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| **UNDER** | **THE COMMISSIONS OF INQUIRY ACT 1908** |
| **IN THE MATTER OF** | **ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES**<br><br>**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** |

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**SIXTH STATEMENT OF EVIDENCE OF ASHLEY HENRY SMITH**
**COMMENTS ON THE EVIDENCE OF:**

- **ROBIN SHEPHERD**
- **JOHN BARRIE MANDER AND**
- **ALAN MICHAEL REAY**

**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 Sixth 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 Robin Shepherd [WIT.SHEPHERD.0001], John Barrie Mander [WIT.MANDER.0001] and Alan Michael Reay [WIT.REAY.0002 ] 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 ROBIN SHEPHERD [WIT.SHEPHERD.0001]**

*FORENSIC ENGINEERING PRACTICE*

PARAGRAPH 20

7. I made every effort to *"critically evaluate those portions of the structure remaining after the collapse"* from the time of my first visit to the site on 5 April 2011. Unfortunately by the time I arrived most of the collapse debris, apart from the north core and the foundations had been moved. I understood that it had been particularly urgent to move the debris because of the fire and the need to check for survivors. Clark Hyland from our team interviewed Graham Frost. I looked very carefully at the photographs and the observations recorded by Graham Frost and many others who had been on the site before me.

PARAGRAPH 21

8. Hyland Consultants carried out the site examination and materials testing, including concrete testing under a separate contract and reported those in the Hyland report (not the Hyland/Smith report). I understand there is to be a session at the hearing where the concrete experts will further discuss the various concrete test results and this may shed more light on the likely concrete strengths. The Hyland/Smith report allowed for a range of concrete strengths because the test results by Hyland were variable. As explained in the Hyland/Smith report overview on page 1, concrete strength was considered to be only one of many factors that contributed (or may have contributed) to the failure.

9. The Hyland/Smith investigation was not locked into pursuing the hypothesis that the columns provided the critical failure initiator. Figure 16 in the Hyland/Smith report, which I prepared, shows all the factors that were considered at the left and right sides under the headings 'demand' and 'capacity'. The 'probable collapse sequence' in the centre of that diagram shows various possibilities involving east side and/or internal column failures and also failure of the diaphragm connections to the north core. As explained in paragraphs 22 to 27 of my fourth statement of evidence, it is acknowledged that the possibility of beam-column joint failure was not given sufficient emphasis in the Hyland/Smith report.

10. The non-linear time history analysis (NTHA) did not *"focus on the columns at the exclusion of a possible alternative weakness."* All of the factors shown in Hyland/Smith Figure 16 were considered in the NTHA.

11. I did not have any *"reluctance to accept the results of the NTHA."* As explained in paragraph 14 of my fifth statement of evidence I relied on the results of the NTHA and the collapse evidence to justify the findings of the investigation, including evaluation of the various collapse scenarios that we reported. From page 2 of the Hyland Smith report *"Many reasonable possibilities existed. In these circumstances it has been difficult to identify a specific collapse scenario with confidence."* I am keeping an open mind even now; because the results of the more refined NTHA which is currently being carried out as directed by the Royal Commission may identify some other possible cause or sequence.

12. StructureSmith and Compusoft Engineering did not *"neglect the possibility that the CTV Building might not have been in an essentially undamaged state at the commencement of the February 2011 shaking."* The NTHA for 4 September 2010 had indicated only minor yielding in the shear walls and in the columns and also partial diaphragm dis-connections from the north core. I felt that the precise level of this damage in September could not be accurately predicted and calibrated from the analysis, given uncertainties about the many contributing factors and their influence, the approximations inherent in the analysis and the limited coverage of the eye witness and post-earthquake damage reports that were available.

13. On the other hand, the NTHA for 22 February 2011 indicated that ultimate compressive strains in columns, limiting beam-column joint capacities and diaphragm connection capacities had all been exceeded by a large margin at many different locations and this result would be unlikely to change if a slightly damaged state had been assumed at the beginning of that analysis.

