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

FOURTH STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH
COMMENTS ON THE
PEER REVIEW REPORT BY WILLIAM T. HOLMES

DATE OF HEARING: COMMENCING 25 JUNE 2012

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INTRODUCTION

1. My name is Ashley Henry Smith. I live in Auckland. I am the director of StructureSmith Ltd, a consulting engineering company specialising in structural engineering.

QUALIFICATIONS AND EXPERIENCE

2. This is my Fourth Statement of Evidence. My qualifications and experience are outlined in my First Statement of Evidence dated 27 April 2012 [WIT.SMITH.0001.1].

EVIDENCE

3. This evidence comments on aspects of the Peer Review report dated 30 April 2012 by William T. Holmes [BUI.MAD249.0372.1] that relate to my involvement in the CTV Collapse Investigation report by Hyland Consultants and StructureSmith (the Hyland-Smith report) [BUI.MAD249.0189].

4. It should be noted that this evidence is my view. It does not necessarily reflect the view of the Department of Building and Housing (DBH), who engaged my company StructureSmith jointly with Hyland Consultants to investigate the CTV Building collapse; or the view of the DBH Expert Panel; or the view of my co-author for the CTV Collapse Investigation report Dr Clark Hyland.

5. I have read and agree to comply with the Code of Conduct for Expert Witnesses, a copy of which is attached and marked **"A"**.

6. I confirm that the matters I am giving evidence about are within my areas of expertise.

COMMENTS ON THE PEER REVIEW REPORT DATED 30 APRIL 2012 BY WILLIAM HOLMES [BUI.MAD249.0372.1]

- **GRAVITY FRAME DUCTILITY AS REQUIRED BY CODE**

7. My interpretation of 'gravity frame ductility as required by code' is given in my third statement of evidence [WIT.SMITH.0003.1].

- **FAILURE OF COLUMNS AS IT RELATES TO BUILDING COLLAPSE**

8. The last paragraph on page 7 of the William Holmes Peer Review report (herein referred to as the Peer Review report) reads as follows:
   "Estimation of the drift to fail a column involves assumptions on the limit of strain in the concrete. A value of 0.004 was assumed and this is considered to be realistic and recommended by NZSEE guidelines. However, values up to 0.007 could possibly be justified. Even at the higher strain level, the drift to cause failure would not increase in proportion for most of the lower level columns. This is because the greater part of the drift capacity was in the elastic deformation for the more heavily loaded columns, and the limited post-elastic behaviour was concentrated in the "hinges" at the top and bottom of the column".

9. Before commenting on the above extract from the Peer Review report, I would like to correct some errors in Tables 11 and 12 in Appendix D of the CTV Building Collapse Investigation report. These errors were advised to the Royal Commission by email on 29 May 2012. I have attached two versions of the corrected Tables 11 and 12. The version attached and marked **"B"** shows the changes from the original Tables 'tracked', and the version attached and marked **"C"** shows the revised Tables with all the changes 'accepted'.

10. The two bullet points below, that I received by email from Derek Bradley of Compusoft Engineering (who carried out the CTV Building non-linear seismic analysis for StructureSmith) explain the reasons for these changes as follows:

    - *Note that the drift at which concrete strain 0.004 was reached is different to that I previously gave you. The reason for that is I made an error when applying the plastic rotation limits to the columns – although the plastic rotation capacity was calculated correctly, I erroneously applied this limit from the column hinge point* *during post-processing without allowing for the plastic rotation that had already occurred between first yield and column hinging. Consequently I allowed for more rotation than the columns were capable of. This mainly affects the upper levels of the building where axial loads are lower; however it does effect the location of the maximum demand capacity ratio for column F2.*

- *In addition, I have revised the column 'first yield ' and hinge ('nominal strength') drifts so that they are represented more accurately i.e. I have used the rotation allowances for each column direction rather than an average of each column direction, and have adjusted for some rounding undertaken previously.*

11. Also note that the pink coloured header for the 'Demand' column in Table 12 should have read "<u>N/S</u> Drift Demand 22 Feb <u>CBGS</u> (%) as shown on the corrected Table 12 attached.

