UNDER	**THE COMMISSIONS OF INQUIRY ACT 1908**
IN THE MATTER OF	**ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURES CAUSED BY CANTERBURY EARTHQUAKES** **KOMIHANA A TE KARAUNA HEI TIROTIRO I NGA WHARE I HORO I NGA RUWHENUYA O WAITAHA**

3rd BRIEF OF EVIDENCE OF CLARK WILLIAM KEITH HYLAND
IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

BRIEF OF EVIDENCE OF CLARK WILLIAM KEITH HYLAND IN RELATION TO THE CTV BUILDING

1. My full name is Clark William Keith Hyland. I live in Manukau. I am Director of Hyland Fatigue + Earthquake Engineering a specialist consulting engineering company.

2. I prepared the report on the CTV Building Collapse Investigation (BUI.MAD249.0189) ("the BCR") for the Department of Building and Housing jointly with Ashley Smith of StructureSmith Ltd.

3. I have read and agree to comply with the Code of Conduct for Expert Witnesses.

Evidence

4. Since the release of the BCR expert witnesses have prepared peer reviews or critiques of the BCR in Statements of Evidence for the Royal Commission.

5. This brief of evidence replies to other expert evidence.

6. The expert evidence was by John Manders (**WITR.MANDER.0001**).

Academic Qualifications

7. PhD in Civil Engineering, University of Auckland, 2009

   BE(Civil), University of Auckland, 1985

   BCom (Management Studies), University of Auckland, 1986

   NZIW Certificate of Welding Engineering, NZ Institute of Welding, 1999

   DipCL (Cross-cultural Communication), New Covenant International Bible College, 1996

Professional Practice Qualifications

8. Registered Engineer (New Zealand), 1989

   Registered Structural Engineer (Papua New Guinea), 1992

   Chartered Professional Engineer, 2004

Professional Service

9. IPENZ CPEng Practice Area Assessor

   NZ Society for Earthquake Engineering: Management Committee Member

   Convenor of Southwest Pacific Earthquake Resilience Workshop, Wellington 2011

   NZ Structural Engineering Society: past-committee Management Committee member

   Auckland Structural Group: past-committee member

New Zealand Standards Committees

10. NZS 3404: 1997 Amendment 2 2007 Steel Structures Standard: Committee Member

    NZS 3404.1:2009 Part 1 – Materials, fabrication and construction: Committee Chair

Joint Australian /New Zealand Standards

11. BD23: Structural Steel Products: Committee member

    AS/NZS 1163:2009 Cold-formed structural steel hollow sections

    AS/NZS 3678:2011 Structural Steel Plate

AS/NZS 3679.1:2010 Structural Steel Part 1: Hot Rolled Sections

AS/NZS 3679.2:2010 Structural Steel Part 2: Welded I Sections

Design Guides Authored

12. SteelDoc: Steelwork Documentation Guidelines

SteelDeck: Design for Point loads on Composite Metal Decks

Structural Steelwork Estimating Guide

Structural Steelwork Connections Guide

Design Guide for Penetrations in Composite Steel Beams

SteelEst: Estimating Software

Software developed

13. COBENZ 97: Steel Composite Beam Design Software

Fatigue design of lighting poles

University Lecturing

14. Structural Design 3 lecturing on Steel structures and tutoring in Engineering Design 1 at the University of Auckland 2002 and 2003.

Continuing Education Seminars for Consulting Engineers

15. Preparation and presentation of technical seminars nationwide on structural steel design, construction and estimating topics typically twice yearly between 1997 and 2009.

Papers Published

16. Cowan, H., Beattie, G., Hill, K., Evans, N., McGhie, C., Gibson, G., Lawrence, G., Hamilton, J., Allan, P., Bryant, M., Davis, M., Hyland, C., Oyarzo-Vera, C., Quintana-Gallo, P., Smith, P., "The M8.8 Chile Earthquake, 27 February 2010", Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 44, No.3, September 2011.

Wijanto, S., Hyland, C.W.K., Andriano, T., "Lessons Learned from the 2010 Canterbury Earthquake and Aftershocks", 2nd International Conference on Earthquake Engineering and Disaster Mitigation (ICEEDM-II 2011), Surabaya, Indonesia, July 2011.

Bothara, J., Beetham, D., Brunsdon, D., Stannard, M., Brown, R., Hyland, C., Lewis, W., Miller, S., Sanders, R., Sulistio, Y. "General Observations of Effects of the 30th September 2009 Padang Earthquake, Indonesia", Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 43, No.3, September 2010.