PARAGRAPH 22

14. I deny that I *"failed to abide by the generally accepted open minded approach to a failure analysis investigation"* and gave every avenue of possible enquiry serious consideration.

*CUMULATIVE EARTHQUAKE DAMAGE*

PARAGRAPHS 32 AND 33

15. I query what types of buildings are referred to in Robin Shepherd's quote *"some spectacular failures of old buildings are attributable to progressive weakening in successive minor shaking".* Were they reinforced concrete buildings similar to the

CTV Building? Without further information about the nature and extent of the so-called *"progressive weakening"* this reference is meaningless.

PARAGRAGHS 38 to 40

16. These paragraphs make reference to a draft Hyland/Smith report, which is not relevant now in my view.

*SEISMIC EXITATION AT THE CTV BUILDING SITE*

PARAGRAPH 46

17. I presume the last sentence of this paragraph was intended to read: "*Since buildings are designed to resist, with a significant safety margin, normal static gravity loading, it was inferred that expected earthquake generated vertical loads were <u>not</u> more than allowed for in gravity resistant design without additional vertical seismic strength provisions"*?

PARAGRAPH 47

18. The Hyland/Smith report does *"address the possible influence of vertical accelerations on the CTV columns"* on pages 217 to 228, and elsewhere.

*DYNAMIC ANALYSIS*

PARAGRAPH 52

19. While the NTHA did *"make use of records made at nearby locations with the consequent uncertainty of their applicability"*, the uncertainty was much reduced when compared with several other time history analyses that I have been involved with or seen reported previously. This is because the records that were used were of the actual ground shaking that was recorded from the particular earthquakes of interest (i.e. 4 September 2010 and 22 February 2011) on a nearby site with similar ground conditions.

PARAGRAPH 54

20. The NTHA was not carried out 'to prove a certain hypothesis rather than to investigate all collapse possibilities without prejudice', as explained in paragraphs 9 and 10 above. The NTHA was set up to best capture the response of the building and to provide information suitable for the review of many hypotheses.

PARAGRAPH 56

21. Refer to my fourth statement of evidence for my comments relating to the review by William Holmes.

PARAGRAPH 57

22. I did not *"focus on a particular scenario at the exclusion of others"*, but considered one scenario more likely, because that scenario fitted best when the results of the NTHA were considered together with all the collapse evidence and the eye-witness observations that were available.

NTHA EXPERT PANEL

PARAGRAPH 59

23. I am also participating in the NTHA expert panel being facilitated by Athol Carr. I reserve the right to modify or add to my evidence following the completion of that process. Note that the NTHA expert panel has agreed that further analyses will be carried out with a segment of the 4 September 2010 record followed by a segment of the 22 February record, to check whether cumulative damage effects are likely to be significant for the CTV Building.

**COMMENTS ON THE EVIDENCE OF JOHN BARRIE MANDER [WIT.MANDER.0001]**

I make reference to page numbers at the top right corner in John Mander's evidence.

PAGE 42

24. Refer to my fourth statement of evidence for my comments on the peer review by William Holmes, and to my fifth statement of evidence for my comments on Nigel Priestley's evidence.

*HIGHER THAN EXPECTED HORIZONTAL GROUND ACCELERATIONS*

PAGE 44

25. Refer to my third statement of evidence for my interpretation of the applicable design codes in relation to columns and beam-column joints. My interpretation is that the CTV building columns and beam-column joints were not designed in accordance with the codes. Chapter 9 of the Hyland/Smith report also identifies other areas where the design and/or the construction did not comply.

PAGE 45

26. The Hyland/Smith report did consider and evaluate the effects of the earlier 4 September 2010 earthquake on the building as explained in paragraphs 12 and 13 above. Other earthquakes, including the Boxing Day 2010 earthquake, had considerably less intensity and so were considered unlikely to have had much effect on the structure.