12. Going back to the Peer Review comments under item 8 above, I can confirm the approximate column drift capacities from the non-linear analysis for the columns at grids F2 and D2 corresponding to alternative ultimate strain criteria of 0.005 and 0.007. The corresponding percentage changes in drift capacity described in paragraphs 13 and 14 below are expected to be 'lower bound' percentage changes because the potential drop-off in stiffness and strength of the columns beyond their peak capacity was not explicitly modelled in the non-linear analysis.

13. For an ultimate concrete strain of 0.005, which represents a 25% increase over the 0.004 ultimate strain value assumed in the Hyland-Smith report, I confirm a corresponding increase in column drift capacity of approximately 20% for level 5 columns and 10% for level 1 columns.

14. Similarly, for an ultimate concrete strain of 0.007 which represents a 75% increase over the 0.004 ultimate strain value assumed in the Hyland-Smith report, I confirm a corresponding increase in column drift capacity of approximately 60% for level 5 columns and 30% for level 1 columns.

15. This may help to give an understanding of the non-linear relationship that William Holmes referred to in item 8 above with his statement: *"Even at the higher strain level, the drift to cause failure would not increase in proportion for most of the lower level columns."*

16. Page 8 of the Peer Review report includes the statement:

"Elwood, Moehle and others summarized a decade of research on the collapse behaviour of columns as defined by loss of gravity support, and introduced a major revision to ASCE 41 in 2007".

17. An extract of the latest draft of ASCE 41, The American Society of Civil Engineers guide for 'Seismic Rehabilitation of Existing Buildings' which, I understand, includes the noted major revision in 2007 by Elwood, Moehle and others, is attached for information and marked **"D"**. Clause 6.3.3.1 in attachment **D** states:

"Without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fibre shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components unless larger strains are substantiated by experimental evidence and approved by the authority having jurisdiction".

Other guidance about usable strain limits is given in clause 6.3.3.1 and C6.3.3.1 of the latest draft ASCE41attached. Overall this latest ASCE guidance is not inconsistent with the ultimate concrete compression strain value of 0.004 assumed in the Hyland-Smith report, as recommended by NZSEE 2006 guidelines.

18. Further work is being done by the CTV Building non-linear analysis expert group, as directed by the Royal Commission, to refine the non-linear time history analysis in an attempt to model more explicitly the various potential failure modes. This will include a review of usable strain limits and beam-column joint capacities. It is hoped that results from those further non-linear analyses will be available for the Royal Commission hearing.

- **CONCLUSION REGARDING COLLAPSE MECHANISMS**

19. At the bottom of page 8, continuing onto page 9 of the Peer Review report William Holmes concludes:

In my opinion, the CTV Collapse Investigation does not describe a building collapse mechanism, but concentrates on column failures as defined by excess rotation at hinges. Such column "failures" can reduce building lateral and torsional stability and definitely should be avoided, but in the presence of an independent lateral system such as shear walls, are not sufficient to cause complete and catastrophic collapse such as occurred in the CTV Building

20. I disagree with this conclusion. I believe the unusual combination of very low confinement and high axial loads in the CTV Building columns, and considering in addition the potential effects of vertical earthquake accelerations and / or possible low concrete strengths, meant that the columns were susceptible to more widespread flexural / compressive failure. This would have been initiated by excess rotation at column hinges, but then would have progressed rapidly upon spalling of the cover concrete to column compression failure and so to rapid vertical collapse.

- **PROPOSED REFINED COLLPASE SCENARIO**

21. On page 9 of the Peer Review report William Holmes states:

"I propose that a global collapse mechanism was caused more by the degradation of the beam-column joints than by the column hinging. This is not necessarily in disagreement with the investigative report, but represents a change in emphasis."