Bothara, J., Beetham, D., Brunsdon, D., Stannard, M., Brown, R., Hyland, C., Lewis, W., Miller, S., Sanders, R., Sulistio, Y. "Building Safety Evaluation Following the 30 September 2009 Padang Earthquake, Indonesia", Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 43, No.3, September 2010.

Hyland, C.W.K., Wijanto, S., "Lessons for Steel Structures from the 2009 Earthquake Damage in Padang", Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 43, No.2, June 2010.

Hyland , C.W.K., Ferguson, W.G, "Steel Fracture Behaviour in the Chilean Earthquake February 2010', International Conference on Structural Integrity and Fracture, University of Auckland, 2010 *(presentation only)*

Hyland, C. W. K., Ferguson, W. G., and Butterworth, J. W. (2007). "Assessment of Cyclic Ductile Endurance of Structural Steel Members." International Journal of Advanced Steel Construction, Hong Kong.

Hyland, C. W. K., and Ferguson, W. G. (2006). "A Fracture Mechanics Based Approach to the Assessment of Seismic Resisting Steel Structures." Fracture of Materials: Moving Forwards 2006, Sydney, 312, pp.89-94.

Hyland, C. W. K., Ferguson, W. G., and Butterworth, J. W. "Recommendations for Improved Material Performance Criteria for Seismic Resisting Steel Structures in New Zealand." International Symposium of Steel Structures '05, Seoul.

Hyland, C. W. K., Ferguson, W. G., and Butterworth, J. W. (2005). "Structural Steel for Seismic Performance." Journal of the Structural Engineering Society New Zealand, 18(1).

Hyland, C., Ferguson, W. G., and Butterworth, J. W. "Assessment of Cyclic Ductile Endurance of Structural Steel Members." Pacific Steel Structures Conference 2007, Wairakei, New Zealand.

Hyland, C. W. K., Ferguson, W. G., and Butterworth, J. W. "Selection of Structural Steel for Seismic Performance." New Zealand Metals Industry Conference 2004, Christchurch.

Hyland, C. W. K., Ferguson, W. G., and Butterworth, J. W. "Effects of Pre-strain and Aging on the Fracture Toughness of Australasian Constructional Mild Steel." Structural Integrity and Fracture 2004, Brisbane.

Hyland, C. W. K., Ferguson, W. G., and Butterworth, J. W. "The Effect of Monotonic Tensile Pre-strain on the Charpy V-Notch Properties of AS/NZS 3679.1 G300 Structural Steel Sections." 2003 Joint Conference of SCENZ / FEANZ / EMG, Institute of Technology and Engineering, Massey University, Wellington, p.59-64.

Hyland, C. W. K., and Ferguson, W. G. "Cyclic Fracture Limit States in Seismic Resisting Steelwork Structures." Proceedings of the Australasian Structural Engineering Conference, 2001, Gold Coast.

Hyland, C.W.K., Clifton, G.C.C., Butterworth, J.W., Stickland, S., “Composite Down-Stand Steel Beam Behaviour with a Profiled Deep-Deck Slab”, Australasian Structural Engineering Conference, Gold Coast, 2001

Summary of Professional Practice

17. I have 27 years of civil and structural engineering experience. This includes 11 years in general consulting engineering, 10 years as Manager of the Steel Structures Analysis Service at the New Zealand Heavy Engineering Research Association, 3 years as Secretary and Manager of Steel Construction New Zealand, followed by specialist consulting engineering focussing on structural fatigue, earthquake engineering and collapse investigation.

Professional Practice Experience Relevant to the CTV Building Collapse Investigation

18. Collapse and Earthquake Damage Assessment and Reconnaissance

PGC Building Site Examination and Materials Testing Report for the Department of Building and Housing.

Forsyth Barr Stair Collapse Site Examination and Materials Testing Report for the Department of Building and Housing.

Assessment of reinforcing steel damage in the February Aftershock for Pacific Steel Group.

Stadium Southland Roof Collapse Investigation report with StructureSmith, and Laboratory Examination and Testing report for the Department of Building and Housing.

Building safety evaluation data management system development and application support to the Christchurch City Council in the aftermath of the September 2010 Earthquake on behalf of the Department of Building and Housing.

Revision of Building Safety Rapid Assessment forms, guidelines and data management software with David Brunsdon drawing on lessons from Padang evaluations, for the Department of Building and Housing, 2010.

Earthquake damage reconnaissance with NZSEE Chile 2010

Building safety evaluation and repair concept development with NZAID/NZSEE team member to Padang, Indonesia, 2009.