PAGE 45-46

27. I note John Mander's finding, based on the results from Bradley (2012): *"Hence it can be inferred that there is a 40% chance that the Darfield earthquake ground motion at the CTV Building site was larger than the design level of ground motion."*

28. I do not know whether the above statement is correct. However, supposing it is true, I cannot see how this supports the numerous assertions throughout the remainder of John Mander's evidence, that the CTV building must necessarily have withstood the design level ground shaking, because surely there was only a 40% chance of that having occurred.

29. Even if the CTV Building had withstood a 'design level earthquake', in my opinion that is not *"testament to the sufficiency of the design, or that it met the aim and objective of the design codes."*

PAGE 46

30. I do not see any justification for John Mander's statement that "... it is evident that the structure of the CTV Building must have also sustained hidden (unobserved and/or unobservable) damage."

*EXCEPTIONALLY HIGH VERTICAL GROUND MOTIONS*

PAGE 47

31. The Hyland/Smith report did not neglect the effect earlier earthquakes had on the structure of the CTV Building, as explained in paragraphs 12 and 13 above.

32. The continuous three span one-way slab system <u>is</u> referred to in the Hyland/Smith report and also in the referenced Compusoft Engineering Non-Linear Analysis report. A sensitivity study was carried out to determine the influence of slab stiffness on the vertical excitation of structural elements and associated design actions as explained

on pages 71 to 73 of the Compusoft report. An interim finding of the Royal Commission NTHA expert group is that that the effective damping assumed by Compusoft for the high frequency vertical vibration modes, at around 1% damping, was low and would be expected to be a minimum of 2%. This will have the effect of reducing the effects of vertical vibrations on the drift capacity of columns.

33. I and others on the NTHA Expert group, consider a "buckled folded-plate failure mode' of the floor slab unlikely as an initiator of the collapse. The floor slab comprised 200mm thick in-situ concrete poured on a 55mm deep profiled metal deck and so would have been quite resistant to buckling.

PAGE 48

34. Referring to paragraph 1, the structure <u>was</u> analysed for the effects of vertical ground motions alone, as explained on pages 217 to 219 of the Hyland/Smith report and on pages 71 to 73 of the Compusoft report. As explained in paragraph 32 above, the adopted 1% damping for vertical vibrations is considered to be low and so the effects of vertical vibrations are likely to have been over-estimated. I consider it unlikely that the *"exceptionally high vertical ground motions were a primary contributor to triggering the CTV Building's failure and subsequent collapse."*

35. Referring to John Mander's section 1.3 "*Lack of Ductile detailing in critical columns.*" My interpretation of the design codes for ductile detailing in columns is given in my third statement of evidence.

36. Regarding John Mander's statement *"It appears to be for these reasons that the structural designer evidently sought a simpler form of construction that avoided the use of copious quantities of transverse reinforcing steel to provide ductility capability."* I do not see that the desire for a simpler form of construction, as justification for interpreting the design codes any differently. Ensuring the ability of the structure to resist seismic movements and to avoid a risk to life should have been the highest priority.

PAGE 49

37. I disagree with the following statement by John Mander:

*"Although it is true that the columns were not provided with substantial transverse reinforcement, this was neither a problem nor a cause of failure within the CTV Building. If it were a cause of the collapse, then there would be a substantial forensic evidence indicating that most columns had significant lengths of cover*

*concrete spalled off, substantial buckling of longitudinal reinforcing bars due to the high axial loads, and diagonal shear cracks."*

The reasons for my disagreement are as follows:

It should be understood that the investigators, and even the USAR engineers who had been on site much earlier during the emergency phase, were unlikely to find the particular columns that initiated the collapse. This is because of the urgency with which the collapse debris had to be cleared in order to rescue people, but also because most of the columns would have exhibited the effects of consequential damage rather than damage which initiated the collapse. By consequential damage I mean the severe impact damage that would have been caused by upper floors falling onto lower floors. It is for this reason that most of the observed column failure surfaces exhibited a spearhead shaped compression failure surface rather than spalled concrete, buckled bars or diagonal shear cracks.