And at the top of page 10:

"As previously stated, the vulnerability of the beam column joints is pointed out in the investigative report, just not emphasised".

22. I accept William Holmes' comment that potential failure of beam column joints was not given adequate emphasis in the Hyland-Smith report. I agree that the "Proposed Refined Collapse Scenario" outlined by William Holmes on pages 9 to 13 of the Peer Review report is a further viable collapse scenario that could have been reported more fully and brought through into the Conclusions and Executive Summary of the Hyland-Smith report.

23. However, I do not agree with the rationale behind William Holmes' "Conclusion Regarding Collapse Scenario" on page 13 of the Peer Review report, which identifies the joints as necessarily the most critical. I believe the columns may have been critical, as explained in paragraph 20 above.

24. The NTHA indicated that limiting column drifts (based on the adopted 0.004 ultimate concrete strain criteria), limiting beam column joint capacities and diaphragm disconnections may have occurred at several different locations at similar times. Further work is currently being carried out by the non-linear analysis expert group, as directed by the Royal Commission, may provide a better indication of whether columns or joints were most likely to have failed first.

25. Assessment of beam column joint capacity was covered on page 228 in Appendix D of the Hyland-Smith report and in Appendix B of the Non-Linear Analysis report by Compusoft Engineering. From page 228 of the Hyland-Smith report I wrote:
Given the greater uncertainties with analysis of the joints, and given the results that have come out of the column analyses, it was decided that limiting the assessment to columns would be sufficient for the purposes of this investigation.

26. The beam column joints in the CTV Building were an integral part of the columns. As explained in my Third Statement of Evidence, my interpretation was that the code requirements for confinement of columns and for confinement of beam column joints was similar.

27. For this reason I did not consider it necessary to further investigate whether the columns or the beam column joints may have failed first. I also considered that making that distinction was not feasible at the time of writing the Hyland-Smith report, given the information that I had to hand at the time.

- **WHAT LED TO DRIFTS THAT CAUSED COLLAPSE**

28. On pages 13 to 15 of the Peer Review report William Holmes discusses disconnection of the floor diaphragms from the north core and concludes on page 15:
These rough estimation lead me to conclude that one or more diaphragms must have disconnected relatively early in the shaking, unloading the tower, leading quickly to the collapse and not allowing further lateral loading to significantly damage the tower.
and
So I conclude that although the building experienced torsion from several sources, it is unlikely that the failure drifts of columns could have been reached without either severely damaging the tower, or having the diaphragm partially or completely disconnect from the tower at a lower level.

29. This conclusion is not consistent with what I found from the non-linear time history analysis, which identified and took into account factors that cannot readily be calculated by hand, including foundation flexibility and rocking of the north core and south wall. Foundation rocking could be compatible with larger column drifts than those calculated by William Holmes and would not result in visible damage to the

north core walls. My conclusion was that column failure could have occurred with or without diaphragm connection failure.

30. If large storey drifts had occurred due to disconnection of lower floor diaphragms from the north core, as promoted by Clifton [BUI.MAD249.0223.1] and William Holmes, I would expect to have seen more out-of plane damage in the collapsed south wall. Note that the south wall would have been present and would have prevented locally high storey drifts because it was one of the last components to collapse, on top of all the other collapse debris.

31. I do not disagree with the overall conclusion by William Holmes on page 17 of the Peer Review report which states:

Based on this review, it is my judgement that the most important seismic deficiencies in this building were the brittle gravity frames and the poor diaphragm, particularly the connections to the north tower walls.

Signed: ....AH Smith....

