction flaws cannot be resolved. Perhaps there were issues with the west wall cement mortar. Perhaps there were issues with the concrete although the concrete hot tub session tends to have disproved that possibility.

Standards and By-laws

84. Construction of the CTV Building was completed under Bylaw 105 (1985 Buildings) ("**Bylaw**"), which was in force in 1986 pursuant to the provisions of the Local Government Act 1974. The Bylaw referred to several Standards and codes, some of which were explicitly incorporated into the Bylaw and others of which were used as references for compliance options available to designers. The codes and Standards most relevant to the issues raised in this hearing are as follows:

(a) NZS 4203:1984: Code of Practice for general structural design and design loadings; and

(b) NZS 3101:1982: Code of Practice for the design of concrete structures.

85. Clause 5 of the Bylaw provides that compliance with the Bylaw may be proven by whether the Standards listed in Schedule 2 (being those set out

in the above-named Codes of Practice) have been met. Specifically, Schedule 2 provides as follows:

> The Second Schedule details those Standards, Standard Specification, Codes of Practice and Appendices which detail means by which the requirements of the Bylaw **may** be complied with. **These documents are not part of the Bylaw.**
> ...
> NZS 4203:1984 Code of practice for general structural design and design loadings for buildings
> NZS 3101:1982 The design of concrete structures –
> Part 1:1982 Code of practice for the design of concrete structures
> Part 2: 1982 Commentary on the design of concrete structures
> [*Emphasis added*]

Legal effect of standards

86. There are two ways in which standards may be incorporated by reference into a bylaw. The first is that some or all of the provisions of the standard may form part of the actual bylaw, thereby becoming binding because they are the bylaw itself. An example of this method is clause 11.2.5.1 of the Bylaw, which repeats clause 3.1.1 of NZS 4203

87. The second is that a bylaw may provide that compliance has been achieved if the requirements of the Standard are met. In that latter case, the Standard is not itself law. Its requirements are binding only to the extent that they are made an express requirement by a bylaw. Generally, the position is that one may comply with the bylaw by other means and so without complying with every part of the Standard. Clause 5, in tandem with Schedule 2, is an instance of this method of incorporation. This use of Standards included by reference has been upheld by the Court of Appeal even where the standard itself was no longer in force.[132]

88. The two Standards in this case were therefore absolutely binding on designers at the time the CTV Building was constructed only to the extent that they were specifically incorporated in the Bylaw (such as clause 11.2.5.1). They were a method which, if followed, entitled the designer to require the Council to accept that the Bylaw had been complied with. Such an entitlement may at times have been part of the differences between engineers and the Council which are said to have arisen.

89. In both cases, subsequent revocation of the Standard by the Standards Council is irrelevant. If provisions of a Standard have been included within the provisions of a bylaw, then they have lost the character of the Standard

[132] *Parlane v Waipa DC* [2007] NZAR 16

from which they came, and are now provisions of a bylaw (and subject therefore to amendment and revocation procedures applicable to bylaws). Where the Standard is simply a measure by which compliance with the bylaw is assessed, subsequent revocation of the Standard may have no effect, depending on the terms of the bylaw. In either case, the effect of later amendment or replacement of the standard will also depend on the terms of the bylaw. The new changes will not be operative unless the bylaw says they will be. The two Standards in this case were superseded by Standard NZS 4203:1992 Volumes 1 & 2 and Standard NZS 3101:1995 Parts 1 & 2.

90. One particular issue that requires mention in this context is that raised in paragraph 370 of the closing submissions of Counsel Assisting. There the submission is made that clause 11.1.5(d) of the Bylaw refers to a 'major earthquake' not a 'design-level earthquake' and that even if it is accepted that 'major' means 'design-level', neither the Bylaw nor the Codes allow the designer to design on the basis that a building is only required to withstand an earthquake at 'design-level' but to collapse in an earthquake only marginally stronger.[133]

91. However, no such check by then engineer was required by the Bylaw or the Code. In fact, in their paper *Design of Concrete Structures* presented at the July 1986 seminar attended by Mr Harding, Park, Paulay, Priestley and Gaerty noted the following:[134]

> In many countries, for example New Zealand, only one level of earthquake load is considered in design, the level being that corresponding to a major earthquake.

92. The Bylaw and Code do not contemplate two (or more) design-level or above design-level earthquakes in quick succession. No one can say what might have occurred to the CTV Building on 22 February 2011 had that been a one-off event without the lead up of the 4 September earthquake and subsequent aftershocks.

93. The submissions of Counsel Assisting go on to state that the underlying purpose of the Bylaw design requirements is to avoid collapse and to minimise the probability of injury and death. Counsel Assisting submit that this purpose cannot be met if the designer seeks to draw a line beyond

[133] TRANS.20120827.CS.81
[134] BUI.MAD249.0519.12

which collapse and death are virtually certain.[135] It is accepted that the purposes identified are implicit in the Bylaw. Even more obviously every structural engineering design always has as its main purpose and objective human safety and then the control of building damage from earthquakes. But the only quantitative means to assess whether these purposes are achieved is via design checks. Design checks already contain safety factors, so something that 'just' passes the Code should not be extremely vulnerable. Passing the Code means an acceptable level of design (with a risk that may be higher than significantly exceeding the Code, but an acceptable risk nonetheless).

Broader context of Standards and bylaws

94. It was the nature of engineering practice in the 1980s that many components of safety and loading were uncertain. The Bylaw and associated Standards were not, in that context, a complete instruction manual. Rather, they were a starting point. They included mandatory requirements that reflected what was known at the time, but other provisions were advisory or were stated subject to qualifiers, for example as to reasonableness or practicality. The Standards and Bylaw were intended to operate in tandem with the growing expertise of professional and expert engineers, and the qualifiers reflect that.

95. This understanding is borne out by the foreword to NZS4203:1984, which states: [136]

> [1] The Loadings Committee's task in drafting this standard was seen mainly to be one of providing a set of minimum design criteria of an effective and economic nature which would not be too difficult for the designer to apply, but at the same time would leave him scope for innovation and imagination.
>
> The committee believes that the requirements of this standard provide a reasonable level of protection to life and property at an economic level of risk, taking into account the relevant seismicity of New Zealand as compared with the rest of the world and the particular building practice and design methods adopted in this country.
>
> [2]
> ...
> Designers should recognize that the precise properties of construction materials and of structural elements made from them are not clearly known. Furthermore, the interaction of these elements in a building frame under load is extremely uncertain, so that the total design technique is one of some degree of imprecision. In fact, the design result depends so much on the nature of the mathematical model of

[135] TRANS.2012.0827.CS.82, paragraph 373
[136] ENG.STA.0018.14

> the building as envisaged by the designer that **the use of more advanced techniques of earthquake analysis can easily lose validity.** [*Emphasis added*.]

96. NZ standards in the 1980s were drawn up to prescribe design methods for particular types of construction; but leaving some degree of freedom to the designer as to how they were interpreted and applied. Evidence for that can be found in the foreword to each of NZ3101 and NZ4203 (above).

97. From the evidence the Commission has heard, it is clear that a number of the engineers saw the application of Standards was something of an art, informed by experience and based on an accumulation of practical knowledge. Yet no such reservation (or requirement for experience) appeared in the bylaws or the general law.

98. To a lawyer, a concept that a person holding the required authorisations to undertake work in a legal sense cannot undertake it because of some “professional experience” overlay requirement – not written but to be inferred from some arcane knowledge of older engineers – is in direct conflict with the authority given by the Bylaw.

99. In addition, there is the issue as to how a “bylaw” (itself subordinate legislation) can incorporate within its mandatory requirements, standards which are not written in that way. In the Christchurch City Council bylaws applicable in the 1980s and the 1990 Bylaw, rather than purport to make the standards provisions of the Bylaw, they are drafted to provide that compliance with the standard is deemed to be compliance with the Bylaw. This however leaves open the possibility of other means of compliance. To ensure that the most important parts of the standards have legal effect, some, but not all, of the provisions in the standards are included in the Bylaw as mandatory requirements.

100. Given that it is clear that in the 1980s there were severe limits on the ability of the designer to achieve certainty with the design methods and computer analyses available (which fact is expressed in the forewords with corresponding cautions), the actual operation of the design engineer in those days was to try to develop designs which had sufficient levels of confidence to them that the structure could be considered compliant and safe. In turn the Council (which in those days had important review and approval roles, but had even less ability to carry out calculations or other checks) then had to form an overall judgment.

101. This leads to a situation where mere compliance with the bylaws may lead to a design which is bylaw compliant but unsafe; and where a safe design may be open to attack as not complying with the bylaws. In addition, seemingly mandatory requirements (e.g. for symmetry) are modified by words like "as far as practicable" and it even becomes a matter of opinion which elements of a structure are identified as being the "main elements" – and so on. In reality, in design and construction, it is not only accepted that the standards will be used as a basis for bylaws, the standards appear to be written for that purpose.

102. This method of tertiary legislation and its adoption of standards as part of bylaws is controversial. The Legislation Advisory Committee of Parliament in Appendix 4 of its guidelines makes general recommendations which can be read as discouraging this. But as noted above the Court of Appeal in *Parlane v Waipa DC*[137] was willing to uphold a bylaw which applied a standard which had been withdrawn.

103. Finally, a bylaw which cannot be understood or applied may be set aside.

Code deficiencies

104. Mr Latham, a structural engineer at ARCL prepared detailed reports showing how the code could have been interpreted. It became evident through the work of Mr Latham and others (primarily Dr O'Leary) that the codes were deficient in a number of respects. At various points they were ambiguous, confusing and contradictory. A summary of some of these possible issues, as highlighted in the evidence, is set out below.

NZS4203:1984

105. Cl 3.1.1: Symmetry "*as nearly as is practicable*". The difficulty in applying this provision was repeatedly shown during the hearing, due to the lack of definition over what was acceptable and what was not.

106. Cl 3.4.7.1: The concepts of regular and irregular buildings were not well defined. There was very limited guidance for when a design for a structure transitioned from regular to irregular.

107. Cl 3.8.1.2: "*Computed deformations shall be calculated neglecting foundation rotations*." The Code is not specific about what component of foundation rotations are to be neglected. Does this require buildings to be

[137] *Parlane v Waipa DC* [2007] NZAR 16

analysed with a fixed base only? Does the design engineer neglect only the global tilt of the structure? Does the design engineer neglect the local foundation rotations?

NZS3101:1982

108. Cl 3.5.14.3: "*Group 2 elements shall be designed to allow* *ductile behaviour…*". It is ambiguous whether the clause is referring to the group 2 secondary element itself requiring ductile behaviour, or the fact that the secondary elements need to allow for ductile behaviour in the primary system, for example the shear walls. Additionally, part (a) of 3.5.14.3 goes on to provide an approach which allows the non-seismic provisions to be used if the elements remain elastic at drifts less than v.delta. Part (a) must be given a purpose, which contradicts the proposition that there is an overarching requirement to design for ductility. The presence of part (a) of 3.5.14.3 leads to the conclusion that no such overarching requirement was ever intended and consequently the designer had a degree of latitude to design either for ductility or non-ductile behaviour as circumstances permitted.

109. Professor Mander referred to the secondary member provision in clause 3.5.14.3 as a possible "loophole" in the Code.[138] He later elaborated that the non-ductile design of secondary elements was, regardless, permissible:[139]

> Now my belief is that if that's in the Code that's, and people can legitimately do that and the Council signs off on it, then that may not be good practice but it's permissible and the CTV building was designed in that permissible way,

110. Cl 9.4.1: Beam-column joints. "*If the joint is also subject to seismic load reversals it shall be checked for compliance with the provisions of 9.5*." This clause is ambiguous. All joints in any building would be subjected to seismic load reversals. If compliance with 9.5 is to be taken as an absolute requirement of clause 9.4.1, all joints would require design using the additional seismic requirements. On that basis the non-seismic section on beam-column joints would become completely redundant. However, the word "checked" suggests something less than such a mandatory requirement. Furthermore, on reading the commentary it can be inferred that the reference to seismic load reversals is intended to apply when inelastic action is occurring around the joint, although this is not entirely

[138] TRANS.20120724.41, lines 21 to 27, TRANS.20120724.103, line 23 to TRANS.20120724.105, line 16
[139] TRANS.20120816.15, lines 28 to 31

clear. Therefore if the beams and columns are remaining elastic around the joint, it must follow that the non-seismic provisions can be used.

111. Considerable time was also occupied at the hearing trying to identify what might have amounted to *"best practice"* at the time of the design of the CTV Building. It is submitted, however, that a focus on best practice adds little to the consideration of the issues. Professor Priestley refers in his evidence to a 1975 book by Park and Paulay and others referred to this book at representing best practice at the time. However, both Park and Paulay were on the code committee that wrote the 1982 concrete standard (NZS3101). Mr O'Leary made this point in his evidence.[140] Dr Reay made the same point when questioned about best practice requirements.[141] Dr Reay described his understanding of best practice as the code, representing the accepted knowledge at the time of design.[142]

112. One example mentioned by Counsel Assisting was redundancy in column design.[143] This is not a requirement of codes even today, so it is difficult to see that this could have been a best practice requirement in 1986.

113. In circumstances where both the loadings code (NZS4203:1984) and the concrete code (NZS3101:1982) were relatively new codes at the time of the CTV design and had been written by recognised leaders, it is submitted that it is generally unhelpful to look significantly further than the codes for best practice requirements at the time of the design of the CTV Building, although it is recognised that there may be exceptions. On some design aspects, there was no code to guide practice, such as soils and foundation engineering. Where any element of best practice is to be considered, it must be judged from a Christchurch perspective. It is submitted that the opinions of engineers who have never practiced in Christchurch should be treated with considerable caution.

Design issues

114. The submissions of Counsel Assisting outline in considerable detail the areas in which it is perceived the CTV Building failed to comply with the Bylaw and Codes of the day. Paragraph 318, in particular, lists the alleged areas on non-compliance at the time of permit. Each issue is addressed in turn below.

[140] TRANS.20120813.5
[141] TRANS.20120815.53, lines 8 to 16
[142] TRANS.20120815.51
[143] TRANS.20120827.CS.97, paragraph 452

Asymmetry

115. This issue was well traversed at the hearing. Symmetry was an aim of NZS4203:1984, not a mandatory requirement. The clause provided:[144]

> "The main elements of a building that resist seismic forces shall, as nearly as practicable, be located symmetrically about the centre of mass of the building."

116. It is submitted that the walls were located symmetrically, one wall to the north, one to the south, and the four finger walls were also central. The clause does not state the elements had to be similar in stiffness or strength even though this may have been the intention.

117. NZS4203:1984 provided commentary to this provision, but there was no equivalent statement in the Bylaw.

118. The Code (NZS4203:1984) provided methods for dealing with irregular buildings and had specific and additional analysis requirements for buildings depending on the degree of eccentricity and whether the building was considered regular or irregular. In doing so, the Code recognised that not all buildings would be, or were required to be symmetrical. It therefore acknowledges by default that it is acceptable to have a non-symmetric building.

119. It is impossible to define "as nearly as is practicable" for all purposes. Therefore non-compliance cannot be assessed against undefined criteria.

120. Dr O'Leary noted in his evidence that there was no practical way to apply the provision.[145] He promoted "drilling down" further into the standard to find out what the quantitative governing criteria are in order to satisfy the overall symmetry requirements.[146]

121. The Code did not specify a clear limit on the acceptable degree of eccentricity. There was no defined point at which acceptable became unacceptable. The DBH report acknowledges that *"most structures require some level of torsional irregularity to satisfy reasonable architectural requirements. There were no clear limits for torsionally irregular structures in terms of compliance requirements."*[147]

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[144] ENG.STA.0018.38, clause 3.1.1; ENG.CCC.0044.124, clause 11.2.5.1
[145] TRANS.20120813.8, line 28 to TRANS.20120813.9, line 8
[146] TRANS.20120813.8, lines 20 to 21 and TRANS.20120813.9, lines 1 to 8
[147] BUI.MAD249.0189.149

122. In the current Code (NZS1170.5:2004) there is still no limit on the permitted degree of eccentricity. The current Code, like its predecessor, provides a requirement to carry out a more in-depth analysis if there is high eccentricity but there is no maximum limit. Quantification of eccentricity limits would be a valuable enhancement to future codes. It is noted that this point has been acknowledged in the recommendations of the Canterbury Earthquakes Royal Commission of Inquiry final report Volumes 1 – 3.[148] Recommendation 35 provides:

> The requirements for regularity in buildings, and for torsion due to the distance between the centre of mass and the centres of stiffness and strength, should be revised to recognise the implications of these parameters on observed behaviour.

123. The asymmetry of the CTV Building was not raised as an issue by the Council or Holmes, which, it is submitted, is representative of the thinking at the time. The Holmes report stated that "the layout and design of the building is quite simple and straightforward..."[149] These contemporaneous Christchurch based assessments are far more reliable than a 26-year hindsight assessment. There are many other buildings in Christchurch that have similar levels of asymmetry and eccentricity to the CTV Building. One such example is Landsborough House. In fact, Mr Henry discussed the asymmetric shear wall layout with Professor Paulay during the design, who agreed that the proposed layout was acceptable.[150] There are, or were, several other buildings in Christchurch that have similar levels of asymmetry to the CTV Building that were not discussed at the hearing.

Connections between the diaphragms and the North Shear Core

124. NZS3101:1982 required the diaphragm connection to be designed for the overstrength actions or the Parts and Portions section of NZS4203:1984, whichever gave the smaller force.[151]

125. While, in hindsight, using a lower force from the Parts and Portions section derived loads does not achieve a rational capacity design approach, it is nonetheless what the Code stated and required. At the time it was considered that using Parts and Portions loads provided an additional factor of safety and it was the appropriate method to use.

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[148] Volume 1: *Summary and Recommendations in Volumes 1-3, Seismicity, Soils and the Seismic design of Buildings*, 2012
[149] BUI.MAD249.0005.11
[150] TRANS.20120801.130, line 14 to TRANS.20120801.131, line 2 and TRANS.20120801.132, lines 13 to 17
[151] Clause 10.5.6.1, ENG.STA.0016.75

126. During the design of the drag bar retrofit in 1990, Mr Banks from ARCL and Holmes agreed on the design load for the drag bars.[152] This was based on Parts and Portions loads. This virtually contemporaneous assessment shows that this was the accepted method at the time and must be the basis for the assessment as at 1986.

127. The analysis of the diaphragm connection to the North core walls is complex. Within the core area, there was no physical evidence that the diaphragm connection failed during the 22 February earthquake.

Non-Seismic Detailing of Columns and Beam-Column Connections

128. Two issues arise in this context:

    (a) Whether the approach was permissible at all; could columns be considered as secondary elements, and did they need ductility regardless?; and

    (b) Did the columns remain elastic at v.delta?

129. As discussed above (paragraph 51) Mr Henry, Mr Hare and others all noted that designing “shear wall protected gravity load systems” was an acceptable and not uncommon method of design. During the design of Landsborough House, Mr Henry had discussions with Professor Paulay, who accepted this design methodology.

130. NZS3101:1982 considered primary frames acting in parallel with stiff shear walls to be secondary elements.[153] Secondary elements could still have been required to be designed with the additional seismic requirements if the drifts were sufficiently large, similarly the limited ductile provisions or the non-seismic provisions could have been used if the drifts allowed.

131. Applying NZS3101:1982, clause 3.5.14.1, with the single exception of Dr Jacobs, all experts agreed that the columns were secondary elements. Despite this, Counsel Assisting, in closing submissions, have submitted that code compliance is a question of law and experts' opinions are not definitive. Counsel Assisting submit that an approach is generally adopted by engineers does not prove it was lawful. These submissions:

    (a) Invite the Commission to disregard expert evidence as to how the Bylaw was in fact understood and applied and complied with at the

[152] BUI.MAD249.0130.12
[153] Clause C3.5.14.1, ENG.STA.0016A.32

relevant time – an issue of fact which is before the Commission to determine and on which the position is clear;

(b) Invite the Commission to adopt a legal interpretation where the issue arises in respect of the meaning of a code (or Standard) of engineering practice, not a statute. The Bylaw provided that compliance with such code would be deemed by the local authority to be compliance with the Bylaw. That does not require the detail of engineering work to correspond exactly to the Bylaw – to the contrary, "deemed" in itself contemplates that there may be a discrepancy between code compliance and the provisions of the Bylaw, and overrides any such difference;

(c) Invites the Commission to make a finding of law (when the Inquiry is one as to fact) and to apply it retrospectively.

(d) Disregards the interpretation provisions of the Acts Interpretation Act 1924 (in force in 1986) and in particular the requirement of s.5(j) that

> Every Act, and every provision or enactment thereof, shall be deemed remedial, whether its immediate purport is to direct the doing of anything Parliament deems to be for the public good, or to prevent or punish the doing of anything it deems contrary to the public good, and shall accordingly receive such fair, large, and liberal construction and interpretation as will best ensure the attainment of the object of the Act and of such provision or enactment according to its true intent, meaning, and spirit.

132. Counsel Assisting also submit that in the event of an inconsistency or ambiguity between the two codes, NZS4203 prevails,[154] and that the Bylaw prevails over NZS4203.

133. NZS4203 referred to columns as 'primary elements'. For gravity loadings, the columns were primary elements. When considering the "more particular than NZS4203" definition in NZS3101,[155] the columns were secondary elements with respect to the lateral load resisting system (whilst still constituting primary elements for gravity loads). The Concrete Code (NZS3101:1982) specifically stated secondary elements included "*such primary gravity-load resisting elements as frames which are in parallel with stiff shear walls*."[156] This is neither unclear nor ambiguous and is not inconsistent with NZS4203. It is just more descriptive. Further, NZS4203:1984 states under the heading of General Design Principles that

[154] TRANS.2012.0827.CS.67, paragraph 290
[155] Clause C3.5.14.1, ENG.STA.0016A.32
[156] Clause C3.5.14.1, ENG.STA.0016A.32

"*Design shall be in accordance with the appropriate material code subject to the general principles of design set out below*." There were no specific requirements set out in NZS4203:1984 for gravity elements acting in conjunction with ductile shear walls.

134. The submission of Counsel Assisting is that the columns required ductility regardless of whether they were a secondary element because in the case of failure there was a risk to life. This requirement was in NZS4203. With respect to this particular clause, "ductility" is not defined.

135. This raises the question of how ductility is to be measured. A common measure of ductility is the ratio of ultimate displacement to the elastic displacement. The authors of the DBH report have calculated the elastic limit and the failure limit of the columns of the CTV Building at Tables 13 and 14.[157] The ratios of the failure limit to elastic limit at each level (descending) are:

(a) Column C/1: 2.4, 1.9, 1.9, 2.0, 2.2

(b) Column F/2: 2.5, 2.0, 1.9, 2.0, 2.1

136. All ratios are in excess of 1, indicating some level of ductility. It can be concluded that the columns <u>did</u> possess some level of ductility, albeit not as much as what they could have had if the detailing followed the seismic provisions. The current concrete code (NZS3101:2006) classifies structures as nominally ductile if they are designed with a ductility factor between 1.0 and 1.25, or limited ductile if they are designed with a ductility factor between 1.25 and 3.0.[158] Therefore the CTV column ductility factors in the range of 1.9-2.5 are consistent with the classifications of structures with some level of ductility in the current codes.

137. It is also noted that the failure limit calculated in the DBH report is probably estimated too low, as the authors assumed failure at a concrete strain of 0.004. This may be the point at which the columns can no longer contribute any lateral resistance however they could possibly still carry the gravity loads at strains of 0.007-0.008. This point was acknowledged by Mr Holmes in his peer review report.[159] Adopting a higher failure strain would increase the ratios above and demonstrate an increased level of ductility.

[157] BUI.MAD249.0189.288
[158] Cl. 2.6.1.2, NZS3101:2006
[159] BUI.MAD249.0372.8

138. It can be noted that it is acceptable to assume gravity frames, acting in conjunction with other lateral load resisting systems such as shear walls, as secondary elements in the current codes (NZS1170.5:2004 and NZS3101:2006).

139. The current Code NZS1170 Part 5 provides a definition for secondary members:

> "members that are not considered to be part of the earthquake resisting system and whose strength and stiffness against seismic actions is neglected. They are not required to comply with all the requirements of NZS1170.5, but are designed and detailed to maintain support of gravity loads when subjected to the displacements caused by the seismic design condition."

140. The secondary elements clauses in NZS3101:1982 remain virtually unchanged in both NZS3101:1995 and NZS3101:2006. The Code still has the same clause which considers primary gravity frames in parallel with shear walls to be secondary elements, demonstrating that there has been no major shift in the classification of secondary elements from the 1980's codes to today.

141. What has changed is the detailing requirements for columns that are not required to be designed with the additional requirements for seismic loadings. This is implicitly a later recognition that, while not known at the time, the previous 1982 Code was inadequate and it has subsequently been rectified. Far more stringent levels of confinement are required for non-seismic columns. This occurred for the first time in 1995, some 9 years after the CTV Building was designed.

142. Finally, it is observed that if the issue of seismic detailing were as clear as Counsel Assisting argues, then the Council and Holmes would certainly have each identified it.

143. Turning to the question of whether the columns remain elastic at v.delta, the first consideration is what was v.delta? Five different analyses have been presented to the Royal Commission; four by Compusoft, one by Mr Latham. The Compusoft analyses included a rigid base model, and models with soil flexibility included using the upper bound, most probable and lower bound soil stiffness values recommended by Tonkin & Taylor respectively. Mr Latham's analysis adopted the soil stiffness as assessed by Ian McCahon

as would have been assessed in 1986, rather than as assessed with the benefit of hindsight.[160]

144. The second issue arising in this context is how to assess the columns and their elastic limit. There are also several methods that have been put forward.

145. Mr Latham's presentation provided a comparison of the different analyses and different methods of determining whether the columns were elastic. [161]

146. The presentation showed that under Mr Latham's static/ERSA analysis, all of the columns remained elastic, using either his column criteria or the criteria used in the DBH report.

147. Adopting the rigid base model by Compusoft, and the column criteria used in the DBH report, all columns except one level on one grid line remained elastic. That is, 120 columns out of a total of 123 remained elastic (98% of the total). Increasing the longitudinal reinforcing from the 6-H20 bars provided to 6-H24 bars in those three columns would have meant that all columns would have remained elastic using the same analysis approach as that used in the DBH report. This would not have required any change to the R6 spiral transverse reinforcing provided.

148. Mr Latham's analysis neglected the foundation rotations. There has been debate about the interpretation of clause 3.8.1.2 of NZS4203:1984 however a literal interpretation requires the foundation rotations to be neglected. If this same approach was used on the Compusoft analyses using the Tonkin & Taylor soil stiffnesses, the drifts would reduce and similar design drifts to those obtained by Mr Latham would be achieved. Accordingly, it might be said that the appropriateness of Ian McCahon's stiffness as against that of Tonkin & Taylor is somewhat irrelevant.