19. Analysis and Strengthening of the Stanford Graduate School of Business after the Loma Prieta Earthquake in 1989, on secondment to Rutherford and Chekene Engineers, San Francisco.

Analysis, evaluation and design for major structural upgrading to compliance with the demands of the USA 1988 Uniform Building Code. This work was initiated after damage to the 5 level building occurred during the October 1989 Loma Prieta earthquake.

The existing concrete shear wall building designed in the early 1960s was analysed using ERSA and strengthened by thickening selected existing shear walls and adding new ones to bring greater regularity and structural symmetry. This then reduced demands on the existing structure to acceptable levels. Upgraded wall thicknesses varied from 450 to 560mm with heavy reinforcement and were constructed using shotcrete.

Cracking in existing walls caused by the earthquake was epoxy grouted prior to the strengthening works. This project was featured in the American Concrete Institute magazine 'Concrete International' May 1992.

20. Experience using the Loadings standard NZS 4203:1984 and the Code of Practice for the Design of Concrete Structures NZS 3101:1982 design standards used in the design of the CTV Building.

Ten years practice designing some or all of the reinforced concrete aspects of the following projects from 1985 to 1995 in New Zealand and Papua New Guinea. City Life Apartments; Quay West Apartments; Hobson Centre; BHP Glenbrook: Cogeneration Plant Turbine Hall and Pipe Bridge; 19 Storey Hotel in Federal St; Skycity: Best Dressed; Skycity Development; Barrys Point Rd Shopping Centre; Teaching Block, St Kentigern College; Balfour Rd Apartments, Parnell; People-Mover:Whakarewarewa concept, Rotorua; Serra Wharf, Vanimo, PNG; Wewak Main Wharf Rehabilitation; Lamana Commercial Development, PNG; Webb Street Apartments; Harbourview Apartments; Nambawan Finance Haus ; Government Haus; Daru Provincial Government Building; Wewak Main Wharf Rehabilitation; Nambawan Finance building; Jackson Airport Redevelopment; 20 storey, Parktower, Port Moresby; 27 Storey PTC Downtown, Port Moresby. Watties Frozen Foods, Gisborne: Addition of corn-cob Conveyor Mezzanines to No. 2 and No. 3 Blast Freezer Tunnels; Pakuranga Shopping Centre Redevelopment; NZ Parliament Buildings Seismic Strengthening Proposals; Princes Wharf Redevelopment proposal; Auckland High Court.

21. Experience using the Specification for Concrete Construction NZS 3109:1980 used in the construction of the CTV Building.

    I was involved in the observation and inspection of reinforced concrete construction specified to comply with NZS 3109:1980 for many of the projects described above. In addition during that time I prepared the specification and undertook engineering observation and inspections of 200 metres of Whenuapai Airforce Base runway reconstruction. I also undertook engineering observation and inspections during the construction of the Waiwera River Bridge replacement, and a multi-level teaching block at Carrington Polytechnic.

22.. Experience using Elastic Response Spectra Analysis (ERSA) in the design of structures.

    I have used ERSA since 1989 on numerous reinforced concrete and steel structures. This includes its use for over 120 preliminary design schemes prepared for consulting engineers around New Zealand including Pacific Tower in Christchurch.

23. Practical work experience relevant to the CTV Building collapse investigation

    I worked as a reinforcing steel placer during the construction of reinforced concrete penstock inlet structures for Ohau C and a weir on the Upper Waitaki hydro-electric scheme at Twizel from November 1980 to February 1981.

24. Research projects relevant to the CTV Building collapse investigation

    While working at the New Zealand Heavy Engineering Research Association I undertook a number of applied research projects involving the development of a laboratory testing programme, testing, analysis of the results, computer modelling and development of design guidance.

    Two of these projects have some relevance to the CTV Building collapse investigation.

    Push-off testing of long shear studs with deep composite steel and concrete decking, at the University of Auckland in 2000.

    Tall building response to serviceability wind loads project including wind tunnel testing with Opus Central Laboratories, in conjunction with sponsored undergraduate and Master's research projects at the University of Auckland from 2001.

