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

FIFTH STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH
COMMENTS ON THE EVIDENCE OF MICHAEL JOHN NIGEL PRIESTLEY

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 Fifth 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 Evidence of Michael John Nigel Priestley [WIT.PRIESTLEY.001.1] 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 EVIDENCE OF JOHN MICHAEL NIGEL PRIESTLEY [WIT.PRIESTLEY.001.1].

NIGEL PRIESTLEY'S AREAS OF DISAGREEMENT
(FROM PAGE 5 OF HIS EVIDENCE)

a. RELIANCE ON ELASTIC RESPONSE SPECTRUM ANALYSIS

7. Quoting from page 12 of the CTV Building Collapse Investigation report dated 27 January 2012 (the Hyland-Smith report)
   "... there remain some issues where interpretations by the authors varied. These have been noted in the report and relate to; interpretation of some collapse evidence and photos; reconciliation of some damage reported compared with that implied by the ERSA and NTHA; limitations of the applicability of ERSA in the assessment of earthquake demands on structures; and interpretation of the requirements for design of secondary elements. While interpretations varied it was recognised that the resolution of these was not critical to the findings of the investigation."

8. In paragraphs 20 to 25 of my first statement of evidence, and in attachment "C" thereto I explained how I was involved in coordinating ERSA for the CTV Building for the purposes of code compliance checks only.

9. StructureSmith and its subcontractor Compusoft Engineering did not have any involvement in the ERSA that used response spectra from the September 2010 and/or February 2011 earthquakes, as reported by Dr Clark Hyland in Appendices E and F of the Hyland-Smith report.)

10. I agree with Nigel Priestley's statement in his paragraph 18, that Elastic Response Spectrum Analysis (ERSA) *"is a method intended for determining the required strengths of structural members to satisfy code-specified seismic input."* Also that ERSA is not suitable for determining the expected response when assessing a building for a particular earthquake, and especially when that response is beyond the elastic capacity of the structure. In this case some form of non-linear analysis is generally required.

11. I generally agree with Nigel Priestley's description of the limitations of ERSA in paragraphs 21 to 26 of his evidence.

12. I do not agree with the circumstances described in paragraph 28 of Nigel Priestley's evidence where he states "As the consultants did not have the expertise in Non-Linear Time History Analysis (NTHA) a decision was made to employ Compusoft, a consulting firm specialising in high-level structural analyses, including NTHA." We always had the capability to carry out the required non-linear analysis from the commencement of the project, with Compusoft Engineering personnel identified in our team from the outset.

13. StructureSmith and Compusoft Engineering had already commenced work on a non-linear analysis model in late April / early May 2011 but were instructed to put a hold on any further non-linear analysis work by Dr Clark Hyland on 11 May 2011. We did not re-start the non-linear analysis until early August 2011, when instructed to do so by DBH, as explained in paragraphs 32 to 35 of my first statement of evidence. I expect this is what Nigel Priestley is referring to in paragraph 27 of his evidence.

14. Regarding paragraphs 31 and 33 of Nigel Priestley's evidence where he states:

"... despite the serious limitations of ERSA I have referred to earlier in my evidence, and the NZSEE recommendation against its use, the consultants have generally chosen to rely on the ERSA results rather than the NTHA results, where these differ.

I would like to clarify that I did not rely on the ERSA results, other than to estimate design actions according to the codes of the day. I relied only on the NTHA and the collapse evidence to justify the findings of the collapse investigation including evaluation of the various collapse scenarios that were reported.

b. DESIGN SEISMIC INTENSITY

15. I was not involved in the preparation of Figure 11, or in the other response spectra plots that were presented in Appendix E by my co-author Dr Clark Hyland. I was involved only in coordinating the various code-based ERSA that have been outlined in attachment C to my first statement of evidence. The variation in scale factor referred to by Nigel Priestley in paragraph 38 of his evidence can be seen, for the code-based ERSA, in attachment C to my first statement of evidence, by comparing the base shears for north-south and east-west directions.