ASHLEY HENRY SMITH

Date:....25 June 2012

Amendments to Tables 11 & 12 - tracked changes version – "B"

Extract from CTV Building Collapse Investigation Report - Appendix D

Level	Axial Load (kN)	Drift Capacity (% of storey height)			E/W Drift Demand 22 Feb CHHC (%)	Ratio Demand / Capacity (Ɛcu=0.004)
		First Yield	Nominal Strength	Ɛcu=0.004		
5	~~324~~320	~~0.71~~0.66	~~0.85~~0.79	~~1.31~~1.17	1.65	~~1.26~~1.41
4	~~681~~685	~~0.85~~0.82	~~0.90~~0.87	~~1.20~~1.14	1.85	~~1.54~~1.62
3	~~1038~~1029	~~no yield~~0.89	0.89	~~1.10~~1.14	1.86	max. ~~1.69~~1.63
2	~~1328~~1317	no yield	~~0.95~~0.92	~~1.08~~1.14	1.76	~~1.63~~1.54
1	~~1682~~1695	no yield	~~0.90~~0.91	~~0.96~~N/A	1.46	~~1.52~~N/A

Table 11 - Drift demand vs. capacity for columns at grid D2

Level	Axial Load (kN)	Drift Capacity (% of storey height)			~~E/W~~N/S Drift Demand 22 Feb ~~CHHC~~ CBGS (%)	Ratio Demand / Capacity (Ɛcu=0.004)
		First Yield	Nominal Strength	Ɛcu=0.004		
5	~~230~~235	~~0.67~~0.74	~~0.86~~0.84	~~1.40~~1.14	2.03	max. ~~1.45~~1.78
4	~~462~~466	~~0.77~~0.81	~~0.94~~0.90	~~1.34~~1.19	2.04	~~1.52~~1.71
3	~~695~~685	~~0.91~~0.90	0.98	~~1.30~~1.26	1.91	~~1.47~~1.52
2	~~910~~924	~~0.91~~0.94	~~0.96~~0.94	1.21	1.85	~~max.~~ 1.53
1	~~1154~~1153	~~0.96~~no yield	0.97	~~1.14~~1.15	1.62	~~1.42~~1.41
Check effect of upper bound spandrel interaction on drift capacity of columns (refer note* below):						
5-S	~~230~~235	~~0.57~~0.63	~~0.73~~0.70	~~1.11~~0.92	est. 2.03	max. ~~1.83~~2.21
4-S	~~462~~466	~~0.63~~0.67	~~0.76~~0.74	~~1.06~~0.96	est. 2.04	~~max.~~ ~~1.92~~2.13
3-S	~~695~~685	~~0.72~~0.76	~~0.80~~0.78	~~1.02~~1.00	est. 1.91	~~1.87~~1.91
2-S	~~910~~924	~~0.81~~0.84	~~0.88~~0.84	1.05	est. 1.85	1.76
1-S	~~1154~~1153	~~0.95~~no yield	~~0.92~~0.93	1.09	est. 1.62	~~1.49~~1.47

Table 12 - Drift demand vs. capacity for columns at grid F2

*Note to table 12: The drift capacity values in the bottom row, denoted Levels "1-S", are from the pushover analysis with full spandrel interaction (i.e. "S" is for spandrels). Potential beam-column joint failure has also not been included in these Figures.

Amendments to Tables 11 & 12

Extract from CTV Building Collapse Investigation Report - Appendix D

Level	Axial Load (kN)	Drift Capacity (% of storey height)			E/W Drift Demand 22 Feb CHHC (%)	Ratio Demand / Capacity (Ɛcu=0.004)
		First Yield	Nominal Strength	Ɛcu=0.004		
5	320	0.66	0.79	1.17	1.65	1.41
4	685	0.82	0.87	1.14	1.85	1.62
3	1029	0.89	0.89	1.14	1.86	max. 1.63
2	1317	no yield	0.92	1.14	1.76	1.54
1	1695	no yield	0.91	N/A	1.46	N/A