149. Mr Latham was questioned about being unconservative with the column criteria by Commissioner Fenwick. However, it is to be noted that Mr Latham's column criteria was more conservative than the criteria adopted in the DBH report.

150. In summary, it can be shown that the columns remain elastic, and accordingly were entitled to be designed without the additional seismic

[160] BUI.MAD249.0460.1
[161] BUI.MAD249.0583.15-17

requirements as outlined in part (a) of Cl. 3.5.14.3 of NZS3101:1982. This is the level of detailing which was provided.

Shear Reinforcement of Columns

151. The code provided cases where the minimum reinforcement was not required. Pursuant to clause 7.3.4.1 of NZS 3101 if the shear demand was less than half the concrete shear strength, the minimum requirements did not need to be met.

152. The columns satisfied this requirement, depending on the assumptions made during the analysis.[162] Further to this, Clause 7.3.4.2 of NZS3101:1982 allowed the *"minimum shear reinforcement to be waived if it could be shown by test that the ultimate flexural and shear strength could be developed when the shear reinforcement is omitted."*

Anchorage of Spirals on Columns

153. The structural specification required all reinforcing steel to comply with the requirements of NZS3109:1980 (Concrete Construction Standard). It also required a copy of this standard to be kept on site.[163] This standard gave the detailing requirements for anchorage of reinforcing which included a hook detail.[164]

154. The specification and the drawings were required to be read together. Therefore the anchorage was specified by reference. There was no requirement to show the direct detail on the drawing.

155. The correct anchorages were in fact provided as shown by a photograph of the column remains.

[162] See WIT.LATHAM.0003.24 and .25, TRANS.20120809.125, lines 2 to 25 (O'Leary)
[163] BUI.MAD249.0199.4
[164] Clause 3.2.2 and Figure 1 of NZS3109:1980

156. The DBH report did not identify this as a design issue.

Adequacy of R6 @ 250mm spirals in cranked splice regions of the columns

157. NZS3101:1982 required that for splices, ties or spirals were to be placed at no more than 150mm from the point of bend.[165]

158. The R6 spiral provided for in the structural drawings had a pitch of 250mm.[166] The greatest distance that a bend could possibly be from a spiral would be 125mm, that being when the bend is exactly half way between the two spiral ties 250mm apart. The 125mm is less than the required maximum of 150mm, therefore the specified detail is in compliance with NZS3101:1982.

159. The DBH report did not identify this as a design issue.

Minimum transverse reinforcement of beam column joints

160. It has been acknowledged that the requirement of NZS3101:1982 for the transverse reinforcement in the beam column joint to be at a maximum of 200mm centres was not satisfied. However both NZS3101:1982 and NZS4203:1984 allowed for testing to be used as an acceptable means of demonstrating compliance.[167] This is what is required for the beam column joint, which, due to its arrangement is difficult to analyse.

[165] ENG.STA.0016.41, cl 5.3.27.1
[166] Drawing S14, BUI.MAD249.0284.15
[167] Clause 7.3.4.2 NZS3101:1982 and 1.2.2 NZS4203:1984

Diaphragm design

161. The Hi-Bond Manufacturer's product literature applicable at the time of the CTV Building design recommended for slabs 151-200mm thick to use 664 mesh, which is what was provided.

162. It is accepted that the slab reinforcement was marginally less than the Code specified minimum if the contribution from the Hi-bond decking is ignored. However, allowing for the Hi-bond decking and areas where the mesh was lapped, the minimum reinforcement levels specified in NZS3101:1982 were met.

Spandrel Panel

163. While it has been argued that no seismic gap was shown on the drawings, the drawings in fact provide for a 10mm gap.[168]

164. The 10mm clearance provided was sufficient to allow for the seismic drifts. This does not appear to be disputed.

How defects arose

165. Throughout the hearing Counsel Assisting have attempted to present a theory they appear to have developed by putting it to witnesses that the design deficiencies arose as a result of claims that there was:

(a) A culture within the office of ARCE of designing buildings to just comply with the codes and no more;

(b) The use of an "inadequately qualified" engineer who "should have been supervised" (or even perhaps never allocated to the job in the first place);

(c) The use of inappropriate or wrongly specified materials (for example Hi-bond);

(d) ARCE (and in particular Dr Reay) forcing the building permit through the permitting process by confrontation and argument.

These matters do not emerge from the evidence but generally the other way around. In addition, evidence tending to match the theory was featured; conflicting evidence gained little attention. It is not clear why

[168] See drawings S25 and, for example, S15 – compare the length of spandrel panel on S25 to clear space between columns on S15

Counsel Assisting have adopted personal theories on these topics and argued towards a personalised "liability" outcome, as opposed to presenting all evidence in the usual way for the Commission itself to interpret.

166. These theories of Counsel Assisting are also evident elsewhere. Dr Reay was criticised at times for an apparent failure to take adequate interest in the Royal Commission proceeding by reading and/or following the evidence of other experts. Quite apart from the fact that Dr Reay was endeavouring to continue to manage a not insubstantial engineering practice throughout, the criticism was unfair as Dr Reay has taken a very active role in the hearing and dedicated significant resources to this case.

167. In any event, none of these arguments as to how the defects allegedly arose can be sustained when the evidence is properly analysed.

Culture

168. The claims as to office culture were raised with various witnesses. Mr Harding first said that the practice within the office was to build no greater than necessary in terms of the strength of the buildings and said that employees would be asked to justify designing a building stronger than Dr Reay considered necessary. The object at ARCE, said Mr Harding, was to comply with the code but to reduce the cost where possible.[169]

169. Dr Reay described the culture in somewhat different terms:

> I consider that we had a culture of quality, that to deliver that quality there were several factors that were important. One of them code compliance, another was buildability. I always had the view that if the building was difficult to build, it would probably not be built well, errors would occur, and there was a culture of delivering quality drawings that could be easily read and were complete in terms of the necessary detail.[170]

170. Mr Horn described ARCE's drawings as "shop drawings" with "*every aspect of it ... pulled to pieces and itemised so you could hand it to a man in gum boots to build it*."[171] Mr Fairmaid independently used the same term, noting that " *pre-cast componentry and structural steel componentry was detailed to a higher degree and that enabled builders to be more accurate about what they were doing in terms of delivery of those components*."[172]

[169] TRANS.20120730.76
[170] TRANS.20120731.134
[171] TRANS.20120806.13, lines 23 to 25
[172] TRANS.20120815.98, lines 15 to 19

171. In response to evidence that he was intolerant of design stronger than necessary, Dr Reay said:

> If I found people designing things that I thought looked like they were just guessing and adding reinforcing or concrete I would ask them to justify it to ensure that they were actually designing what they were doing and not guessing what they were doing.

172. Mr Harding later accepted under cross-examination that his evidence regarding Dr Reay's intolerance of overdesign or the inclusion of unnecessary design elements was an attitude Dr Reay had in relation to efficient design not an attitude as to compliance of design.[173]

173. Mr Smith's evidence was that when he joined the firm in November 1987 it was a very quiet professional office which was necessary so everyone could concentrate on the manual calculations but otherwise there was nothing out of the ordinary.[174]

174. Mr Horn agreed in principle with Mr Harding's sentiments noting an emphasis on achieving efficiency.[175]

175. Mr Strachan described the culture within the office as relaxed. He compared the environment to Powell Fenwick where it was *"real head down, bum up, no talking, tight control, whereas Alan wasn't"*.[176] On the issue of building no stronger or more expensive than necessary Mr Strachan referred to the practice more as *"the end result of a series of developments that fine-tuned those buildings"*.[177]

176. Mr Fairmaid refuted Mr Harding's evidence of a general philosophy not to include anything that couldn't be justified saying that in reality it was more about buildability. He said:

> I think the perception might have been less reinforcing in concrete wall panels but the reality was that the building systems enabled buildings to be built very efficiently.[178]

177. Pulling together these various threads of evidence it is submitted that there is no basis to level any criticism at the practices of ARCE in terms of its design philosophy. Designing to code was clearly the prime objective. The fact that the firm had a policy of design efficiency focussing on buildability is not a reason for deficiencies in the CTV design, not least because there is

[173] TRANS.20120730.111, lines 10 to 12
[174] TRANS.20120806.50
[175] TRANS.20120806.8
[176] TRANS.20120806.35, lines 30 to 32
[177] TRANS.20120806.37, lines 1 to 3
[178] TRANS.20120817.98, lines 27 to 30

no evidence of any such outcome in any other work and substantial evidence of engineering design awards received.[179]

Suitability of Mr Harding for the job

178. The suitability of Mr Harding for the job has been discussed above. Mr Harding believed he could do the work and wanted the challenge. Dr Reay considered that Mr Harding was appropriately qualified and experienced for the job and Mr Harding in what he said and did confirmed that. The departure of Mr Henry had left current projects incomplete – the appointment of Mr Harding was intended to replace Mr Henry, not take on a trainee.

179. Dr Reay's evidence was that if Mr Harding had expressed any reservation about taking on the job, ARCE would not have taken the job on at all.[180]

Inappropriate or wrongly specified materials

180. There was no evidence to support the contention that inappropriate or wrongly specified materials were used for the job. The Hi-bond product was singled out for special attention. Mr Harding suggested it may have been selected because of some association between Dr Reay and Fletchers.[181] He later conceded that he was not privy to how the decision to use Hi-bond came about.[182] Dr Reay firmly rejected any suggestion that Hi-bond was selected because of his association with the Fletcher organisation. Hs evidence was that he had no recollection of playing any part in the decision to use that product.[183] No other witness supported Mr Harding's suggestion.[184] Mr Scott ended any uncertainty – he said that the decision had come from Williams.[185]

181. Dr Jacob's evidence was that the use of Hi-bond in this application in terms of span and reliance may have been something commonly done in the mid-1980s but would not represent current practice.[186]

182. There was no contention that any other product used in the CTV Building was inappropriate or wrongly specified.

[179] TRANS.20120716.29, lines 30 to 31; TRANS.20120815.21, lines 13 to 14; TRANS.20120815.86, lines 22 to 25
[180] TRANS.20120815.30, line 32 to TRANS.20120815.31, line 2
[181] TRANS.20120730.121, line 30 to TRANS.20120730.122, line 2
[182] TRANS.20120730.125, lines 1 to 3
[183] TRANS.20120731.112, lines 14 to 32
[184] For example the issue was pressed by counsel assisting with Mr Scott who said he did not associate Alan Reay with that product for any particular reason and it was not recommended to him – TRANS.20120808.108, lines 20 to 31
[185] TRANS.20120808.104, TRANS.20120808.108
[186] TRANS.20120809.50, lines 17 to 25

Influence over permit process

183. Some witnesses believed (almost all of them from anecdote or hearsay) that the building permit for the CTV Building was somehow pushed through by the influence of Dr Reay. Mr Henry said in his evidence that ARCL did not like the scrutiny of Mr Tapper and would go to Mr Bluck to override Mr Tapper.[187] Dr Reay denied, on multiple occasions, that this occurred,[188] even as a general proposition, and certainly not on this job.

184. Mrs Tapper's evidence that her husband went *"on and on about the CTV Building"* proved to relate to a period of approximately one week, at the end of which Mr Tapper attended a meeting where he clearly intended to present his views, and commented (jokingly or not) that he might lose his job.[189] Whether he was being light-hearted or serious, that evening he told his wife that the issue was resolved and he never mentioned it again.[190] Mrs Tapper's evidence was hearsay, and in her initial brief it was significantly overstated. Once she had the opportunity to state it her way it gained a completely different character.

185. In the light of the memorandum of counsel for the Christchurch City Council dated 22 August 2012, it might not even have related to the CTV Building.

186. Assuming that it is more probably the CTV Building, two important additional facts emerge. First, it is then clear that the building and its compliance had such exceptional attention from Mr Tapper that it must have been scrutinised at permit stage with almost military thoroughness. Next, once he signed the structural consent on the permit form on 10 September 1986, he cannot have continued to hold concerns, as he did not mention it again and Mrs Tapper said that in later years he went into the Building for filming with Grey Power.[191]

187. Mr Peter Nichols mentioned that Dr Reay could go over the head of the engineer assessing the bylaw compliance and speak to Brian Bluck directly.[192] Dr Reay accepted that he had direct contact with Mr Bluck on

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[187] TRANS.20120802.52, lines 20 to 26
[188] TRANS.20120731.126, lines 16 to 21, TRANS.20120807.106 to TRANS.20120807.110
[189] TRANS.20120802.81, line 21
[190] TRANS.20120802.81, line 29 to TRANS.20120802.82, line 3
[191] TRANS.20120802.82, lines 1 to 6
[192] TRANS.20120806.66

occasion, but he was not alone in this respect,[193] and he explained the many other matters that he had occasion to speak with Mr Bluck about.[194]

188. Mr Nichols' evidence in respect of the CTV Building related to a conversation with Mr Bluck in a nearby street (with an obscured view of the partly built building) when Mr Bluck assured Mr Nichols that he had *"carried out due diligence and had been convinced by Alan Reay that his reservations were unfounded"*.[195] Mr Nichols expanded on this in cross-examination when he said that he understood Mr Bluck's reference to due diligence to mean that *"he had been pretty thorough about having it checked"*.[196] In addition, Mr Nichols referred to Dr Reay convincing him in respect of the "innovative" design concept, but it is not known whether that related to this building, another building (e.g. Landsborough), or a more general discussion on construction techniques.

189. All witnesses were agreed that Mr Bluck would not be overridden. He was described variously by different witnesses as:

(a) Professor Mander: *"...a man to be revered in the city. He knew everything, and he is the sort of person that would've had all this knowledge in his head. He would've known the types of buildings that he would've permitted and had somebody said to him, "Hey, these buildings could've been in trouble," he would've been the sort of person that would've advised the mayor that there may be some problems here, we should be extra cautious*."[197]

(a) Mr Henry: "*Bryan Bluck usually came and he would be very forthright with what he said. He would make it absolutely clear that what, if he thought he had something to say*."[198]

(b) Mr Nichols*: "Bryan Bluck held a civil engineering degree and was a registered engineer with considerable experience in structural engineering. He had held that position for many years, certainly more than I am able to quantify and was almost an institution in his own right. During the period I worked with Bryan I was aware that his acknowledged expertise was being utilised by his periodic appointment as a committee representative responsible for the*

[193] TRANS.20120814.98, lines 23 to 28
[194] TRANS.20120807.106, lines 16 to 21
[195] TRANS.20120806.63, lines 32 and 33
[196] TRANS.20120806.71
[197] TRANS.20120723.17, lines 11 to 17
[198] TRANS.20120802.108, lines 29 to 31

preparation of a number of New Zealand Standard building bylaw documents." [199] In response to a suggestion that Mr Bluck would not have known enough about the technical details of the code to determine whether the aspect queried by Graeme Tapper met the code or not, Mr Nichols said: "*Well I strongly disagree with it. I can't imagine that the gentleman knew Bryan as well as I did but if it was even vaguely true then we wouldn't have had one catastrophic collapse, we would have had a multitude of them because Bryan had the overall responsibility for approving them and obviously he got it right most of the time*."[200]

(c) Dr Reay: "*Bryan Bluck knew all the engineers around Christchurch. He knew the strengths and weaknesses of the Christchurch structural engineers. He would have weighed up the complexity of the proposed building with those qualities of the design engineer in determining the extent of a structural review. ...Mr Bluck and Mr Tapper were both dedicated and competent engineers. Mr Bluck had been at the council for many years. I am unsure of Mr Tapper's background, but his role at the Council was secondary to that of Mr Bluck*."[201] Dr Reay later described Mr Bluck as "*a very competent engineer*."[202]

190. Details of Mr Bluck's professional credentials attest to his qualifications and extensive experience.[203] He was made a Fellow of the Institution of Professional Engineers in 1994.

191. When Mr Hare went to visit Mr Bluck in 1990, Mr Bluck identified, in considerable detail, four possible issues for Mr Hare. Mr Hare's note records that they discussed easements, construction of the fire escape, and the vehicle entrance.[204] Mr Hare also recalled a discussion about fire egress.[205] Clearly Mr Bluck had a detailed knowledge about the building. He never mentioned any issue with the structural design or the permit process. An appointment was made for the meeting.[206] Mr Bluck as a careful person would have obtained the building file before or at the meeting with Mr Hare (though the latter did not recall if he did).

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[199] TRANS.20120806.59, line 34 to TRANS.20120806.60, line 8
[200] TRANS.20120806.70, lines 28 to 33
[201] TRANS.20120807.83, lines 19 to 29
[202] TRANS.20120807.110, line 6
[203] BUI.MAD249.0339 and BUI.MAD249.0339A
[204] WIT.HARE.0001.47
[205] TRANS.20120816.55, lines 17-18
[206] Mr Hare's note records "2-15" and Mr Hare accepted this likely meant he went to see Mr Bluck at that time TRANS.20120816.89, lines 26 to 31

192. Put simply there is no reliable evidence to support a contention that Dr Reay exercised any influence over Mr Bluck or Mr Tapper in relation to the permitting of the CTV job. To the contrary, the evidence demonstrates that the Council did a thorough review of the building and was satisfied that a permit should issue. Further, the chronology of events proves that the authorisation process followed a prompt standard practice incorporating a detailed review by the Council.

Date	**Description**
Tuesday 26 August 1986	Structural drawings received by the Council[207]
Wednesday 27 August 1986	Mr Tapper writes to ARCE with queries and requirements (sent to ARCE's PO Box address)[208]
Thursday 28 August 1986	
Friday 29 August 1986	
Saturday 30 August 1986	
Sunday 31 August 1986	
Monday 1 September 1986	Mr Harding received Mr Tapper's letter of 27 August 1986 (note pencil note in the top right-hand corner)[209]
Tuesday 2 September 1986	
Wednesday 3 September 1986	
Thursday 4 September 1986	
Friday 5 September 1986	Mr Harding replies to the Tapper letter of 27 August 1986[210]
Saturday 6 September 1986	
Sunday 7 September 1986	
Monday 8 September 1986	
Tuesday 9 September 1986	
Wednesday 10 September 1986	Mr Tapper signs off on the structural aspects of the building in the permit documentation[211]

193. In Mr Harding's later letter in relation to fire issues he states there had been a discussion with the Council at the permitting stage.[212] It is logical that there was a review, discussion and decision on the permit application in the period between 5 September (when the reply to the Tapper questions was sent) and 10 September 1986 (when the sign off occurred).

[207] BUI.MAD249.0141.8
[208] BUI.MAD249.0141.14
[209] BUI.MAD249.0141.14
[210] BUI.MAD249.0141.01 (at an unknown time after sending his letter, Mr Tapper received the requested calculations – note the margin note ("*rec-d a day or two after letter sent*" on BUI.MAD249.0141.14)
[211] BUI.MAD249.0141.8
[212] BUI.MAD249.0152.3

Post-construction – drag bars

194. In 1990 Holmes conducted a review of the CTV Building at the instruction of Schultz Knight to assess the CTV Building for the possible purchase by the Canterbury Regional Council ("**CRC**"). In carrying out its review, Holmes inspected four floors of the building and other areas, reviewed the structural plans and calculations, did their own calculations and met with Mr Bluck at the Council. Mr Bluck identified areas that called for close review in the building, including the fire escapes, easements and a possible issue relating to welding. Apparently no mention was made of any structural or general concerns.

195. Holmes found one issue: the connections from the slab to the north shear wall.[213]

196. Although noting that limited time had been available for the report, the report was complete and on its face in final form. Holmes concluded in respect of all other structural matters: [214]

> The layout and design of the building is quite simple and straightforward and generally complies with current design loading and materials codes.

197. Dr Reay did not understand the report to be a draft.[215] Neither did his co-director Mr Banks, who investigated the issue and designed the repair. ARCL was entitled to, and did, rely on Holmes' conclusions as to the state of the building as final conclusions. The report as presented to ARCL had no disclaimers. From 15 February at the latest Holmes knew ARCL had the report and were working on it.[216] The submission of Counsel Assisting that ARCL was not legally entitled to rely on the report[217] has no factual or legal substance.

198. The Prime West receivers as the owner of the building were informed about the issue that had been identified by Holmes within a day or two of learning of it.[218] From that time both the owner and the prospective owner CRC were aware of the issue.

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[213] BUI.MAD249.0130.8
[214] BUI.MAD249.0130.5
[215] TRANS.20120817.115, lines 1 to 2
[216] TRANS.20120816.62, lines 7 to 14;TRANS.20120816.84, lines 25 to 31
[217] TRANS.20120827.CS.126, paragraph 610(a)
[218] BUI.MAD249.0129.27

199. Counsel Assisting have labelled it "puzzling" that Dr Reay "did not tell Mr Banks of David Harding's inexperience."[219] But Dr Reay did not consider Mr Harding inexperienced, and in relation to load path issues, Mr Harding certainly was experienced. This exemplifies the “personal theories” error into which such submissions fall, contradicted by the facts.

200. ARCL notified its insurer as an abundance of caution and prepared preliminary calculations for remedial works.[220] The possible sale to CRC did not proceed. The receivers did not instruct or permit repairs. The building remained unoccupied and in early 1991 new owners were reported in *The Press* to have taken over.[221] To back up what should have been knowledge from the vendors, ARCL brought the issue to the attention of the new owner. In particular:

(a) Each of Mr Banks and Dr Reay formed the view that they would tell the new owner, as they stated in evidence.

(b) It was the preliminary view of ARCL's insurer that nothing further should be done,[222] but ARCL persisted regardless. In the face of this view, legal advice was sought. However both Mr Banks and Dr Reay (independently) confirmed that they were determined to notify the new owner and seeking advice was in order to ensure ARCL's insurance position was preserved.[223]

(c) ARCL did proceed to notify the new owners and further correspondence ensued.[224] Others should have advised the new owners but did not (the receivers, Holmes, real estate agents). Counsel Assisting criticise Dr Reay and Mr Banks for not contacting the receiver during this period.[225] This ignores that the vendors should have disclosed and ARCL did not know they had not. ARCL was throughout waiting for direction from the receivers to repair. Only when there was public reference to a sale, without ARCL being instructed to repair, did it occur to Dr Reay and Mr Banks that the vendors might not have disclosed.

[219] TRANS.20120827.CS.125, paragraph 609
[220] BUI.MAD249.0130.15 to 20
[221] BUI.MAD249.0438.1
[222] BUI.MAD249.0227.6
[223] TRANS.20120817.75, lines 15 to 21; TRANS.20120817.108, lines 18 to 19 and TRANS.20120817.139, lines 24 to 28
[224] BUI.MAD249.0129.50
[225] TRANS.20120827.CS.128, paragraph 619

(d) Alone of all the engineers that had had involvement with this issue, ARCL followed through and made sure the issue was dealt with.

(e) The claims of Mr Robertson that engineers other than ARCL were ethically correct in knowing of the issues but doing nothing cannot be sustained either generally or in the context of the duty of engineers to design safely and without risk to life. If his interpretation and opinion has any wider currency, the Commission should put the position beyond doubt.

(f) Finally, Counsel Assisting exaggerate the risk, which was a risk if the building was occupied (which it was not) if a major earthquake occurred, and the repair had not been done before occupancy. At that time and indeed at all times leading up to the earthquake sequence starting in September 2010, buildings in Christchurch that were less than 10% of the new building standard were occupied. The Christchurch City Council policy required an upgrade of these "earthquake prone" buildings on a schedule under which the most urgent case required an upgrade within 15 years. Grant Wilkinson formerly of Holmes stated he was "comfortable" with the timeline from discovery to repair.[226]

201. Neither ARCL nor the building owner, not the contractor obtained a permit for the retrofit works. In the circumstances of the time, none was perceived to be required. Dr Reay's evidence is that he believed, based on his experience in dealing with Mr Bluck over many years, that Mr Bluck (and so the Council) would regard the works as part of the original job and that no permit was required.[227]

202. Indeed under the current Building Act 2004,a permit is not required in these circumstances. Clause (ag) of Schedule 1 of the Building Act 2004 provides that no consent is required for:

> the alteration to the interior of any non-residential building (for example, a shop, office, library, factory, warehouse, church, or school), if the alteration does not—
> (i) reduce compliance with the provisions of the building code that relate to means of escape from fire, protection of other property, sanitary facilities, structural stability, fire-rating performance, and access and facilities for persons with disabilities; or
> (ii) modify or affect any specified system

[226] TRANS.20120816.112, line 27 to TRANS.20120816.116, line 25
[227] TRANS.20120817.109, lines 20 to 24

203. This may well reflect the actual practices which preceded the Act. Whether a consent was required at the time is open to doubt. The Christchurch City Building Bylaw 1990 provided that no person could erect or commence to erect any building without first obtaining a building permit from the Engineer.[228] Building was defined in the 1990 Bylaw as follows:[229]

> *"Building" in addition to its ordinary and usual meaning, means any thing or part of a thing constructed or erected whether temporary or permanent, movable or immovable, but for the purposes of this bylaw does not include [listed exclusions].*

204. On one interpretation, this extraordinarily wide definition (in particular, "part of any thing") could capture virtually any form of work within a building, for example building a shelf into a wall or replacing wall linings. Council permit staff must have exercised a discretion as to when a permit would be required. Dr Reay's recollection of Mr Bluck's likely approach may well be correct.

205. Considerably more liberal definitions and extensive exclusions were provided in the 1991 and 2004 Building Acts, with the focus shifting from a "thing or part of a thing" to a "structure".

206. Finally in this context, reference is made to the cross-examination of Dr Jacobs, which demonstrated that, despite the best expertise, competence and care, connection loading requirements can be overlooked or miscalculated in first designs and need to be added later.[230]

207. The 1991 repair marks the end of the involvement of ARCL with the building.

Subsequent inspections

208. In the mid-1990s Mr Tyndall inspected the CTV Building following an earthquake on the Alpine Fault. He did not find any issues.

209. In 1998 or 1999 Mr Mitchell carried out a desktop review of the structural drawings of the CTV Building in order to assess suitability of the building for occupation by Opus. Mr Mitchell concluded that the interconnections between the floor diaphragm and the shear core were not as strong as they should have been and that there was a lack of alternative load paths in the event that the primary load path failed. In fact, the issue identified by Mr Mitchell had been responded to in 1991 by the addition of drag bars

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[228] ENG.CCC.0045.18
[229] ENG.CCC.0045.11
[230] TRANS.20120813.104, line 1 to TRANS.20120813.106, line 8

although Mr Mitchell was not to know this. The points relevant now are that Mr Mitchell did not identify any other issues from his desktop review of the structural drawings and he did not find it necessary to pursue the issues that he had identified in any way.