Signed: ______________________________ Date: 11 July 2012

Clark Hyland

Contents

REVIEWS

In this Section

Review of Statement of Evidence John Mander WIT.MANDER.0001

WIT Ref	Issue	Comments
.42 para1	Were there faulty assumptions and incorrect conclusions in the Hyland / StructureSmith CTV Building Collapse Report? Do Priestley.0001 and Holmes.MAD249.0372 back up the assertions of John Mander?	It is not established by John Mander what the faulty assumptions are, why they are faulty and what incorrect conclusions were made in the Building Collapse Report ("BCR"). John Mander makes many assertions which are quite different to the comments made by Nigel Priestley and William Holmes about the BCR. Refer to Hyland .0001 for responses to Priestley.0001 and to Holmes.MAD249.0372.
.43 para8	Did collapse occur because of column euler buckling over four floors?	Buckling is not possible in a braced frame with columns of this dimension even if pin ended between floors. To extend over a number of floors the floor slab would have had to totally separate from the columns on all sides. The level of lateral restraint required to restrain a column from buckling is typically only 3% of the axial compressive action in the column.
.44 para1	Was column failure the cause of the collapse as found by the Building Collapse report?	That conclusion appears to fit the collapse evidence, eyewitness testimony, material testing and analyses the best. Four scenarios were considered and all led to failure of a column that caused progressive collapse of the Group 2 frames. The South Wall and the North Core were found to have a reserve of strength beyond what had been applied to them by the February Aftershock prior to the collapse.
.44 para3	Was the building designed and constructed in compliance with the applicable design and building codes?	The BCR p.109 to 116 sets out a number of non-compliance issues found in the design and construction.
.44-46	If the building survived the September Earthquake did that prove it had satisfied the design expectations of NZS 4203:1984 and NZS 3101:1982?	NZS 4203:1984 and NZS 3101:1982 do not refer to survival of a specific earthquake as a means of compliance. The standard sets specific earthquake loads, actions and drifts that the structure is required to be able to resist without collapse with a 10% probability of exceedance in a design life of 50 years or once in 475 years. The CTV Building suffered only minor structural damage after the September Earthquake according to all those who inspected it. Only one column at Level 4 at F4 was found to have a crack of all the columns inspected between Level 1 and Level 5. Some hairline cracking in the South Wall and North Core occurred. This shows that the building response to the September Earthquake did not come close to exceeding the seismic loads it was designed for. Considering an axial compression load of approximately 250kN in the Level 4 corner column based on BCR Fig 161 a drift of

		around 0.6% would be required to achieve yield in reinforcing and some permanent crack width, whereas it would have been capable of sustaining nearly 1.6%. None of the other columns however on that level were found to have any obvious cracking and flexible sealant is seen on the west side of the column in the C PG photo. It is therefore an indication that the column may have been in contact with the Spandrel Panel on the west face, developing a short column enhancement of bending and shear. Similar cracking would also have been expected in the same column at Level 5 given the drifts were similar and the axial compression less, but none was found. This is further evidence that the Level 4 F4 column was being affected by contact with Spandrel Panels on the east face. It may have been a precursor of the collapse initiation occurring on that line in the February Aftershock. Structural damage appeared to be sustained at the connection of the column C18 to the overhead wing wall on Line D/E as seen in the photos by Peter Higgins. However this is an indication of a design non-compliance issue. The column being connected into the North Core was required to be designed as part of the North Core system or be separated from it. If connected in then the column and connection was required to be subject to capacity design to ensure it's failure did not occur before the North Core achieved its capacity. In the February Aftershock the connection failed and the column did collapse however this occurred as a consequence of failure starting in the south of structure. The column itself was not dependent on the connection to the North Core to sustain the axial loads on it so once the connection failed increased flexibility of the core is understandable but the redundancy remaining was sufficient o allow a redistribution of stresses.
.46 para2	Could there have been major hidden (unobserved and/or unobservable) damage that therefore justified giving the CTV Building a red placard?	John Mander claims the CTV Building must have sustained hidden (unobserved and/or unobservable) damage. If no significant structural damage was able to be observed there was no significant structural damage. The damage observed is summarised in BCR p.49 to 57. Detailed inspections were carried out by a suitably qualified engineer David Coatsworth. Inspections by others for other purposes such as firefighter Andrew Ayers (wit. Ayers.0001), an experienced quantity surveyor Leonard Pagan (wit.Pagan.0001) and two experienced concrete crack repair contractors Graeme Smith (wit.GSmith.0001) and Peter Higgins (BUI.MAD249.0453) preparing estimates for repairs including an inspection inside the Elevator shaft of the North Core found nothing remarkable about the damage at a structural level. The appropriate placard appears to have therefore been applied.
.46 para2	Should a knowledge of the level of recorded ground motions have influenced the rapid assessment of the CTV Building and subsequent inspections?	The level of ground motion recorded has no relevance to structural damage inspections. Structural damage inspections identify damage caused by an earthquake. The low levels of structural damage observed indicate that the response of the CTV Building to the September Earthquake was well below the design level response.
.46 para3	Was the building lively because of hidden damage sustained in the	There was no hidden damage except perhaps some possible structurally insignificant slippage in the Drag Bar bolts that would not have affected their strength.

