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**

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 a peer review and a critique.
6. The peer review was by William Holmes (**BUI.MAD249.0372**), and the critique was by Nigel Priestley (**WIT.PRIESTLEY.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: 24 June 2012

Clark Hyland

Contents

REVIEWS

In this Section

Review of Statement of Evidence William Holmes BUI.MAD249.0372

Review of Statement of Evidence William Holmes BUI. MAD249.0372

WIT Ref	Issue	Clark Hyland's Comments
.2 para1	Review based almost totally on the documents available for review. No significant calculations or viewing of collapse debris.	In summary William Holmes ("WH") appears to put the case for Scenario 3 Level 3 diaphragm pull out from North Core as the collapse initiator. This is a reasonable scenario. However the drift demands at Level 3 not being enough for detachment when the masonry wall is considered effective at Drag Bar capacity. WH also considers collapse may have initiated with the A1 column buckling at the beam column joint at Level 4. The A1 scenario is discussed further later in this response however it is not likely to have been an initiator but rather as part of the progression of the collapse from east to west along Line 2 and may be that the beam pulled out of the joint leading to the column kicking out because of loss of support rather than from buckling. The collapse evidence doesn't support the idea that the Level 3 slab could break away from the North Core and fall onto the Level 2 slab then have the Line 3 frame collapse. Refer BCR Fig 95 which shows the north end of the Level 3 slab still close to the Level 3 floor level and what appears to be Hibond perhaps still attached at Level 3. The Line 3 beams are reported by Heywood to have displaced northwards. In fact careful examination of BCR Fig 99 shows the Level 2 slab on the ground and its supporting beam at Line 3 rotated southwards. The Level 3 slab is still leaning against the North Core and its supporting Line 3 beam can be seen rotated southwards but pulled closer towards the North Core than the Level 2 bema. The level 2 slab that it is laying on has its south ward end pushed over the top of its supporting Line 3 beam. The lie of these members therefore indicates that the Level 2 and 3 slabs and support beams rotated downwards about their North Core attachment as the columns on Line 3 collapsed evident by the southward rotation of the beams. The Level 3 slab then hit the Level 2 slab knocking its north end to the ground at Level 1 and forcing its south end over the top of its supporting beam on Line 3. Also refer to Heywood evidence of northward movement of the slabs between Line 3 and 4 as they fell and no northward displacement of the slab ends at Line 1. The northwards rotation of the Line 2 beams shows that their vertical support was lost and they rotated downwards prior to breaking off the Line 1 frame. The tension developed in the slab between the South Wall and the North Core eventually pulling the slabs apart perhaps at Line 3. WH does not appear to accept that a flexural hinging mechanism is sufficient for collapse to initiate in a braced frame situation. This is theoretically the case if the hinges were points only. However Fig 101 shows that hinging of a 400 mm diameter column could extend over 800 mm based on a 60 degree cone shape of the compression failure surfaces. So a localised mechanism could occur at the heads or the mid-height splice end locations, enhanced by Spandrel Panel contact, sufficient for significant loss of vertical load carrying capacity to occur.

.2 para4	High attention to detail to eyewitness reports, site examination and testing to back up the most likely collapse condition of Line F or 1 caused by excessive drift	Line 1 is less likely to have led to progressive collapse due to vertical load carrying element of South Wall.
.3 para1	Structure of the report made it difficult to follow and justifications for the conclusions difficult to piece together.	Due to nature of joint authorship and changing requirements for structure by DBH Project Manager and Panel. Varying views had to be expressed and formats and material added and removed on a number of occasions in the interest of getting as much consensus as possible.
.3para2	Identifies a new collapse scenario	
.3para3	Commentary language is informative not mandatory	Agreed.
.3para4	NZS3101:1982 cl3.5.14.3(f) 50% or 55%? Table 1 and 2 confusing	The 50% requirement in that clause is the limited ductile detailing limit which is 50% to 100% of the K/SM drift. The K/SM or $\nu\Delta$ drift for non-seismic detailing is 55% of S=5 refer fig 162. 1986 relates to the time of the design. The language is simplified at DBH request for the Tables 1 and 2 (ie for public readership that may not recognise the subtlety of ductile and limited ductile requirements) and is the fully elastic rule for which the additional seismic detailing requirements were required ie limited ductile ch 14. The more detailed description is in Appendix F and Fig 162 which shows that limited ductile requirements were triggered.
.5para3	Correct interpretation of Group 2 requirements	Agreed as noted in App F.
.5para5	What seismic detailing was required?	The ch14 provisions were required which specified more stringent minimum detailing requirements for limited ductility
.6para1 and 2	Were limited ductile detail provisions triggered for other	Not clear what WH is saying here. The ratio of D/2 to C/1 east west drifts can be derived from Tables 15 and 16. For L4 in Feb ULS D/2 of 1.79% compared to 2.42% for C/1 gives a ratio of 3/4

	columns ?	If the columns remained elastic at 0.5 $v\Delta$ then would have been elastic at 0.25 $v\Delta$. The code approach required that the later the column exhibited inelastic initiation then less additional seismic design was required. For example the elastic deformation limit of column C/1 in Table 13 at L4 is 0.73% the K/SM=2.75 or $v\Delta$ drift is 0.79%. Therefore the column would not remain elastic at that drift and could not be assumed to be able to be designed for the "non-seismic loading" provisions. However because the elastic deformation limit is greater than 0.5 $v\Delta$ it would only have to satisfy the Ch14 limited ductile design requirements. Note that the Summary on p.261 and 262 describes the matters succinctly. The lack of respect for structural symmetry and the relative flexural weakness of the South Wall relative to the North Core meant that the check was unconservative and greater caution was required to assess the $v\Delta$ drift.
.6para3	"Gravity" framing . Elastic limits for hooks	Gravity framing is not the correct term for the Group 2 elements. These were still required to be designed for the drifts imposed of $v\Delta$. If elastic requirements were satisfied for the columns or beams then hooks could be designed without reference to the additional seismic requirements. The members still had to satisfy the demands from the imposed $v\Delta$ drifts (cl 3.5.14.3). 2D frame analyses could be done using the imposed displacements related to the $v\Delta$ drifts. When this was done it was clear that the flexural and shear demands exceeded the code requirements. However this did not explain the collapse sequence observed as it was known that the shear provisions in the Code were conservative due to the many unknowns related to shear capacity of concrete columns in combination with flexural/axial actions and the effects of confining steel. However if the drift compatibility analysis using 2D frame analysis had been done by the designer then it would have been apparent that significant shear and ductility would have been required in the columns and beams-column joints.
.6para4	"members not designed for seismic loading" terminology may have confused designers into not considering the Group 2 requirements	This may have happened but was an incorrect interpretation of the standard. The definition of "Not seismic loading" included elastic design of S=5 for the primary frame or satisfied the elastic criteria at $v\Delta$ displacements of the Group 2 frames. This is similar to wind or gravity design with no dependence on additional ductility to offset inelastic design actions. 2D frame analysis was able to be undertaken using the $v\Delta$ displacements as prescribed displacements. This is a pragmatic method of analysis to estimate the requirements for satisfactory ductility and strength to be built into the Group 2 frames.
.6para5	Minimum shear and anchorage requirements for "members not designed for	This may have happened but was an incorrect interpretation of the standard. The definition of "Not seismic loading" included elastic design of S=5 for the primary frame or satisfied the elastic criteria at $v\Delta$ displacements of the Group 2 frames. This is similar to wind or gravity design with no dependence on additional ductility to offset

	seismic loading"	inelastic design actions.
.7para2	Conclusion: "not designed for seismic loading"; limited ductility requirements; hooks etc	The conclusion reflects incorrect interpretation of the requirements of Group 2 elements; "not designed for seismic loading" and how that affects design of hooks etc. The definition of "Not seismic loading" has included elastic design of S=5 for the primary frame or satisfied the elastic criteria at νΔ displacements of the Group 2 frames. This is similar to wind or gravity design with no dependence on additional ductility to offset inelastic design actions.
.8para8	Collapse mechanisms	It is agreed that a collapse mechanism does not necessarily develop in a building with lateral support provided by shear walls. However this was subject of much debate with some DBH Panel members. The evidence of hinging of columns on perimeter frames at mid height or adjacent to column bar splices found from the collapse examination was opposed by Nigel Priestley and significant pressure was applied to remove reference to three hinges forming in columns due to short column effects. However this is still noted in the Figs 17 and 18 as a contributory cause. It was also acknowledged that the shear demands on the columns were significant at the drifts imposed and well exceeded code levels. The criteria of 0.004 concrete strain was accepted as a reasonable point at which load carrying capacity would be lost. This is considered reasonable given recommendations of the NZSEE guidelines use that limit and recognition that the concrete used was uncrushed river gravels that are recognised to have lower strains to compression failure and that the confinement provided was less than those used in the development of the NZSEE guidelines. The preferred failure sequence is consistent with witness observations and can be justified by analysis. The collapse sequence that was observed appears to be most consistent with column failure on Line F progressing into overload of Line 2 columns. The possibility of an internal column failing due to combination of low concrete strength and vertical acceleration remains possible as described in Scenario 2. It is also possible that both Scenario 1 and 2 happened together, however the Drag Bars set a bound on what the maximum drifts may have been on Line 2 or 3 at the onset of a Scenario 2 collapse initiation.
.9para 2-4	Beam-column joint initiation	Beam-column joint failure can be shown not to have preceded column hinging at the drifts associated with Line F column failure benchmarked to 1% associated with Drag Bar failure. Opening moments simultaneous with that level of drift were not sufficient at the underside of the beams at the connection to develop sufficient tensile forces in the bars and hence initiate failure of the hooks. However it can be seen that once collapse developed the beams column joints were the most fragile elements and would have pulled apart as was seen in the collapse debris.
.9para5	Beam-column joint initiation	The conclusion that Line 2 and 3 joint collapse initiated the collapse (modified Scenario 2) is not consistent at the drifts at which Drag Bar failure was required without significant vertical acceleration or low concrete strength. This indicates short column effects on the perimeter frames which increased shear and drift demands on the columns