210. In or about 1999 to 2000 an internal stairwell was installed between Levels 1 and 2 to provide access between the two floors of the CTV tenancy. The design work for the stairwell installation was carried out by Mr Falloon. In carrying out this design work Mr Falloon had to identify and review load paths within the building and how the stairwell would impact on those load paths and the connections between, in particular, the south shear wall and the floor slabs. Mr Falloon did not identify any issues with the load paths in the building.

211. Counsel Assisting's criticism of ARCL (which is then focussed on Dr Reay) is dependent on the assumption that he can reasonably be expected to have found matters other than the load path issue which each of Harding, Tapper, Bluck, Hare, Mitchell and Falloon did not identify in their respective assessments. Dr Reay readily acknowledged that the load path issue was obvious on looking at the plans, which in itself tends to prove he did not look at them in 1986. In respect of all other non-compliances or weaknesses now identified, even if there had been a review by Dr Reay in 1986 it cannot be assumed that this "permit-level check" would have picked such issues up when no other engineer did. The closing submission of Mr Elliott that it was "inconceivable that Dr Reay did not know" about these alleged defects[231] is unsustainable in these circumstances.

Change of Use

212. The CTV Building underwent a number of changes of use after its original construction. The most significant changes were as follows:

(a) The Going Places Tenancy on Level 3 in 2001;

(b) The Kings Education tenancy on Level 4, date unknown.

(c) The Clinic medical centre on Level 5 in 2011.

213. Brief comments on these issues only are offered. A detailed analysis was presented by counsel for the Christchurch City Council in opening submissions. That described actual practice of the Council, and shows they

[231] TRANS.2012.0828.CS.5

did not consider the changes of use required either building strengthening or new consents. As the latter two were never checked, that is at best a tentative (and probably erroneous) view.

214. These (and other) tenancies represented new opportunities to review the structure. As neither the owner nor prospective tenants applied after the initial Going Places change, the checks required were not done.

215. The Going Places tenancy was correctly identified as a change of use and processed as such by the Council.[232] The Council needed to be satisfied that in its new use the building would have complied with the provisions of the building code for, inter alia, structural behaviour, as nearly as is reasonably practicable to the same extent as if it were a new building.[233] A desktop assessment by the Council that the building was "reasonably modern", frame/ shear wall building[234] appears to have formed the basis for the decision that no further structural review was required.

216. The timing of the commencement of the Kings Education tenancy is not known and no building consent application was lodged with Council. Regardless, as acknowledged by Council in its opening[235] the new tenancy would have amounted to a change of use under either Act.

217. Mr Drew's medical practice, The Clinic, moved into level 5 following the red stickering of its former Gloucester Street premises as a result of the Boxing Day earthquake. The Council has no record of the new tenancy. However, on the basis that level 5 was previously used as either a physiotherapy clinic or office space this was not seen as a change of use for the purposes of the Building Act 2004. However depending on its scale and the range of medical services provided it may have been a change of use (to "health facilities") under the District Plan.

218. At paragraph 256 of their submissions,[236] Counsel Assisting have criticised the approach of Dr Reay and ARCL to the change of use issues. What is claimed by them does not logically follow. The 1991 works were one opportunity for Council review, if a permit was in fact required; each change of use was another. The evidence clearly supports the importance of change of use reviews and shows that material changes of use were able to

[232] BUI.MAD249.0151C.29, BUI.MAD249.0151C.44, BUI.MAD249.0151C.52, BUI.MAD249.0151C.60
[233] Section 46(2)(a) Building Act 1991
[234] BUI.MAD249.0151C.37
[235] Paragraph 90
[236] TRANS.20120827.CS.60

occur without the involvement of the Council. Each change alters the demands on the building which may create ongoing new risks.

219. The evidence is that the building was not being well managed through this period. There is no evidence of any maintenance programme, with the possible exception of lifts. An example of absence of control is the fact that an unknown number of holes were drilled in unknown locations. This issue is not “light relief” (as Counsel Assisting submit). Every such structural invasion raises serious issues. The witness who voluntarily came forward was unreasonably brushed aside by Counsel Assisting, who it is now clear thought him and his issues a joke. They were not, but the extent of slab and beam weakening from such works cannot now be resolved.

September 2010 earthquake

220. On 4 September 2010 the CTV Building sustained the first major earthquake, a magnitude 7.1 earthquake centred near Darfield. This earthquake was described by Professor Mander as a "design level" earthquake.[237] He supported that statement by reference to spectral response acceleration results.[238] Dr Bradley's analysis also showed that the September earthquake was "essentially equivalent to a design ground motion for structures with a vibration period of 1 second."[239] As noted above, in their 1986 seminar paper, Park, Paulay and others equated a design level earthquake to a major earthquake.[240]

221. The extent to which the CTV Building was damaged in the September earthquake cannot now be ascertained with certainty. The second NTHA conducted by the panel convened by the Royal Commission identified likely damage in the September earthquake as follows:

(i) Inelastic behaviour of the line F columns (i.e. yielding of bars).

(ii) Likely disconnection of level 4 drag bars.

(iii) Some inelastic behaviour in beam-column joints (but not exceeding peak capacity).

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[237] TRANS.20120723.12, line 8
[238] TRANS.20120723.20, lines 15ff
[239] WIT.BRADLEY.0004.2
[240] BUI.MAD249.0519.12

(iv) Some (minor) inelasticity at the base of the North and South walls.[241]

222. Additionally, there are many indicators of some damage. It seems likely that there was some slab separation from the north shear wall. It is also likely that there was south shear separation.[242] This inference is from analysis, however such insight is important because visually such damage is unlikely to have been directly observable.

BUI.MAD249.0386B.8 [32]

[241] See paragraphs 264 and 265 below for remaining perceived shortcomings with the second NTHA.
[242] See for example the photographs at BUI.MAD249.0386B.8 [photograph [32]) and, on a different floor, BUI.MAD249.0386.10 [photograph [40] – adjacent to the black shoe shown in the photograph annexed to the evidence of Mr Pagan

BUI.MAD249.0386.10 [40]

223. It is also quite possible that in parts the slab delaminated from the Hi-bond during or as a result of the September earthquake, but this could not be seen without pulling up carpets, which did not occur. Observed from the floor below, Mr Coatsworth considered the Hi-bond looked satisfactory, but this would not identify any delamination above the Hi-bond. Professor

Priestley identified the possibility of cracking on the floor mesh, and noted that this damage may not be visible to an inspecting engineer.[243]

224. The reports of the occupants in the building are difficult to ignore. They report, almost universally, increased liveliness, discomfort and noise following the September 2010 earthquake. In **Schedule 4** annexed, all such evidence has been extracted from the transcript as a reference. The suggestion that the building did not sustain any serious damage in the September 2010 earthquake is simply unable to be reconciled with these many and varied observations.

Post September assessments

225. On 5 September 2010 there was a Level 1 rapid inspection of the CTV Building by a team which included a CPEng engineer. The building was given a green placard with no damage noted.[244]

226. On 7 September 2010 the building received a Level 2 rapid assessment by three Council inspectors, none of whom were engineers.[245] The building was again green-stickered with no damage noted.

227. There seems to have been a general misconception amongst tenants about the nature and result of the post-September assessments.

228. Kings Education reported in its newsletter that "*Civil Defence engineers inspected the structure and have informed us that it is safe to enter. They have also advised us to get the school underway again as quickly as possible so that things can return to normal*."[246] There is no evidence of any such recommendation being given.

229. The 7 September inspection was wrongly understood by some to be an inspection by three engineers. Mr Wood, the managing director of CTV reported to staff that the building had been inspected by three engineers.[247]

230. Following acceptance of his proposal, Mr Coatsworth was engaged to carry out an independent structural assessment. Mr Coatsworth's proposal to Mr Drew recorded that the basis for the inspection was: [248]

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[243] TRANS.20120711.69, line 28 to TRANS.20120711.70, line 3
[244] BUI.MAD249.0136.1
[245] BUI.MAD249.0137.1
[246] WIT.BREHAUT.0001.3
[247] BUI.MAD249.0388.1
[248] WIT.COATESWORTH.0001A.1

> I understand that the building owners are interested in having an independent structural assessment carried out.

231. In any event, no detailed structural analysis was performed. Mr Coatsworth did not review the structural plans, a critical omission in our submission. The plans were not available at the Council but Mr Coatsworth made no attempt to obtain the plans from ARCE or to follow up with a structural review when the plans became available from the Council. Mr Coatsworth reported on various minor incidences of damage within the building but raised no issues of significant concern and the building continued to be occupied in reliance on Mr Coatsworth's findings.

232. Undoubtedly both the Level 2 assessment and Mr Coatsworth's assessment gave a false assurance to the building occupants. Occupants were reassured that the building was safe to occupy while in fact no detailed structural review of the building had been conducted and therefore no assessment had been made of the structural capacity of the building following the significant earthquake on 4 September 2010.

September 2010 to February 2011

233. Between 4 September 2010 and 22 February 2011 a number of relevant events occurred:

    (a) Mr Drew failed to act on recommendations of Mr Coatsworth in his report. Mr Coastsworth recommended further investigation of the pin board lining on the south wall of level 1 and the western wall. Mr Coatsworth also recommended that a security fence be erected around the south wall fire escape to protect against injury from falling plaster. This, too, was not actioned by Mr Drew.

    (b) There were changes of occupancy which may have been unauthorised changes of use (discussed above).

    (c) The building sustained many small quakes and a larger one on 26 December 2010. Following a Level 1 Rapid inspection on 27 December 2011 a green sticker was assigned by the Council.[249] The Level 2 inspection form included a prompt for the inspector to recommend that a Level 2 or detailed engineering evaluation be carried out. Unfortunately, having regard to the nature of the building and the proximity and intensity of the Boxing Day earthquake, this box

[249] BUI.MAD249.0167.1

was not ticked. Accordingly, there was no further follow up from the Council.

(d) Tenants (especially after 26 December 2010) complained of noise, floor movement, a hump in the floor, cracked windows, broken cement. An experienced contractor Mr Reynish judged from a wall to frame gap that the building had gone out of square:[250]

> While I was on level 6 I noticed large gaps around the perimeters of the windows along the eastern side and part of the south side of the building... The join between the steel window frame and the concrete window opening is generally filled with silicone but in some places the steel window frame had pulled completely away from it and you could feel a draft..
> In my opinion the concrete building had moved but the window had stayed square, the metal, the metal in the glass of the window frame was square and stayed still and the building had moved and that's what it looked like to me.

(e) The "building manager" Mr Drew perceived a need for further engineering review but after one phone call did nothing more.

(f) The building on the Les Mills site next door was demolished. The effects of the removal of the old building, including the excavation of foundations along the west wall (after which the building movement became greater) and the vibrations from the wrecking ball are unknown.

234. For the Boxing Day earthquake the evidence of two witnesses is particularly relevant. Jo-Anne Vivian, describing the state of the building after this event reports being "shocked at the extent of the mess" and refers to, and produces photos of filing cabinets having fallen over, shelving emptied on to the floor and ornaments broken.[251] Ms Vivian contacted the Council to raise a concern about cracks in a structural pillar[252] but withdrew her request for an engineering inspection after being assured by Mr Drew that the building had been inspected by an engineer after the Boxing Day quake.[253] Mr Drew says he was relying on the Council green sticker.[254] Council records are consistent with Ms Vivian's evidence.[255] It is submitted that particular weight can be attached to her perception of damage requiring engineering review. In addition what she reported to Mr Drew was damage

[250] TRANS.20120702.135, lines 10 to TRANS.20120702.136, line 26
[251] TRANS.20120702.99, lines 28 to 30, BUI.MAD249.0382
[252] TRANS.20120702.106, lines 21 to 26
[253] TRANS.20120702.107, line 21 to TRANS.20120702.108, line 21
[254] TRANS.20120702.46, lines 1 to 34
[255] BUI.MAD249.0310.1 and .2

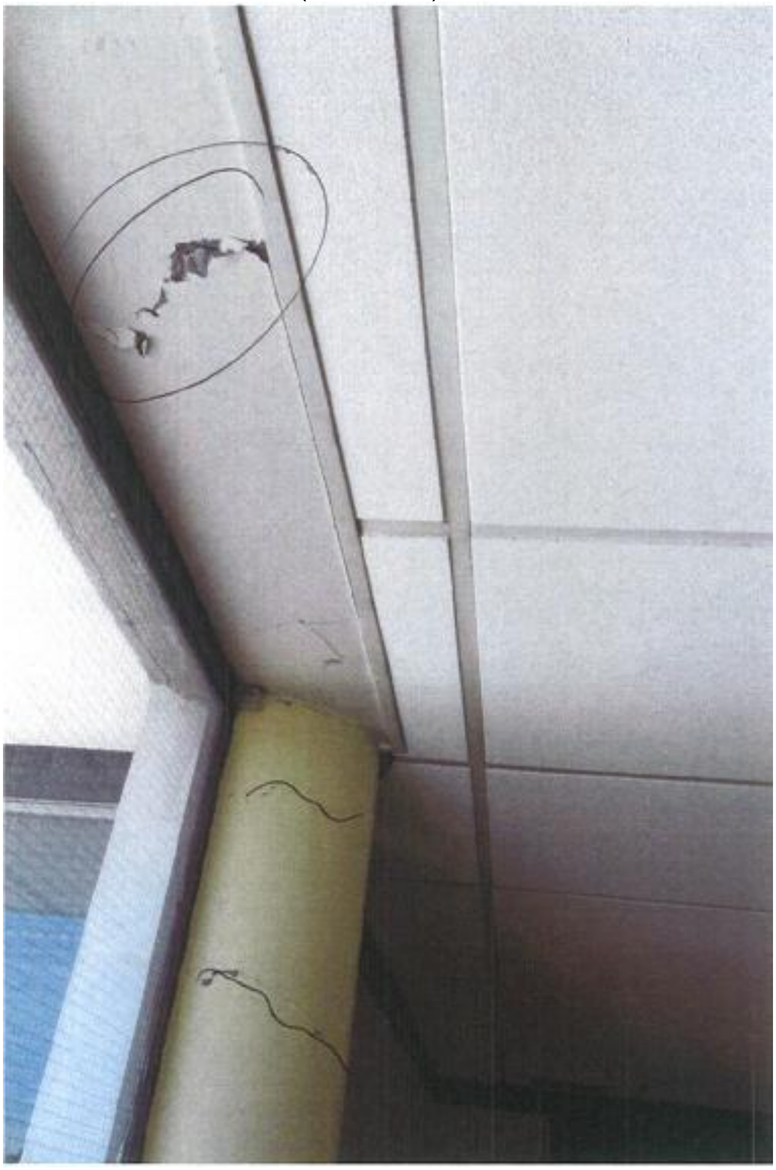
from the 26 December quake – a clear signal of more damage than that which Mr Coatsworth had reviewed.

235. The second important new evidence is that of Mr Higgins, confirmed in a photograph he took in February 2011.[256] When compared with photographs of the same area taken by Mr Coatsworth in October 2010,[257] a marked change in the level of damage can be observed. Mr Coatsworth confirmed that he would have taken a photo of this damage if it had been there when he visited the building.[258]

WIT.COATSWORTH.0001H.35 (October 2010)

[256] WIT.HIGGINS.0001.9, BUI.MAD249.0454.1
[257] WIT.COATSWORTH.0001H.34 and .35
[258] TRANS.20120704.54, lines 12 to 23

WIT.COATSWORTH.0001H.34 (October 2010)



BUI.MAD249.0454.1 (February 2011)

236. The CTV Building had now been through a design-level earthquake, many aftershocks including a major aftershock,[259] and was damaged. Its remaining resilience in the face of a further major aftershock cannot now be assessed but was clearly significantly reduced from design levels.

237. In his second statement of evidence Professor Mander discussed the concept of low cycle fatigue in the context of the Canterbury earthquake sequence between September 2010 and February 2011.[260] Professor Mander observed that there were 15 earthquakes greater than or equal to M5.0 from 4 September 2010 to 22 February 2011. Focussing on the five with the highest recorded Peak Ground Acceleration, he noted that all five events have notable spectral response in the T = 1s period range which will produce ongoing cumulative demand on a structure with periods in the range of 1 to 2 seconds. After analysing the response spectra in the context of NZS4203 capacity requirements, Professor Mander concluded:[261]

(a) The CTV Building was exposed to cyclic demands considerably greater than what one would expect to observe back at the time structures were designed in the 1980's.

(b) It would have been prudent for all concerned to have been suspicious about the ability of the CTV Building, designed as it was in 1986, to have with withstood the earthquake sequence without a material loss of fatigue capacity in fatigue-prone regions such as column bars and also its associated loss of strength in the concrete damage-prone elements, in particular the beam-column joints. Only a structural analysis with references to the building plans, seismic and other information could allay those suspicions.

(c) Building survival to the excessive demands of the Canterbury earthquake sequence can only be attributed to a measure of over-strength. Ductility is not a substitute for strength.

238. Under questioning, Professor Mander emphasised that the type of damage resulting from low cycle fatigue may not be visible and more sophisticated techniques, such as ultrasonic tomography, may be necessary.[262]

[259] A full list of major aftershocks is at BUI.MAD249.0502
[260] WIT.MANDER.0002
[261] WIT.MANDER.0002.10
[262] TRANS.20120724.64 to TRANS.20120724.65

239. On 22 February 2011 at 12.51pm the magnitude 6.3 earthquake struck, tragically lending to the collapse of the CTV Building and the loss of 115 lives. A fire broke out in the remains of the building in the aftermath.

Collapse hypotheses

240. The discussion which follows on collapse hypotheses and concrete has been prepared with the assistance of Dr Bradley, utilising also the work of Professor Mander, Mr Haavik and the evidence hot tubs.

241. During the hearing into the failure of the CTV Building there have been several collapse sequences proposed by different experts. We now set out a comparative analysis of the principal scenarios.

242. The DBH report identifies four collapse initiators.[263] These four scenarios are:

(a) Scenario 1: Column failure on line F or Line 1 resulting from column failure (excessive drift). This is the authors' (Hyland and Smith) preferred scenario. It is noted that this scenario is with or without the influence of spandrel interaction, although the level at which the identified column failure occurs is noted as the one at which spandrel interaction would have occurred. Based on the authors' interpretations, this is consistent with collapse debris and eyewitness reports.

(b) Scenario 2: Failure of line 2 or 3 columns due to excessive axial load. The authors cite results of non-linear pushover analysis as evidence in support, as well as possibly low strength concrete, and high vertical accelerations.

(c) Scenario 3: Level 2/3 detachment from the north core. The authors cite the difficulty in assessment of diaphragm forces. They also cite interpretation of evidence that the level 3 slab did not disconnect, and hence postulate that this scenario is less likely than scenario 1 or 2.

(d) Scenario 4: Drag bar disconnection (L4 or L5) on line D or D/E.

243. The DBH report scenarios make no mention of beam-column joint failure. The focus in these scenarios is on exterior columns, whereas the interior columns had greater gravity loading. A conventional strength hierarchy

[263] BUI.MAD249.0189.124

analysis of the CTV structural elements illustrates that the beam-columns are critical, followed by the columns and then beams.

244. In the DBH report there is a large emphasis placed on several eyewitness accounts. It is important to note both the potential unreliability of eyewitness reports (particularly in isolation in such devastating situations), as well as the possible inability of eyewitnesses (both non-technical and even technical) to distinguish between large deformations in the structure that result from:

(a) the initiation of collapse; and

(b) the consequent large deformations once a collapse mechanism has formed (particularly in the case of eyewitness views external to the structure, in the event of failure due to structural element on the interior of the structure [e.g. not on lines 1 or F]).

245. A number of expert witnesses offered a critique of the Hyland/Smith collapse analysis in the DBH report and proffered their own collapse scenarios. The experts' respective positions are discussed below.

Mr Holmes

246. Mr Holmes noted that the DBH report focused on column hinging, and that, with independent shear walls, cannot alone lead to a collapse mechanism. Mr Holmes emphasised the failure of the beam-column joints (particularly at interior joints). He noted that there was little evidence of beams attached to columns, indicating joint degradation (whether column failure happened first or not).[264] Unlike columns, joint failure could be sudden and complete, due to a lack of joint reinforcing or confinement and hooked beam bars in the joint (particularly in lines 2,3 and A). This could lead to a global collapse mechanism. Column failure would lead to sideways collapse (but there was no evidence of this), joint failure would lead to gravity collapse.

Professor Priestley

247. Professor Priestley criticised the authors of the DBH report for excessive reliance on the ERSA analysis. He noted that the problems arising from this excessive reliance included that the analysis is not inelastic, it produces an underestimation of higher modes and an overestimation of torsional

[264] BUI.MAD249.0372.9

effects and that it is inappropriate when using recorded spectra compared with design spectra.

248. Professor Priestley observed that the DBH report largely focuses on the spandrel panels as the collapse initiator (i.e. Scenario 1). He considers this scenario unlikely for the following reasons:

(a) A gap was intended;

(b) Spandrel/beam connection failure would occur prior to hinging;

(c) Photos indicate spandrel diaphragm failure, but no column hinging.

249. In relation to the possible line F failure, Professor Priestley's position was that calculations show that a line F failure would not overload line E and that the line E columns have a lower axial load than line D columns.

250. Professor Priestley also comments on the issue of interior columns. He observed that interior columns have higher gravity loads and drift demands (on level 3). With vertical acceleration effects the demand/capacity ratio for D line columns becomes even greater relative to F line. In Professor Priestley's view, interior column failure would result in catenary action, likely causing failure of the adjacent columns; line F column failure would not.

251. Professor Priestley observed that the DBH NTHA suggested drag bar failure between the floor and the north core prior to column failure. He notes that the drag bar capacities in the DBH report were too high, based on bolts failing in shear. Professor Priestley's calculations suggest that the bolts would fail in flexure (at ~50% shear capacity).

252. Professor Priestley believes that the torsional eccentricity of the CTV Building was overpredicted by the DBH ERSA relative to the NTHA. Similarly, he noted that the effect of the masonry infill panels was overpredicted by the ERSA analysis.

253. Professor Priestley was reluctant to accept the DBH report NTHA analysis results. He believed that the analysis did not recognise the importance of beam-column joint capacity and that there was inadequate modelling, combined with post-processing.

254. Professor Priestley's hypothesised failure sequence was as follows:

(a) Failure of the floor diaphragm/north core connection early in the shaking;

(b) Distress to the beam-column joints;

(c) Spalling at base of beam-column joints, reducing vertical and lateral column capacity, which, combined with large vertical loads (including vertical acceleration effects) would result in explosive failure of the columns and the joints.

Professor Mander

255. Professor Mander emphasised the inconsistencies between the DBH report and the evidence provided by Mr Holmes and Professor Priestley. In this respect, Professor Mander supported other opinion that the DBH report is by no means definitive. In many respects, the collapse hypotheses of Professor Mander were similar to that of Mr Holmes. Professor Mander made the following key points:

(a) Spandrel panels and line F failure:[265] Professor Mander notes that the panels and line F columns were not significant in the collapse. He observed that these columns were more lightly loaded.

(b) Sideways or vertical collapse: Professor Mander pointed to the fact that the DBH report focuses on sideways collapse, while Mr Holmes and Professor Priestley both postulate gravity load failure (i.e. vertical collapse); which Professor Mander also supports.

(c) Postulated collapse modes: All of the collapse modes considered by Professor Mander involve some form of buckling of the internal gravity columns. The logic for this is that these columns have the greatest axial loads and therefore both the internal columns and beam-column joints are most vulnerable (as also noted by Mr Holmes and Professor Priestley).

(d) Collapse mode in East-West direction:[266] Professor Mander's postulated collapse mechanism in the East-West direction is based on disconnection of the beam and the line A (west) wall.[267] Once this unseating occurs, additional load is transferred to the line B columns and combined with a small eccentric interstorey displacement

[265] WIT.MANDER.0001.52
[266] WIT.MANDER.0001.81
[267] Figure 3.1, WIT.MANDER.0001.82

buckling over two storeys may occur.[268] Once one column fails it overloads other columns, creating a domino effect.[269]

(e) Collapse mode in North-South direction (north movement): The mechanism for failure resulting from the structure displacing in the North direction is based on the fact that when the structure moves to the North, the 'flexible' frames want to displace further than the stiff North core and as a result, the floors go into compression. Professor Mander asserted that this floor compression, combined with separation of the traydeck (Hi-bond) and slab concrete, and vertical accelerations, could cause the floor slab to develop a failure mechanism at midspan.[270] This action will lead to greater shortening of the slab, meaning larger displacement of the gravity column at that location. This slab failure would transfer additional load to the internal columns, and, combined with the column displacement, lead to buckling over two storeys.

(f) Collapse mode in North-South direction (south movement): The mechanism for collapse due to south movement is based critically on the lack of a drag bar(s) on the lower storeys. When the structure displaces to the south the flexible gravity frames will try to displace further than the stiff north core. If the drag bars maintain their connection then this will result in a differential displacement of the lower floors of the gravity frame and the north core.[271] This may lead to unseating of the floor slab at these lower levels. Similar to the above hypotheses, this action would transfer additional gravity load to the internal columns, and, combined with displacements, would lead to buckling. If the drag bars in the upper floors failed, then the buckling may have occurred over more than two floors.

256. During his oral evidence, Professor Mander noted that his collapse hypotheses do not imply that the hypotheses of Mr Holmes and Professor Priestley are incorrect, just that these additional hypotheses should be added to the list of possibilities.

257. As noted above, Professor Mander also postulated the importance of cumulative damage effects on the CTV structure. In support of this contention, he pointed to the increased 'liveliness' of the floor system and

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[268] Figure 3.2, WIT.MANDER.0001.83
[269] Figure 3.3, WIT.MANDER.0001.85
[270] See Figure 3.4 upper panel, WIT.MANDER.0001.87
[271] Figure 3.4 lower panel, WIT.MANDER.0001.87

the fact that damage to elements such as the beam-to-slab connection and beam-column joints could not be observed based on visual inspections. The hypothesised collapse scenario put forward by Professor Mander does not require that these assertions in relation to cumulative damage are actually correct. However, if correct, the cumulative damage would have increased the vulnerability of the CTV Building to all the postulated failure modes of Professor Mander, Mr Holmes, Professor Priestley, and Hyland/Smith in the DBH report.

258. In his third brief of evidence Professor Mander[272] described the experimental results and analytic modelling of the axial compression tests conducted on the full-scale column remnants extracted from the CTV Building. Those results show from the inverse modelling analysis that the concrete material exhibited "softer" behaviour than would usually be expected from concrete in a pristine (unused and undamaged) condition. It was shown from historic tests that were conducted in a similar fashion to the CTV Building tests, that increasingly large (stress) amplitude cyclic axial compression is a cause of concrete softening.