Table 11 - Drift demand vs. capacity for columns at grid D2

Level	Axial Load (kN)	Drift Capacity (% of storey height)			N/S Drift Demand 22 Feb CBGS (%)	Ratio Demand / Capacity (Ɛcu=0.004)
		First Yield	Nominal Strength	Ɛcu=0.004		
5	235	0.74	0.84	1.14	2.03	max. 1.78
4	466	0.81	0.90	1.19	2.04	1.71
3	685	0.90	0.98	1.26	1.91	1.52
2	924	0.94	0.94	1.21	1.85	1.53
1	1153	no yield	0.97	1.15	1.62	1.41
Check effect of upper bound spandrel interaction on drift capacity of columns (refer note* below):						
5-S	235	0.63	0.70	0.92	est. 2.03	max. 2.21
4-S	466	0.67	0.74	0.96	est. 2.04	2.13
3-S	685	0.76	0.78	1.00	est. 1.91	1.91
2-S	924	0.84	0.84	1.05	est. 1.85	1.76
1-S	1153	no yield	0.93	1.09	est. 1.62	1.47

Table 12 - Drift demand vs. capacity for columns at grid F2

*Note to table 12: The drift capacity values in the bottom row, denoted Levels "1-S", are from the pushover analysis with full spandrel interaction (i.e. "S" is for spandrels). Potential beam-column joint failure has also not been included in these Figures.

exercised where flexural deformation capacities are determined by calculation. FEMA 306 is a resource for guidance regarding the interaction between shear and flexure.

6.3.3.1 Usable Strain Limits

Without confining transverse reinforcement, the maximum usable strain at the extreme concrete compression fiber shall not exceed 0.002 for components in nearly pure compression and 0.005 for other components unless larger strains are substantiated by experimental evidence and approved by the authority having jurisdiction. Maximum usable compressive strains for confined concrete shall be based on experimental evidence and shall consider limitations posed by fracture of transverse reinforcement, buckling of longitudinal reinforcement, and degradation of component resistance at large deformation levels. Maximum compressive strains in longitudinal reinforcement shall not exceed 0.02, and maximum tensile strains in longitudinal reinforcement shall not exceed 0.05. Monotonic test results shall not be used to determine reinforcement strain limits. If experimental evidence is used to determine strain limits, the effects of spacing and size of transverse reinforcement and of low-cycle fatigue shall be included in the testing procedures, and results are subject to the approval of the authority having jurisdiction.

C6.3.3.1 Usable Strain Limits

The reinforcement tensile strain limit is based on consideration of the effects of material properties and low-cycle fatigue. Low-cycle fatigue is influenced by spacing and size of transverse reinforcement and by strain history. Using extrapolated monotonic test results to develop tensile strains greater than those specified above is not recommended. The Caltrans Seismic Design Criteria (Caltrans 1999) recommends an ultimate tensile strain of 0.09 for #10 bars and smaller and 0.06 for #11 bars and larger, for ASTM A706 (Grade 60). A lower bound is selected here considering the variability in materials and details seen in existing structures.

6.3.4 Shear and Torsion

Strengths in shear and torsion shall be calculated according to ACI 318 except as modified in this standard.

Within yielding regions of components with moderate or high ductility demands, shear and torsional strength shall be calculated according to procedures for ductile components, such as the provisions in Chapter 21 of ACI 318. Within yielding regions of components with low ductility demands and outside yielding regions for all ductility demands, calculation of design shear strength using procedures for effective elastic response such as the provisions in Chapter 11 of ACI 318 shall be permitted.

Where the longitudinal spacing of transverse reinforcement exceeds half the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed not more than 50% effective in resisting shear or torsion. Where the longitudinal spacing of transverse reinforcement exceeds the component effective depth measured in the direction of shear, transverse reinforcement shall be assumed ineffective in resisting shear or torsion. For beams and columns, ~~in which perimeter hoops are either~~ lap-spliced ~~or have hooks that are not adequately anchored in the concrete core,~~ transverse reinforcement shall be assumed not more than 50%