		perhaps in conjunction with vertical acceleration and lower than expected concrete strengths as the most likely Scenario. It is not clear why there would be an eastward throw or why the upper levels would drop as a unit onto those below if central column failure initiated collapse.
.11	Graeme Frosts observations of failed beam column joints	These observations are accepted and the collapse of the Line 2 and 3 columns before detachment of the slab from the North Core is accepted as part of Scenario1 however it appears that at the drifts benchmarked by the capacity of the Drag Bars that short column effects and loss of capacity along Line F is the most likely initiator. The observation of the upper levels falling as a unit is not explained by beam-column joint failure. But is by Line F initiation. I know Graeme Frost well at a professional level and have discussed this with him and he accepts that he did not undertake any analysis in relation to his comments but could see how Scenario 1 could be an explanation of his observations.
.12	Column 1A buckling	Fortune 's observation are noted including the large westward sway which is consistent with masonry infill engagement with et concrete frame online A. BUI Mad 249.0222.2 shows the Level 3 to 4 column in one piece lying kicked out southwards from the corner. It does not have a mid-height fracture as thought by Fortune. An explanation may be that the beam pulled out of the beam-column joint at Level 3 causing the column to kick outwards and tear away from the lower portion of the column. This is a consequence or progression of collapse further east rather than initiated by 2 storey column buckling at 1A. The observation of a column kicking out is consistent with Stephen Gill's observation that it appeared to have been kicked out, but up a couple of levels. It is also consistent with collapse having progressed from east to west and fragility of beam column joints. If collapse had initiated at this corner column then a tilt to the west would presumably have occurred rather than one to the east. Nilgun Kulpe reported a tilt of the Level 6 floor at the south west corner towards the east.
.13para 3	Conclusion regarding collapse : preference for beam-column joint initiation	Preference for beam column joint collapse initiation does not recognise the shear and column demands at collapse consistent with Drag Bars not failing as being greater than the demands on the beam column joints at the same drifts. There were a number of fragilities in the structure that if one thing had not failed others would have due to the lack of compliance with the requirement to design for the additional requirements for seismic loading as required to Group 2 elements. Significant improvements would have resulted if the Group 2 elements had even been designed for the $v\Delta$ drifts using the non-seismic provisions for elastic behaviour. However it appears that no attempt was made to design them for the drift demands required according to David Harding. Rather they were

		assumed to have been "pin ended columns" supporting continuous beams.
.13para5	What drifts led to collapse: Scenario 3 favoured by WH	Comparative drifts simultaneous with Drag Bar capacity achievement indicate insufficient demand at the Level 2 and 3 connection to the North Core at the amenities area when the west infill wall is included in the assessment. Otherwise a credible option. However the idea that the L3 slab could pull away and then not drop to the ground is difficult to reconcile. Also consider Heywood's view. Prof Clifton used an unscaled ground motions approach and assumed no restraint provided by the west infill masonry . The collapse evidence doesn't support the idea that the Level 3 slab could break away from the North Core and fall onto the Level 2 slab then have the Line 3 frame collapse. Refer BCR Fig 95 which shows the north end of the Level 3 slab still close to the Level 3 floor level and what appears to be Hibond perhaps still attached at Level 3. The Line 3 beams are reported by Heywood to have displaced northwards. In fact careful examination of BCR Fig 99 shows the Level 2 slab on the ground and its supporting beam at Line 3 rotated southwards. The Level 3 slab is still leaning against the North Core and its supporting Line 3 beam can be seen rotated southwards but pulled closer towards the North Core than the Level 2 beam. The level 2 slab that it is laying on has its south ward end pushed over the top of its supporting Line 3 beam. The lie of these members therefore indicates that the Level 2 and 3 slabs and support beams rotated downwards about their North Core attachment as the columns on Line 3 collapsed evident by the southward rotation of the beams. The Level 3 slab then hit the Level 2 slab knocking its north end to the ground at Level 1 and forcing its south end over the top of its supporting beam on Line 3. Also refer to Heywood evidence of northward movement of the slabs between Line 3 and 4 as they fell and no northward displacement of the slab ends at Line 1. The northwards rotation of the Line 2 beams shows that their vertical support was lost and they rotated downwards prior to breaking off the Line 1 frame. The tension developed in the slab between the South Wall and the North Core eventually pulling the slabs apart perhaps at Line 3.
.14 para3	Simplified sets of drifts	I agree with this approach and was undertaken but not included in the BCR.
.14para4	NTHA drifts are considered high in general. Disconnection included means they were upper bound	The NTHA modelling of disconnection did not allow for failure to progress into the slabs at the amenities area between wall C and C/D so limited effects of the disconnection need to be recognised. It is not clear what damage levels WH is referring to in the NS walls, there appears to have been a northward drift at the upper levels particularly above Level 3 based on the survey done in the SEMT Fig 57 consistent with around 1% drift at the eastern end of the North Core if an elastic component of drift is added in.
.15para 1	Drifts at Line F collapse compared to North Core drifts	No mention of the Drag Bars occurs in the WH review when they signal an upper bound demand on the structure prior to its collapse.

.15para4	Blockwall on Line A: Even if built as detailed interaction would have occurred at large drifts. Damaged after collapse initiating?	WH appears to recognise the effect on response without getting involved in what connection was there. The issue of collapse occurring prior to Drag Bar failure would limit what demand was placed on the masonry infill. Two way diagonal failure of the wall panels seen in Fig 72 show at least 1 reversing cycles of severe damage.
.15para5	Spandrel Panel interaction: Little evidence given that Spandrel panels and or connections to the slabs were strong enough to cause column failure	This was not reported however it was clearly shown that Spandrel Panels were found by analyses to induce short column effects and increase flexural and shear demands on the column heads depending on the amount of gap between them and the columns and the level of inter-storey drift that occurred. Equivalent static drift displacement analyses of 2-D frames using cracked section properties estimated from Cumbia analyses, with prescribed drifts with varying spandrel panel gaps showed this. In conjunction with the low levels of spiral reinforcing below minimum shear reinforcing levels at the tips of the columns vertical reinforcing steel splices a vulnerability to development of mid-height hinging or failure was also found to be a realistic explanation for the failure damage observed such as in Fig 106 (refer Park & Paulay 1975 p.421-422). Development of a mid-height hinge in conjunction with the base and head hinging would have led to sudden loss of vertical load carrying capacity in perimeter columns and shedding of load onto adjacent columns leading to their overload and progression of the collapse. Due to the very low levels of spiral confining reinforcement in the columns it was also considered that at the attainment of a concrete compression strain of 0.004, used in the Cumbia and NPA drift capacity analyses, the columns would also have suffered loss of load carrying capacity. The column failure shown in BCR Figure 101 shows that development of a mid-height and column head hinges could and did occur due to contact with Spandrel Panels. The difficulty was only in knowing what level of gap actually existed at any particular column and Spandrel panel as no information was available on the as-built gaps. No specific Spandrel Panel to column gap was specified on the drawings, only an end to end spacing of Spandrel Panels (BCR Fig 8 and 179). The failure pattern observed in the columns seen in BCR Figure 101 shows that Spandrel Panels were able to force failure in the columns above the Spandrel Panels. Calculation of Spandrel Panel restraint effects is indeterminate to some extent. Gaps may have been compromised at both ends of multiple panels allowing direct bearing of columns on panels. However 2D frame analysis at drifts of 0.5% and gaps of 3 mm found there was sufficient increase of bending at column heads to cause overload. The level of restraint required at the columns from the Spandrels was between 125 and 150 kN at Levels 3 and 4 respectively. There were 4 TCM 20 fixing the panels to the floor (BCR Fig 8) even if only two of these were fully effective the restraint and ignoring direct bearing to the adjacent columns there was a minimum of was 233 kN resistance available. This shows that there was more than

		enough restraint available in the connections to allow overload from short column effects caused by Spandrel Panel contact at low levels of drift along Line F. The inside faces of the panels was timber framed and lined. There was no need to ensure hinging developed at the top of the panels for Scenario 1 due the possibility of a deep hinge forming at the head. But this could not be ruled out as having occurred at the tips of the compression splices as previously noted.
.16para2	Exceptionally intense ground motion	The intense ground motion did not translate into damage in the September Earthquake indicating that the building did not respond perfectly to the ground motion and the resulting demand levels on structural components were not close to code level demands. A design earthquake is not specified as being required to be resisted by a structure rather that it satisfies specific response loads with a specified return period or with 10% probability of exceedance in 50 years.
.16para4	Elastic Response Spectra Analysis: what is reason for comparative inter-storey drifts in Appendix F?	These show the comparative drift levels in a quantitative way that could not be derived from the spectra themselves. They also show the effect of the masonry infill on the relative drift levels to north south and east west seismic demands. For east-west attack the drifts on Line F are significantly greater when the masonry infill wall is affecting response. This is with the use of the partially stiffened properties used in the ERSA agreed with StructureSmith. Comparative demands on Line F, 1 and 2 could also be derived by reference to it.
.16para16	NLTHA: the input ground motion is the greatest unknown. The P695 studies in the US have documented significant difference in response of structures to different earthquake records even when scaled to the same intensity. The same studies also estimated uncertainties in predicting collapse from other sources including modelling	The difference in response to different records scaled to same base accelerations is well known. In addition in this case the NTHA was using ground motion records, and the erroneous assumption was made that these would lead to accurate prediction of collapse without calibration to the observed damage. WH acknowledges the effect of modelling assumptions on ability of models to predict collapse.
.16para6	Vertical ground motions	Vertical ground motion effects were difficult to quantify. The issue of calibration of the response applies to the vertical effects as much to the horizontal however there is nothing to calibrate these to. Reports of vertical jolts were made by Cammock, Kulpe, Horsely and Godkin. They were significant enough to almost raise people from their seat or feet and cause filing cabinets to bounce around. However they also appear to have been distinct motions separate to the large horizontal movements. Though Stephen Gill reported being thrown up and down