259. The concrete softening effect in the context of the CTV Building is ascribed to the cumulative effects of previous cyclic damage from earlier earthquakes, particularly those events that had high vertical components in the ground motions at frequencies greater than some 3 Hz, as demonstrated in the second evidence brief of Professor Mander.[273]

260. Concrete softens even further due to the damaging effects of large sidesway motions when present.[274] Softer concrete (which means a smaller than normal Young's modulus), gives rise to the greater possibility of column stability failure in the form of the collapse mechanisms advanced by Professor Mander in his first brief of evidence.[275]

Second NTHA

261. At the direction of the Royal Commission an expert panel was convened to develop consensus on the NTHA that was performed by Compusoft Engineering Limited as part of the DBH report.

[272] WIT.MANDER.0003.6 to .7
[273] WIT.MANDER.0002.4 to .10 and BUI.MAD249.0531A.11 to .12
[274] WIT.MANDER.0001.50
[275] WIT.MANDER.0001.79 to .88

262. Following the first meeting of the expert panel it became immediately apparent that many of the assumptions in the NTHA model used in the DBH report were inappropriate in view of the potential failure mechanisms which may have occurred and therefore additional analyses were suggested.

263. The revised or ‘second’ NTHA contained several improvements over the original NTHA, which include:

(a) The beam-column joints are now modelled with a moment-rotation spring (previously rigid).

(b) The effect of axial load on the column capacity was explicitly considered via a PMM (Axial Load (P) – Moment – Moment) interaction.

(c) The cumulative effects from the 4 September 2010 Darfield earthquake were explicitly considered in several analyses.

(d) All four ground motions in the CBD were used to improve the reliability of the analysis results.

(e) Only the analysis case in which the masonry infill panel interaction on the west was negligible was pursued.

(f) The floor diaphragm out of plane behaviour allows for the potential for nonlinearities in the connections to the internal beams.

264. Despite the improvements in the model there remain significant limitations which need to be borne in mind when scrutinizing the results obtained. In order of inferred importance these include:

(a) <u>Beam-column joints:</u> There is significant uncertainty as to the peak capacity of the joint; and the constitutive model does not consider degradation of joint strength over successive cycles of loading, which will occur following the peak joint capacity being reached (as a result of cracking). The second point is particularly important, because such degradation effects are considered in the fibre-modelling of the column elements, and therefore this impairs the ability of the analysis to allow for beam-column joint failure, prior to column failure. The beam-column joint model also does not consider the time varying effect of axial load, which is known to be significant as a result of significant vertical accelerations.

(b) Drag bars: The drag bar strengths used in the revised NTHA analysis are likely an upper bound (as noted in the joint panel report). BECA provided information on their view for the drag bar strengths. Given that analyses for the 4 September 2010 earthquake illustrate that failure of drag bars was likely to have occurred (not to mention failure early in the 22 February 2011 ground motion), then the use of a lower (and arguably more realistic) value will likely indicate a greater predominance of drag bar failure than the current analyses already illustrate.

(c) Beam bar pullout: was not explicitly modelled in the revised NTHA. Post-processing of the analysis results suggest that beam pullout demands exceed their capacity.[276] As a result, this failure mechanism should be modelled explicitly in any revised analyses, to allow for effects subsequent to this failure to be considered, and understand whether it is important in the global failure of the CTV structure.

(d) Bar buckling: As a result of a lack of confinement in the joint, rebar buckling is likely to occur at small axial strains (particularly the pair adjacent to each side without the adjoining beams). This buckling effect is considered to assist in breaking off the "wings" in the precast units leaving the joint completely exposed to rapid failure. This was not modelled.

(e) Concrete strength: There was debate on the uncertainty in concrete strength. Initial analyses used $f_c' + 2.5MPa$, while the revised analyses have used $1.5f_c'$. As several potential failure mechanisms are not directly related to concrete compression/tension capacity, then the variation in concrete strength may result in a different sequence of local failures, leading to the global collapse mechanism.

(f) Foundation soil: The effects of the foundation soils were considered simplistically using linear springs. As soil nonlinearity occurs at infinitesimal strains then some level of plastic deformation in soils is always occurring. The explicit modelling of soil nonlinearity would result in the ability of the foundations to have differential settlement, allowing redistribution of forces in the structure, which maybe significant in leading to additional distress in several critical elements.

[276] BUI.MAD249.0547.7, 3(f)

(g) Large displacements: The analyses utilize small displacement theory (with a "PDelta adjustment"). Hence, geometric nonlinearities are not explicitly considered, which are likely significant given the high axial loads on vertical load resisting elements. Differential vertical deformations as a result of foundation settlements and/or initiation of beam-column joint or column collapse will lead to redistribution of loads which may overload other elements, and are not currently considered. It is noted that the neglect of large displacements does not allow for the possibility of 'buckling-type' failure that is postulated as a possibility in evidence of Professor Mander.

(h) Sensitivity studies: It is conventional in numerical analyses to consider the sensitivity of the problem to uncertainties in input variables. In the points noted above such sensitivity studies should be conducted by considering alternative plausible models and/or uncertainties in their input parameters in order to assess the resulting variability in the analysis results.

265. Furthermore, NTHA results can never be considered determinative as they are entirely dependent on inputs and assumptions made in modelling the structure.

266. The revised NTHA results suggest (bearing in mind the above limitations):

(a) During the 4 September 2010 earthquake:

(i) Inelastic behaviour of the line F columns (i.e. yielding of bars).

(ii) Likely disconnection of level 4 drag bars.

(iii) Some inelastic behaviour in beam-column joints (but not exceeding peak capacity).

(iv) Some (minor) inelasticity at the base of the North and South walls.

(b) During the 22 February 2011 earthquake

(i) Drag bar disconnection at all floors early in the analysis (in all four analysis cases).

(ii) Column failure in the lower levels of the structure.

(iii) Potential pull out of the beams (based on post-processing).

(iv) Considerable damage to the beam-column joints (note earlier limitation that modelling did not predict degradation adequately).

267. Attached as **Schedule 1** is a table summarising the experts' views on the key issues affecting the collapse of the CTV Building and the insights from the second NTHA on each issue.

Concrete

268. Concrete evidence was presented by way of a hot tub. The participants were Mr Haavik, Dr Mackechnie, Mr Gaimster, Professor Mander, Dr Hyland and Dr Bradley.

269. **Consensus**: It became immediately clear that all witnesses except for Dr Hyland were in general agreement that the testing performed was unacceptable within the context of the significance of this hearing. Dr Hyland's cores did not comply with the concrete testing standards (neither ASTM nor NZS) as the specimens were too small.

270. **Core strengths**: Mr Haavik noted that his core strengths were ~2-55% higher than those of Hyland. This maybe the result of several factors:

(a) testing of damaged specimens;

(b) testing parallel vs perpendicular; and

(c) larger diameter specimens.

Mr Haavik's testing was in line with accepted practice and compliant with standards for testing and hence produces far more reliable results than those presented in the DBH report.

271. **Column C18**: Mr Gaimster and Dr Mackechnie vigorously argued that column C18 (also referred to as D/E4) was damaged even to the eye (i.e. macro-cracking and also fire damage). Dr Hyland attempted to deny this, but these comments were without basis. Comments as to damage were applicable to all specimens tested by Dr Hyland (because they were obtained transverse), but not Mr Haavik's (because they were taken down the core of the column).

272. **Number of specimens**: Only specimen C18 contained 6 samples (the industry minimum), with all other cores having only two samples per column

specimen. Thus, Mr Gaimster pushed the point that the results are unreliable because of limited samples.

273. **Schmidt hammer testing**: There was significant criticism of the use of Schmidt hammer hardness testing in lieu of directly obtaining concrete core samples. This is because the correlation between hammer results and core strengths are invariably poor. Adding to this generally poor correlation, there were only 6 points used in the correlation (codes say minimum of 9),[277] and also there were several mix designs (each of which would have a different correlation); and finally because of significant weathering during the 26 year life of the structure, and damage to the column surfaces from collapse/fire, the correlation of hardness and core strength will be even more variable. Despite all the above comments, Dr Hyland was unmoved.

274. **Orientation of aggregate**: It was noted by Mr Haavik that petrographic examination of some cores illustrated horizontal lamination of the aggregate. Dr Hyland noted that this may be the reason for the difference in the Hyland/Smith Cores (transverse direction) and Haavik (longitudinal direction), and that this may have an implication for the loading of the columns in shear (which he thought implies transverse loading). However, Dr Mackechnie noted that this lamination of aggregate would unlikely occur throughout the depth of the column (because of the laminar flow during placement), and only likely at the mid-height of the column. Professor Mander also added the comment that concrete structures do not carry shear force in conventional methods that are taught in introductory mechanics (only applicable for steel structures). Instead, concrete carries shear in a 'truss' mechanism, in which the concrete can act as compression 'struts', and the steel (both longitudinal and spiral transverse) as 'ties'. Because of the large spacing of the spiral reinforcing steel, the compression struts would have an angle near vertical. Thus the concrete compression in shear would be closer to the vertical angle than the horizontal. Hence, the comments by Professor Mander and Dr Mackechnie rebut the speculation by Dr Hyland that his low strengths may be an issue for shear loading.

275. **Interpretation of results**: There were also criticisms of the interpretation of the results in the DBH report with reference to specified concrete strengths. The comments are elaborated in the evidence of Dr Bradley, Dr Mackechnie, and Mr Gaimster. In all cases these three pieces of evidence

[277] BUI.MAD249.0373.5: BS EN 13791:2007 Assessment of in-situ compressive strength in structures and precast concrete components. (2007). London, United Kingdom: British Standards.

suggest that the values are consistent with the specifications (irrespective of whether the testing was flawed as noted above). Dr Bradley, in particular, concluded that there was no credible evidence to suggest that the observed concrete strengths in the DBH report were lower that the specified concrete strengths.[278]

276. **Summary**: The Commission should rely on the evidence of five independent witnesses, practising within their field of expertise, who assert that the concrete strength was not an issue in the CTV structure or its collapse.

277. ARCL's comments on the draft DBH report, prepared predominantly by graduate engineers Mr Latham and Mr Urmson[279] identified the very issues with the DBH report concrete testing that were ultimately identified by expert engineers before the Royal Commission.[280] However no material changes were made to the DBH report in response to these comments before it was finalised.

278. At paragraph 257 of the submissions of Counsel Assisting, it is submitted that on the evidence available to the Royal Commission it will not be possible to reach a firm conclusion on concrete strength. This is not accepted. The inadequate sampling, testing, interpretation and reporting procedures by the authors of the DBH report justify the conclusion in that report being rejected by the Commission. That is what each of Dr Mackechnie, Mr Haavik, Professor Mander, and Mr Gaimster have done. In contrast, the detailed work done by ARCL and its staff and experts, confirmed by the evidence from Dr Mackechnie and Mr Gaimster, enable the Commission to reach a clear conclusion on concrete strength, namely that there was no evidence of understrength concrete at the CTV site.

Probable Collapse Theory

279. The Commission's Terms of Reference include to enquire into why the CTV building failed severely.[281] This is probably the single most important issue.

280. It had been anticipated that Closing Submissions of Counsel Assisting would propose an answer and that these submissions would respond to it.

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[278] WIT.BRADLEY.0003.32
[279] TRANS.20120716.45, line 11
[280] BUI.MAD249.0195, BUI.MAD249.0195A
[281] Terms of Reference, page 2, paragraph (a)(i).

It has not been possible to take an overview of all the evidence until it was completed.

281. In fact, while the Opening of Counsel Assisting identified a number of questions and issues, presumably aimed at providing the information necessary to answer the central question why the CTV Building failed severely, in the Closing Submissions this essential question remains unanswered.

282. Those submissions propose that it seems unlikely the Commission will be able to reach a definitive view on the precise order of the collapse sequence and that the consensus of expert evidence is that there are several "critical structural weaknesses" in the building, with one or more plausibly the initiating event.[282]

"Critical structural weaknesses"

283. There is no formal engineering definition of "critical structural weakness". It was not a term used in 1986 or 1990.[283] The term "Critical Structural Weakness" (CSW) does not appear to have been in common use prior to the publication of *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes* by the New Zealand Society for Earthquake Engineering (2006).[284]

284. The term can reasonably be taken to mean some element or detail that, when/if it fails, results in disproportionate consequences (i.e. partial or complete collapse). It can arise from various sources including construction, design and inadequacies in the design codes themselves. It does not necessarily mean non-compliance with the original design code.

285. The import of paragraph 19 of the Closing Submissions is that despite the acknowledgment that the forces to which the CTV Building was subjected on 22 February 2011 were well above a design level earthquake, the vulnerabilities in the building referred to as "critical structural weaknesses"

[282] The Closing Submissions, paragraph 19, TRANS.20120827.CS.7.
[283] Mr Banks XMN TRANS.20120817.45, line 14.
[284] New Zealand Society for Earthquake Engineering (2006), *Assessment and Improvement of the Structural Performance of Buildings in Earthquakes*. The definition used in that publication (page 3-22) states: "A critical structural weakness (CSW) shall be deemed to exist if any of the features shown in Table IEP-3 exist". Table IEP-3 (page 3-14) relates primarily to issues regarding building layout, proximity to neighbouring structures, site stability and so on - although there is a reference (of uncertain scope) to "Other Factors" which rely on engineering judgment.

are proposed as material contributors to the collapse. In this regard at paragraph 19 of the Closing Submissions states:[285]

> The CTV Building is the only building in Christchurch, designed to the 1982 and 1984 Codes that suffered a complete[286] and catastrophic collapse on 22 February. Earthquakes search out weakness in structures and there were a number of critical structural weaknesses in this Building which the earthquake found.

286. The submission implicit in this, that the CTV building failed severely because it had alleged "critical structural weaknesses", says nothing about why the building collapsed in the way that it did and minimises to an unrealistic degree the important effects that both:

(a) the 4 September 2010 design level earthquake; and

(b) the 22 February 2012 above design level earthquake, with its extremely high vertical accelerations,

had on the collapse of the building.

287. The closest Counsel Assisting's Closing Submissions come to proposing a collapse scenario involves somewhat nebulous reliance upon “critical structural weaknesses”, with no single possibility gaining precedence (e.g. at paragraph 19):

> The consensus of the expert evidence is that there are several critical weaknesses that could have been the initiating event, but they are triggered in such quick succession, with only split seconds separating them, that if it was not one it would be another. A separation of the floor diaphragm from the North Shear Core continues to feature as a strong possibility and the most recent Non-Linear Time History Analysis (**NLTHA**) supports this, but it remains no more than one of several candidates.[287]

288. While referring to "several" critical weaknesses, the Closing Submissions do not here specify what these are, what they are relative to the initiating event or what the initiating event is.

289. Relative to the actual collapse of the building, not one matter has been proven to be a critical structural weakness contributing to the collapse.

290. Considering the level and range of evidence presented to the Commission, this lack of an answer is an important omission. At least an attempt at a more definitive answer should be made. With hesitation, since this work

[285] TRANS.20120827.CS.7.
[286] The north shear wall remained standing.
[287] TRANS.2012.0827.CS.8.

has had to be done unexpectedly and in a compressed time frame an attempt at analysing the evidence and proposing a conclusion on the probable reasons is now put forward.

Answering the TOR question "why the CTV building failed severely?"

291. If the question "why the CTV building failed severely?" is to be answered the best route to that answer must be the best possible understanding as to how it collapsed. There are many limitations on the scope and reliability of the evidence before the Commission relevant to this, including poor retention and cataloguing of elements of the building with consequent forensic limitations and the significant divergence of approach and views amongst the writers of the DBH reports and the experts who gave evidence. That is not however sufficient reason to avoid an attempt at answering the essential question.

292. It is submitted that the question is capable of an answer with an acceptable degree of certainty, in a manner consistent with the relative consensus that can be seen to have developed about where the collapse initiation began. The submission that follows builds upon the opinions expressed by, in particular, Professors Priestley and Mander and Mr Holmes as to where the collapse began.

293. The route to answering the question is to focus some key relevant evidence discussed in **Schedule 2**. Once answered, then on the basis of the submissions now made, that answer puts into perspective the true effect on the collapse of what Counsel Assisting has referred to as "critical structural weaknesses".

294. The **Schedule 2** analysis has been developed since the evidence by looking back at all the evidence, as a comprehensive analysis for submissions. The present discussion, and **Schedule 2** analysis, is now offered on the basis that it can be of assistance to the Commission when Commissioners come to the task of answering the “essential question” asked in the Terms of Reference.

295. In particular, after reviewing all the proposed collapse scenarios already summarised, the most credible analysis suggests there was an initiation of the collapse through disconnection of the suspended floor slabs from the southern shear wall, as explained in **Schedule 2**. This approach establishes direct linkage of this southern shear wall and slab disconnection

to the conclusions reached by Professors Priestley and Mander and Mr Holmes that the column and floor collapse seem to have been initiated from the internal columns.

296. If the **Schedule 2** analysis of key evidence of disconnection of the suspended floor slabs from the southern shear wall leading on to column and floor collapse in the vicinity is adopted, other important observations can be made. In particular:

(a) There is no other single factor that stands out to the same extent on the evidence as the obvious collapse initiator;

(b) As such, it is more probable than not that this disconnection was the initiator of the collapse; and

(c) Consequently, it is feasible to suggest that had the slabs not disconnected from the southern shear wall as now appears to have been likely, the building may not have collapsed so completely in the 22 February earthquake.

297. In conclusion it is submitted that:

(a) The collapse probably initiated from the southern shear wall (leading directly to column and slab failure in the immediate vicinity followed by all elements save the north shear core);

(b) The Closing Submissions have not identified any "critical structural weaknesses" that had a causative effect in the immediate collapse initiation;

(c) The columns in the vicinity that are likely to have failed under the lateral load of the collapsing section of floor slabs were all demonstrably compliant with the design code as it was in 1986;

(d) The disastrous collapse may not have occurred without the initial trigger of the southern shear wall connections disconnecting; and

298. On this analysis the CTV Building failed severely in part because of the damage caused by the 4 September 2010 design level earthquake which, without repair, meant the building could not withstand the exceptional vertical and other forces experienced on 22 February 2011.

Other matters

299. It is acknowledged that a separate hearing is to be held on the issue of post-earthquake inspection processes. ARCL has no oral evidence to add to that hearing.

300. Professor Mander's evidence included a recommendation that after an event such as the September 2010 earthquake, all buildings of a particular type should be red stickered – he advocated a cautious "guilty until proven innocent" approach.[288] This approach differs from current thinking in important respects. ARCL and its experts have since undertaken further work on a methodology to implement Professor Mander's recommended approach. This is set out in **Schedule 3**. The discussion in **Schedule 3** is a proposal of general application after having heard all evidence in the CTV hearing. It has been developed principally by Dr Bradley.

Why did the CTV Building collapse when other buildings did not?

301. The question as to why the CTV Building collapsed when others did not can only be answered by considering a number of factors, the most important of which are discussed below. They each relate to potential special vulnerabilities that the CTV Building may have had.

301.1 The physical characteristics of the site at 249 Madras Street and its location relevant to the epicentre of the 22 February earthquake may have made it relatively earthquake prone:

(a) However, any opportunity to determine its vulnerability in that respect has been lost. It will never be possible to say with any certainty how the CTV Building site reacted on 22 February 2011. It is unfortunate that no other person or organisation saw it as necessary to place a recording instrument on the CTV site following the 22 February earthquake as readings from the larger aftershocks on 13 June and 23 December 2011 may have provided some enlightenment. ARCL was the only party that took this step and delays with getting permission from CERA meant it was not in place until March 2012 and these key events were missed.

(b) The readings from the ARCL instrument of minor aftershocks post-March 2012 supported the inclusion of the higher REHS reading from

[288] TRANS.20120723.16, line 32

22 February 2011 in collapse analyses, but these minor aftershocks were insufficient to draw any meaningful conclusions about the response of the site on 22 February.

301.2 There was damage prior to 22 February 2011:

(a) To the southern shear wall. Refer paragraph 222 above.

(b) There were many observations about abnormal behaviour of the building. Refer schedule 4.

(c) The building survived the earthquake it was designed to survive and could not have been expected to have survived a further, stronger, earthquake event. (It should have been red-stickered after 4 September 2010 but was not.)

(d) The reasoning in (c) above is emphasised further if it was the case that the CTV Building was relatively vulnerable consequent upon the factors referred to each of the points made in this paragraph 301.

(e) As otherwise predicted from the revised NTHA analysis. Refer paragraph 265 above.

301.3 There were construction faults, in particular Mr Brooks' evidence (and the photograph) showing a beam not connected properly to the shear wall. Refer paragraphs 79 to 81 above.

301.4 There was not as much remaining plastic deformation capacity in the joints in the CTV Building as might have been expected due to the cumulative plastic deformation effects upon the reinforcing bars. This effect has been shown in other buildings, many of which were themselves very close to failing. The IRD Building was discussed in evidence. Reports relating to seven other buildings where the phenomenon was observed were provided to the Royal Commission on 13 August 2012.

301.5 The design of the building was permitted by the Code. The Code allowed design that was in relative terms weaker than it could have been. Deficiencies with the code are discussed in paragraphs 104 to 113 above.

Conclusion

302. The 22 February 2011 Christchurch earthquake was one of New Zealand's worst natural disasters. Indeed, the size of the area it impacted, the number of persons affected, and the injuries and deaths which it caused probably make it the worst. In turn, the collapse of the CTV building was the worst single event in that disaster.

303. This Royal Commission has been established so that we may learn from what occurred, identify the causes, and – so far as is humanly possible – ensure that we are better prepared and better protected in any future quake. The TOR detail that, and the questions to be answered.

304. Royal Commissions have a proud and very lengthy history. The origin of Royal Commissions can be traced back to eleventh century England with William the Conqueror's appointment of an 'inquiry' to prepare the Domesday Book of land ownership. In New Zealand the response to the most serious disasters has been Royal Commissions from which have come improvements in public safety.

305. Each Commission learns from the past so that we may look forward and do better. Those involved in these investigations feel many emotions – anger at what has occurred; pain and sadness at the death and injury which has occurred; often disbelief at the accident causes found. Disasters do not distinguish between those who die and those who live – the innocent injured, those who should have prevented the disaster, and those who have erred are all victims in the effect on their lives.

306. The Commission investigation process is rigorous, independent, and searching. But the process is neither a pillory at which those alleged to have erred are paraded and humiliated; nor a time of atonement. Punishment and apology are both relevant to the disaster, but not to the purpose of the investigation or its outcome.

307. For ARCL and Dr Reay, the collapse of the CTV building was stunning and then distressing. Their regret at this was stated in opening, expressed again when Dr Reay first gave evidence, and again as a later personal statement of apology. It detracts from these apologies, and achieves nothing else, for counsel for the families to suggest cynicism in response. It is also wrong in fact.

308. Our submissions have therefore dealt with the matters for investigation, and proposed answers. It is the outcome of that which is of such vital importance to the future. To the families and friends of those who died; to the injured and all who have cared and will care for them; to those impacted by the collapse in any way, and to the engineering profession of which Dr Reay and ARCL are proud to be a part, a pledge is made to continue to work to ensure that such an event will not occur again.

Dated this 3rd day of September 2012

HB Rennie QC

WJ Palmer

KM Paterson

Counsel for ARCL

Schedule 1 – Collapse hypotheses summary

<table>
<tr><th>Structural detail and hypothesised role in collapse</th><th>Hyland/Smith</th><th>Holmes</th><th>Priestley</th><th>Mander</th><th>NTHA insight</th></tr>
<tr><td>Failure initiated due to excessive drift in columns on line F or line 1.</td><td>Preferred Scenario (1)</td><td colspan="3">Disagree</td><td>Not clear</td></tr>
<tr><td>Spandrel beams on lines 1 or F critical in collapse</td><td>Agree</td><td colspan="3">Disagree</td><td>Not modelled</td></tr>
<tr><td>Disconnection of slab to north core initiator to collapse</td><td>Considered unlikely (scenarios 3 and 4)</td><td colspan="3">Agree</td><td>Drag bar failure likely on level 4(?) in Sept 4</td></tr>
<tr><td>Beam column joint failure critical in collapse</td><td>Largely not considered (columns considered critical)</td><td colspan="3">Agree</td><td>Not clear as simplified modelling used</td></tr>
<tr><td>Interior columns more critical than exterior due to high vertical loads and ground motion</td><td>Disagree</td><td colspan="3">Agree</td><td>Not clear due to simplified beam-column joint modelling</td></tr>
<tr><td>Euler-type buckling of columns over two or more storeys</td><td>Not considered</td><td>Noted indirectly on page 14 sentence 1</td><td>Considered possible, but unlikely (in oral testimony)</td><td>Considered as important</td><td>Not modelled (as small displacement theory used)</td></tr>
<tr><td>Beam-pullout</td><td>Not considered</td><td colspan="2">Considered possible, but unlikely (in oral testimony)</td><td>Considered as important</td><td>Not modelled explicitly, but post-processing suggests likely to some extent</td></tr>
</table>

Schedule 2 – Probable collapse initiation mechanism

1. Based on the NTHA Panel findings that emerged at the hearing, it is likely that the drifts resulted in cracks and affected the reinforcing connecting the floor slabs to the southern shear wall.

   (a) Based on the NTHA Panel results, the 4 September 2010 earthquake caused the CTV Building to be subjected to a north inter-storey drift of approximately 1.5% and a south inter-storey drift of approximately 1%.[289]

   (b) Diagram D1 shows in cross-section the likely cracking between the southern shear wall and the floor slabs due to the drifts.[290] (Diagram D1 is drawn as a cross-section through the southern shear wall at a point other than where the fire door is. If drawn the fire door would be located to the right of the crack. Photographs of the fire door in the evidence off Messrs Coatsworth and Pagan discussed below show how the door has been rebated from the shear wall.[291])

   (c) The effect of the 1% south drift is to induce a crack in the top of the floor slab of approximately 1.5 – 2mm, and the north drift of 1.5% to induce a 3mm withdrawal of the hi-bond deck from the shearwall. (For a 200mm thick slab, a 1% drift results in a 2mm crack at the slab surface.) Examples of the cracks likely to have occurred if the NTHA Panel predictions are correct, were observed by Messrs Coatsworth and Pagan in their inspection after the 4 September earthquake:

      (i) Reference is made to the cracking damage at the 3rd floor fire escape sill (possibly the level 4) (WIT.COATSWORTH.0001E.15).[292] There is another view of the same crack damage from a different angle (WIT.PAGAN.0001.43 - photo 32).[293] From the photographs the crack shown parallel to the door sill can reasonably be said to be at least 2mm wide.