c. ROLE OF SPANDREL PANELS ON EAST AND SOUTH FACES ON THE CTV COLLAPSE

16. I comment on several of the points raised by Nigel Priestley in section 44 of his evidence as follows:

- Regarding paragraphs 44b & 44c, the NTHA analyses that I based my conclusions on considered two cases, a lower bound case with no spandrels and an upper bound case with no gap and very stiff spandrels. Regarding the actual gap that may have been present it should be noted that the spandrel-to-column gap was not a clear gap but was filled with an elastomeric sealant which is likely to have lost a proportion of its elasticity over time.
- I disagree with paragraph 44d. My calculations showed that the connections between the spandrels and the supporting beams may have been considerably stronger than calculated by Nigel Priestley and could have provided restraint up to the limit of the spandrel strength.
- I agree with the first sentence of paragraph 44e i.e. that the capacity of the spandrel end diaphragms was likely to be less than the connection capacity. However, I do not agree with the second sentence of paragraph 44e, because I would say that a significant force could be transmitted at the point of contact between the column and spandrel, as shown in Nigel Priestley's own calculations on slide 13 of his presentation [BUI.MAD249.0402.13]. I performed similar calculations myself, with similar results. This potential force on the columns from the spandrels may not have been sufficient to cause the columns to fail directly, however it may have progressively weakened the columns, leading to premature failure by other mechanisms.
- Regarding paragraph j, i and iii, my calculations show that approximately 70% of the full axial force in the line F columns could be transmitted to line E, compared with 60% calculated by Nigel Priestley. This is sufficient axial load transfer to cause the line E column to then be the most heavily loaded, i.e. with approximately 30% more axial load than in the line D column, which was the most susceptible column in Nigel Priestley's view. In addition to increased axial load the line E columns could also have been subjected to high bending, shear and twisting actions as a result of line F column failure.

d. EXTERIOR COLUMNS VS INTERIOR COLUMNS AS COLLAPSE INITIATOR

16. Regarding paragraphs 46, 47 and 50, please refer to the amended Tables 11 and 12 attached and marked **"B"** including revised demand / capacity ratios. The amendments are explained in paragraphs 4 to 9 in my Fourth Statement of Evidence. The amended demand / capacity ratios in attachment "B" indicate that high level grid F columns were the most susceptible, ignoring the effects of vertical earthquake actions and / or possible low concrete strength. I agree with Nigel Priestley's comments that combination with vertical earthquake accelerations and / or lower concrete strength would alter these demand / capacity ratios and would tend to make the lower level interior columns more vulnerable.

17. Regarding paragraphs 50 and 51 where Nigel Priestley discusses catenary action. I would like to point out that the arrangement of collapse debris, with the east side structure falling out onto cars parked in Madras Street, is not consistent with catenary actions from an internal column failure, as promoted by Nigel Priestley because an internal column failure would have pulled the grid F structure inwards.

e. CONNECTION FAILURE BETWEEN FLOOR DIAPHRAGM AND NORTH CORE

18. Regarding paragraph 59, regarding the calculated capacities of the drag bars. Dr Clark Hyland calculated the values on page 266 of the Hyland-Smith report, based on the FIB 2011 reference. I calculated similar, though generally slightly lower values, by less rigorous methods by referring to the Ramset resource books. I considered the values in the Hyland-Smith report to be upper bound capacities. Even with these upper bound capacities the NTHA indicated failure of the drag bars and so this was considered as a possibility in the various collapse scenarios that were reported.

19. Regarding paragraphs 60 to 62, regarding the capacity of the floor connection to walls on lines C and C/D. The expected strength of this slab connection is somewhat uncertain, because; we know now with the benefit of hindsight that steel mesh can be brittle, we don't know how much the Hi-Bond metal deck contributed, and because we must consider in-plane tension, bending and shear actions and also out-of-plane actions acting on the floor slab simultaneously. Based on the NTHA, I concluded that connection failure between the floor diaphragm and the north core was a possibility and this was considered in the various collapse scenarios that were reported.

f. TORSIONAL ECCENTRICITY

20. Regarding paragraphs 63 and 64, I agree with Nigel Priestley's comments about torsional eccentricity and ERSA. However it should also be noted that, with or without the masonry wall at the west side, the NTHA also showed considerable torsional eccentricity was present in the structure, and this had the effect of magnifying diaphragm connection forces and column drifts.

g. MODELLING OF MASONRY INFILL ON LINE A

21. I am comfortable that the modelling that was carried out for the NTHA, with and without the masonry infill walls, as reported in Appendix D of the Hyland-Smith report gave an adequate insight into the potential interaction effects.

h. CONSULTANTS RELUCTANCE TO ACCEPT NTHA RESULTS

22. Refer to paragraph 14 above for my view.

i. DISPLACEMENT CAPACITY ERSA ANALYSIS

23. Refer to paragraph 7 above and also to paragraph 6 in my third statement of evidence.

THE CAUSES AND LIKELY SEQUENCE OF COLLAPSE OF THE CTV BUILDING

24. Regarding paragraph 83, for the reasons explained above, I am still of the view that the columns (including perhaps beam-column joints) on line F are most likely to have acted as the failure initiator. However, I cannot be certain because the NTHA indicated that limiting compressive strains in columns, limiting beam column joint capacities and diaphragm connection failures occurred at several different locations at similar times.

Signed: AH Smith

ASHLEY HENRY SMITH

Date: 25 June 2012

Amendments to Tables 11 & 12

"C"

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.