		and sideways on the roof of Les Mills and Godkin reported seeing the floor undulating from the South Wall as the building collapsed. There were reports of surface undulations on the ground coming from the southeast prior to the collapse occurring. The exact quantification of the effect however is beyond analysis methods at this point of time without there having been motion recorders or a "black box" in the building at the time to back analyse.
.17para4	Most important seismic deficiencies were the brittle gravity frames and poor diaphragm particularly the connections to the North Core	This is agreed though the diaphragms themselves were robust and thick. However the connections to the North Core were of limited capacity. However they do not appear to have caused the collapse but would have if collapse had not occurred prior to them being overloaded.
.17para6	Should have used the requirements for limited ductility	Agreed that the Ch 14 requirements for Group 2 columns would have been required as discussed earlier in this review. Recommendation is good for other buildings of the time be reviewed. The issue of precast log and shell beams is also good.
.18 2	Use of thin toppings, collector design and diaphragm forces	Thickness not relevant to CTV with thick topping; collector design recommendation good; also determination for diaphragm design forces.
.18 3	Infill walls and panels lacked adequate separation	Agreed
.18	Recommendations for better drift assessment and modelling and limits	Agreed

Comments on Statement of Evidence WIT.PRIESTLEY.0001

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WIT Ref	Issue	Clark Hyland's Comments
C.9	Issues of disagreement between Nigel Priestley and the Collapse report were identification of critical columns, the influence of the spandrels and when in the collapse sequence the separation of the floor slabs and the North Core occurred	I don't know which other Panel members disagreed on these matters.
C.10	Divergence with Panel Report	It is not clear what those areas of divergence were but my understanding was that the Panel agreed with the conclusion of the Collapse Report that Scenario 1 was the most likely collapse Scenario. This was what was presented by the Department during the public release of the reports that Nigel Priestley attended. Scenario 1 involved Line F column failure initiation with reduced drift capacity due to short column effects from contact with Spandrel Panels, and possible low concrete strength and vertical accelerations. Scenario 2 was the next most likely being failure of an interior column on Line 2 or 3 with low concrete strength and vertical acceleration effects. Nigel Priestley has shown he prefers Scenario 4 requiring failure of the Diaphragm slabs adjacent to the North Core followed by internal column failure.
C.14	Nigel Priestley agrees with much in the two reports particularly the Panel Report	The Panel Report ("PR") draws on the Building Collapse Report ("BCR") with some modifications by Panel members. Nigel Priestley was Vice Chair of the Panel, the chairman Sherwyn Williams was a non-technical chair. Nigel Priestley's approval was apparently required for the reports to be accepted by the Department. No separate investigation was undertaken by the Panel. I understand that the DBH Project Manager David Hopkins largely drafted the Panel Report.
C.15	Extent of contribution of each contributing factor to the collapse	It is not clear what he means by extents.
C.16.a	Undue and inappropriate reliance on ERSA for assessing performance of CTV building in Sep 11 and Feb 12	Conclusions of the collapse were based on a combination of eyewitness testimony, physical debris, study of collapse debris photos, physical testing and various analytical techniques. Analytical techniques included use of various computer modelling software packages and engineering calculations. Analyses using Cumbia and Nonlinear Pushover Analysis ("NPA") found the column drift capacity compared to

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		code drift demands were low. The NTHA found that drifts along Line F of 1 % occurred simultaneous with Drag Bar failure at Level 4. A witness at Level 4 Lifts at the time of the collapse reported collapse initiating at the south end of the building with vertical floor undulations, prior to a violent west-east lurch. This indicates that the collapse occurred before full seismic response demands were placed on the structure. It also indicates that collapse occurred with some capacity reducing effects such as contact with Spandrel panels, low concrete strength and vertical accelerations. Comparison of column drift capacities to Code drift demands required use of ERSA as prescribed by the Standards. ERSA was therefore used for code compliance checks, comparison of relative magnitude of elastic response of the three events, and interrogation of NTHA results. It was the first method of analysis to use in the investigation due to its speed and common use amongst consulting engineers. The model was developed by Compusoft staff seconded to StructureSmith so that the NTHA model could be developed easily from it using SAP. NTHA was also planned to be used and initial development occurred early. However this was put on hold after initial development had occurred and the focus was put on getting the ERSA model correct. The ERSA results clearly indicated problems with the compliance of the columns and capacity of the Drag Bars relative to the requirements of the design standards. Failure of the Drag Bars was found to be close to development of flexural yield in the South Wall. Suitable means for quantifying drift capacity were explored. It was recognised that shear demands on the column heads would exceed code limits when the Group 2 frames were subjected to prescribed displacements derived from the ERSA. However a collapse mechanism could not be directly justified on that basis as the columns were restrained top and bottom by the floor slab diaphragms. Nigel Priestley suggested use of Cumbia software developed by Kowalsky in the USA. This proved to be very useful. It was recognised that the accurate assessment of shear capacity of concrete columns subject to combined axial compression, and flexure is still able to be accurately determined with current knowledge of concrete behaviour. So use of the limiting compression strain of 0.004 was agreed with the DBH Panel in line with the NZSEE guidelines. An alternative NPA method of assessing the column drift capacities was also developed by StructureSmith which gave reasonable agreement to the Cumbia results. All computer based analytical techniques must be interpreted using engineering judgement accounting for the limitations of the method, the modelling and the inputs, and the level of calibration to actual behaviour. The limitations of ERSA were understood. ERSA identified the relative weakness of the South Wall, the weakness of the Drag bars and the effects of the Line A masonry infill with and without interaction with the structure. Comparative peak drift levels were also

		compared at various points at critical drifts identified by the column drift capacity analyses to help prioritise relative collapse scenarios. 3D equivalent static analyses and hand calculations were also used to improve insight into behaviour of the structure.
C.16.b	Misleading information about the design seismic intensity for the CTV Building	No attempt was made to mislead. Design seismic intensity is not a term used in the design standards. Design spectra were applied in accordance with the Standards of the day.
C.16.c	Excessive emphasis on the role of Spandrel Panels on the east and south faces of the CTV Building initiating failure	Spandrel Panels were found by analyses to induce short column effects and increase flexural and shear demands on the column heads depending on the amount of gap between them and the columns and the level of inter-storey drift that occurred. Equivalent static drift displacement analyses of 2-D frames using cracked section properties estimated from Cumbia analyses, with prescribed drifts with varying Spandrel Panel gaps showed this. In conjunction with the low levels of spiral reinforcing below minimum shear reinforcing levels at the tips of the columns vertical reinforcing steel splices a vulnerability to development of mid-height hinging or failure was also found to be a realistic explanation for the failure damage observed such as in Fig 106 (refer Park & Paulay 1975 p.421-422). Development of a mid-height hinge in conjunction with the base and head hinging would have led to sudden loss of vertical load carrying capacity in perimeter columns and shedding of load onto adjacent columns leading to their overload and progression of the collapse. The physical evidence of columns fractured close to the termination zones of the vertical column reinforcement indicates this occurred. Due to the very low levels of spiral confining reinforcement in the columns it was also considered by the Panel that at the attainment of a concrete compression strain of 0.004, used in the Cumbia and NPA drift capacity analyses, the columns would also have suffered loss of load carrying capacity. The column configuration was apparently worse than any of those reported by Nigel Priestley and used as the basis of the NZSEE guidelines. The column failure shown in BCR Figure 101 shows that development of a mid-height and column head hinges could and did occur due to contact with Spandrel Panels. The difficulty was only in knowing what level of gap actually existed at any particular column and Spandrel panel as no information was available on the as-built gaps. No specific Spandrel Panel to column gap was specified on the drawings only and end to end spacing of Spandrel Panels (BCR Fig 8 and 179).
C.16.d	Exterior columns were more likely than interior columns to	The design level drifts were found to be highest on the perimeter columns along Line 1 and F, particularly when account was made for the stiffening effect of the masonry infill on the Line A or west face. Initiation by failure of