      (ii) These photographs shows a crack which is located on approximately the inside face line of the southern shear wall,

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[289] Second (incomplete) draft NLTHA report by Compusoft. Figures 35 & 36, BUI.MAD249.0552.67.
[290] Diagrams D1 to D10 **attached** are the drawings and photographs referred to in the remainder of this Schedule as such diagrams.
[291] See in particular WIT.PAGAN.0001.43, photo 032 which shows the door about 6-10cm inside the plastered shear wall with the crack shown running from the shear wall edge as drawn in D1. Although due to the angle it is not shown so clearly, WIT.COATSWORTH.0001E.15 shows the same door rebate and crack running from the southern shear wall edge.
[292] **Attached**.
[293] **Attached**.

which is consistent with crack induced by the inter storey drift referred to above. These cracks are most likely to have existed on at least levels 3 to 6.

(iii) There is another smaller but consistent crack with the same location and orientation shown on another floor (level 4 or 5?) at WIT.PAGAN.0001.45 (photo 40).[294]

2. With the drifts expected from the NTHA Panel work, and the cracking observed on the inside face of the southern shear wall after the 4 September earthquake, it is likely that demand created by both the 4 September and 22 February earthquakes exceeded the capacity of the reinforcing connecting the southern shear wall to the floor slabs to withstand the combined forces of those two earthquakes. It is noted:

(a) The drift induced cracking of the magnitude noted above is relevant to seismic capacity reduction of the floor slabs. Professor Priestley noted in his evidence that crack widths of only 2mm are required to induce mesh fracture.[295] Under cross-examination he said further, with reference to the experience of the Clarendon Tower, that 1.5 to 2mm cracks would be sufficient to cause fracture in the floor slab.[296]

(b) Across Christchurch a phenomenon has been observed whereby unexpected seismic resisting capacity reduction has also occurred in the reinforcing bars due to cumulative plastic deformation being limited to the immediate vicinity of a single crack. This phenomenon is present in a new building directly across Cashel Street from the CTV Building in a building formerly occupied by the IRD which has resulted in that building, built in 2007 to 100% of NBS, losing a significant part of its seismic resisting capacity.[297]

(c) To confirm this phenomenon, a Non Destructive Testing Report, prepared by Holmes Solutions, has been provided for another relevant building labelled as Building B in the vicinity of Moorhouse Avenue and Lincoln Road, and has been provided to the Commission.[298] In relation to that report:

[294] **Attached**.
[295] Professor Priestley's statement of evidence at paragraph 80, WIT.PRIESTLEY.0001.24.
[296] TRANS.20120712.19.
[297] Holmes Solutions reports as provided to Counsel Assisting in response to an email request dated 15 July 2012 (at 11.49am).
[298] Holmes Solutions reports as provided to Counsel Assisting by letter dated 13 August 2012. Also see TRANS.20120716.35.

(i) Building B has floor slabs connected directly to shear walls, similar to the floor to the southern shear wall connection of the CTV building.

(ii) Seismic resisting capacity of the reinforcing has been tested for Building B and following the Canterbury earthquakes was determined to be approximately 50% of the undamaged (design) seismic resisting capacity of Building B.[299] (The level of shaking at the site of Building B in the 22 February earthquake was approximately half of the shaking levels in the CBD.)[300]

(d) This observed phenomenon, whereby unexpected seismic resisting capacity reduction has occurred in the reinforcing bars due to cumulative plastic deformation being limited to the immediate vicinity of a single crack, would be one "critical structural weakness" that is reasonably certain to have contributed to the collapse initiation mechanism discussed in this Schedule.

(e) It is feasible that the seismic actions experienced at the CTV site could have been greater than those at the recording sites, or at least in the upper bound of the range. In particular, the vertical uplift described by some eyewitnesses was not fully evident in the seismic records from the recording sites.

(f) The combined effects of:

(i) the 4 September 2010 earthquake forces, which created the floor cracks observed by Messrs Coatsworth and Pagan after 4 September;

(ii) the likely diminished Seismic resisting capacity of the floor to wall reinforcing steel; and

(iii) the high vertical accelerations on 22 February 2011 which, based on a floor vibration period of 0.25 seconds, results in an additional ± 1 G loading,[301]

are likely to have initiated failure of these wall to floor connections.

[299] See Tables 3 (page 16), 4 (p17) and 5 (p19) of the Holmes Solutions report on Building B (report 108222) dated May 2012
[300] Per records from the recording station at 20 Moorhouse Avenue. Source GNS Strong Motion database (**attached**).
[301] Dr Brendon Bradley's Figure 8, WIT.BRADLEY.0003.44.

(g) These effects will have been exacerbated by the horizontal seismic displacement and the secondary connection effects described by Dr Arthur O'Leary (discussed in paragraph 5(b) below).

(h) The Heywood photos (WIT.HEYWOOD.0002.13 and WIT.HEYWOOD.0001.41)[302] show that levels 2 and 3 must have collapsed with a hinging action on line 2, prior to the collapse of level 4 and the general developing collapse. This arises as discussed below.

(i) The partial collapse of the floor slab at level 3, as shown in diagrams D2 and D3 induces actions on the column at lines 2/D, and possibly 2/C, as the floor slab drops down against the column pushing it northward as shown in diagram D2 (stage 2b). The collapse forces at this point would be sufficient to fail the column in bending, coupled with a buckling failure, resulting in the initiation of a vertical collapse mechanism of the columns, floors and other elements above and adjoining it.

(j) The disengagement of the south wall from the floor system at level 3 would have shed some of the gravity support of the floors in that region; the weight normally supported at the south wall region then had to be transferred to the columns connected to the disconnected floor slab. The columns along line 2 and C or D would have had to bear that extra weight at the same time as they were subjected to lateral load from the collapsing level 3 floor.

3. John Trowsdale refers to the separation of the floor slabs from the southern beams in his conclusion in 39 of his brief. (WIT.TROWSDALE.0001.7):

> Based on what I saw at the Building site the concrete floor slabs appeared to have separated quite cleanly and completely from the beams at the southern end of the building.

This observation is consistent with the collapse initiation mechanism discussed above and below.

4. In support of such disconnection, the way the beams and floor slabs adjacent to the southern shear wall came to rest suggests a failure mechanism affecting levels 2 and 3 beams and floor slabs from a hinging effect on column line 2. In this respect it is observed:

[302] Copied in diagram D8.

(a) Mr Heywood's photographs (WIT.HEYWOOD.0002.13) and (WIT.HEYWOOD.0001.41) show the relationship between the levels 2, 3 and 4 floor slabs and the south wall. The Heywood photographs are repeated as diagram D8 with labels attached by ARCL. The 400mm measurement shown on the shear wall is the approximate width of the southern shear wall. The 300mm, 375mm and 400mm measurements to the right of the southern shear wall are the approximate measurements from the northern face of the southern shear wall to the face of the floor slabs or steel beam supporting the floor slab.

(b) Levels 2 and 3 have separated from the wall and are lying some 400mm approximately between the southern shear wall and the edge of the separated floor (noting Mr Heywood's marking of each floor in his photograph). Level 4 is hard against the southern wall and at an upturned angle against the wall.

(c) The difference between levels 2 and 3 and level 4 indicates that level 3 separated and hinged along line 2, which is illustrated in diagram D2.

(d) The disconnection of the level 3 floor slab from the southern wall would have resulted in a progressive disconnection of the floor slab from the beams which connect into the southern wall on line 1, or the disconnection of those beams from the southern wall with the level 3 slab attached.

(e) Mr Frost's photograph (WIT.FROST.0001.52)[303] shows the end of the level 3 beam substantially inside the line of the southern shear wall, which is consistent with the separation between the level 3 floor slab and beam and the southern shear wall at the western end of the southern shear wall. Levels 2 and 3 have been marked by ARCL on diagram D7.

(f) The probable collapse mechanism for level 2 is shown in diagram D4.

(g) The possible extended collapse mechanism of level 3 is shown in diagram D5.

303 Copied in diagram D7.

(h) In three-dimensional form the probable collapse mechanism of stage 1 (also referenced in diagrams D2 and D3) is shown in diagram D6.

(i) Diagrams D9 and D10 show the cleanly broken ends of the levels 2 and 3 fractured floor slabs through the fire door opening (at level 1).

5. Dr Arthur O'Leary's evidence is consistent with the reasoning outlined above. In particular:

(a) Dr O'Leary refered in cross-examination to the disconnection of the southern wall and floor as a likely collapse scenario initiation.[304]

(b) Dr O'Leary went on to say: [305]

> Well I don't believe it was necessarily triggered by east-west earthquake, I think it could've been triggered by north-south earthquake and this is, I haven't done some calculations to this but if you look at level 2 and its attachment to the south shear wall you've got to develop quite a high load in the north-south direction to pull that wall over so it remains connected to the building in its weak direction, just as slab's standing up, you've got to develop quite a high load to pull that wall over so it conforms to the building deformation. Now if you get a very high pulse to the north I have a suspicion that the reinforcing into that wall could not develop that load, and I think, my suspicion also is that the, only starters would've been effective in trying to develop that load. And the other thing that makes me suspicious of this is that if you have a look at a photograph by Mr Heywood. He shows a photograph of three slabs stacked one on top of the other, close to the south shear wall, and they're about that far from the south shear wall….If you do your sums looking at the slab length on a diagonal, it looks to me as though those slabs rotated about grid line 2, one floor. I haven't quite worked out why some of them only rotated one floor not two or three, but that's another issue. I think what – and there's a further thing that makes me worried about the south wall is that there seems to be very little evidence that it actually resisted any significant loads from a pulse to the east. It only appears to have resisted a significant pulse to the west. So there's something fishy going on with that south wall, and I think, I think that south wall had one or two of those slabs at level 2 or 3 torn away from it, from the first quite high pulse to the north.

(c) Some or the uncertainties referred to by Dr O'Leary in his consideration of this issue are addressed in the collapse initiation scenario that has been explained in detail above.

6. In summary of the above discussion:

[304] Dr O'Leary, TRANS.20120813.19, lines 4 to 11.
[305] TRANS.20120813.68, line 30 to TRANS.20120813.69, line 29

(a) There was failure of the steel reinforcing connecting the southern shear wall to the floor slabs. It is highly likely that damage to this element was caused on 4 September 2010.

(b) Evidence of such damage was available upon inspection post 4 September (cracking as noticed by Messrs Coatsworth and Pagan) but its significance was not recognised.

(c) Damage to the reinforcing in this part of the building was neither recognised nor repaired prior to 22 February 2011.

(d) If such failure was not complete after 4 September, it became so during the 22 February earthquake.

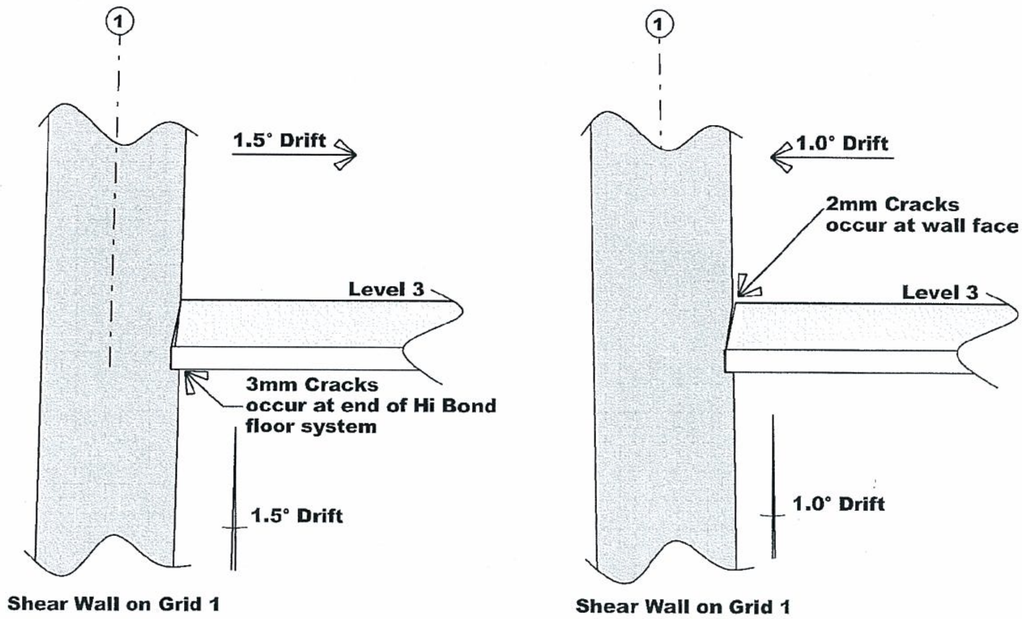
(e) Such reinforcing failure in the connections of the floor slabs to the southern shear wall caused the slabs at that point to disconnect from the southern shear wall and collapse.

(f) Because of the manner in which the southern beams and floor slabs for levels 2, 3 and 4 were laid down during the collapse, as shown in Mr Heywood's photographs (diagram D8), it appears that there was a hinging effect on the beams at line 2 commencing at level 3 (as shown in diagram D2).

(g) Once the collapse of the floors in that area occurred it set in motion the collapse of the columns on line 2 (by the bending/bucking failure of the column/s). This occurred at the same time as the columns were subjected to additional high axial loads resulting from the very high vertical accelerations.

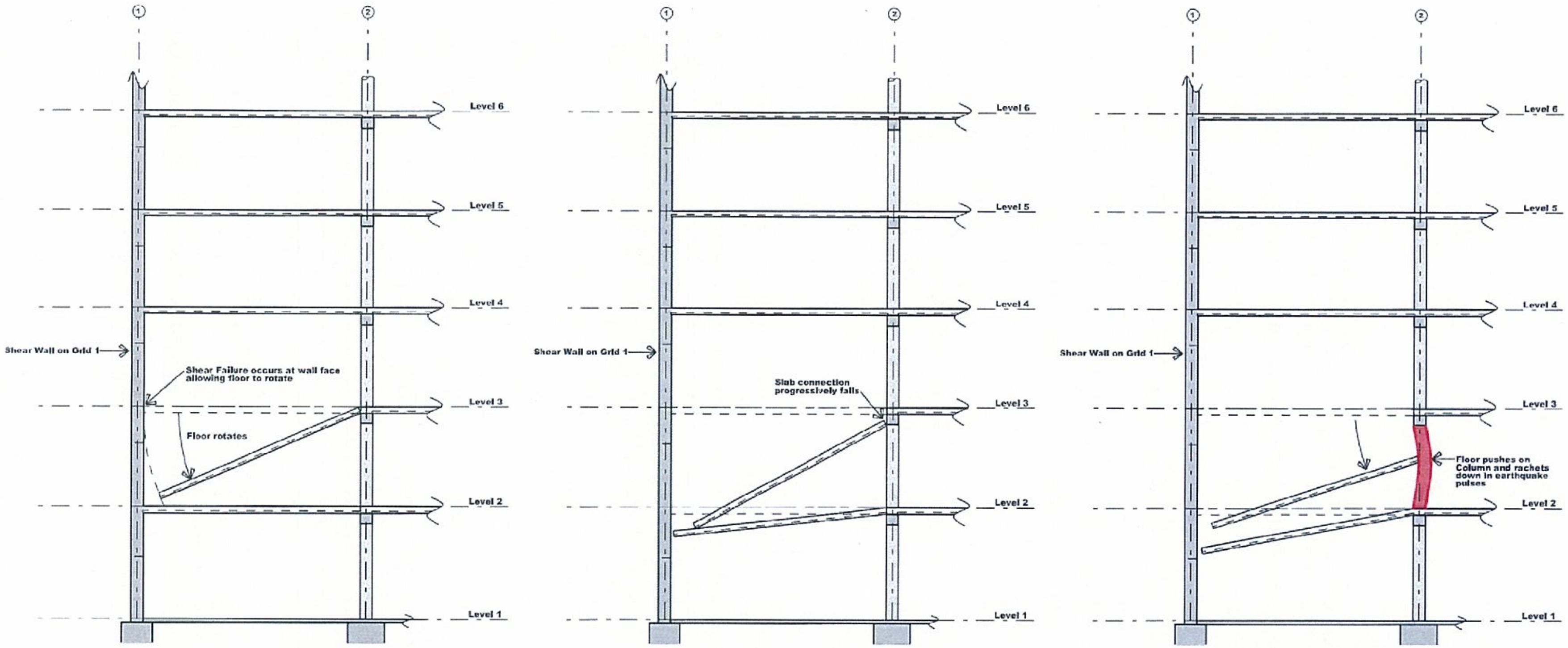
7. The chain reaction that followed brought the building down.

DIAGRAMS. D 1



Level 3 - Wall / Floor Connection

September 4th Earthquake



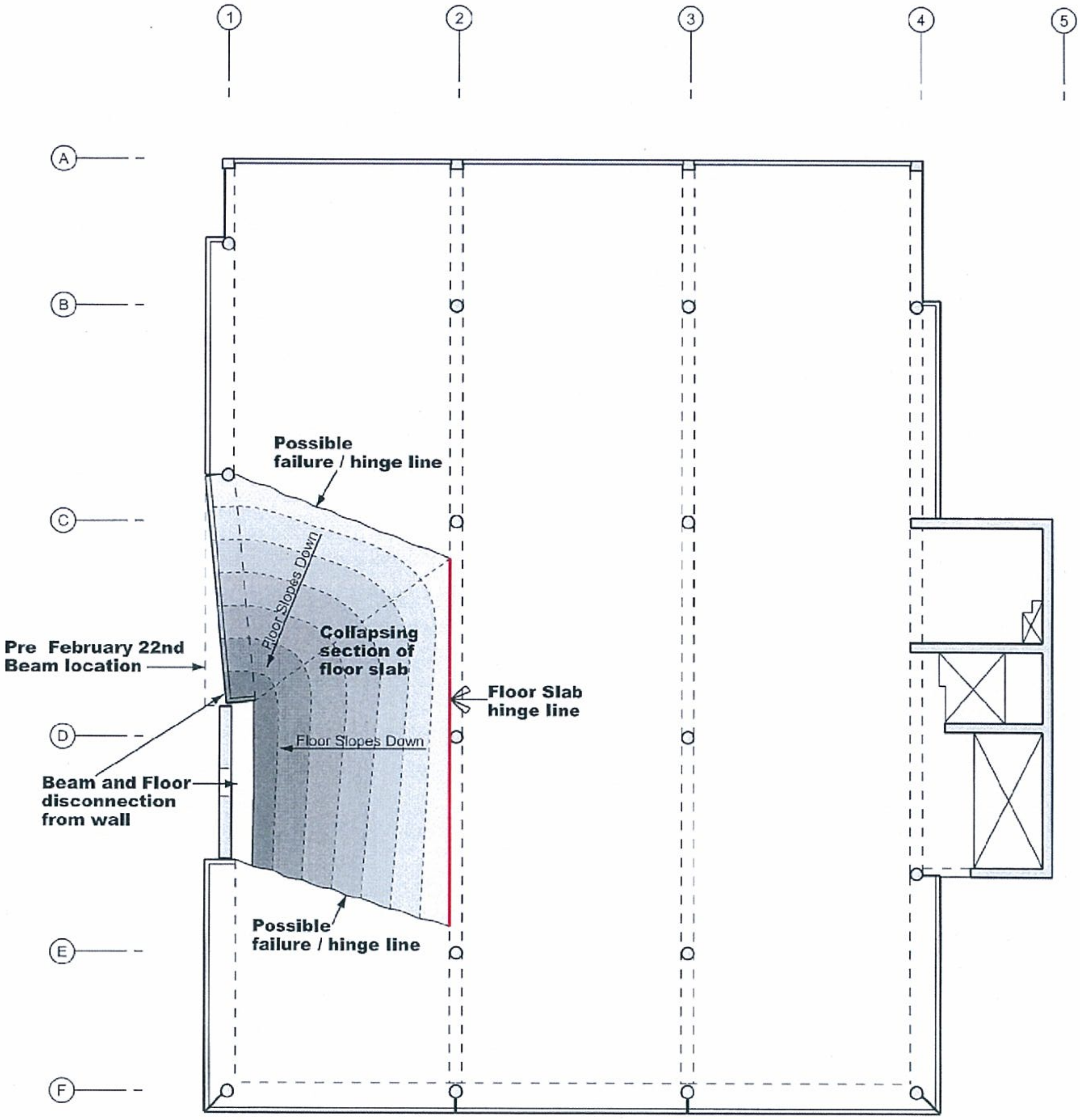
Probable Collapse Mechanism - Stage 1
February 22nd Earthquake