	September Earthquake?	The building was inspected by a number of people on a number of occasions. Coatsworth, Pagan and Smith prepared professional accounts of the damage and found the damage to be minor and unremarkable. The damage at the connection of the head of the column C18 into the North Core wing wall on Line D/E however would indicate that shear failure had occurred in the connection and therefore some reduction of stiffness of the North Core would not be unexpected. No structural drawings were available to the inspectors so it was not recognised that the column head was connected into the North Core rather than a lintel beam. Tenants reported that the building was known to be lively some years before the September Earthquake. Ron Godkin reported that a bank tenant had moved out eight years earlier apparently because of the liveliness. Peter Brown, Nilgan Kulpe, Peter Millar all reported that the building was lively prior to the earthquake. Calculations show that the floor had a natural frequency of 4.1 Hz and was susceptible to footfall vibration in open plan areas. The reports of groaning of the structure (wit.Fortune.0001) may indicate slippage in the Drag Bar bolted connections which had 2 mm oversized holes. The damage to non-structural partitioning may also have had some effect on increasing lateral flexibility, however that is not relied on for structural design purposes.
.46 para1	Can floor slabs be excited by high frequency vertical motions?	The floors would have been susceptible to high frequency low displacement response. The actual level of response of the building to the vertical ground motions from the September earthquake and the February Aftershock is difficult to determine in much the same way as the horizontal response is difficult to determine without calibration to the observed damage. Partitioning will also inhibit vertical response of floors and damp it.
.47 para3	Did vertical accelerations in the September Earthquake break the "fixed-end" condition to make the slabs function like three simply supported units with vibration induced deflections amplified by up to 500%	The displacements required to fracture the slab topping reinforcing saddle bars would have had to have been large and would have left permanent and observable damage. Such displacements are not compatible with vibration response at 4 Hz. The composite slab itself was very ductile. The decking is controlled rolled to give it high tensile strength. It does not rely on chemical bond to develop its behaviour. See also comment made on Frost. Refer BS 5950.4 :1994 Code of Practice for the Design of Composite Slabs with Profiled Steel Sheeting. This is the standard used for testing of composite deck products until superceded by Eurocodes. Hibond utilises Cl 4.6.3 Plain Re-entrant Angle Profiled Sheets (Fig 4) and Cl 4.6.4 Embossed profiled sheets (Fig 4c and 4d) mechanical bond mechanisms . Moment capacity for full shear connection between the concrete and the sheeting, resulting in sheeting tensile fracture or concrete compression failure should be treated as an upper bound to the capacity of a composite slab (refer cl6.3). The typical modes of failure are shown in Fig 6. Longitudinal slip normally occurs prior to shear failure at supports. The typical failure surfaces on slabs identified by Heywood (Fig 47) did not fail by shear on the internal beams but pulled

		away from the supporting beams. On the outer beams the slabs failed in negative flexural tension and shear. End bearing requirements were for not less than 50 mm seating of the sheeting on the concrete with 70 mm minimum total bearing including concrete (cl 4.7 and Fig 5). The ARCL drawing S15 complied with this. When testing composite slabs for product performance the chemical bond component is deliberately broken by initial testing of 10000 cycles between 50 and 150% of the anticipated Ww (γf) applied load for which 1.5 Ww if exceeded by higher static test loads becomes the lower bound on load capacity Wc (cl 8.2.2.4(d)). The slab is deemed to have satisfied the initial dynamic test if the maximum deflection does not exceed Ls/50. For the CTV slabs Ls= min(7500, 7500-350-2x70= 7010. So the deflection limit of Ls/50 for dynamic load capacity performance = 140 mm . It can therefore be assumed that the CTV Hibond slabs could have sustained vertical displacements of at least 140 mm without collapsing by any of the modes of failure. Vibration analysis of floors to rhythmic excitation using ATC-1 “Minimizing Floor Vibration” and applying it to seismic excitation shows that the peak response is governed by the level of damping. With 5 % damping the peak response is 0.63g at 4 Hz. At that level of loading the floor slab was not overstressed in bending at the supports or midspan. If the floor slab was somehow overloaded to Ls/50 limit the response of the floor would reduce to 0.37g and the loading would reduce to just under 60 % of the peak load. In conclusion there is no basis for failure of the floor slab by vertical response to the earthquake. But significant vertical accelerations well above human comfort levels can be explained. Some minor slab cracking damage was reported by David Coatsworth after the September Earthquake at the connection of the slab to the South Wall similar to typical shrinkage flexural cracking at supports (BCR Fig24). This may have been pre-existing. No cracking was reported through the vinyl flooring stuck to the concrete slabs at Level 4 though a number of tenants on that level were concerned with humps in the flooring and its unevenness. However none reported any cracking. There is therefore no evidence or reasonable quantifiable basis that “fixed end” fracture of the slabs occurred in the September Earthquake. Discomfort and concern can be caused to occupants at quite small accelerations for floors (0.4 to 0.7% g in offices). People are well known to become more sensitive to floor vibration when stressed or in quieter places, but can cope with 4 to 7% g when involved in rhythmic activity. This is accounted for in the design of floor slabs for vibration serviceability as described in ATC-1 Design Guide 1, Minimizing Floor Vibration.
.47 para	Could floor vibration may have led to buckling of the slab and been a	Interesting proposition due to the slenderness of the slab.