	initiate failure	central columns such as considered in Scenario 2 tended to rely on lower than specified concrete strengths occurring in low level columns and concurrent vertical acceleration effects at peak drifts. The probabilities of these occurring was lower than those of east face columns failing. However this could not be totally discounted. In the initial stages of the investigation a low concrete strength central column failure initiation was considered the most likely scenario. However a number of eyewitnesses reported that they observed the upper levels of the building collapsing as a unit onto the floors below. Eyewitness 6 Dennis May who was aligned with the east face of the building reported a slight tilt to the east of the upper levels, then the upper levels falling straight down (BCR Fig 60). There was evidence in the photos taken immediately after the collapse that the there was an eastward throw of the debris. These observations are best explained by initiation in the failure of upper level east face columns. It was also found to be consistent with analysis results. It is also interesting to consider that the only column damage reported found by anyone after the September Earthquake in columns from Level 1 to 5 occurred at F4 at Level 4. A crack was photographed just above the Spandrel Panel. Does this indicate that the Spandrel Panel had interfered with the column's north-south displacement in the September Earthquake at this location? Was it a precursor of what was to happen at that level in the February Aftershock?
C.16.e	Rejection of connection failure between the floor diaphragms and the North Core as a high-probability failure initiator	The scenario proposed by Nigel Priestley was called Scenario 4 in the Collapse Report. Study of the collapse evidence photos, site examination and witness testimony however found that the floor slabs had remained attached to the North Core until after collapse had developed at the south end of the building. Photos showed the slabs that had been attached to the North Core leaning down to the ground indicating that the column supports to the floor on Line 3 had collapsed prior to the slabs collapsing and then detaching from the North Core. If they had detached from the North Core prior to Line 3 collapsing then the slabs would not be expected to be laying in such a way against the North Core. Inspections of the North Core on two occasions found that the threaded anchor bolts that had connected the Drag Bars to the slabs were still upright behind the tip of the lift well wall. This indicated that the slab had rotated downwards about the tip of the walls of the North Core after it lost support on Line 4, effectively prying the slab off the bolts. This meant that disconnection of the floor slab diaphragm from the Drag Bars had not occurred prior to column collapse initiating elsewhere in the CTV Building. Photos taken immediately after the collapse showed that the Level 6 slab remained supported by the Drag Bars despite loss of supported on Line 4 from the collapse of column C18. The Level 5 slab could be seen rotated

downwards onto Level 4 where two eyewitnesses had been standing during the collapse (BCR Fig 164). One of the Level 4 witnesses located in front of the lifts at the time, saw collapse initiating at the south end of the CTV Building. Another next to him recollected partitions separating along the top surfaces to the underside of the ceiling immediately before the collapse.

Calculations using internationally recognised limit states methods and measured material properties from the CTV Building Site Examination and Materials Testing Report ("SEMT Report") found that the critical limit state for the Drag Bars was shear failure of the threaded studs. The studs were found not to have sheared however. This could be seen from photos of the Drag Bars after the collapse prior to then being removed during the recovery operations and during the two inspections I made of the North Core (refer letter to Royal Commission of 7 September 2011 and Fig 163).

Assessment of the failure limit states of the floor diaphragm adjacent to the North Core in BCR Appendix G found that the weakest failure limit state was at the Drag Bars rather than in the slab at the ends of the H12 saddle bars for in plane floor bending and tension.

Out of plane fracture of the slab at the ends of the H12 saddle bars without loss of vertical support along Line 3 would have required very large northward rotations to have occurred at the North Core. However the northward set of the North Core found in the Site Examination was not compatible with that level of rotation. Yielding of the walls in the north-south direction was found by calculation to precede uplift of the foundations.

The sets or leans measured on the North Core are shown in Fig 57 of the SEMT Report. The displacements are similar at the west and east end of the North Core at RL 6040 mm, which is just below L3. This may indicate a stiffening effect of the masonry infill on the west face which came up to the underside of Level 4. Between Level 3 and Level 5 and from Level 5 to Level 7 the lean is 1.6 times greater at the east side of the North Core.

Based on 62 mm set at RL 12320 (L5 13374) and 33 mm at RL 6040 (L3 at 6890mm), this would place a limit on the inelastic inter-storey drifts at the North Core of approximately 0.45%. Review of the NPA pushover curves in Fig 124 indicates that the elastic component of drift could have been a similar amount resulting in total combined elastic and inelastic drift along Line F compatible with 1% derived from the NTHA. The RL=0 was taken on the kerb across Madras street from the site.

The rotation and set on the North Core may also have been partly caused by the southwards collapse of the South Wall onto the collapsed structure and by a north and eastwards push as the building collapsed.

It therefore appears that the rotation demands in the Hibond slab adjacent to Line 4 would therefore have been well below what would be required to cause fracture of the Hibond floor slab. the slab was much stronger when

		southwards displacements were imposed on the structure due to the presence of the Hibond decking. In summary the failure mechanism proposed by Nigel Priestley described as Scenario 4 in the BCR was checked through but was not found to be viable at the drifts found to have been sufficient to cause collapse initiated along Line F in Scenario 1.
C.16.f	Excessive emphasis on torsional eccentricity based on ERSA findings	The effect of the masonry infill needed to be seriously considered after statements from Leonard Fortune who had been working on the face of the infill masonry wall on the two days immediately preceding and at the time of the collapse (BCR Fig 149 and 150). A review of the collapse evidence photos (BCR p.239-241) also indicated that contact had occurred. The observation by witnesses of failure above Level 4 where the west wall terminated was also indicative of some change in response at that level caused by the west wall interaction. The wall was modelled in ERSA with partial interaction using a propped cantilever model of the three panels (BCR p.240-241). This was found to be sufficient to cause a major shift of the centre of stiffness and increase in torsional response of the structure in ERSA. If the panels had been modelled to act monolithically their stiffness would have been even higher and would have increased the torsional response further. The design shear capacity of the 2.3 m masonry panels if constrained sufficiently to develop their shear capacity was calculated to be 220 kN for Grade B masonry 140 mm thick all cells filled,D12 @ 600 cc EW, according to the masonry structures standard NZS 4230:2004. This is twice the capacity used as the upper bound in the NTHA model. The upper bound for shear demand on the masonry was 402 kN using vgmax criteria in the same standard. This is 4 times the capacity used as the upper bound in the NTHA model. The NTHA model for the masonry infill used a similar approach, however it used a much lower level of maximum stiffness based on cantilever flexural yielding limiting the stiffness, then with degrading stiffness such that at 20 mm inter-storey displacement the capacity was a maximum of 100 kN shear / 2.3 m wide panel to zero at 35 mm displacement (BRC cl.10 p.203). The input properties for the NTHA model were made based on advice received from Nigel Priestley. This meant that the masonry infill stiffness was significantly less than that if the wall was constrained by the boundary elements to develop its shear capacity. It also meant that at the critical drifts of 1% or 32.5 mm at which the columns were found to be susceptible to collapse the masonry wall in the NTHA would have had little stiffness and effect on torsional response. The NTHA model with masonry infill did not adequately allow for the possible level of constraint of the structure by the masonry that may have occurred In Padang Indonesia in the aftermath of the 2009 earthquake there, as a volunteer with NZAID I observed many heavily damaged reinforced concrete frame buildings with unreinforced masonry infill panels. Where these panels were constrained by reinforced concrete boundary elements it appeared that what would have been otherwise weak brick infill material with a cement plaster covering could, via compression field behaviour, provide relatively

		high levels of shear resistance to portions of structures. They tended to exhibit dominant diagonal cracking across the panels from corner to corner indicative of diagonally acting shear or tension orthogonal to the diagonal crack (Hyland Wijanto NZSEE Bulletin 2010). The low level of shear stiffness and capacity with degradation used in the NTHA model would explain the Nigel Priestley observation that the effect of the wall on torsional response in the NTHA was not as significant as seen in the ERSA. Whereas in fact it appears to have been an outcome of his modelling input requirements for the infill wall. Observations after the collapse showed that the wall had in fact failed in diagonal shear not in flexure as postulated by Nigel Priestley. This showed that the wall had developed its full shear capacity prior to collapse rather than flexural yielding behaviour (BCR Fig 153 and 154). Compression field behaviour is more likely to have occurred making the wall even stiffer than that used in the ERSA. Only small amounts of mortar infilling the gaps would have been necessary for this to occur due to the constraint to the panels provided by the beam and column boundary elements to the infill masonry.
C.16.h	Reluctance to accept the results of the NTHA where these did not agree with the consultants view of the collapse sequence.	The NTHA could not be treated with the same level of credibility as the physical collapse evidence, testing results and eye witness testimonies. Its reliability was dependant on the input records and modelling assumptions used, and the level of calibration it had with observed damage. It has to be interpreted with engineering judgement and is n approximation to reality at best. The modelling of the masonry infill wall appears in the NTHA does not appear to have been consistent with observed performance. The use of earthquake ground motion records which by necessity were at some distance from the site meant that the actual ground motion of the site could not be guaranteed to match any of the records exactly. The response of the structure to the ground motion could also not be guaranteed to be exact. Some level of calibration to observed behaviour of the real structure was necessary but could not be agreeably achieved. It is not acceptable scientific practice to rely on the output of a model such as this without carefully calibrating its performance to physical observations of damage. My experience with calibration of computer software to real performance of structural components is that calibration is always essential. A meeting was held in Auckland with the DBH Project Manager, David Hopkins, the Panel Chair Sherwyn Williams and Ashley Smith to discuss the calibration of the NTHA over three days. During that time it became the collective view that there was wide range of calibration possible for the NTHA results. David Hopkins was to communicate the findings to Nigel Priestley. The NTHA did not include a memory of deformation damage caused by previous cycles due to the need to reduce