PAGE 50

38. Regarding John Mander's statement *"It is contended that the lack of joint shear reinforcement is one of the principle contributing factors to the CTV Building's collapse."* That may be correct and is being further investigated by the NTHA expert group. My comments on potential beam-column joint failure and the reasons why that was not further investigated in the Hyland/Smith report are covered in paragraphs 21 to 27 of my fourth statement of evidence.

39. I disagree with the following statement by John Mander, for the reasons explained in paragraph 36 above:

    *"But in the 1980's at the time of design, such columns and joints would have been considered an expensive and unnecessary luxury that would minimise the developer's profit margin."*

PAGES 50 TO 52

40. Regarding *"1.4 Low concrete strength in critical columns."*

    My comments on concrete strength are given in paragraph 8 above.

41. Regarding John Mander's statement: *"In spite of the so-called low concrete strength in the critical columns, Compusoft wisely chose to ignore this advice from Opus and used specified strength + 2.5MPa."*

In fact Compusoft were instructed to use specified strength + 2.5 MPa for columns in the NTHA at the DBH Technical Workshop meeting on 16 August 2011, as explained in my first statement of evidence at paragraph 37.10.

PAGES 52 TO 53

42. Regarding *"1.5 Interaction of perimeter columns with the spandrel panels."*

I have already commented about the potential role of the spandrel panels and exterior vs. interior columns as potential collapse initiators in my fifth statement of evidence, at paragraphs 16 and 17. The potential upper and lower bound effects of the spandrels were modelled as described in the first bullet point of paragraph 16 in my fifth statement of evidence. It is not possible to model the spandrel effects directly using gapping elements because the size of the actual gaps that existed is not known.

PAGE 53

43. Regarding *"1.6 Separation of the floor slabs from the North Core."*

I do not agree that separation of the floor slabs from the North Core is a necessary feature of the failure modes proposed in the Hyland/Smith report, for the reasons explained in paragraphs 28 and 29 in my fourth statement of evidence.

44. I also do not agree with the statement *"But on the other hand, ARCL can feel vindicated because the structure survived the design level Darfield Earthquake without collapse, which was the main aim of the design"*, for the reasons explained in paragraphs 27 to 29 above.

PAGES 53 TO 54

45. Regarding *"1.7 Accentuated lateral displacements of columns due to the asymmetry of the shear wall layout."*

I do not agree with the statement by John Mander that *"...the ... results agree remarkably well with the NTHA; they do not indicate that the translational displacements are significantly amplified by lateral torsional (eccentricity effects."* The accentuated lateral displacements of the columns due to the asymmetry of the shear wall layout can be clearly seem by comparing the maximum storey drifts for the frame on line 1 vs. the frame on line 4, as shown in figures 127 and 128, and in figures 131 and 132 in Appendix D of the Hyland/Smith report.

46. Regarding *"1.8 Accentuated lateral displacements due to the influence of masonry walls on the west face."*

I would not say that the Darfield earthquake left the west part of the CTV Building badly cracked. Rather internal linings were reported to be cracked, and daylight could apparently be seen through some joints, where I expect some areas of mortar and/or sealant had worked loose. I found no evidence that *"damage had been caused by demolition of the adjoining building"*. The so called *"deteriorated condition of the west wall"* was not the reason why repairs were being undertaken. I understand the sole reason for that work was to waterproof the portion of the wall that had been exposed to the weather by the removal of the adjoining building.

47. John Mander's hypothesis involving unseating of the east-west beams at the west face may be considered further by the NTHA expert group. This is a specific type of beam-column joint failure. I discussed beam joints on pages 21 to 27 of my Fourth Statement of Evidence.

PAGE 55

48. Regarding *"1.9 Limited robustness and redundancy."*

I do not agree with John Mander's statement that a *"one-way slab system is relatively unusual in the modern era of buildings"*. Quite the reverse. In my experience one-way slab systems are the most common slab systems in New Zealand in the modern era (including the 1980's).