Probable Collapse Mechanism - Stage 2a
February 22nd Earthquake

Probable Collapse Mechanism - Stage 2b
February 22nd Earthquake

PROBABLE COLLAPSE MECHANISM
CTV Building

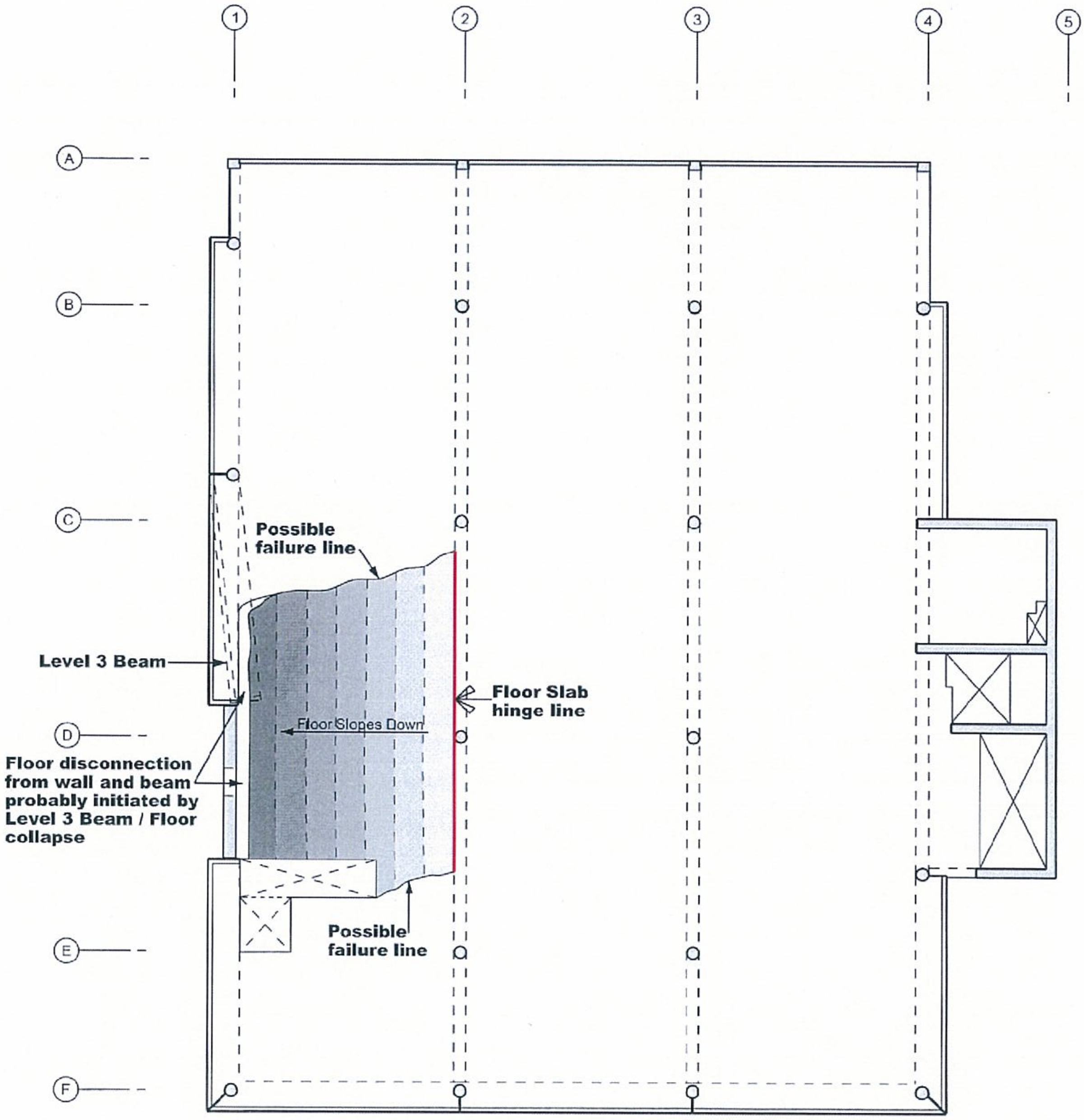
D3



Probable Collapse Mechanism - Stage 1

CTV BUILDING - LEVEL 3

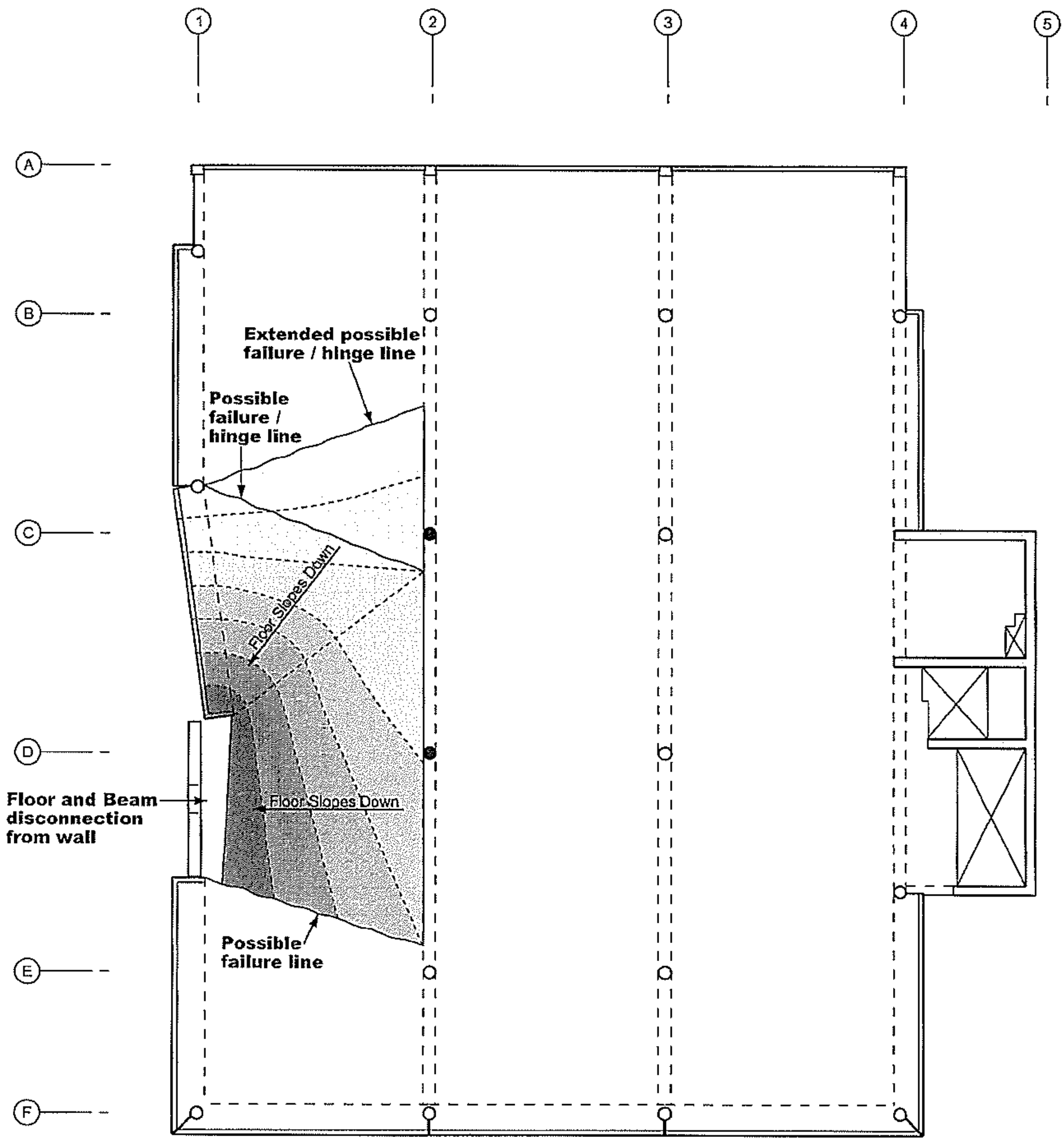
D4



Probable Collapse Mechanism - Stage 2

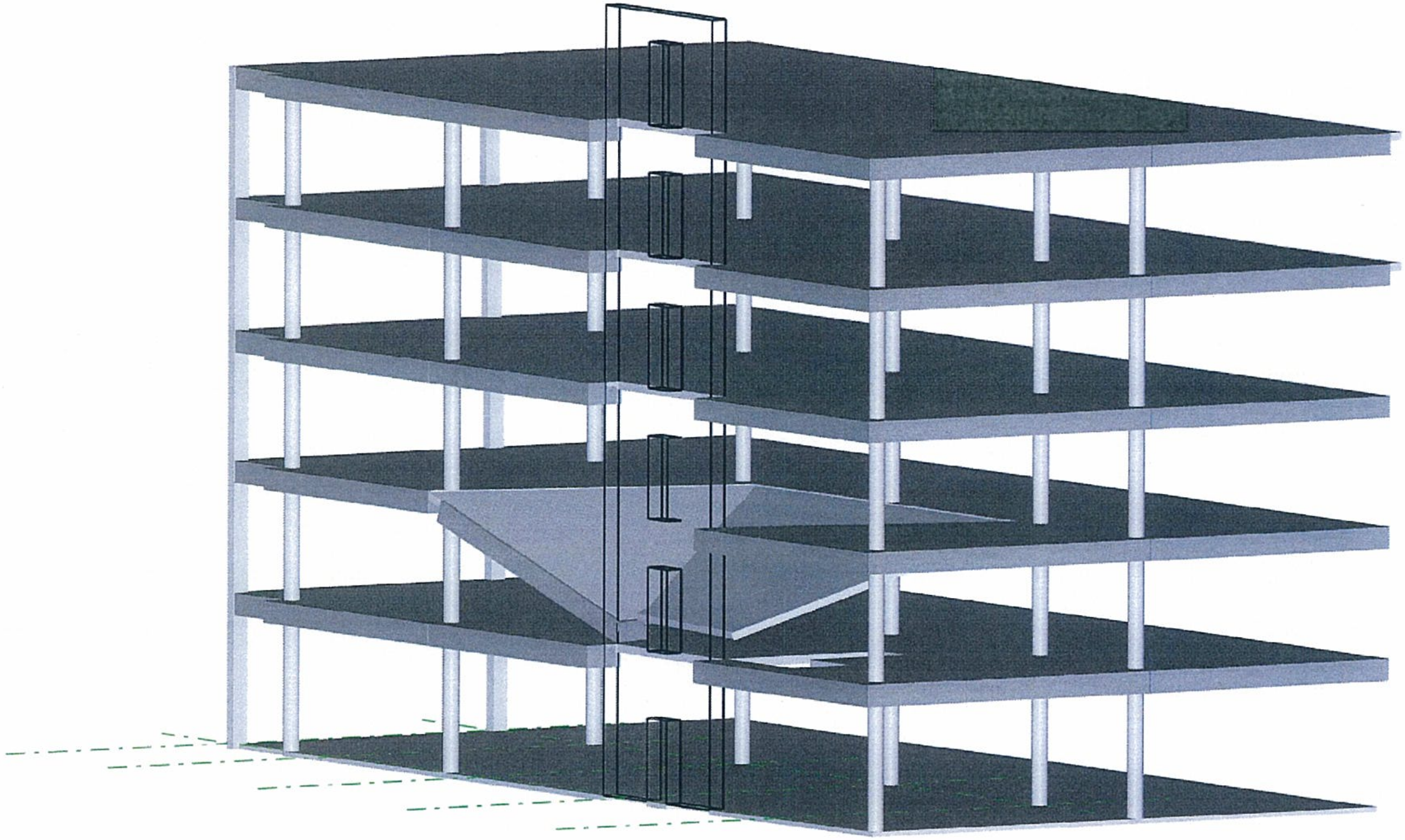
CTV BUILDING - LEVEL 2

D5



Probable Collapse Mechanism - Stage 3
Possible extended collapse of Level 3 floor

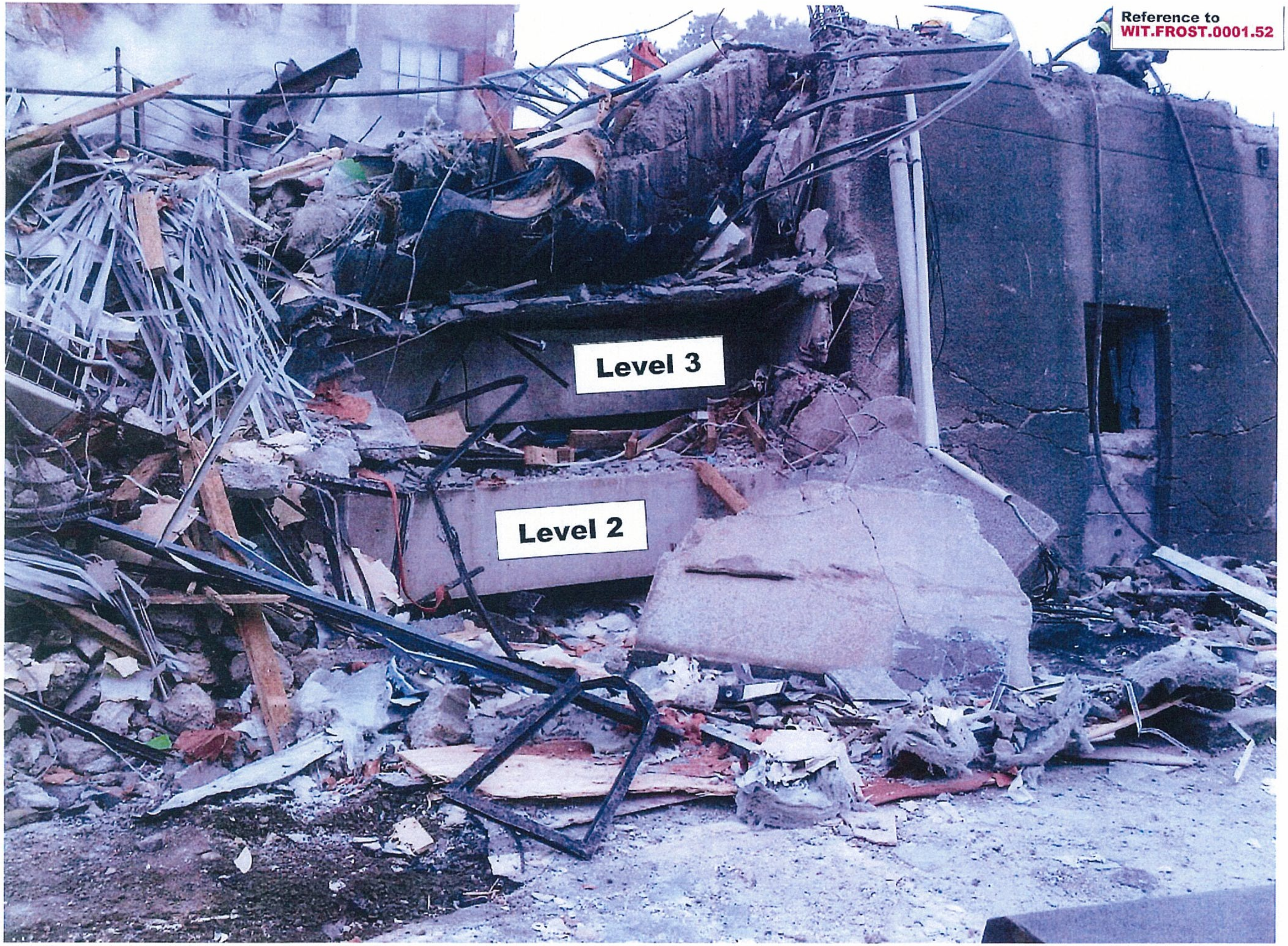
CTV BUILDING - LEVEL 3



D6

Probable Collapse Mechanism - Stage 1

CTV BUILDING



D7

D8

Figure 12

Southern edges of floor slabs from Levels 2 (L2), 3 (L3), 4 (L4) and 5 (L5) immediately to the east of the South Wall on Line 1 (7:20 AM 25 Feb 2011).

D9

D10

1/2

Site: 20 Moorhouse Avenue

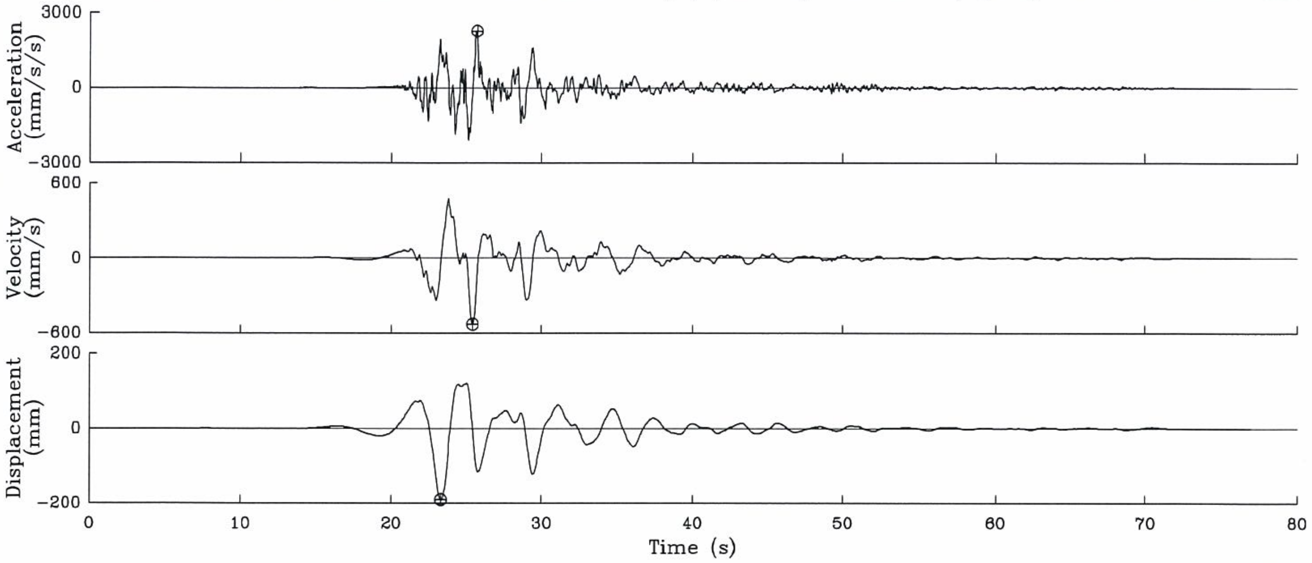
20110221_235142_D06C

Event: 10 km south-east of Christchurch 2011 February 21, 23:51:42 UT

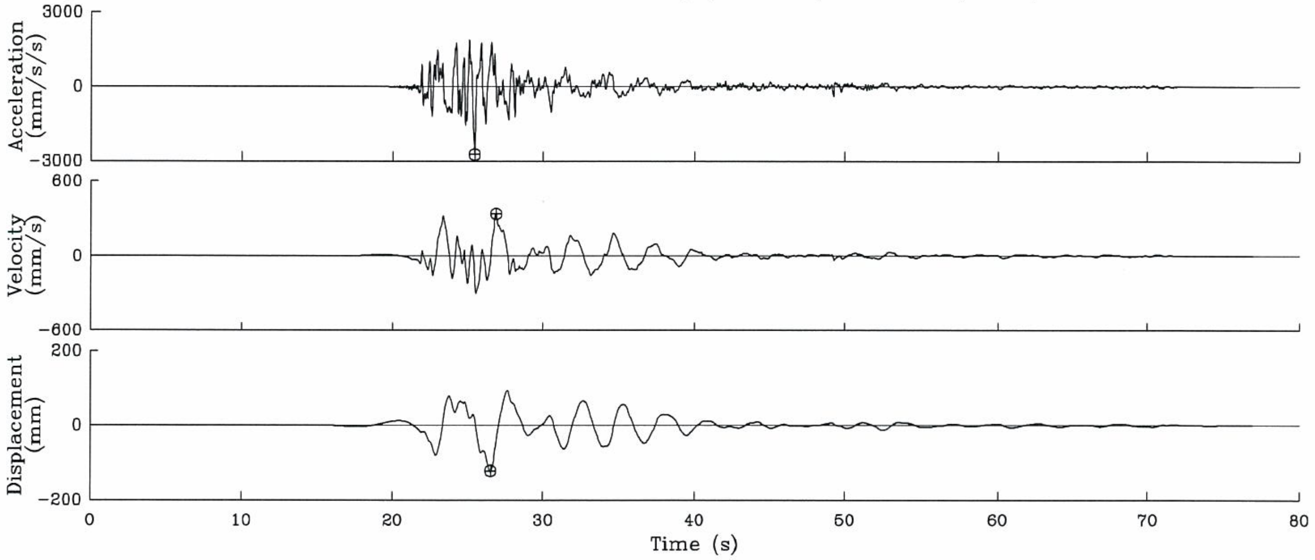
Dist 7km Depth 6 km Ml 6.34

Band-pass filter transition bands are 0.10- 0.25 Hz and 24.50-25.50 Hz

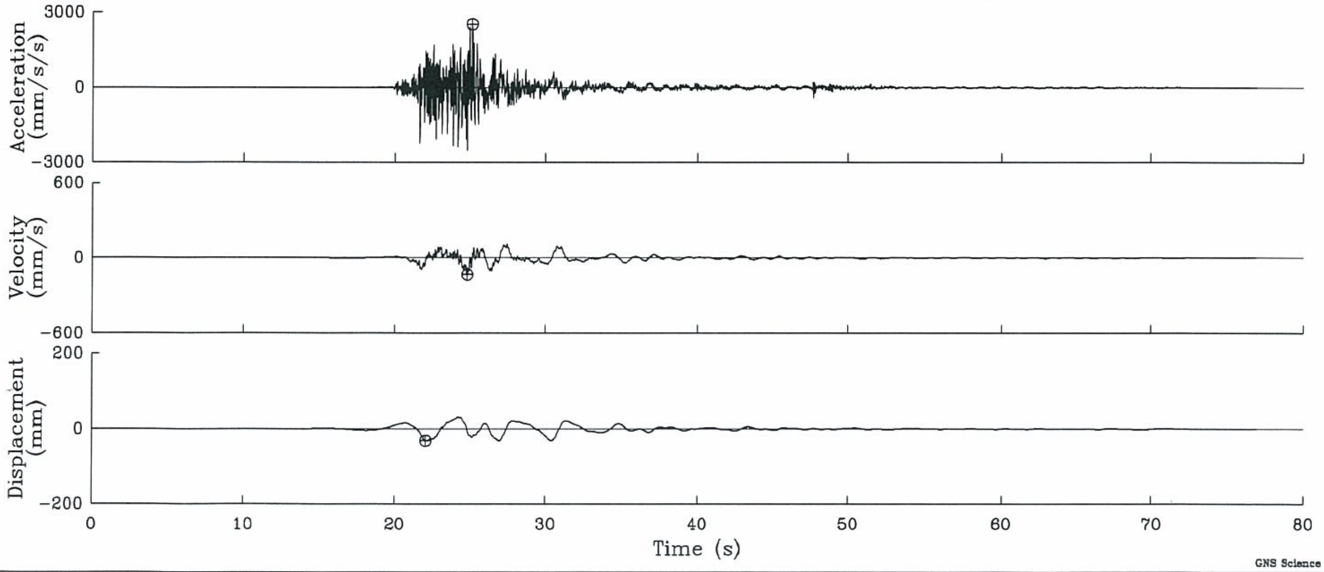
Component W ⊕ Peak values: acceleration 2243.8 mm/s/s, velocity -533.42 mm/s, displacement -192.200 mm



Component S ⊕ Peak values: acceleration -2758.4 mm/s/s, velocity 334.44 mm/s, displacement -123.400 mm



Component UP ⊕ Peak values: acceleration 2516.4 mm/s/s, velocity -136.48 mm/s, displacement -32.800 mm



GNS Science

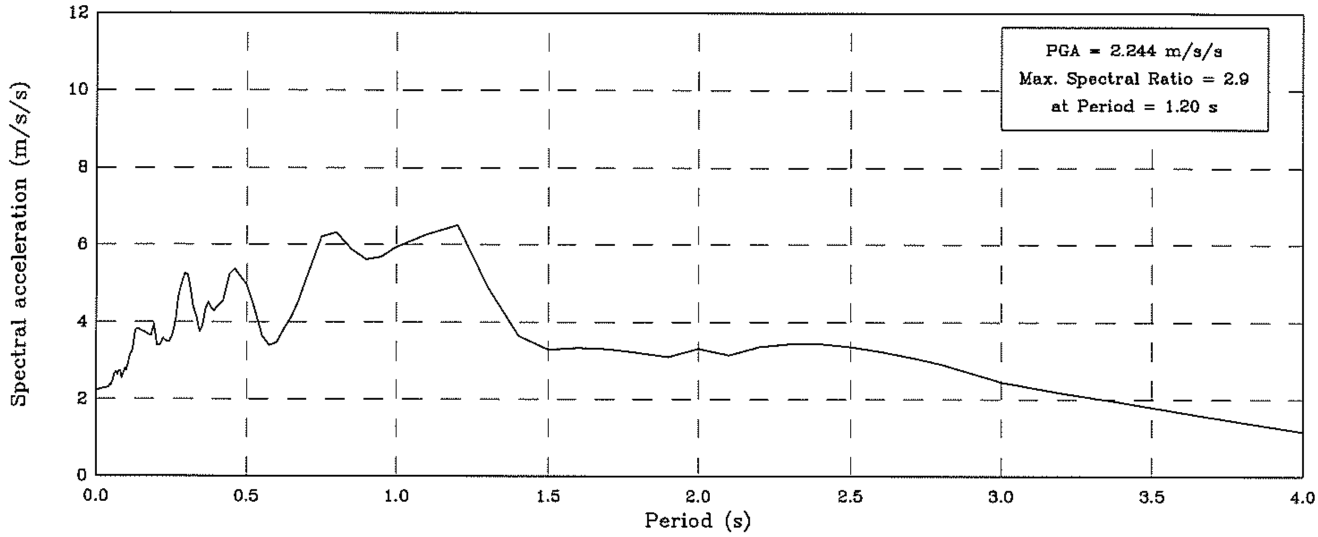
2/2

Site: 20 Moorhouse Avenue
20110221_235142_D06C

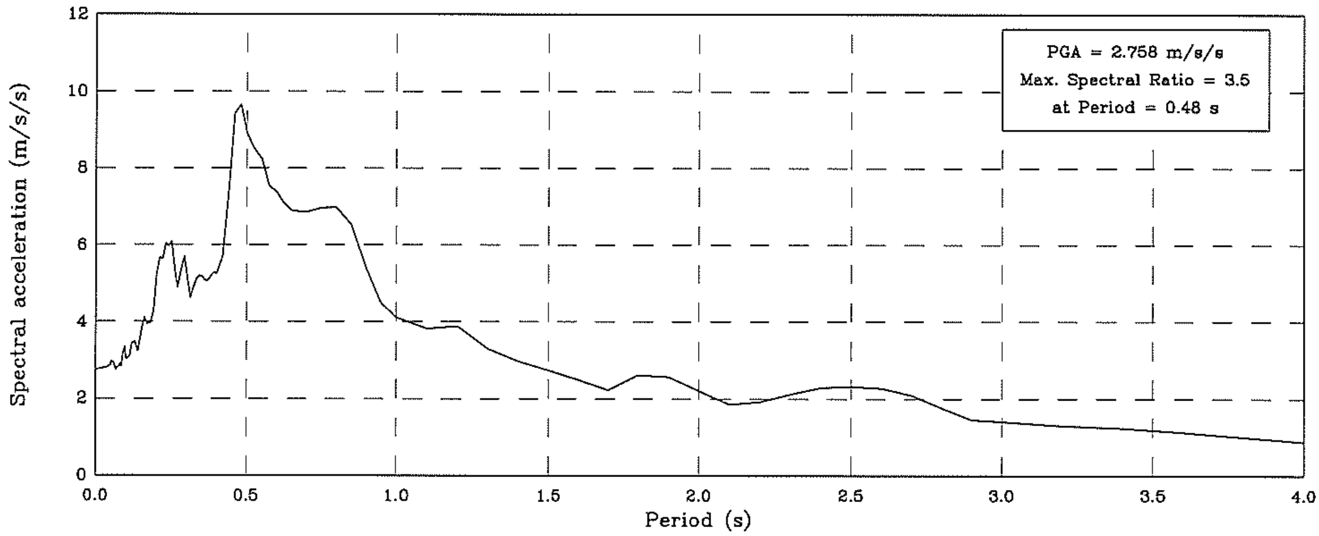
Event: 10 km south-east of Christchurch 2011 February 21, 23:51:42 UT
Dist 7km Depth 6 km Ml 6.34

Band-pass filter transition bands are 0.10- 0.25 Hz and 24.50-25.50 Hz

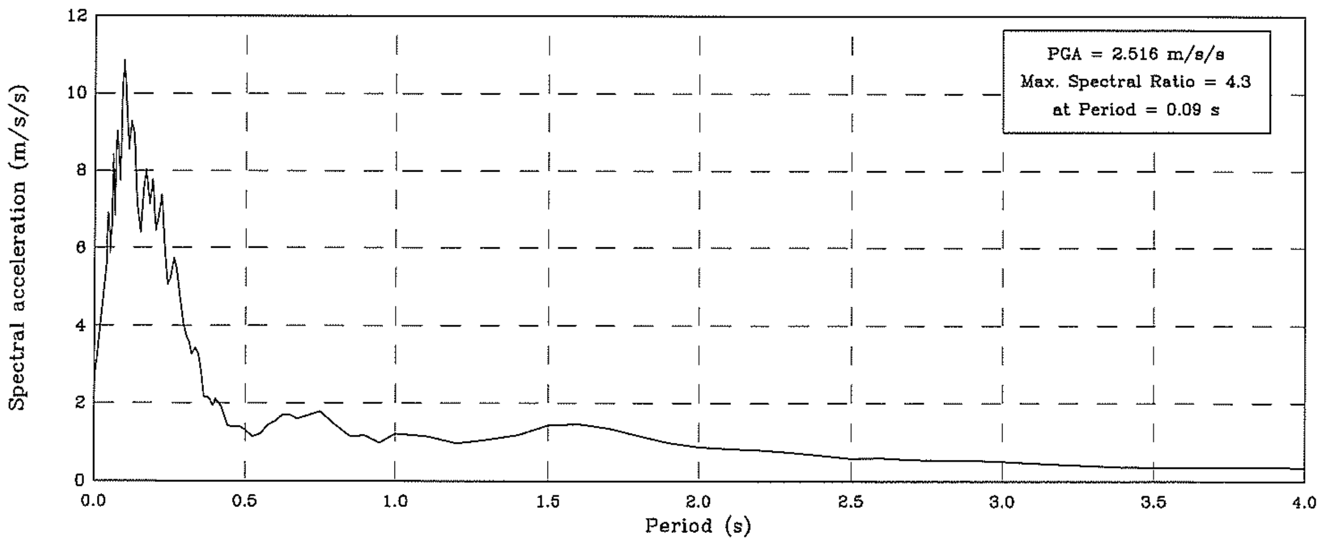
Component W Acceleration response spectrum. Damping value: 5 percent of critical



Component S Acceleration response spectrum. Damping value: 5 percent of critical



Component UP Acceleration response spectrum. Damping value: 5 percent of critical



GNS Science

075 3rd Fl fire escape sill

1/3

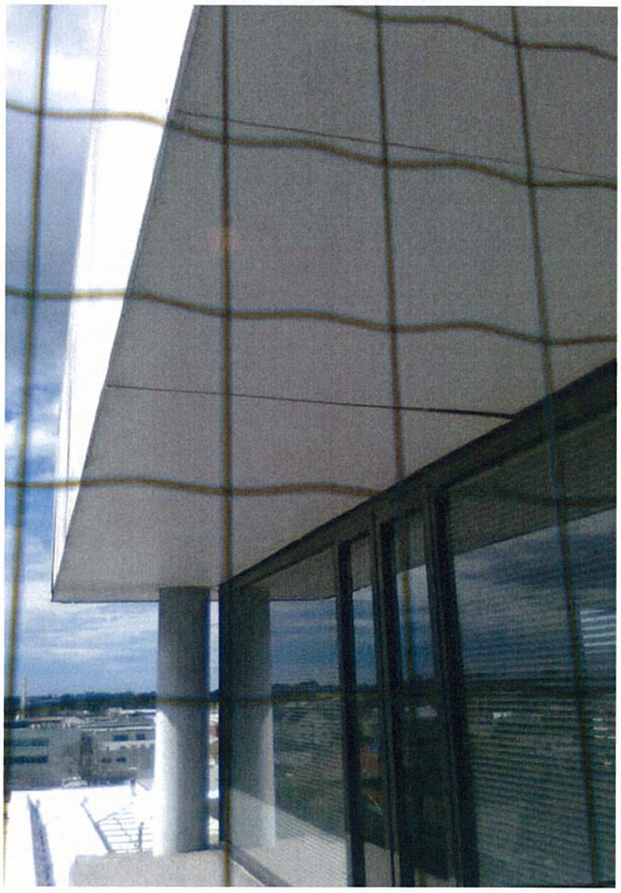
029

030

031

032

037



038

039

040

Schedule 3 – Post-earthquake assessment recommendation

1. As a result of different ground motion intensity levels (relative to seismic design loading standards), and different building resilience/performance levels, it is beneficial to have a hierarchical system for post-earthquake inspection with different degrees of complexity. The seismic intensity of the ground motion that the building is subjected to can be most simply quantified in terms of the ratio of the observed elastic-pseudo spectral acceleration at the building's fundamental period to that of the current (NZS1170.5:2004) New Zealand Loadings Standard. Such information is often available immediately after an earthquake event from nearby strong motion station recordings. This is shown on the left hand column of the table below.