		computation times. This meant that the stiffness of each element was restored to 100% on each unloading cycle. This is another aspect that requires some care when interpreting the NTHA results. The inability to reliably calibrate the NTHA meant that it was not possible to reliably use the NTHA results, to determine the sequence of failures and magnitude of total drifts and actions imposed on the structure. It was however useful in an informative and subjective sense.
C.16.i	The methodology for the 'Displacement Compatibility " Analysis used in Appendix F	The displacement compatibility analysis was required to check compliance with the Standards and used the ERSA analysis methods prescribed by the standards. A generous approach was taken in terms of checking the drifts at which elastic behaviour would cease in the columns and requirements additional to those required for non-seismic loading were required to be applied when analysing the Group 2 frames. Normal design practice would have been more severe.
C.18	ERSA not suitable tool for analysing structures	Many tools can be used to assess structural behaviour including simple hand calculations. Experienced engineers know the limitations of these methods and use them appropriately. The ERSA was used and interpreted appropriately in the investigation. It was used first as described previously to identify compliance with codes and identify weak aspects of the structure with a view to use of NTHA if necessary to refine findings. Use of eyewitness and collapse debris evidence was essential to determine scenarios of the collapse. Computational modelling is necessarily reliant on calibration to actual observed behaviour, or prescribed code limits as previously discussed. Once calibrated much additional information can be gleaned from computer models. Comparison of performance with code expectations requires use of ERSA. Use of ERSA is a common and an acceptable means of analysing structures in design standards around the world. Development of the application of the method continues by international researchers. International experts such as Fajfar recognise it as an analytical used by design practitioners and are developing improved ways to utilise ERSA utilising increased knowledge of inelastic behaviour determined from academic research. Refer to his abstract of his paper at the recent International earthquake engineering conference ICEEDM 2011 describing developments in the Eurocode 8 N2 method: "The N2 method is a pushover-based method which has been implemented in the European standard Eurocode 8 for nonlinear seismic analysis. It is, like other original pushover-based methods, based on the assumption that the structure vibrates predominantly in a single mode. This assumption is not always fulfilled, especially in the case of high-rise buildings and/or torsionally flexible plan-asymmetric buildings. The extended N2 method has been developed, which takes into account higher mode effects both in plan and in elevation. The extension is based on

		the assumption that the structure remains in the elastic range when vibrating in higher modes. The seismic demand in terms of displacements and storey drifts can be obtained by combining the results of basic pushover analysis and those of standard elastic modal analysis. In the paper, the extended N2 method is summarized and applied to a test example which represents an actual 8-storey reinforced concrete building."
C.19	Utilisation of NTHA in the investigation	NTHA was initially planned to occur following ERSA, subject to the findings. The modelling had been started early in the process however the cost and time consumed in it was seen to be excessive so with the approval of the project manager the NTHA modelling was halted until the ERSA could be completed. The ERSA results were compelling that column detailing deficiencies meant column heads would have been overstressed by design level actions. However no clear mechanism of the collapse compatible with witness observations was obvious. Spandrel Panel contact and torsional irregularity at Level 4 due to the west wall infill indicated a possible explanation It was also not clear how to accurately model the drift capacity of the columns and effects of spandrel panels. Nigel Priestley suggested use of Cumbia software. This proved to be a very useful approach. The NTHA development was restarted by StructureSmith at the request of the project manager while I was overseas in Indonesia and out of effective contact for two weeks. It then progressed as a DBH Panel led analysis with significant direction from Nigel Priestley and Rob Jury. It became a point of polarisation that led to many contentions up to the end of the project. NTHA analyses for assessment of structures are usually scaled. For example according to NZS4203:1992, "at least three different earthquake records of acceleration versus time" and "scaling shall be such that over the period range of interest for the structure being analysed, the 5% damped spectrum of the earthquake record does not differ significantly from the design spectrum for the limit state being considered". In doing this the ground motion record is calibrated to the design response spectra which itself has been based on a process of calibration assessment in the development of the loadings standard. This process of moderation or calibration of earthquake spectra in hazard models A moderated design spectra was developed. This can perhaps be seen in the difference between the GNS hazard model described by Stirling in 2000 at the World Conference of Earthquake Engineering and the eventual spectra used in earthquake loadings standard NZS 1170.5:2005. Stirling's paper describes 1 in 500 spectral accelerations for the CBD close to those recorded in the February Aftershock, whereas the design spectra accelerations in the loadings standard are lower. No scaling of the ground motion record was used in the CTV Building NTHA. In this case the scaling would have needed to have been done based on assessment of damage predicted by the model and the damage observed. No agreement could be achieved on calibration of the NTHA so it was agreed with David Hopkins in the interest of completing the report for Panel sign off to set the record as a benchmark 100% and ensure interpretation of the

		results recognised this limitation.
C.20	Suitability of ERSA for assessing seismic performance of buildings (NZSEE Guidelines)	Elastic analysis techniques are the basis of most structural analysis around the world. New Zealand and International standards recognise use of ERSA as a useful design analysis method when used in conjunction with capacity design or ductile detailing principles and maintenance of compatible levels of inelastic demand in the primary seismic resisting system, this is achieved through application of structural symmetry and drift limitations. Greater emphasis of setting and checking of drift limits has occurred in more recent standards. These limitations are discussed in the BCR p.234 to 236. The recommendations of the NZSEE study group reported in the Assessment and Improvement of the Structural Performance of Buildings in Earthquakes published in 2006 ("NZSEE Guidelines") section 4.3.2 are focussed on the assessment of pre-1975 buildings that tended to exhibit low levels of ductile design and reinforcing. The concepts of ductile design and use of computational software such as ERSA and THA only gained momentum in New Zealand design standards after the large earthquakes in California in 1971. Application of the recommendation to a 1986 building required to be designed using ERSA and detailed for ductility and capacity design therefore appears to be out of context. The NZSEE Guidelines do not have the same status as a Standard and have not been through the rigorous development and review process of a New Zealand Standard. Inelastic analysis methods are gaining more credibility but are still not widely used due to their expense, the high level of computational power and time required and lack of universal agreement on input parameters and their calibration between different analysts. They can imply a level of output accuracy that is not justified by the level of accuracy of the input and modelling assumptions. There is a lack of clear and tested guidance on methods of benchmarking modelling to ensure reliable performance between one model and another and real structure behaviour.
C.21	Limitations of ERSA	The limitations of ERSA are recognised. However the torsional response of the NTHA model was affected by modelling assumptions which do not appear to have been consistent with the observed performance. It is noted last para p.235 of the BCR that ERSA was only used for the following purposes: 1. Compliance checks consistent with NZS 4203:1984. 2. Comparative assessment of the level of demand implied by the earthquake spectra records compared to design

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		spectra. 3. Distribution of demand prior to inelastic deformation developing. NTHA was used to investigate inelastic performance but was limited due to inability to calibrate it successfully to the damage observed in the September Earthquake and the February Aftershock.
C.23	Principal Lateral force resisting elements	Both the North Core and the South Wall acted together as primary or principal lateral force resisting system tied together by the floor slab diaphragms. In the east- west direction bases shears on the North Core and South Wall were similar prior to development of inelastic demand. With eccentricity applied 10% south of the centre of mass as required by the Standards, the east west shear demand on the South Wall was greater than on the North Core due to combined direct and torsional shears.
C.25	Assessment of peak floor diaphragm inertia actions	ERSA gives peak shears in wall elements for a given spectra. The difference in shears above and below a diaphragm using ERSA requires consideration of each mode of response. Current versions of some ERSA software do these calculations. Traditionally design engineers have reviewed static analysis results and compared these to the ERSA results to assess how shears varied then used an enveloping approach to capture upper bound actions. Derek Bradley advised that ETABS software used for the ERSA (email 29/5/12): "It will give the maximum difference across the cuts that occur. It is difficult to find specific references to what they do as CSi have not documented everything fully. I found something on section cuts that may be of assistance to you. https://wiki.csiberkeley.com/display/kb/Section+cut" Assessment of the simultaneous demand on elements during an earthquake is difficult and problematic in NTHA also due to the level of calibration required to input records and the modelling assumptions necessary for practical applications. The NTHA showed that the capacity of the Drag Bars connections to the North Core were indicated to be exceeded simultaneous with drifts of 1% along Line F. The relationship between the drift along Line F and attainment of Drag Bar connection capacity appears to be largely a geometrical relationship, which is what would be expected, and therefore not affected by the magnitude of the loading. This is illustrated by the similar levels of drifts along Line F simultaneous with Drag Bar connection failure that were found to occur in NTHA runs using the September Earthquake and the February Aftershock records. The response of the CTV building February Aftershock, based on the comparative ERSA analyses (BCR

		p.259) was found to be approximately 2.2 times greater than it was to the September Earthquake, but the Drag Bar disconnection drifts were similar.
C.28	Engagement of Compusoft for Computer Modelling	The ERSA model was developed deliberately to allow ease of transfer into SAP to run the NTHA. The results from the ERSA indicated Drag Bar and South Wall weaknesses. The difficulty of determining drift capacities of the columns was not initially resolved, however Cumbia was found to help this. The exact effect of Spandrel Panel contact and vertical acceleration effects was also not able to be resolved. However it was generally apparent that Scenarios 1 and 2 were the most likely collapse scenarios. A meeting of the Panel as a whole when Nigel Priestley was absent did not find the NTHA results critical to the conclusions and debate was over whether the additional analysis effort was necessary as it was taking much longer than expected and was holding up release of the report.
C.29	Advantages of NTHA	These are recognised.
C.30	Limitations of NTHA	These are also recognised. The model did not model accumulated stiffness degradation due to excessive computing time required. It could not model large non-linearity effects such as occur after initiation of collapse in one location. No calibration of the model to damage observed was undertaken. Input records were from seismographs over 750 m from the site. Modelling of the masonry infill contact effects do not appear to have been consistent with observed damage. No modelling of the effect of Spandrel Panel contact was attempted due to modelling complication. No effect of gross separation of the diaphragms from the North Core could be made. For example immediately following Drag Bar disconnection the remaining connection of the floor slab to the North Core at the amenities area should have become overloaded and would have failed resulting in collapse of the floor slab at that location. However the model was set to ignore that consequence and continue running after Drag Bar disconnection for practical computing reasons.
C.31	Reliance placed on NTHA results	The NTHA results were informative but could not be relied upon for determining sequence of failure and magnitude of drifts and other actions. ERSA was used for Code compliance checks and assessing the comparative magnitude of response of the CTV Building to the three events. Primary conclusions on collapse were drawn from the column displacement compatibility analyses using Cumbia and NPA. Assessment of drifts at Drag Bar failure on Line F utilised NTHA as this was considered to be a result independent of the magnitude of the input records (Refer discussion in C.25). Assessment of diaphragm section analysis limit states and equivalent static analysis to estimate comparative drift demands in the structure under