PAGES 56 TO 59 - *GROUND MOTIONS*

49. Tim Sinclair of Tonkin & Taylor has commented in his second statement of evidence, about the evidence of Brendon Bradley that relates to ground motions.

PAGES 60 TO 64 – *CONCRETE TESTING*

50. My comments on concrete testing are covered in paragraph 8 above.

PAGES 68 TO 69 – *COLUMN PERFORMANCE ANALYSIS*

51. Regarding John Mander's statement *"It should be noted that this aspect of the vertical horizontal load coupling was not correctly modelled in the Compusoft analysis. In its modelling, Compusoft reported the level of axial load and moment, but did not adjust the moment-axial load failure surface accordingly."*

As explained in paragraph 15 of Derek Bradley's second statement of evidence the shear walls resisted approximately 80-90% of the total seismic shear force in the north/south and east/west directions and the shear walls also controlled the overall seismic displacements. The NTHA focussed on capturing the response of the building as accurately as possible by modelling the shear wall elements as non-linear shell elements. It was not intended to adjust the moment-axial load failure surface for columns, because, as explained in Compusoft report and in Appendix D of the Hyland/Smith report, M-M hinges rather than P-M-M column hinges were used. However, the effects of varying axial load were certainly considered and taken account of in the processing and interpretation of results for columns.

52. It was recognised that Figure 15 of the Hyland/Smith report displayed some results for column moments that exceeded, by up to 9%, the intended column moment capacity envelope. Isotropic hardening of the column hinges, which is an unintended effect of the modelling, lead to this increase. This effect was not significant in terms of the overall building response or in the assessment of column drift capacities. It was not significant because the columns have only a small rotation capacity past yield (as a result of non-ductile detailing) and so the resulting increase in column moment capacity, at the stage when column failure was predicted to occur, was of the order of only 1% to 3%. The effect on the overall seismic response is expected to be negligible. A different type of column hinge is currently being used by the NTHA expert group to verify this. (Interestingly those outer points on Hyland/Smith Figure 15 would be within the interaction surface for 1.5 x 35MPa = 52.5 MPa concrete, as proposed by John Mander for any further NTHA runs).

53. The effects of varying axial loads on columns were certainly considered and taken account of in the processing and interpretation of results. This was done via the non-linear pushover model, where joint rotations were monitored and related back to the source moment-curvature plots to calculate the column drift capacities shown in Hyland/Smith Tables 11 and 12 and in Figures 139 to 142.

54. I disagree with John Mander's statement *"Therefore, the clearly demonstrated modelling inaccuracies of the failure criteria puts a large cloud of doubt over most of the results in the Hyland/Smith report, and in particular the interpretation of the results"*, for the reasons outlined in paragraphs 51 to 53 above.

PAGE 81

55. *"3.2 Collapse mechanism under east-west shaking."*

The possibility of beam column joint failure, including that indicated by John Mander in his Figure 3.1 is being further investigated by the NTHA expert group. Regarding Figures 3.2 and 3.3, I cannot understand how those columns could buckle while the floors remained connected to the south wall. My interpretation of the collapse evidence was that the floors were pulled off the south wall after the vertical collapse of internal columns. I also did not see or hear any evidence of the building deforming in the manner depicted in Figures 3.2 to 3.4.

PAGE 85

56. Regarding "*3.3 Collapse mechanism under north-south shaking*", as shown in Figure 3.4, I comment as follows:

    57. For the northward collapse mechanism, as explained in paragraph 33 above, I and others on the NTHA Expert group consider a "buckled folded-plate failure mode' of the floor slab unlikely as an initiator of the collapse.

    58. As for the southward collapse mechanism, with lower level floors disconnecting from the north core, I have commented on the reasons why that is unlikely in paragraphs 29 to 30 of my fourth statement of evidence.