3	primary contributor to triggering the collapse?	Large displacements and high axial compressive actions would have been required to cause the slabs to buckle, plastically deform and collapse. The slab may have been slender in terms of column strut assessment criteria in NZS 3101:1982 cl 6.4.10.1. However the axial loading in the slab from diaphragm actions to force buckling failure need to be checked. Pc could be in the order or 200 kN/m . Most critical condition would be adjacent to North Core as loads try to go into the north Core and not at south end. For base shear of 1996kN s=1 for 08STATXDUCT the greatest demand would be at level 6 of approx 798 kN or 61/kN/m over 13 meters. At this level of loading this does not appear to be sufficient for a moment magnifier greater than 1.0 to be applied. So loading would have to be getting to s=3 levels to be getting critical. Needs more detailed assessment. It is not considered in the structural design of earthquake resisting buildings as slabs like the one used in the CTV Building could sustain large displacements without failing and the level of axial compression through the diaphragms would not have been particularly high. The levels of displacement however may have caused significant damage in the non-structural partitioning leading to damping well above the 5% assumed and led to significantly reduced response. It would be interesting to investigate this effect further. There was no evidence of slabs having collapsed at midspan due to buckling in the collapse debris and there is no reasonable quantifiable basis for it to have occurred, based on preliminary calculations.
.48 para2	Was the deviance from customary ductile column detailing acceptable?	The CTV columns did not comply with requirements for Group 2 frames in NZS 3101:1982 . There appears to have occurred in other buildings in Christchurch designed ignoring the requirements for Group 2 frames according to John Henry.
.48 para3	Was the CTV Building revolutionary for the time?	There does not appear to have been anything particularly revolutionary about the design. However important aspects of the knowledge of reinforced concrete design of structures for earthquakes current at the time appear to have been ignored.
.49 para4	Was the lack of adequate transverse reinforcement in the columns neither a problem nor a cause of failure?	The lack of confining and shear reinforcement reduced the drift capacity of the columns. Increased confining and shear reinforcing would have increased the ability of the columns to sustain the drifts imposed without failing even with short column effects from contact with the Spandrel Panels.
.50 para4	Was there a lack of forensic evidence of column failure modes?	Evidence was found of column hinging at the base and heads and also at the mid-height in vicinity of the termination of the vertical reinforcing steel and Spandrel Panels.
.50 para 1	Did fatigue damage exist in the beam-column joints?	There is no basis for high cycle or low cycle fracture damage having occurred in the joints. High cycle fatigue occurs at low stresses and requires hundreds of thousands of cycle of loading. Low cycle fracture requires severe loading reversal in the plastic range of the materials and notching to occur typically from buckling of the reinforcing steel. There was no observable damage to the beam column joints at Level 1 to underside of Level 6 after the September Earthquake found by Coatsworth, Peter Higgins or Graeme Smith meaning significant plastic deformation had not occurred, so