		seismic loadings. The collapse Scenario 4 was developed based on the NTHA results. More refinement occurred to all the scenarios as time went on. However Scenario 4 was not found to be as likely to have occurred as Scenario 1 or 2 for the reasons previously discussed.
C.32	Points of greatest divergence and concern	The conclusions of the most favoured collapse Scenarios 1 and 2 do not rely particularly much on either ERSA or NTHA results except as previously stated.
C.33	Validity of using NZS 4203:1984 design spectra	Code compliance checks had of necessity to be undertaken using NZS 4203:1984 spectra
C.35	Design Spectra in BCR and Panel Reports	BCR Fig 11 and 145 show scaled NZS 4203:1984 spectra scaled by 1.48 to give equivalent of 90% static base shear for T=1.03 s for the east-west response of the CTV structure with North Core and South Wall. It is not clear what is shown in Figure 4.10 of the Panel Report. However my understanding was that it was simplified to show schematically the relative scale of things only for non-engineers.
C.36	Panel Report Fig 4.10	As above
C.38	N/S ERSA Scaling factor	The north-south scaling factor of the design spectra was 1.14
C.39	Application of scaling factor to design spectra	Application of a scaling factor to the input spectra is normal procedure in ERSA software such as ETABS. It is normal to scale the input spectra to determine design actions at varying levels of ductile response for the same design spectra.
C.40	Comparison with earthquake records	The design spectra were correctly scaled.
C.41	Variations in records between Fig 11 and Panel Fig 4.10	The 4 sites considered for the NTHA were determined prior to knowledge of the closer Police Station and Westpac sites having data available. The CHHC CCCC and CBGS were used in the NTHA . REHS was however considered by the geotechnical consultant Tonkin & Taylor to be inappropriate for the CTV Building due its surface geology being significantly different (BCR p.274). It was decided no change would be made to the input record sites used for the NTHA , when it became known that the Police Station and Westpac records were available. This was due to time pressures already on getting the NTHA results. However for the development of the ERSA spectra the closest sites to the CTV Building were used.

		This is because the closer the site the more closely the records would likely correspond to what was felt at the CTV Building site. The ERSA spectra developed were only used to compare relative magnitude of elastic response of the CTV Building to each of the three events considered (BCR p.259-261). The Panel Report fig 4.10 uses the term "principal directions" for the spectra. However what is shown is in fact only the largest of the two orthogonal components ie North-South or east –west measured at the sites. This is not strictly the principal direction of response. The orientation of seismographs varied at each site. The relative magnitude of the records in each direction at each site varied and also varied between sites. The term "principal direction" which is normally associated with the direction of the greatest magnitude is unlikely to have been in either of the directions measured at the sites. The BCR Figure 11 spectra were therefore developed using the resultant of the two orthogonal spectral accelerations recorded at each site and averaged between sites to get a better estimate of the maximum spectral accelerations measured.
C.42	Method of determining the averaged spectra and application	Reason for using different sites for ERSA comparative assessment is discussed in C.41 above. Police, Westpac, CHHC and CCCC records were used to develop the averaged spectra. CBGS and REHS were excluded as they were free field spectra. The free field spectra appeared to have higher response in the lower period range. This may be due to less kinematic rocking of the structures housing the seismographs according to discussions with John Zhao at GNS in early September 2011. The larger building seismograph records tend to reduce low period responses. It may also be the case that the free field instruments were more affected by surface waves that occurred at longer periods and hence have a more enhanced bump in the spectra at 3.5 seconds than the seismographs located in buildings. The Police building had natural period of 1.0 s close to that of CTV Building. CHHC and CCCC likely to be less than 1 s. Westpac first mode period is not known but probably greater than 1 s. The spectra derived are the average of the resultant of the orthogonal responses at each period for the sites (BCR p.233). The spectra derived were applied to the CTV Building ERSA in the same manner as the design spectra. No combination of EW and NS loading was attempted as the purpose was only to identify relative magnitude of response of the structure in each direction to the three events considered.

C.43	Spandrel Panels effects	The development of collapse scenarios and mechanisms towards the final report was a developmental process and a number of theories were considered and refined or discarded in the process. Regular reporting was made to the Panel throughout the process and adjustments made based on feedback received. Spandrel Panels contact with columns causing a short-column effect appears to have been a contributing factor to the collapse. It has been shown that interference from the Spandrels would reduce the drifts at which column head failure would occur. The lack of adequate spiral confining reinforcing steel at the termination of the vertical laps in the column reinforcing is also seen to be a vulnerability that may have lead to compressive flexural failure initiating at the locations observed in some of the columns (BCR p.111 and Fig 106) . That the Spandrel panels had sufficient capacity to force column failure above the Spandrel panels is demonstrated by the column failure seen in BCR Fig 101. The level of gap between any particular Spandrel Panel and a column is not known however and no specific seismic separation gap was specified. Bill Jones who was involved with the construction when interviewed indicated that the Spandrel Panel ends were likely to have been lined up for visual effect rather than for any seismic separation.
C.44	Spandrel Panel Effects	Spandrel Panel interaction is considered to have been a contributor to the collapse. However the columns had drift capacities that were low. Drifts sufficient to cause their collapse at Levels 3 and 4 were estimated to be as low as 1.15 to 1.25%. Concrete strengths appear to have been low in some cases based on column testing results (BCR Table 6 and Fig 118) . The effect of vertical acceleration effects and reduced concrete strengths can be assessed using BCR fig 160 and 161. Beam column joint detailing may have also contributed to a reduction in drift capacity however at the low drift levels indicated to have occurred at collapse by the Drag Bars having not disconnected, it did not appear from calculations that significant tension forces would develop in the bottom reinforcing adjacent to the column heads.
C.44.a	Specified gap between Spandrel Panels	A discussion on the gaps is on BCR p. 110 and 111. The gap specified on the drawings was between the ends of adjacent panels not between end of panel and the columns. There was no requirement shown to set out the panels at any distance from the column faces (BCR Fig 179). The Concrete Structures Standard required Group 1 elements like the Panels to be separated from the columns allowing for the seismic drifts and construction tolerances. A 0.83% drift would have required a gap of 7 mm. A 1.51% ultimate drift would have required 12.7 mm. Construction studies in the UK incorporated in BS 5606:1990 indicate that without special attention the expected as-built condition would have been between 0 and 22 mm gaps. Indicating that it was quite possible for some Spandrel Panels to have been placed hard up against a column. The columns at Level 2 to 4 were found to have been vulnerable to failure at drifts as low as 1.15 to 1.25 %

		without Spandrel Panel interference. This sets a limit on the structural performance of the CTV Building. The level of spiral reinforcing was found to be lower than that used in the development of the NZSEE Guidelines for limiting compressive strain to 0.004 (BCR p.221). The concrete used in the columns was uncrushed river gravels which may have had lower strains at peak stress than concrete using crushed aggregates. The columns may therefore have performed worse than predicted by the Cumbia analyses.
C.44.b	Spandrel Panel gaps used in analyses	Analyses showed that zero gap gave an upper bound effect of contact with the Panels. At greater gaps the effect reduced.
C.44.c	Stiffness of contact of Spandrel Panels	In developing the report analyses of various sorts were done to attempt to assess what effect various gaps would have on the frame behaviour. Columns were modelled using cracked section properties appropriate for the level of displacement. Drifts were imposed at each floor and at Spandrel Panel contact level commensurate with the gap assumed in each case by using prescribed displacements of nodes. This is a commonly used analysis technique by practicing design engineers. The results found that even with small amounts of contact the column head bending and shear actions became elevated above non-contact cases with the same inter-storey drifts.
C.44.d	Ability of Spandrel Panels to deform columns	The failure pattern observed in the columns seen in BCR Figure 101 shows that Spandrel Panels were able to force failure in the columns above the Spandrel Panels. Calculation of spandrel panel restraint effects is indeterminate to some extent. Gaps may have been compromised at both ends of multiple panels allowing direct bearing of columns on panels. However 2D frame analysis at drifts of 0.5% and gaps of 3 mm found there was sufficient increase of bending at column heads to cause overload. The level of restraint required at the columns from the Spandrels was between 125 and 150 kN at Levels 3 and 4 respectively. There were 4 TCM 20 fixing the panels to the floor (BCR Fig 8) even if only two of these were fully effective the restraint and ignoring direct bearing to the adjacent columns there was a minimum of was 233 kN resistance available. This shows that there was more than enough restraint available in the connections to allow overload from short column effects caused by Spandrel Panel contact at low levels of drift along Line F. The inside face of the Spandrel Panels were infilled with timber framing and lined There was no need to ensure hinging developed at the top of the panels. But it did occur in at least one case photographed and at the tips of the compression splices as previously noted.
C.44.e	Spandrel end diaphragm collapse	There was sufficient resistance from the two TCM20 bolts along the edge of the floor so that any flexibility out of plane of the end diaphragms of the panels could be accommodated without them losing their ability to transmit