2. The seismic resilience/performance of buildings in terms of discrete performance groups should be information which is held by the local territorial authority. The current performance expectations could be referenced to those implied by the current seismic design standards.

3. Note that the measure of seismic performance is not the same as the strength (i.e. it is not %NBS). This is shown on the top row of the table below.

4. On the basis of the relationship between the ground motion intensity and building resilience it is proposed that the following actions may be required (where numerical percentage values are given for illustration only):

- **If the ground motion intensity is less than 10% of the seismic design intensity then no action is required for any structures. No action is also required if the ground motion intensity is less than a certain fraction of the building's resilience.**

- **A visual inspection with building plans would be required when the ground motion intensity is approximately half of the building's resilience level.**

- **A visual inspection with plans, and direct examination of critical structural weaknesses required when the ground motion intensity is just below the inferred building's resilience level.**

- **A visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing is required when the ground motion intensity is approximately equal to the inferred resilience level.**

- **For particularly vulnerable buildings which are likely unsafe to enter (if not collapsed) as a result of the ground motion intensity significantly exceeding their resilience–external visual inspection shoring and other make safe actions required before internal inspection is permitted.**

5. For all structures which require any internal inspection, an external only inspection can first be used to triage buildings. Such external inspections would ideally use a different placarding system which is binary in nature, either (i) the building is safe to enter by an engineer for an internal visual inspection, or (ii) not safe to enter (so shoring or make safe actions would be required prior to internal inspections).

Post-inspection hierarchy of actions for internal inspection of structures

		Building resilience relative to current importance level 2 criteria				
		<30%	30-60%	60-100%	100% of IL2	Importance level 4
Ground motion Intensity relative to 500 year design Level	10-30%	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses required	Visual inspection with plans required	No action required	No action required
	30-60%	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses required	Visual inspection with plans required	No action required
	60-100%	Building likely unsafe to enter (if not collapsed) – External visual inspection shoring and other make safe actions required before internal inspection	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses required	Visual inspection with plans required
	>100%	Building likely unsafe to enter (if not collapsed) – External visual inspection shoring and other make safe actions required before internal inspection	Building likely unsafe to enter (if not collapsed) – External visual inspection shoring and other make safe actions required before internal inspection	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses including non-destructive testing required	Visual inspection with plans, and direct examination of critical structural weaknesses required

Schedule 4 - Transcript references to the state of the CTV Building after the 4 September and 26 December 2010 earthquakes

Witness	Transcript reference	Extract
Nilgun Kulpe	TRANS.20120625.91, line 26 onwards	Okay, so I noticed a small crack in the foyer by the lifts. ... It ran vertically on a slight diagonal. From memory I would say it was approximately 1.5 metres in length. It's a real estimate, it's not maybe the correct length. It ran above the window and underneath the window as well. I felt really scared being in the CTV building after September 4th. I felt like the building was sick and that it wasn't safe. In aftershocks I would always go to the nearest doorframe. I would do that even if I was seeing clients, which was a bit embarrassing if they were [missing]
	TRANS.20120625.92, line 9 onwards	The CTV building moved a lot as a result of the demolition and it was very difficult to do trauma counselling when the building was moving as much as it was. ... It made the whole of the CTV building shudder and I was sitting right there on that side so each time there was a pull the whole building felt like it was moving with myself in it.
	TRANS.20120625.92, line 29 onwards	In aftershocks the building seemed to sway a lot more and it just felt weaker. In the bigger aftershocks file cabinets would fly across the room and bookcases would fall down.
	TRANS.20120625.93, line 6 onwards	... after the Boxing Day earthquake I noticed that one of the pillars or columns was cracked. This was outside the lifts... I can't be sure if it was a result of the Boxing Day earthquake, but I didn't notice it before then. I noticed it about two or three weeks before the 22nd February earthquake.... I remember that there were cracks in the foyer area and that that got worse over time. I remember wondering whether they were just surface cracks or something more.
	TRANS.20120625.93, line 26 - 27	The building just seemed to be under constant stress.
	TRANS.20120625.94, line 6	We were told not to worry, but I was worried. At staff meetings some of us would ask if we could temporarily relocate to another building because we felt unsafe.
Elizabeth Cammock	TRANS.20120625.102, line 23	Two cracks had appeared on either side of the elevators... I can't be sure when these cracks first appeared because the elevator was not something I used often, but I know they were more pronounced following the Boxing Day earthquake. We talked about the damage by the elevators at staff meetings.
	TRANS.20120625.102, line 30	I saw people downstairs fixing the entrance way after the September earthquake. I'm not sure if this had anything to do with the earthquake but I know that they had to return and redo some of the tiles after the Boxing Day earthquake. This was the only damage I was aware of after 4th of September.
	TRANS.20120625.103, line 14	I remember after the 4th of September earthquake the CTV building would vibrate as a result of the demolition [next door to the West]. I don't remember this before the earthquake. Every time part of the building was brought down, and that's the building next door, the building would vibrate. It felt like the building was hollow. As the demolition moved closer to us the building would shake more and more. It seemed like it was happening right next door.
	TRANS.20120625.104, line 6 onwards	When I returned to work after the Christmas break... There were cracks that had appeared up the stair well in the north core... I only noticed cracking around the 4th and 5th floors. I remember feeling really uncomfortable walking up the stairs and feeling unsafe. The lights were not working and these were never replaced. Plaster and small bits of debris had come off the walls too and no one cleaned this up.
	TRANS.20120625.104,	I remember the cracks by the elevator were more pronounced after the Christmas break. I can't remember how

	line 20 onwards	wide they were but you could see them very easily, even from a distance of approximately ten metres away. I remember they ran from floor to ceiling and were very distinctive. It looked like someone had slightly pulled the wall apart. I wondered how the building could be safe with these cracks in both corners.
	TRANS.20120625.106, line 16 onwards	Q. Now the issue of the cracks that you observed before Boxing Day and after, were they vertical or were they inclined? A. The cracks in the corners by the elevator shafts were floor to ceiling. Q. And they were vertical. A. Yep ... Q. And you said you could see those from a distance of 10 metres. A. Approximately, approximately that distance 'cos they were quite, they were quite deep cracks. They weren't superficial they were deep cracks, yeah, deep cracks. Q. In terms of my third of a pencil, a quarter of a pencil or is that too wide, to get an idea? A. Um, maybe a third to a quarter of a pencil, yes, yeah. I mean you could certainly see that ...
Kendyll Mitchell	TRANS.20120625.110, line 4 onwards	During that visit I felt the building rock as the digger worked, but I wasn’t too worried about it. The digger was only moving across the section, it wasn't actually digging. There was also vibration at one point and Betty said it would have been a truck going past. Hayden was with me at the time and he didn’t like it. After we had left the appointment Hayden said the building should not be rocking like that just because a digger was moving next door.
Phillippa Lee	TRANS.20120625.115, line 26 onwards	There was internal cracking along the west wall on level 5. Some of these cracks were quite large, approximately one metre in length. The cracks started from the bottom and went diagonally, went up diagonally. These were visible from at least three metres away... Faye had put white tape on some of the cracks, but the cracks had grown past the tape by about two inches.
	TRANS.20120625.116, line 19 onwards	I did not notice the floor moved when people walked past. Initially the reception desk was not fixed to the floor so it used to wobble when it was touched or leaned on. Many patients thought it was from an earthquake but it was later secured and did not move after that. The building really shook during an aftershock. A lot of the staff did not like being in the building because of this... The neighbouring building to the west was being demolished and the banging would make, would really make the building shake. However, the demolition felt quite violent and we were surprised at how much the building moved.
Ronald Godkin	TRANS.20120625.121, line 17	Following the September earthquake there were a number of cracks that appeared on the 3rd floor.
	TRANS.20120625.122, line 8 onwards	On the eastern side of the student common room there was a big glass wall, this one here, and that had a very large crack that developed about the middle. So all this wall along here was glass. It had a doorway into the student common room there and a doorway in here and that’s where it is there.... It had gone from the top to bottom.... There was also a big crack which ran from ceiling to the floor in an internal partition wall between the tutors’ room and the audio visual room. Now this is the audio visual room here and the crack was in this part here.... we would go into that room fairly regularly after each earth shake or aftershock and see if there was any further movement of the width of the crack and that crack developed possibly around about the September earthquake but it was plaster only and it was about the width of my finger. In other words, it was about 50 millimetres. Finally, there was a hump in the floor which I've referred to in more detail in my evidence and this was the hump here... I now realise that each of these areas of damage run together effectively in a straight line going from here through here
	TRANS.20120625.124, line 28 onwards	The damage that Brian drew to my attention included a crack under the window on the western wall... and a crack in the tutors’ room... because there was some concern being expressed by the staff that some of the cracks were

		increasing.
	TRANS.20120625.126, line 9 onwards	The hump... ran east to west across the foyer of the building at the point marked 4 on the plan. It first appeared following the September earthquake but as we experienced more earthquakes the hump appeared to get bigger and become more and more noticeable. It was a matter of particular concern to me and to other staff. The effect of this hump was sufficient to cause a pencil to roll across the receptionist's desk, which did not happen before the September earthquake.
	TRANS.20120625.126, line 28 onwards	After Boxing Day there was more damage. This was on the western wall adjacent to where the Les Mills building was being demolished. It occurred about two to three weeks before the 22nd of February earthquake and during the course of the demolition. Both of these damages were at ground level and could be seen from the ground floor car park.
	TRANS.20120625.127, line 5 onwards	The damage noted as 5 involved a concrete non supporting wall at the end of the car park – that one there – and had completely collapsed eastwards, so it collapsed this way. The wall marked 6 was between two supporting pillars here and it had dropped and separated from the floor above by about 20 millimetres or so, but did not collapse. Neither of these walls were load bearing.
	TRANS.20120625.128, line 2 onwards	In about early to mid January, and then again just before the 22nd of February earthquake, there were serious water leaks into the third floor. I spoke to John Drew about this and I was advised that it was the result of work on the fourth floor with the heat pumps.... There was also problems in the male toilets, the male toilet was here and the male toilets were said to have been involved with blocked sewer lines.
	TRANS.20120625.129, line 18 onwards	So we had major ongoing damage with the glass along here. Anything that was not reinforced glass ended up by getting cracks quite regularly and this, this glass here was forever being checked...
Margaret Aydon	TRANS.20120626.1, line 33 onwards	I did notice visible damage to the building, both inside and out, when I joined in early October. The damage was noticeable right from the beginning.
	TRANS.20120626.2, line 30 onwards	There was a crack in the video room, which ran down the wall attached to the pillar. I have marked this as "1" on the plan. I was particularly concerned with this crack because it was very noticeable. The room was painted in a dark paint and you could clearly see the plasterboard underneath. The crack was approximately 1 cm in width. It ran vertically down the pillar and was approximately half the length of the pillar... There was also damage to a glass partition in the canteen area... There were many other areas of damage in the building, but these seemed superficial. For example, there were ceiling tiles that had moved and had been left. There were obvious cracks down the corners in rooms next to the lift, but these things were common in buildings at that time so were less of a concern to me. These areas of damage were included when the other issues of damage were discussed.

	TRANS.20120626.4, line 10 onwards	Staff members were reluctant to come back into the building following the Boxing Day earthquake because shelving had still not been secured to the walls after the September earthquake, as promised... The aim had been to clear as much excess material as possible to clear the tops of the shelving units so things would not fall, let alone dislodge in an aftershock... The crack in the video conference room had increased. It was the full length of the pillar running ceiling to floor. At the top of the pillar, close to the ceiling, it was approximately 5cm in width. You could now put your hand right through the wall. It was also clear to me that whatever was causing the reception desk to slope had increased. I covered reception a lot for the two weeks after Christmas so I had first hand experience of how bad it had really got and it definitely seemed to get worse following Boxing Day. We had to use Blu-Tack or rubber-bands on my pencils and pens to stop them rolling off the desk.
	TRANS.20120626.5, line 9 onwards	On two occasions we had water running down the wall outside of the toilets causing flooding on the floor... The first occasion was probably around the middle of October, but I can't be sure of the exact date. This was repaired but the flooding occurred again and this time the flooding was much worse. I think the second occasion was some time after Boxing Day.
	TRANS.20120626.5, line 28 onwards	When they removed the adjoining wall you could see that there were holes in the wall of the CTV building. It was especially visible when you were in the underground carpark because the carpark was dark and you could see light coming through the holes. I came to the conclusion that when the ties were removed they had left holes in the wall of the CTV building. The CTV building moved a lot after the removal of the wall. I didn't notice any difference in the movement in the building from foot traffic, but there was an increased movement when dump trucks drove past and from aftershocks. There was a big change in the way the building felt. We would bounce constantly when the digger work was being carried out at the demolition site. Some days the movement of the building was so bad I felt seasick. I used to joke with Brian and other staff about it because sometimes we wouldn't know whether it was an aftershock or the diggers. The whole feeling of the building was somehow different. It was very difficult to put into words. It was just a constant sense of bobbing around. Some days were much worse than others. We soon realised that the movement we felt was approximate to the size of the diggers working on the demolition site. The bigger the digger the more the building bounced. When I was sat at my desk or stood in the accounts room the feeling was like being on a trampoline, just gently but constantly bobbing up and down. Yet for the most part an aftershock would send you from side to side.
Maryanne Jackson	TRANS.20120626.10, line 13 onwards	After the September 2010 earthquake I never felt safe in the building. I only liked to be on level 1. I felt uncomfortable when I was on level 2. The CTV internal staircase to level 2 was on my immediate right when sitting at reception. In an aftershock it would shake badly, moving in and out. The windows would move in and out also. I ran out of the building each time there was a big aftershock. I just did not feel safe.
	TRANS.20120626.10, line 26 onwards	The building always shook when trucks went past. However, this was a lot more noticeable after the September and Boxing Day earthquakes in 2010, especially on level 2. You could feel the floor moving when people walked down the corridor on level 2. After the September/Boxing Day 2010 earthquakes there was cracking on the studio walls and in the Sales Office. There were also cracks on the west wall in the Sales Office, about 5mm in size and you could see daylight through them. I also understood there was similar cracking in the Archives Room. There were three big windows broken in the building after Boxing Day 2010.

Tom Hawker	TRANS.20120626.13, line 32 onwards	Before 22nd of February 2011. After the September 2010 earthquake I noticed internal cracking in the master control room situated on level 1. This jagged crack went up on a diagonal and was about one metre long.
	TRANS.20120626.14, line 20	I occasionally worked in the archives in the north-west corner of level 2. After the Boxing Day earthquake there were cracks in the corners of the walls in that area. You could see little holes, about pin size, with daylight coming through them.... Um, there was also daylight coming through the northern wall 'cos there was a window as such in the corner but the light was definitely coming through the western wall.... I remember there were also cracks in the internal plaster walls and gaps in the brick further along the west wall where daylight shone through, but I am unsure of the exact locations of these.
	TRANS.20120626.15, line 31	After Boxing Day I definitely noticed the floor felt less stable. If a truck drove past the building would shake. I could feel it moving slowly, but could distinguish it from an aftershock... if you, a truck drove past it wasn't as sort of violent as an aftershock. An aftershock would sort of shake a lot more than a truck driving past. This was more noticeable on level 2. In some areas you could feel the floor moving when people walked past, especially on level 2. There was much discussion amongst staff as to whether the building was safe. A lot of people did not like being in the building at all and the general consensus was that it did not feel safe. Sam Gibbs had stood in about the same place I was on 22nd of February, which I discuss below, during a large aftershock. Sam told me he thought the building was going to collapse then the way it was rocking back and forth. There was some concern amongst the CTV staff about the fact that we were on the bottom two levels of the building. People were concerned that if the building did come down, there were a lot of floors above us. After one particularly nasty aftershock, Jo Giles got really upset as she thought at that time the building was going to collapse.
	TRANS.20120626.26, line 15	When I was in the building during aftershocks at that time, you know, I felt as if the building was going to come down, just the way the motion of the building was, was happening and it was sort of going, a back and forth motion.
Penelope Spencer	TRANS.20120626.33, line 26 onward	After the September 2010 earthquake the building also had several noticeable cracks. On Level 1 there were cracks in the plasterboard on the south wall of the master control room. One crack went diagonally up the wall and was about 50 centimetres long. After Boxing Day this crack went almost the whole length of the wall, about 2 to 2.5 metres in length, that's an approximation... Q. Firstly, considering before Boxing Day, you've said how long it was but can you say how wide the crack was? A. It was just like a superficial crack, there was no real width to it, it was just a noticeable crack you could say. Q. And what about after Boxing Day how wide was it then? A. I still don't think there was any width to it, maybe a millimetre... It was just a crack that ran along the wall, um, and after Boxing Day earthquake the crack had several other cracks leading off it.
	TRANS.20120626.34, line 22 onwards	There was a significant crack around the internal pillar between the cafeteria and the staff computers on level 2. I would probably say it was actually a gap. I never actually saw a crack there.... Here the carpet along the edge of the internal wall had separated from the pillar by about one centimetre. This had previously been attached before the September 2010 earthquake...
	TRANS.20120626.35, line 22 onwards	There were cracks in the internal walls on level 2. Cracks along walls, um, along the corridor by the editing room went straight up the wall and were 10 to 20 centimetres long. I am sure these cracks also grew bigger after the Boxing Day 2010 earthquake.... I am sure there were also other cracks around windows and door frames but I do not recall where they were. I think the windows may have been broken along the east wall of level 2 as well.
	TRANS.20120626.36, line 2 onwards	I never saw any cracking on the outside of the building. I do not think anyone at CTV felt safe in the building. Sometimes we would joke around and jump into the cafeteria on level 2 which would make the floor shake. The floor also shook every time someone walked down the corridor on level 2. This did not happen prior to the

		September 2011 earthquake.
	TRANS.20120626.36, line 15 onwards	During the demolition of the building next door in January 2011 the building constantly shook. It felt like we were having aftershocks all day but it was just the building shaking.
Peter Brown	TRANS.20120626.130, line 27 onwards	In or around 2008 a fitness centre occupied level 5 of the building. You could feel the building shake whenever people were using the fitness equipment. The movement that we felt was the only concern I'd had in the building before the December the 4th earthquake. After December 4th earthquake cracks had appeared on levels 1 and 2. I have prepared a floor plan of level 1 and level 2.
	TRANS.20120626.131, line 13 onwards	Okay. Photograph 1 shows a crack running alongside the pillar on the western wall of level 2. This crack is marked as 1 on the plan of Level 2 B.
	TRANS.20120626.131, line 30 onwards	I understand from colleagues that I have since spoken to that this crack slowly increased in size with each large aftershock. Photographs 2 and 3 show a horizontal crack that appeared along the ceiling in the northeastern end of the building. It was approximately two metres in length and four millimetres in width. This crack is marked as 2 and 3 on the Plan B. It appeared to me as though the ceiling had been forced down because you could see the plaster board had compressed by about four millimetres. I understood this was a result of the movement of the building during the earthquake forcing the ceiling up and down.
	TRANS.20120626.132, line 22 onwards	Photographs 4-6 show two vertical cracks that appeared in an internal wall in the north-eastern end of the building. I have marked them as 4-6 on the Plan B.
	TRANS.20120626.133, line 4 onwards	Photographs 7-11 show the mess on Level 2 resulting from the earthquake. The area where each photograph was taken has been marked accordingly on the Plan B.
	TRANS.20120626.133, line 16 onwards	Photograph 12 shows two cracks in the north wall near the stair well on Level 2. One ran horizontally up the gib board under the ceiling about 600 millimetres in length. The other ran vertically from the ceiling about a metre in length and joined the horizontal crack at the ceiling. I have marked this crack as 12 on the Plan B.
	TRANS.20120626.133, line 28 onwards	Photograph 13 shows two cracks in the southwest corner of the building on Level 2. No measurements were taken but with every aftershock more and more daylight could be seen coming through the crack nearest to the pillar. I have marked these cracks as 13 on the Plan B.
	TRANS.20120626.134, line 8 onwards	A. Areas of cracked glass developed along the eastern wall. These are marked 14 on the Plan B. Q. On the eastern wall. A. Eastern wall. The cracks got worse with each large aftershock so we put gaffa tape across them to help keep the window together in case it shattered. Each of these windows was replaced as soon as a glazier became available. I took photographs of the replacement of one of the windows. These photographs are attached and marked D. The replaced window is marked as 14a on the Plan B. Q. And that's nearest the south-eastern corner. A. Yep.
	TRANS.20120626.136, line 4 onwards	There were other areas of damage that were not photographed. A crack had appeared between one of the pillars and the wall at the northwest end of level 2. I have marked this as 15 on the plan B. The crack ran from floor to ceiling and daylight could be seen through it. It would have been at least 25 millimetres wide after the September earthquake and seemed to increase slightly with each large aftershock.
	TRANS.20120626.136, line 16 onwards	Q. And how much of that crack could you see daylight through? A. We could see daylight through it all. Q. Through the whole crack? A. Yeah, yeah. It was only about 25 millimetres you could've almost put your hand in it, that's how I kind of judged. I didn't actually measure it as such.

		Q. And is that how it was, you say 25 millimetres wide after the 4 September earthquake? A. Yeah. Q. And you said it seemed to increase slightly? A. Yes others had told me later, that's all, that it increased. I noticed between the time, for a time I left it had increased slightly with the other aftershocks. Q. So that's between 4 September and? A. December.
	TRANS.20120626.137, line 1 onwards	A crack appeared along the south wall of level 1. I have marked its approximate position as 1 on the plan for level 1 A. This crack ran vertically from the ceiling nearly to the floor and was about two metres in length.
	TRANS.20120626.137, line 11 onwards	Q. And can you be any more specific about where it was? A. It was on the, um, in the room they call the master control room. It was on the, um, in the plaster on the what do you call that wall the? Q. Southern wall? A. Yeah southern, yeah the wall that took the strength, you know the? Q. Shear wall? A. The shear wall by the fire escape yeah. Q. So it was a crack in that wall or in the plaster that covered that wall? A. Ah, it was in the plaster but, no, it was also on the outside and I think I mentioned further on, do you want to wait till then or do you want me to say now?
	TRANS.20120626.137, line 28 onwards	Sometime after the 4 September earthquake cracks appeared along the northern wall in the carpark on level 1. These were along the join where the concrete blocks met the ceiling. They were approximately two metres in length. I am not sure what caused the cracks but they may have been a result of aftershocks. I have marked their approximate position as 2 on plan A. The cracks appeared to get worse over time.
	TRANS.20120626.138, line 7 onwards	I did not notice any other damage. An inspection of the building was carried out after the 4 September earthquake. Reference to this is made in an email sent out to all staff on the 9th of September from 10 Murray Wood, the managing director. This is attached and marked E.
	TRANS.20120626.140, line 13 onwards	There was a crack in the plaster again on the outside wall and again the same issue is that you could see in that how far it went without taking the plaster off, it was an unknown quantity.
	TRANS.20120626.141, line 29 onwards	We all, but mainly senior staff, talked about the damage we saw, especially when we were in areas where the damage was clearly visible. The room in the southwest corner of level 2 was one of these areas; the cracks marked as one and 13 were common topics of conversation in B. We were all quite concerned about being able to see outside through the gaps in the wall.
	TRANS.20120626.142, line 2 onwards	I did not notice movement from people walking around the floor. However, I thought that the building moved more easily whenever trucks went past. It is hard to compare this movement with what it was like before September the 4th, because we had a lot more trucks going past as a result of demolitions that were going round about, going round the place. They began to demolish the building next to the CTV building in about October 2010. The building had been used as a car park. It was approximately eight metres in height. I do not know why they were demolishing it. They used excavators, large trucks and jack hammers. I think sometime early in the demolition a ball and chain swinging from a crane was used to knock the walls down. The CTV building was quite sensitive and would shudder and shake a lot as a result of the digging and falling masonry next door. This caused a lot of anxiety among the staff. I was told that the building was designed to move between the lift shaft and the opposite

		part on the south wall. I think I was told this by David Coatsworth, during his inspection of the building after the 4 September earthquake. I repeated this message to CTV staff. We all assumed that this was why the building moved with the demolition next door.
	TRANS.20120626.144, line 2 onwards	Well the cracks. There was a crack on this wall here that I pointed out there but in the stairwell there was cracks showing around the well itself and one of the things, and the crack that I was pointing out there continued through but they seemed to think it was the way the floor was poured that caused that rather than being any actual damage itself.
	TRANS.20120626.144, line 12 onwards	I was able to distinguish between the movement felt in the building by the demolition and from aftershock. Aftershocks tended to have a wave rolling motion whereas the demolition was a shuddering vibrating motion. However, I would not be surprised if some smaller aftershocks were masked by the movement already felt in the building during the demolition. I retired at the end of December, in December 2010. I did not visit the building again.
	TRANS.20120626.150, line 1 onwards	Q. Yes you've got a wealth of information there which is very interesting. I'm particularly interested in this crack you've marked 1 on A, the first floor, and possibly might need the view of the second floor as well. Now that crack, was that plaster on the actual structural wall or was it alongside the structural wall? A. It was actually on that structural wall.
David Bainbridge	TRANS.20120626.118, line 22 onwards	Building Damage. On my second visit to the building I noticed some damage to the building which with my having building experience really concerned me. I discuss my observations below.
	TRANS.20120626.118, line 29 onwards	When walking out of the lift onto level 6 I noticed the internal column in the foyer had cracks in it. This column was along the north wall in the corner where the east side of the north core met the wall. I have marked this column with the number 1 on the plan and if we look at the plan we can see the number 1 that's been put in marking that column.
	TRANS.20120626.119, line 2 onwards	Approximately three-quarters of the column circumference was visible from the floor to the ceiling. The column had at least three large cracks around it of approximately five millimetres in thickness and other hairline cracks of about two to three millimetres in thickness. These ran all the way around the column at about 20 centimetre intervals. There were flakes of concrete three of which were about the size of a 50 cent coin at the base of the column. You could also see blow-out marks on the column. From what I observed it looked like the column had performed to its maximum capacity in the previous earthquakes but would fail in any further events. It definitely did not look like it could withstand another large earthquake. I was more concerned because this column was on the perimeter of the building. I have been shown a photograph of a column by counsel assisting the Royal Commission. This photograph is annexed hereto and marked 0001.6. I can confirm that this column is the column I am referring to in the foyer marked with the number 1 on the plan
	TRANS.20120626.119, line 23 onwards	I observed one other column while I was in the building. This was on the corner of the consultation room on the west side of the building. I have marked the location of this column with the number 2 on the plan. This column had cracks in it also however I was with our counsellor Ann Malcolm when I walked past this column so I never got the opportunity to examine it closely and we can see 2 on the room 3 along from the north side on the western wall.
	TRANS.20120626.119, line 31 onwards	Water Damage. When I looked at the top of the column in the foyer I could see around the bulkhead where the column attached to the ceiling. There was water damage on the plaster board. This was a brownish discolouration on the ceiling panels that went about a metre inwards from the column. This indicated to me that the building may have been damaged exposing it to the elements and causing water to enter the building.
	TRANS.20120626.120,	Exterior Spandrels. As I was waiting for the lift to leave I looked out the window in the foyer on the east side of

	line 5 onwards	level 6. I saw one of the spandrels on the exterior of the building. This panel was between levels 5 and 6. I have marked its location on the plan with the number 3. And if we look on that plan, bottom right of the plan, we can see you've put "3" indicating the spandrel
	TRANS.20120626.120, line 25 onwards	The spandrel was exposed aggregate which was a feature of the building design. I could see the interior and exterior sides of the end of the spandrel that was closest to the north core. I observed that this spandrel was uneven and not in line with the building by about 100 millimetres. It was damaged and chipped at the end. From what I observed I believe it had experienced significant movement prior and had at some stage been compressed against the north core.
	TRANS.20120626.122, line 2 onwards	Damage to the column in level 6 foyer. I am advised by counsel assisting that the photograph of the column in Level 6 foyer referenced 0001.6 in my earlier statement of evidence is a photograph from the Earthquake Damage Report for Madras Equities Limited dated 6 October 2010. The damage I observed on the column appeared worse than that shown in the photograph. There was more cracking and bits of concrete had come off the column. Some of the cracking had grown in size, especially the large cracks referred to at paragraph 8 of my earlier statement of evidence. At paragraph 8 of my earlier statement of evidence I also use the term "blowout marks". By this I mean the concrete and paint on the column directly above or below the cracking is dislodged from the column, exposing the aggregate inside the column. From my experience in columns, and from my observations of this column, I believe these blowout marks resulted from the cracks that went the entire way through the column as the column was moved or compressed in later earthquakes. The movement or compression forces the paint and concrete to dislodge. I had never seen a column with this much damage before and could only put the damage down to the earthquakes.
	TRANS.20120626.123, line 7 onwards	Q. Now in terms of the section of column that can be seen and the damage that's shown on it, how does that fit in with your recollection of the column you saw in February? A. The column I saw in February was – had worse damage on it, more damage sorry. Q. So more cracks or wider cracks? A. More cracks and wider cracks on three main parts of it. Q. And what about the area below that we can't see in that photograph? A. Yes there was one other major crack. There was three major cracks that I saw, it's probably about roughly about 800mm off the floor. Q. And when you said three major cracks, can we see two of them in that? A. Yes you can. Q. So those two that we can see, your recollection is they were wider when you saw it in February? A. That is correct.
	TRANS.20120626.123, line 29 onwards	Q. Can you say anything about the damage that we can see or the cracks in the one on the left first? A. The cracks are still the same just the other ones that were added, and which you probably can't see, were the hairline cracks that were every 200, going on vertically up the column. Q. That we can't see any in either photograph? A. No, or they were pretty fine so that they were there. Q. But in terms of the cracks that we can see and that are marked on that photograph on the left, do they – are they consistent with your recollection of the cracking? A. Yes, yes they are, that's what stood out when I walked through the foyer.