		low cycle fracture had not occurred. Similarly the small number of cycles associated with Earthquake responses of structures means high cycle fatigue could not have resulted from the September Earthquake. For example even 100 x 10 second earthquakes at 4 hz would be equivalent to only 40,000 cycles of loading. The slabs were typically tested to 10,000 cycles by BS5950.4:1994 requirements for a severe stress range just to break chemical bond prior to static load testing, of between 50 and 150% of the anticipated Ww (γf) applied load for which 1.5 Ww if exceeded by higher static test loads becomes the lower bound on load capacity Wc (cl 8.2.2.4(d)). 40,000 cycles of much low stress loading from many small aftershocks is therefore not a significant fatigue loading. It is interesting to consider the response that may have occurred to aftershocks from floor excitation effects.
.50 para2	Did beam –column joints the trigger of the collapse?	Gravity closing moments meant that at drifts necessary to initiate column failure in Scenario 1 there was no opening moment causing tension in the bottom reinforcing steel. However the fragility of the beam-column joints likely contributed to the rapid progression of the collapse once it had started in the perimeter columns.
.50 para4	If the NZS3101:1982 limited ductile design requirements for the Group 2 frames had been complied with, would the building have survived the February Aftershock?	The requirements for Ch 14 limited ductile design of the Group 2 frames extended into the design of the beam column joints and the beams, as well as to the columns. Application of these requirements would have significantly improved the drift capacity of the columns, reduced the potential for beam-column joint failure and progressive collapse. It would also have reduced the likelihood of column fracture occurring at the termination zones of the vertical reinforcement in the east and south frames. However it would not have prevented the short column effect enhancement of the bending and shears in the columns along the south and east faces due to Spandrel Panel contact. However the greater resilience of the Group 2 frames would likely have meant that the columns could have endured further into the February Aftershock and probably beyond it. It appears from witness Statements that the collapse initiated near the peak response of the building after 7 to 12 seconds (wit.Jackson.0001). So a little more resilience may have got it through the Aftershock. It also needs to be recognised that the CTV Building had other vulnerabilities. The Drag Bars were vulnerability and are estimated to have only been able to remain connected to the North Core up to simultaneous drifts of 1% occurring along Line F and not long after flexural yielding of the South Wall. Once they disconnected collapse is then likely to have occurred in a Scenario 4 manner. So collapse may still have occurred but by a different mechanism .
.50 para4	Were properly confined beam-column joints considered an expensive and unnecessary luxury in Christchurch in the 1980's ?	The University of Canterbury was a world leader in the development of seismic resisting reinforced concrete design and construction at that time. I was designing concrete structures in Auckland during the 1980's and was aware of the requirements for seismic design such as properly confined beam-column joints. Engineers in Christchurch would have also been aware of these requirements.
.51	Is the in-situ strength of concrete	Three samples taken from a batch of concrete at the batching plant or during the pour do not define the in-situ strength.

para1	formally defined as the average crushing stress of three 100 mm diameter by 200 mm long cylinders tested at 28 days after pouring of concrete?	These are used to assess the conformance of the concrete supplied with the specification. The average of three tests are then compared using a statistically based approach to the production standard to assess rejection or acceptance. This is because concrete production like produces a statistical range of properties for a specified mix. Acceptance and rejection of a particular batch of concrete therefore needs to consider the range of properties that are acceptable to occur for a specified mix and the ability to determine conformance with that using a small testing sample. The assessment of in-situ strength is more complex and requires a consideration of sample locations and statistical analysis. The assumption of the concrete design and construction standards is that if concrete is supplied in conformance with the production standard NZS 3104 and then placed and cured in accordance with NZS 3109 the lower 5% in-situ concrete strength will be not less than the characteristic or lower 5 percentile strength specified by the designer. NZS32104:1983 which was applicable at the time of the CTV Building construction did not specifically define the specified 28 day strength as the 5 percentile strength but was close to it. In later revisions the characteristic or 5 percentile strength became more clearly defined.
.51 para1	Is the target strength the mean strength of concrete batched to achieve a specified 28 day strength?	The target strength is a measure set so as to give an appropriate level of confidence that a batch will not be rejected based on a specified amount of sample testing set out in the concrete production standard NZS 3104. It gives a particular level of confidence that the batch is conforming to the range of properties expected of the mix design.
.51 para1	Should only 100 mm diameter by 200 mm long cylinders allowed to be used for concrete testing?	It is preferable to use a consistent size of cylinder with consistent length to diameter relationships. Industry standard practice has been in New Zealand to set the use of100 x 200 cylinders. In the UK however 100 x 100 cubes are tested rather than cylinders. The concrete testing standard NZS 3112:Part2:1986 specifies the core diameter to be no less than 4 times the nominal maximum size of the aggregate. For 19 mm aggregate this is 76 mm. For investigations such as that required on the CTV Building the extraction of smaller diameter cores is sometimes required such as on the slabs. Bartlett and Macgregor found from an analysis of strength data from 1080 cores tested by various investigators that the strength of 50 mm cores was on average 6% less than the strength of a 100 mm core. For larger diameter the core the difference became negligible. The shape and size of the specimens affects the results. Cube strength are higher for the same concrete. Similarly tests undertaken on cylinders with length to diameter ratios less than 2, as used by John Mander will give higher test strengths. The reason is that concrete fails in a combination of columnar and shear cone mechanisms. A change in shape affects the orientation and failure surface patterns, and confinement within the specimen. To compare one batch of concrete cylinder test results to another it is best to maintain the same l/d ratio. Unfortunately the concrete testing by John Mander ignored this and the results have had to be adjusted using an empirical approach which reduces their usefulness. The core l/d ratios ranged from 1.09 to 2.0. International standards disagree on the strength correction factors that should be used to adjust for length to diameter ratio. BS1881 requiring a cylinder with l/d =1.0 to have its strength reduced to 80% of the test strength to give comparable results to l/d =2 specimens, whereas ASTM C 42M requires it to be reduced to 87% (AC 214.4R Table 6.2). The use of l/d =2 is the