		column restraint actions sufficient to elevate column head flexure and shears.
C.44.f	Photos of broken Spandrel Panel end diaphragms	Other photos show the diaphragm ends intact particularly those of Line F immediately after the collapse. Refer BCR fig 77 and 78. The panel in the photo Nigel Priestley refers to may have been damaged during the collapse or retrieval (refer Graeme Frost photo of strop damage to Spandrel Panels during lifting by crane).
C.44.g	Evidence of column failure at top of panel	BCR Figure 101 shows column failure at top of Panel. However the more common evidence was failure in proximity to the ends of the column compression reinforcing and at the heads and bases.
C.44.h	Failure at tops of splices	Failure at top of splices is the most common seen. The lack of sufficient confinement spiral reinforcing steel meant that initiation from combined compression and flexure could have initiated there (BCR p.111). Refer also Park and Paulay 1975 p. 421 – 423.
C.44.i	Figure 17	BCR Figure 17 shows contact initiating at the top of Spandrel Panels. Figure 18 shows schematically the development of column head and base hinges and possible column splice end hinging.
C.44.j	Line F to E load transfer	Frame and Vierendeel effects mean that axial loading could increase significantly onto the adjacent Line E columns once Line F columns lost load carrying capacity, leading to their overload ahead of columns on Line D.
C.45	Torsional response difference between ERSA and NTHA	Collapse appears to a have initiated at the south end of the building prior to a final west to east lurch based on the witness testimony of Ron Godkin who was located at the Level 4 lifts. He saw a stapler flying westwards and smashing internal office window after seeing the south floor begin to collapse and undulate. Margaret Aydon who was in an office at the north and east side of the CTV Building on Level 4 recalled being flung westward immediately before the collapse. Inertia effects mean the floor would have been accelerating sharply eastward after moving westward to cause the stapler and Margaret Aydon to be thrown westward. Refer C.16.f for discussion of the different modelling used for the wall in each analysis. The NTHA modelling of the masonry infill on the west face was much softer and allowed major degrading of the wall at drifts close to critical for the columns. This may have resulted in the response being something close to no infill at critical drifts in the NTHA models with masonry infill included.
C.46	Collapse initiator columns	Ashley Smith should answer this as it refers to the NTHA and NPA
C.47	Effect of concrete strength	The effect of low concrete strength would be more for the highly loaded interior columns, but also a factor for the east face columns as can be seen by comparing the drift capacities determined for columns with the same compression load but different concrete strengths in BCR fig 160 and 161.

C.49	Line 1 drifts compared to Line F	Line 1 columns were recognised as having similar problems with loss of column capacity as the Line F columns. However the potential for collapse to develop from those failures on Line 1 was less due to the presence of the South Wall. The South Wall would have acted to hold up the beams connected in to it even though the columns at the ends of the same beams may have been losing load carrying capacity. This was observed to have occurred by Trowsdale and Heywood. On the east face there was no such vertical load carrying element to inhibit progressive collapse development. Eye witness 6 Dennis May's testimony was that there was slight tilt to the east of the top part of the building before it came straight down (BCR p.139 Fig 60). The debris was also found to have been thrown slightly eastwards as seen in BCR Fig 1. On balance therefore Line F collapse initiation is more convincing than Line 1 collapse initiation though severe column damage may have been sustained on Line 1 at a similar time.
C.50- 51	Lateral Catenary effects to columns	The Line F collapse mechanism is described in Fig 17 to 20. It is based on progressive load transfer as elements become overloaded like a pack of cards.
C.52	Graham Frost assessment	The Collapse report collapse mechanism encapsulates Graham Frost's assessment. He was interviewed by the author early on in the investigation. He has corresponded since release to his satisfaction with the eventual favoured Scenario 1 after correspondence on the influence of the beam-column joints. The Line F initiation scenario rapidly transitions to an internal column collapse as shown in BCR Fig. 19 and 20. The difference is that the eastward throw and low drift initiation is better explained by east face initiation.
C.55	Why Drag Bars were not installed on L2 and L3	ARCL calculations for the Drag Bars indicate that they followed the "Parts and Portions" procedure in NZS 4203:1984 cl 3.4.6.3 and 3.4.9 for diaphragm design. They showed that no connection was needed at L2 and L3 except at the amenity area slab connection on Line C and C/D. This "Parts and Portions" procedure did not consider ultimate drift demands that could occur and used the structural coefficient associated with yield of the South Wall as the base load factor. Cl 3.4.6.3 required "allowance shall be made for any additional forces in such members that may result from redistribution of storey shears" but gave no guidance on what that meant or how it should be done. The Drag bars were found by the ERSA to have limited strength and unable to sustain full ULS demands of the design spectra at SM=4.0 load levels ie 5 times the design level actions required for strength.
C.58	NTHA failure sequence	The uncalibrated NTHA indicated Drag Bar failure prior to column failure. However this was on the basis of no Spandrel Panel interaction, nor reduced concrete strength and no vertical acceleration effects. The modelling of the masonry infill has previously been described as problematic and unconservative also. These factors combined with the inability to satisfactorily calibrate the NTHA model and inputs to observed damage meant that the model

		could not be used to accurately predict sequence of failure. Drag Bar failure was found not to have occurred until after the collapse had developed in the south end of the CTV Building (as described in Appendix G). The conclusion is therefore that column failure occurred prior to Drag Bar failure due to combination of Spandrel Panel interaction, possible low concrete strength, vertical acceleration and masonry infill response effects. The column drift capacities were shown to be close to those associated with Drag Bar failure of 1% determined from the NTHA. There was found to be sufficient reduction of drift capacity from the above effects to reasonably place collapse at drifts less than 1% associated with Drag Bar failure.
C.59	Drag Bar capacity	The Drag Bar capacity has been calculated using current international limit state design procedures incorporating manufacture's data and material test data from the SEMT Report.
C.60	NTHA sequence of failure	The inability to calibrate the NTHA input to observed damage and lack of modelling of Spandrel Panels, concrete strength variation and calibrated vertical acceleration means that it cannot be used to determine failure scenarios with any confidence. North South tilt behaviour necessary to fail the floor diaphragm was investigated in Scenario 4 assuming no masonry infill on the west face. However it was found that the drifts necessary to cause Line F column failure with Spandrel Panel interaction were much lower and would reasonably have preceded Scenario 4. In addition cracking of the HIbond slab at the location proposed would not have necessarily led to a collapse condition. Collapse initiated in the slab at the location proposed should have led to the slab falling away from the North Core not against the North Core as was found to have occurred (Fig 165).
C.61	Slab Fracture calculation	The forces determined from the NTHA are unreliable in terms of magnitude due to the lack of effective calibration of the model. The drifts at which collapse appears to have occurred are much less than those required to achieve the actions quoted.
C.62	Contribution of Hibond decking	The Hibond decking should be allowed for in calculations of the tensile capacity of the slab at the 1200 mm location . It acted as external reinforcing and develops strength past the critical location. It provided vertical support through this effect. It would have provided effective vertical support even with top face cracking while the slab remained supported at each end at Lines 3 and 4 beams. Hibond does not rely on chemical bond for development of its actions into the concrete slab. It relies on mechanical bond. It is normal to see Hibond decking debonded from the concrete during demolition as observed by Graham Frost. The slabs leaning against the North Core however also show that the Hibond was still in contact to those slabs (BCR Fig 165).

C.63	Torsional Eccentricity	The amount of torsional stiffness was not possible to determine exactly from the NTHA or ERSA because of limitations of the modelling, calibration and analysis procedures. However it was found that the columns on Line F could initiate collapse at drifts compatible to those needed to cause failure of the Drag Bar connections. The contribution of Spandrel Panel contact appears to have reduced the drift necessary for collapse to initiate in the columns. Northwards twist measured on the North Core above Level 3 may be indicative of a change of torsional stiffness above Level 4 from termination of the infill masonry.
C.64	NTHA torsional response	As previously noted the NTHA modelled the masonry infill as a much softer flexural yielding dominated elements rather than shear dominated. Degradation of the masonry infill modelled led to the degradation of the model to something close to a model without infill at drifts of over 1% which were critical for failure of the columns on Line F. Collapse evidence shows shear failure of infill panels occurred indicating that the assumptions used for the masonry infill properties for the NTHA were not appropriate.
C.65-69	Contact between masonry panels	The observations of the condition of the masonry wall after the September Earthquake based on witness testimony are described in detail in BCR p. 239 to 241. These were taken into account with the modelling of the masonry panels for ERSA. However the NTHA modelling appears to have used a separated cantilever approach. Leonard Fortune and Bruce Campbell testified that there were no gaps between the columns or between the panels observable from the outside face he was working on. This indicates mortar had filled the gaps on the outer surface and had not been dislodged by the September earthquake. Full contact along the horizontal joint between the precast beam and the masonry is seen on the inside in BCR Fig 151 after the September Earthquake. The Precast beam may have been installed onto the masonry wall or the masonry installed up to the underside of the beam later. Leonard Fortune describes on BCR p.239 that there were some horizontal gaps between the top courses and the undersides of the beams on the outside. On BCR p.240 it is noted that a nominal bead of masonry may have been applied to the outer face in an attempt to achieve fire rating to the boundary wall. This may have been seen as acceptable practice at the time. Alternatively the mortar tailings may have filled the gaps. The report notes that there appears to have been gaps in some places and not in others on the wall. Confinement of the masonry infill by the surrounding concrete columns and beams appear to have led to compression field behaviour in the wall, with very little shear demand on the vertical joints and the mortar infilling