PAGE 90 ITEM 4.5

59. I disagree with the statement *"There are several assumptions and various aspects of the Hyland/Smith report that bring into question the veracity of the claims and conclusions."* I have commented on the peer review by William Holmes and on Nigel Priestley's evidence in my fourth and fifth statements of evidence.

**COMMENTS ON THE SECOND STATEMENT OF EVIDENCE OF ALAN MICHAEL REAY [WIT.REAY.0002]**

60. Regarding paragraph 12 "Suspended Floors." I understand that the CTV Building had concrete floors poured on <u>Hi-Bond</u> (not Bondeck) profiled metal decking.

Non-Linear Time History analysis

61. Regarding paragraphs 20 and 21, on cumulative damage, I have commented on that in paragraphs 12 and 13 above.

62. Regarding paragraph 26, the NTHA carried out for the DBH report used specified concrete strength + 2.5 MPa for columns.

63. Regarding paragraph 31, I have made some relevant comments on concrete strength and the concrete testing carried out by Haavik and CTL in paragraphs 8 and 40 above.

64. Tim Sinclair of Tonkin & Taylor has commented in his second statement of evidence, about the evidence of Brendon Bradley that relates to ground motions.

65. Regarding paragraph 35, there is likely to have been some variation in the widths of the gaps between spandrels and columns due to normal construction tolerances.

Change of Use

66. Regarding paragraph 41, according to the 1992 loadings code, I calculated the applicable reduced seismic design live load to be 0.81kPa for school classrooms and 0.45kPa for offices. I agree with the other factors listed in that paragraph by Alan Reay.

67. Regarding paragraph 42, the gravity load factors and the design seismic spectrum were also different in the 1992 code when compared to the 1984 code. Those factors would also need to be taken into account before a substantial change to the seismic and gravity loads could be confirmed.

COLLAPSE CONSIDERATIONS

Reinforcing strain hardening

68. I do not know what possible effects strain hardening could have caused. I expect it would be more relevant for the shear walls; however the shear walls were not the cause of the CTV Building failure, apart from their asymmetry. It was found that for many of the highly loaded columns in the CTV Building, concrete at specified strength + 2.5 MPa would fail by crushing before any reinforcement yielded in tension and therefore strain hardening would not have been relevant. I am aware that the floor mesh could have been brittle and may have fractured. Floor diaphragm disconnection form the north core has been considered as a potential collapse initiator in the Hyland/Smith report.

Vertical acceleration

69. The NTHA expert group is looking further into the potential effects of vertical accelerations. Refer also to my comments under paragraph 32 above.

South wall lateral load resistance

70. The effects of the gravity loadings and vertical seismic accelerations have been accounted for explicitly in the NTHA by modelling the wall elements as non-linear layered shell elements.

Building modifications

71. Damage caused by holes drilled through beams was considered as part of the DBH investigation. However, given that the beams were not a significant part of the seismic resisting system, then holes drilled through beams would not have diminished the seismic resistance of the building. Also, beam failure is unlikely to have been an initiator of the collapse because I expect it would have exhibited more localised failure, as compared to the rapid progressive collapse that occurred.

72. Regarding paragraph 67, the Hyland/Smith investigation did take account of, and the NTHA did explicitly model, the floor penetration through the Level 2 floor slab for the internal stair that was installed in circa 2000. The level 2 floor penetration for the stair did not encroach onto the south wall and the remainder of that wall, composite with the grid 1 beams, was considered adequate to transfer seismic forces into the shear wall.

Cumulative damage resulting from aftershocks

73. Regarding paragraph 72, the potential effects of the 4 September 2010 earthquake and the 22 February 2011 aftershock were considered and taken account of as discussed in paragraphs 12 and 13 above. The other aftershocks resulted in significantly lower ground shaking intensities and were considered unlikely to have much effect on the performance of the CTV Building.

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

**ASHLEY HENRY SMITH**

Date:....3 July 2012....