		recommended and base for international standards and the conversion factors to adjust test results. ACI 214.4R-03 notes: "strength correction factors are less accurate as the magnitude of the necessary correction increases for cores with smaller l/d. Thus corrected core strength values do not have the same degrees of certainty as strength obtained from specimens having l/d of 2"
.51 para3	Were the cores diameters used in the SEMT report testing too small?	In the testing used in the Site Examination and Materials Testing report care was taken to maintain the l/d ratio as close as possible to 2.0 to minimise use of empirical correction factors. The use of smaller diameter cores was preferred over the use of cores with l/d ratios significantly less than 2.0 due the reliability of the results being better than using larger diameter cores with low l/d values.
.51 para3	Were the test results wrong because some were lower than the minimum specified strength?	The test results are what they are and show low concrete strengths occurred in some of the columns. The results indicate some of the concrete columns were non-compliant with the concrete construction standard.
.51 para3	Did fire affect the strength of the concrete test results?	The tell tale sign of fire having affected the strength of concrete is the colour. All the cores tested for the SEMT had typical concrete colour and were therefore not affected by fire. All the columns tested showed no outward sign of fire except the 400 square column at C18, as the paint was still intact on most indicating no significant heating had occurred to reduce their strength. Concrete is also known to regain its strength after fire.
.51 para3	Can a visual inspection of concrete determine the strength of concrete more accurately than test results?	Visual inspection can only identify qualitative aspects of concrete quality. Accurate assessment of strength requires the use of non-destructive or destructive testing procedures.
.51 para4	Were the process and procedures used for sampling and testing the concrete columns appropriate in the SEMT Report?	Columns were selected at random from the debris pile. As a consequence it is reasonable to conclude that a random distribution of columns from all levels would have been present among the columns tested. They were identified and marked with test locations for coring and Rebound Hammer testing. All testing was undertaken in accordance with the appropriate standards and recommendations.
.51 para5	Was there statistical significance in the SEMT results showing the concrete to have less strength than would be expected and less than specified in some cases?	The distribution of results was compared statistically to the population distribution for concrete manufactured in accordance with the minimum requirements of the production standard and found to be low. This is an appropriate way of testing statistical significance where the statistical properties of a specified population is known.
.52 para2 -4	Can conclusions be drawn about the column strengths based in isolation on the 8 core tests	Test results typically require some care in interpretation. The results should be considered in conjunction with the other 19 concrete core and Rebound Hammer results. It is not appropriate to ignore results that don't suit a desired result. The irregular l/d ratios used for the CTL tests need to be considered also and the results may need to be corrected. CTL

	undertaken by CTL?	Thompson do not state what correction they have applied. Mr Haavik called Geoff Jones to discuss this (wit.GJones.0001). Perhaps he was concerned about their results. The eight core tests are of only four columns. Whereas the SEMT tested 26 columns by utilising Rebound Hammer testing. The Rebound Hammer testing data also needs to be compared and combined if possible to give a more complete picture of the concrete strengths
.52 para52	Do the CTL test results justify using 1.5 times the specified strength in analysis of the columns?	Use of 1.5 specified is useful if there was no testing to justify a different value. However the combined results with the SEMT would be a better basis to make a decision. It is not possible statistically to conclude from the concrete strength results of the four columns whether they were from a concrete with different mix designs with 28 day strength of 25, 30 or 35 MPa given the range of concrete strengths that will be expected from batches made to the same mix design for each type. The densities indicate that they may all have been from the same mix design. The densities of the concrete CTL Thompson tests are on average 2361 kg/m3. I am told that Christchurch concrete has consistent densities for each mix design due to the consistency of the aggregate density. Concrete with 28 day strength of 25 MPa apparently has a typical density of 2380 kg/m3.