		them. The ERSA model reported did not consider this as a monolithic element but used a cantilevered masonry panel but with more typical masonry properties (BCR p.241). It was therefore also a softer modelling of the masonry than what perhaps occurred.
C.70	NTHA modelled masonry as soft flexural elements	As noted the modelling of the masonry in the NTHA as flexural elements that degraded in stiffness at critical drifts was not an effective upper bound. Damage would have been expected in the wall after September if this had been the case.
C.71	Strength and stiffness of masonry in ERSA	As noted the ERSA modelling appears conservative if perhaps less stiff than what the damage indicated occurred.
C.72	Limitations of NTHA and ERSA results	The use of both the NTHA and ERSA were limited in terms of assessing absolute magnitudes of behaviour of the building in response to the earthquake. The NTHA was used to assess the drift at which Line F failure initiated on the basis that the Drag Bar provided an upper bound on drift prior to collapse. This was considered to be a geometric property largely unaffected by magnitude or scaling of input records. Displacement compatibility analysis using Cumbia showed that Spandrel Panel contact and/or possible low concrete strength and/or vertical acceleration effects likely contributed to column collapse initiation along Line F. There was no calibration of the NTHA model with the damage observed in September and February. The ERSA was limited to Code compliance checks as it was the basis of the design standard requirements. It was also used for comparing the relative elastic response of the structure to the three events considered.
C.73	Beam-column joint capacity	On BCR page 93 it was recognised that the beam-column joints were vulnerable. However, the assessment methods for joints such as those in the CTV building are less well defined and the results are likely to be more variable than for columns due to the various "factors that contributed (or may have contributed) to the failure"(refer to BCR page 1 for the list of factors). At drifts of between 0.9 and 1.3% at which interior columns may have exceeded the 0.004 concrete compressive strain criteria (Refer BCR Fig 139) 2D frame analysis showed that there didn't appear to be sufficient opening moment developed at the face of the interior columns to overcome the gravity closing moment on interior columns. The hook lengths shown on the drawings for the interior columns were close to those necessary to develop yield

		of the H28 bottom steel according to the Concrete Structures Standard so it would be reasonable to expect them to have been able to sustain some tension from opening moments if that had occurred prior to collapse initiating. On Line F the hook development was less at F2 and F3 than at the corner columns, which had similar development to the interior beam hooks. The F2 and F3 hooks may not have been able to develop tensile yielding of the bottom steel. However at drifts necessary to exceed the 0.004 concrete compressive strain criteria the amount of net opening moment at underside of the beams appeared less than that required to yield the H24 bars. It therefore seems more likely that column failure initiated before pullout of the beam hooks on Line F. However the Line F beam-column joints further increased the likelihood of collapse developing along that face.
C.74	Compusoft beam-column joint assessment	The lack of calibration and lack of Spandrel Panel interaction in the NTHA model means that it is not possible to determine from the NTHA whether the beam column joints occurred prior to column failure. At the drifts at which column failure occurred as noted in C.73 the beam columns joints are unlikely to have preceded column failure. The significant uncertainties in the NTHA modelling method and calibration meant that the column drift capacity and demands could not be reliably determined from it. Instead the Cumbia and NPA were found to be more reliable.
C.75.a	Displacement compatibility basis	The displacement capacity of the columns is developed using Cumbia/Matlab (BCR p.253-255). Code compliance checks must be done using ERSA as this is the basis for allowable drifts of the code provisions. The limitations of this approach are described in the BCR Appendix F.
C.75.b	Inter-storey drifts estimation	Inter-storey drifts were calculated using the point drifts option in ETABS this calculates the maximum inter-storey drift at each floor, which may not necessarily occur simultaneously at all levels. A check was done to determine the difference in point drifts and the difference of maximum floor displacements. This found that the drifts derived each way were identical when rounded to three decimal places along Line F.
C.75.c	Stiffness based on Codes of the day	The stiffness values for compliance checks must use the recommendations of the Codes of the day. However significant latitude was allowed within the codes for ERSA so a lower bound stiffness was used for the columns compared to what may have been used by design engineers. This would have increased the drift capacity above what would normally have been assumed at the time.
C.75.d	Beam and column frames not included in Code compliance	The Code of the day allowed the analysis of primary frames alone, excluding the secondary of Group 2 elements. The Group 2 frames were then required to be designed for displacement compatibility with the displacements of the primary frame. This was the basis of the design and therefore the basis of code compliance checks. The drifts

	checks	were to be applied and the columns and beams designed for the actions derived irrespective of whether elastic, limited ductile of ductile design requirements were necessary.
C.75.e	Compliance checks using ERSA drifts	The drifts determined using ERSA were required to be checked against elastic behaviour of the columns and satisfaction of the structural stiffness criteria by the standards of the day and hence compliance checks must use ERSA.
C.77	Best practice test	There is no requirement for a designer or builder to meet "best practice to current state of knowledge" . Such a standard cannot be determined or measured. Reasonable current practice of the day is the accepted test. It is generally accepted to be compliance with the standards of the day, which reflect the consensus of an expert committee of practitioners and academics in the field and has been subject to the internationally recognised Standards NZ process of consultation and public comment before being published.
C.79	Failure of floor diaphragm in September	No Drag Bar or partial floor diaphragm failure in the September Earthquake is likely. Failure was found not to have occurred in the larger February Aftershock. Sectional analysis in Appendix G and summarised in Table 18 showed the weakest location for in plane bending and tension failure to be at the Drag Bars connecting the slab to the walls. Only the uncalibrated NTHA indicates Drag Bar failure could have occurred in the September Earthquake. However this is because the NTHA model was not calibrated to the damage observed and the limitations of the modelling, so it over predicted damage. The ERSA comparative analyses showed that the September Earthquake caused approximately half the response of the February Aftershock. If failure did not occur in February Aftershock then failure would not have occurred in the smaller response to the September Earthquake. Drifts at collapse in February Aftershock appear to have been less than 1% based on the fact the Drag Bars did not fail prior to collapse. The same level of drift to cause Drag Bar failure was estimated by the NTHA in September earthquake indicating the calibration problems with the NTHA. Charles Clifton also undertook sectional analysis and came to the same conclusion but identified that failure could have initiated at Level 2 or 3 in the February Aftershock. This is described in the report as Scenario 3. Calibration of his analysis was also not able to be undertaken. His analysis was based on comparative diaphragm actions to input ground accelerations from a study of NTHA results by Andrew King. However it did not account for calibration of the input ground motion to actual building response. Cracking at the junction of the walls and the elevator walls was observed at Level 6 after the September

		Earthquake. However the Drag Bars at Level 6 were still fully connected to and supporting the Level 6 floor after the collapse as seen in Figure 31. One witness Genelou de Guzman reported the building on Level 4 appeared to be new and undamaged prior to the February Aftershock. Ron Godkin reported in my interview with him that the building was well known to be lively in the years prior to the September Earthquake. He understood that a bank tenant had moved out because of the liveliness. David Millar and Nilgun Kulpe reported the floor on Level 6 being lively on occasions in the meeting room when people were moving along the corridors. Floor vibration analysis showed the floor to be sensitive to footfall vibration in open plan areas. It is therefore likely that the September Earthquake and the demolition of the next door building heightened people's awareness of the movement of the building rather than this being due to Drag Bar failure.
C.80	Fracture of HRC mesh	Lino floor covering reported to be at Level 4 would have shown up large floor cracks. None were reported.
C.81	Low crack widths observed after September Earthquake	The column capacity analysis indicates that cracking of the concrete columns would have started at drifts of between 0.15 and 0.25% (BCR Fig 159). Concrete yield occurred at drifts of between 0.45 to 0.82% depending on floor level. Compression cracking, and permanent damage to the concrete would be expected to begin to be visible in the columns that had suffered drifts greater that level. No concrete cracking or damage was reported at column heads after the September Earthquake except at C18 at Level 6 which was connected into the North Core (refer David Bainbridge). It is therefore likely that drifts along Line F in September Earthquake were less than 0.45 to 0.8% at the respective floors. This is consistent with light cracking being observed in the upper Level 6 columns and no failure of the Drag Bars having occurred in the September Earthquake. The Level 6 columns however were cantilevered attachments to Level 6 and therefore would have responded to the earthquake behaviour of the Level 6 floor rather than as columns at Level 1 to 5
C.82	Causes of Collapse	The Panel agreed with the conclusions of the Collapse Report that Scenario 1 was most likely followed by Scenario 2. Nigel Priestley is proposing Scenario 4 in his evidence leading to an internal column failure as most likely.
C.83	Review of Scenario 4 rationale	It has been shown that the drifts necessary to cause column failure along Line F were less than those necessary to cause the diaphragm failure at the North Core required by Scenario 4. The collapse evidence does not support failure of the slabs at the North Corte preceding Line F column failure due to the fact the Drag bars were found to have not failed and the slabs fell against rather than away from the North Core. It has been shown that the drifts associated with Scenario 1 Line F column failure were not sufficient to cause significant opening moments and failure at the beam column joints though these were fragile. Failure of columns at the termination of the column vertical reinforcing steel is consistent with flexural compression failure being induced by Spandrel Panel contact of the columns. Spandrel panels were shown to be sufficiently strong to force failure of columns above the spandrel panels in collapse evidence photos. Only the uncalibrated NTHA indicated that diaphragm failure may occur prior to column failure. However the modelling limitations meant neither the Spandrel Panel influence, nor the effect of

		lower concrete strengths or vertical accelerations was allowed for and no calibration to the damage observed was able to be done. This made the NTHA results informative but unreliable in terms of identifying sequence of failure or magnitude of structural response.