		Q. All right, and is that photograph more consistent with what you saw? A. Yes pretty much, yep.
Leonard Fortune	TRANS.20120626.74, line 27 onwards	A digger was used to flatten out the ground and a wrecking ball was used to break up big hunks of concrete. They would drop the ball from a height of about 6 metres and as it landed you could feel it through the building. It felt like there was an earthquake. The building would jump and creak and made really weird groaning noises. You could see the mortar from the wall we were working on just breaking off as the building shook. I had to go inside the building to run a lead and noticed that it made weird groaning noises inside, even without the demolition going on next door. I am not sure what caused these noises. I had said to my workmates, Bruce and James, that I didn't think people should be working in there. It just didn't feel right.
	TRANS.20120626.75, line 23 onwards	A. And when you heard those noises was demolition going on? A. Um, not at that stage. I'm pretty sure the digger driver himself was actually on a break. Q. And was there any aftershock occurring when you heard the noises? A. No, no. Q. So nothing untoward was occurring but you heard noises. A. Yeah. Q. What about when the demolition was going on, you said that you heard noises, were they the same noises or different? A. Um, slightly different than what I'd heard in the building. Um, they were more, more the sound you would associate with an earthquake that a building would make. Q. You mean the building moving? A. Yeah.
Jo-Ann Vivian	TRANS.20120702.99, line 13 onwards	I understood from speaking to Moira and Pablo Godoy the clinical leader that the filing cabinets had fallen over, shelves had fallen down and there were files all over the floor.
	TRANS.20120702.99, line 27 onwards	I visited the building with my husband Mark Vivian on Sunday 2nd of January 2011 and was shocked at the extent of the mess. Most of the filing cabinets had fallen over. Shelving had emptied onto the floor and some pictures and ornaments had broken.
	TRANS.20120702.100, line 1 onwards	We did a general clean up and picked up several large filing cabinets that had fallen in a southerly direction towards Cashel Street emptying their contents onto the floor in the process. I took a number of photos for insurance purposes. They are attached.
	TRANS.20120702.101, line 3 onwards	Q. And in the second paragraph do you refer to the – you say the pillars on the fifth floor lobby are cracked, new damage and there is no evidence the building has been inspected, so I suggest a call is made to the Council in the morning for that to happen? A. Yes.
	TRANS.20120702.101, line 22 onwards	When I went into the building with my husband on 2nd January I had noticed some very visible cracks in the column outside the lift on the Madras Street wall of the lobby. The column was half inside and half outside of the building on that wall so I was concerned that the cracks might indicate structural damage. I have marked the location of this column as, "1" on the floor plan.
	TRANS.20120702.102, line 18 onwards	The cracks were in a spiral pattern and some of them appeared to go right around the column to the outside. They were approximately one centimetre in width and one to 1.5 metre in length. They were wide enough to concern me, however it didn't look like the column was about to fall down either. I believe that the column was painted dark red, I now know it wasn't and you could clearly see the plaster underneath. I had not remembered seeing this damage on any of my previous visits to the building. It looked like it went beneath the paint layer into the structure

		of the column which made me think it needed checking further.
	TRANS.20120702.102, line 28 onwards	My husband Mark also commented to me about cracks around the lift well doors. I remember the conversation about the cracks but neither Mark nor I recall exactly where they were.
	TRANS.20120702.16, line 104 onwards	So the bottom right, I would say that the cracks I saw were more extensive than that. They were the kind of cracks you walked out of the lift and they just hit you in the face and the staff who saw them and the family and friends who were with me on the day of the clean up, all commented on them. They looked worrying.
	TRANS.20120702.105, line 15 onwards	The right-hand photo, well actually they're both, the top crack on the left, the one going down on an angle, I just remembered that they were on an angle and they were sufficient for me to be worried.
	TRANS.20120702.111, line 3 onwards	"In discussions with staff over the following weeks I was aware of some concern about cracks, the movement of the building in aftershocks and effect of the demolition next door.
	TRANS.20120702.113, line 3 onwards	Q. In s 14 you refer to the cracks as being one centimetre wide and one to one point five metres in length? A. Yes. Q. Was this in the plaster or how far did those cracks extend. Could I have stuck my pencil, blunt end first, into the cracks and how far would it have gone? A. Um, Q. What one centimetre wide would enable me to put my pencil in? A. Yes I would think you could have in some of them. Q. It was just through the plaster was it or was it right into the concrete or are you not...? A. They were solid, I guess solid, I don't know which bit is plaster and which bit is concrete. They seem to me to be going significantly into the body of the pillar. I'm familiar with very superficial cracks and they looked a little more than that.
Graeme Smith	TRANS.20120702.116, line 8 onwards	The lifts were stopped and I was able to get inside the lift shaft in that area. I noticed horizontal and vertical cracking. There was horizontal cracking at each level of the approximate location of the construction joint of each floor. There was also cracking about half way up each floor which appeared to correspond with the landings for the stairs and the adjacent stairwell. Both types of horizontal crackings were present the full height of the western and northern walls of the lift area of the north core but not in the eastern wall. There were two vertical cracks that ran the length of the lift shaft. One was approximately one metre from the western side of the left shaft and the second was approximately 1.5 metres from the western side of the lift shaft. Both were in the range of 0.2 to 0.5 millimetres wide. I recall that this vertical cracking was just in the north core of the north, the northern wall of the north core. None of the cracks that I referred to in the north core had any spalling.
Peter Higgins	TRANS.20120702.122, line 16 onwards	The cracks that I observed in the stairwell walls were generally horizontal and were consistent with construction joints as illustrated in photograph 3 in the CPG report.
	TRANS.20120702.122, line 22 onwards	My recollection is that there was a thin plaster render over the concrete in the stairwells which had cracked with the joint movement and this render would need to be removed along the crack line for setting up and injection of the construction joints. I saw cracks on both sides of the stairwell as well as in the north shear wall.
	TRANS.20120702.123, line 6 onwards	have recorded six horizontal circumferential cracks in this column with concrete spalling in the overhead lintel beam adjoining this column approximately 1200-1500mm out from the face of the column above the window. I have provided the Royal Commission with a scanned photograph which I took of this column and the adjoining beam and I have drawn along the lines of the cracks with a pen.
	TRANS.20120702.125, line 4 onwards	So the damage that you've circled in your photo is separate damage that you saw on the 14th of February? A. In addition to the, to that photo, ah, cracks that you're referring to, yes. Q. Can I take you back to your brief please, paragraph 13?

		A. I have also recorded that the level 2 beam on the north elevation over the entry off Madras Street had five near vertical or diagonal cracks in it above the glass entry area of approximately one metre (in length). This is shown in photograph 5 of the CPG report.
	TRANS.20120702.125, line 20 onwards	My notes also record that the south shear wall had one fine, near vertical or diagonal crack in the wall adjacent to the fire escape landing of approximately two metres. This is the same area shown in photograph 2 of the CPG report.
Stephen Kissell	TRANS.20120702.130, line 20 onwards	I noticed a crack in the foyer on level 6. The location of this crack is marked on the plan attached and marked A. ... It was underneath the window in the eastern wall by the lift. It ran from under the windowsill diagonally towards the corner of the lift. Its width was approximately three millimetres but it's hard to recall exactly how wide it would have been. I've seen a lot of damage in buildings but thought that this crack was serious because of its width and because it was on a diagonal. It appeared as though there wasn't a lot of strength in the wall which concerned me. I've drawn a sketch of the crack that I saw, this is attached and marked B. The location of the crack and its dimensions is approximate only.
	TRANS.20120702.131, line 25 onwards	With this being said I confirm that the crack I saw on level 6 ran under this window in a downward diagonal direction towards the lift. Unfortunately the placement of the crack is out of shot in the photographs of level 6. However I can confirm that the crack I saw underneath the window was similar to, if not slightly worse, than the crack that can be seen horizontally across the pillar in the photograph of level 6.
	TRANS.20120702.132, line 2 onwards	Q. Can you just tell us anymore as to why you were concerned about this crack you saw in the wall? A. As I was standing there in that area I noticed quite a few cracks, not just in the column but also around the lifts, the doors of the lift. Q. In the plaster? A. Yes. Q. Right, but this crack in particular that you've spoken of – A. It's the way it was running of, knowing that the lift shaft is the strong part of the building and that that was an external wall, that was all.
Phillip Reynish	TRANS.20120702.135, line 9 onwards	While I was on level 6 I noticed large gaps around the perimeters of the windows along the eastern side and part of the south side of the building. I have marked on a plan the areas I am referring to with crosses. This is attached and marked **"**A."
	TRANS.20120702.135, line 17 onwards	The join between the steel window frame and the concrete window opening is generally filled with silicone but in some places the steel window frame had pulled completely away from it and you could feel a draft. I have drawn an example of what I saw. This is titled figure 1 in the attachment marked "B".
	TRANS.20120702.135, line 22 onwards	Q. Just so explain that and take us through that please? A. Well the internal bit is what I drew is the window sash, the window frame itself, and the external bit is the – would be the opening in the concrete, external concrete wall and the gaps down the side is sort of an indication of what I saw on most of the windows where there'd be, the gap around the window would not be uniform, it'd look like it was pulled away on one side and it consequently had stretched all the silicone which seals the window from the elements, stretching (inaudible 16:51:03) those gaps. Q. And on that one you've indicated 5 to 10 millimetre gap at the bottom and 20 millimetre at the top? A. Yeah that'd be approximately for most of them, yeah. Q. So was it similar in most of them that it was a bigger gap at the top? A. Yeah, tended to be, tended to look like that which made me think that the opening was not square, I'd imagine

		that the window would have stayed square otherwise the glass would have broken in the window itself so I assumed it to mean that the concrete was not square, the opening wasn't square.
	TRANS.20120702.136, line 19 onwards	In my opinion the concrete building had moved but the window had stayed square, the metal, the metal in the glass of the window frame was square and stayed still and the building had moved and that's what it looked like to me.
	TRANS.20120702.136, line 29 onwards	The gap on the right-hand side of the window frame and it appeared to be larger at the top than the bottom estimated the gap at the top of the window to be around about 20 millimetres and the gap at the bottom to be 5 or 10 millimetres. This gap concerned me because I took it to mean that the building was no longer square and it was leaning away from the stairwell and the lift tower. I also noticed that the building would vibrate quite a lot when a truck would go past. This didn't concern me because I hadn't been in the building before and I didn't have anything to compare the movement to, however it did seem livelier than I would have expected.
Leonard Pagan	TRANS.20120703.2, line 24 onwards	I did see that both the north and south shear walls had hairline cracks in them. These were diagonal.
	TRANS.20120703.5, line 7 onwards	Q. Right, and if we can go to the next page please, top left, is that a photo of the top of that same column and showing the lintel that the columns, above the column? A. Correct. Q. And some damage or cracking to that area? A. Yeah.
	TRANS.20120703.6, line 21 onwards	There was a partition wall on level 3 or 4, I cannot recall exactly which floor that had quite a significant crack in the plaster board. Paragraph 15. The western side of the building was the worst in terms of damage to the plaster board.
	TRANS.20120703.8, line 3 onwards	Q. Yes somewhere you I think you mentioned that there was more damage on the western wall in terms of the plaster, towards the western wall. Can you confirm that and tell me at what levels was that at, was that at level 3 was it? A. I can't confirm exactly which level. I believe it was the upper level so perhaps from three upwards yes we did notice more cracking along the, the plaster on the inside of the western wall, the plaster board on the inside of the western wall.
David Coatsworth	TRANS.20120704.9, line 26 onwards	Um, they had, um, commented that the floors, um, well there was deflections in the floors and the vibrations in the floors.
	TRANS.20120704.10, line 7 onwards	Q. Can you tell the Commission about that? A. Um, she just simply said that the building moved quite a lot in an earthquake. Q. Did she talk about – A. Or an aftershock.
	TRANS.20120704.14, line 2 onwards	At most levels there were some diagonal shear cracks in the walls around the bathrooms and stairwell, for the most part measuring less than 0.2 millimetres in width, but with three measuring up to 0.3 millimetres. For example, in the toilets in the north shear tower on the fifth floor I saw a single fine diagonal crack on each of the east and west walls. None of the cracks I observed in these areas was large enough to indicate failure or yielding of the wall.
	TRANS.20120704.14, line 31 onwards	I observed minor cracking along part of the length of the construction joints in the walls and stairwells at several floor levels. However these cracks measured generally less than 0.2 millimetres in width but with a few up to 0.35 millimetres in width. While this constituted minor structural damage, once again, it was not of an order that would signify yielding of the shear wall. I observed minor cracking in the stairwell walls at most levels. For example, I saw

		horizontal cracking in three of the walls of the stairwell between the fourth and fifth floors. There was also a minor diagonal crack in the northern wall approximately 500 millimetres below the roof level. Although the cracking that I observed was indicative of minor structural damage, because the cracks were very fine I consider that the reinforcing steel had not yielded, that the aggregate in the concrete was still interlocking and that the general integrity of the concrete walls was not compromised.
	TRANS.20120704.16, line 13 onwards	The north-east corner column immediately above the third floor spandrel exhibited some very fine, minor cracking. Similar hairline flexural cracking was evident in the north-east column above the fourth floor spandrel panel. As a result of observing this damage, I further inspected the column in the north east corner at other floors but observed no damage. At the top storey, the first column west of the north-east corner of the building exhibited some cracking, the appearance of which was accentuated because the paint had chipped off at the cracks. I did not record the width of these cracks, but my recollection is that they were less than 0.2mm. I took photographs of the column and the cracks. The first column in from the south west corner on the south side of the building at the top storey also exhibited some fine cracking. I recall these also as being less than 0.2mm and, again, I took photographs of the column and cracks.
	TRANS.20120704.16, line 26 onwards	In a number of places I observed gaps of approximately 7-8mm at ceiling level between the plasterboard wall and structural columns. At the floor level however there was no gap. The movement of the plasterboard was, in my view, caused by building sway. At floor level the plasterboard was fixed to the floor slab, which was fixed to the column. When the column leaned during the earthquake it pushed on the plasterboard wall, causing the gaps that I saw at ceiling level. Similar effects but to a lesser extent were evident where partition walls adjoined the shear walls. This was not evidence of structural damage.
	TRANS.20120704., line onwards	The first floor beam on the north face of the building in the span between the north east column of the building and the adjacent column had two fine diagonal cracks. Because these were so fine I did not consider yielding had taken place.
	TRANS.20120704.18, line 3 onwards	At the ground floor I saw significant cracks in the plasterboard lining. I was also able to identify a fine diagonal crack on the outside of the south shear wall at the ground storey. This crack was quite fine so I did not consider it to indicate structural yielding.
	TRANS.20120704.19, line 2 onwards	I noticed the high points over the beams and the sags in between but I would have expected to have seen more significant deflections if the floor had yielded.
	TRANS.20120704., line onwards	The non-load bearing at ground storey in the stairwell exhibited some non-structural damage. Differential movements between the block wall and the structure had peeled off the gypsum plaster lining on the block wall. There was damage to internal framing and linings on all floors which varied from minor cracking in joints between plasterboard sheets to diagonal cracks in the sheets. There was one broken window on the east wall at the third floor most likely due to the earthquake, and the rubber seal had come loose on another east wall window at the ground floor. I observed no other damage to the windows. None of the damage to the spandrel panels, the concrete block panels, the internal framing and lining or the windows was of structural significance.
	TRANS.20120704.27, line 3 onwards	During this telephone call I discussed with Mr Bull my observations of the cracking present within the CTV building and in particular the diagonal shear cracks in the order of 0.05 millimetres to 0.35 millimetres as well as the cracking in the horizontal construction joints above and below floor slabs. Mr Bull advised that cracks of less than 0.4 millimetres still retain aggregate interlock within the concrete and observed that code designs allow for some cracking. He was not surprised that there was cracking at the construction joints. He said he thought that cracks of the type I had described should be fine but agreed that for peace of mind cracks larger than 0.2 millimetres should be injected with an epoxy resin. A copy of my note of this telephone conversation is annexed at attachment 7.

	TRANS.20120704.31, line 3 onwards	I emailed John Drew that afternoon confirming my findings and my view that the CTV building remained structurally sound. By this I meant that the capacity of the building to resist gravity and lateral loads had not been significantly reduced. I emphasised, however, that it was inevitable that where cracks had been opened by the initial earthquake, subsequent shocks would work the joints and open them further. I accordingly recommended that arrangements to repair the walls by epoxy injection be made as soon as practical. A copy of this email is annexed at attachment 12.
	TRANS.20120704.31, line 21 onwards	However I did state in my report that there were no obvious structural failures. In my email to John Drew, dated the 19th of October 2010, I said that the building was still structurally sound. I did not in either my report or my email recommend that it be vacated. I saw no reason to do so. I considered that with the limited damage observed, the capacity of the building to resist gravity and lateral loads had not been significantly reduced. In my opinion the building performed well in the September earthquake, sustaining only minor structural damage. As an engineer, however, I do not use the term 'safe' because it is too broad and imprecise. It is simply not possible to say a building will be safe under all circumstances. While I understand that a concern has been raised during the Royal Commission's hearings that a layperson might misconstrue a finding that a building had not been damaged as meaning that the building was safe in this broad sense, it was not my intention to imply this.
	TRANS.20120704.45, line 19 onwards	I recall the lady on the top floor making the comment that the building moved quite a bit and I recall not the exact words but the essence of what the Kings Education people were talking about with regards to floor deflection.
	TRANS.20120704.58, line 19 onwards	You also refer to unrelated to this photograph, that there was cracking in the ground floor southern shear wall and you recommended inspection of that. What did you anticipate was the cause of the cracking? A. In the shear walls? Q. Mmm? A. Earthquake loads on the structure. Q. You recommended further inspection but you never undertook that further inspection did you? A. No. Q. Are you aware of whether anyone else did? A. No.
	TRANS.20120704.68, line 11 onwards	A. Basically or thereabouts. Q. So when you went to inspect it you expected to see damage? A. Yes. Q. And in fact that's what you found; you observed considerable damage to the linings and finishings? A. Yes. Q. And as we've just looked at before you noted minor structural damage, albeit that you didn't see any obvious structural failures? A. Yes.
	TRANS.20120704.98, line 2 onwards	Q. So looking at the damage that's circled on that lintel, if that's the right word, does that, did that concern you looking at that. I appreciate you're only looking at a photo? A. Um, yes it's more substantial. Q. And why does it concern you compared to the other crack, apart from being more substantial? A. Um, I guess it's away from the influence, the direct influence of the column and any bending effects that there might have been just at the top of the column, you know, the original crack that I saw that's close to the column, um, um, I interpreted to be an effect of that, of rotations or displacements that had gone on there and that had not only caused that crack but also the cracks in the column, but the one that you're looking at here now, this one

		under the circle, is something different. Q. What, you're unsure of what the mechanism might be? A. Well... Q. Is that what you're saying? A. We know now from the photos of the building after collapse that that corner of the wall actually pulled out of the wall. Q. Right, but if you'd seen that in an inspection – A. The circled damage? Q. Yes. What would you have concluded do you think? A. Um, I would have been looking on the outside of the wall above the window to see if it was apparent there and whether it was more extensive than just on the soffit of that piece of wall. Q. Okay, but not so in relation to the crack you saw? A. No, I believe that the crack that I saw was relatively minor.
	WITNESS STATEMENT REFERENCE	
Marie-Claire Brehaut	[3]	A number of areas of damage had appeared on the 3rd floor. Attached is a floor plan of the Building I have prepared marked **"A"** (the Plan).
	[4]	There was a crack on the wall that separated the teachers' room from the AV room. I have marked this as **"1"** on the Plan. It spanned the entire length from the floor to the ceiling and was about one millimetre wide. I had always been able to hear muffled voices between the rooms, because the walls were so thin, but when the crack appeared I could actually hear what was being said in the next room.
	[5]	There was a hairline crack running down the internal wall next to the elevators. I have marked this as **"2"** on the Plan. I only have a vague memory of this. The walls were a deep blue and I remember there was a whiteness to it, that may have been the plaster coming through, but I can't be sure.
	[13]	… The most memorable aftershock was one that made Sandra Hii and I both immediately get under our desks. I could hear the students screaming throughout the Building. The Building moved a lot and seemed to sway in an east to west direction. There was a bit of up and down movement as well. When we looked at GeoNet I remember being really surprised at its size because it felt a lot bigger than what was recorded.
	[15]	The CTV Building moved a lot during the demolition of the building next door. The movement felt like a shudder usually accompanied by quite short bursts of movement. The movement coincided with dull thudding sounds. It made my colleagues Sandra Hii, Beth Pettigrew and Margaret Aydon feel unwell.
	[18]	After Christmas I covered Reception for Sandra Hii who was away on holiday. I noticed that if I wasn't holding onto my pen it would roll down the desk from west to east. I have marked the position of the Reception desk as **"4"** on the Plan. It would start off slowly and then speed up. It hadn't done this when I had covered Reception in November. I commented to the few staff members who were working over Christmas that the floor must not be even. I also showed the pen rolling to my colleague, Ron Godkin, when he paused in reception to comment to me that he thought the floor was on a lean.

Malcolm Harris	[5]	A number of areas of damage had appeared on Level 2 after the earthquake on 4 September 2010. Attached is a floor plan of the Building I have prepared (marked **"A"**).
	[6]	I sat along the western wall of Level 2. I have marked this position as **"1"** on the plan. I could see approximately 15-20 cracks in the wall from my desk. I have marked these as **"2"** on the plan. They were very noticeable and ran basically from floor to ceiling. You could see daylight through some.
	[7]	I remember a number of cracks had appeared in the office of Joanne (Jo) Giles, a CTV host. Her office was along the southern wall. I have marked this area of cracks as **"3"** on the plan. These were also numerous and ran from floor to ceiling. Jo would mark the cracks with a felt pen and add to the number as new ones appeared.
	[8]	It was noticeably noisier when trucks or buses went by and the floor moved when people walked down the corridor. Once the demolition of the building next door started it shook constantly with the movement of the diggers and the use of the demolition ball was as severe as a large quake.
	[9]	There was a noticeable change in movement in the Building whenever a bus or truck went by as well as the demolition. The building shook noticeably after the 4 September earthquake and severely after the Boxing Day earthquake.
	[12]	They would use a wrecking ball and every time it hit the building the CTV building shook as well. In some ways the shakes from the demolition were worse than the aftershocks, especially once the demolition got to the foundation level. Even the trucks and the grader moving around the demolition site would make the Building shake. At times the crew in the studio would have to stop filming because of the shaking.
	[13]	An aftershock would last a few seconds whereas the demolition was continuous. If it wasn't the wrecking ball it was the digger moving or the articulated trucks coming onto the site to remove the material. In all cases our building shuddered and cracks opened up.
	[14]	The cracks along the western wall (marked **"1"**) appeared to get bigger and wider once the demolition of the building next door started. New cracks appeared in the toilets, above where Rob Cope-Williams, Mandy Uriao and I sat, along the western wall. Nothing was done about the cracks as far as I'm aware.
	[15]	The cracks in Jo Giles' room got bigger as a result of the demolition too. She would note the date and increase the line as the cracks became longer.
	[16]	All of the sales staff had to clean plaster and dust off their desks daily from the movement of the walls. We expressed concern to Murray Wood, but I'm not sure what was done. I tried to convince the sales staff that it was safe and the movement was only natural with the demolition going on. I didn't know this for sure though.