FIRST STATEMENT OF EVIDENCE OF ALAN MICHAEL REAY

1. My full name is Alan Michael Reay. I reside in Christchurch. I am a Chartered Professional Engineer and a Company Director.

   I have asked to be heard because I feel people deserve to know all the aspects of why the building failed.

2. Primarily this statement of evidence deals with factual matters. I will file a supplementary statement of evidence dealing with issues on which an expert opinion is required. Insofar as this evidence can be considered expert evidence, in accordance with the requirements of Rule 9.43 of the High Court Rules, I confirm that I have read the Code of Conduct for expert witnesses and that my evidence complies with the Code's requirements.

3. Matters on which I express an opinion are within my field of expertise.

4. I am a Director of Alan Reay Consultants Limited ("**ARCL**"), an affected party in this Royal Commission hearing.

Qualifications and experience

5. I hold a Bachelor of Engineering with First Class Honours (1965, University of Canterbury) and Ph.D in Civil Engineering (1970, University of Canterbury). My Ph.D thesis was on the dynamic characteristics of civil engineering structures.

6. I am a currently a Fellow of the Institution of Professional Engineers and hold the following memberships:

   (a) New Zealand Concrete Society;

   (b) New Zealand Society for Earthquake Engineers;

   (c) American Concrete Institute;

   (d) Heavy Engineering Research Association of New Zealand;

   (e) Association of Consulting Engineers New Zealand;

   (f) Tilt Up Concrete Association, USA.

7. After completing my qualifications, I commenced work as a structural engineer with Hardie and Anderson. Around two years later, I began business on my own account in 1971, as Alan M Reay, Consulting Engineer ("**ARCE**"). ARCL was incorporated in 1988. I have practiced under this corporate structure ever since. I have also lectured in steel structures at the University of Canterbury in the early 1970's.

8. My full resume is **annexed** to this statement.

Department of Building and Housing ("DBH") reports

89. In early May 2011 I met with Clark Hyland and Ashley Smith following Dr Hyland's request to meet so that he could advise us of the information that he considered would be helpful to their investigation of the collapse of the CTV Building.

90. He advised that he was being employed by the DBH. He also advised that there had been an Expert Panel appointed to provide review and advice regarding the report he would present which he indicated was to be provided by July 2011.

91. I received further requests from Dr Hyland for information, which I responded to **[BUI.MAD249.0439, BUI.MAD249.0130A.1]**.

92. Further information was sought by Dr Hyland in August and I telephoned him for clarification. Dr Hyland advised then or in another conversation around the same time that he could not say what was in his report but suggested there were serious construction deficiencies. I took from this that he had not found any design deficiency.

93. When I provided further information to Dr Hyland in response to his requests, I was surprised by his reply which was to thank me for the information as he had overlooked it **[BUI.MAD249.0440.1]**.

94. The draft DBH report into the collapse of the CTV Building was finally received by ARCL on 8 December 2011 **[BUI.MAD249.0122, BUI.MAD249.0125, BUI.MAD249.0126]**. ARCL, as an affected party, was given until 22 December 2011 to make comment. ARC provided a detailed response on 22 December 2011 **[BUI.MAD249.0195, BUI.MAD249.0195A, BUI.MAD249.0195B]**.

95. ARCL requested and the DBH agreed to provide a copy of the Final Report to ARCL 24 hours prior to release to the media on 9 February 2012. On 3 February 2012 Dr Hopkins advised me by email that the Materials and Testing section of the report was unchanged from the draft so that report would not be included in the early release **[BUI.MAD249.0442.1]**.

96. ARCL received the final report ahead of public release, as promised, with a covering letter addressed to another affected party **[BUI.MAD249.0443.1]**. In the event, although the email from Dr Hopkins was dated after the date of the final Materials and Testing report that report was in fact substantially changed when finally issued.

97. ARCL wrote to the DBH regarding the above issues and others on 24 February 2012 **[BUI.MAD249.0444.1]** and received an apology from the DBH on 5 March 2012 **BUI.MAD249.0445.1]**.

98. I remain dissatisfied about many aspects of the final DBH reports. I will address my concerns in my supplementary statement.

SECOND STATEMENT OF EVIDENCE OF ALAN MICHAEL REAY

DBH Report

17. The DBH Report was originally due for release in July 2011. Extensions of time were granted and the provisional DBH Report was finally made available to ARCL in early December 2011 in a draft format. The DBH permitted ARCL two weeks within which to review it. ARCL had previously (in early October 2011) sought information from the DBH under the Official Information Act 1982. The DBH refused to release any information to ARCL until the draft DBH Report was issued.

18. As noted in my first statement, ARCL commented in detail on the draft DBH Report **[BUI.MAD249.0195, BUI.MAD249.0195A, BUI.MAD249.0195B]** which was eventually released in final form on 9 February 2012.

19. I remain dissatisfied with aspects of the final DBH report. I summarise some of my key concerns below.

Non Linear Time History Analysis

20. The cumulative damage and fatigue effects on the structural elements should be included in the modelling and have been insufficiently accounted for in the analyses run as part of the DBH Report.

21. Related to this point is the effect of each aftershock on the deterioration of the CTV Building and its progressively increasing fragility to further large earthquake events (also discussed further below).

22. To date, there have been no experimental studies to corroborate the computational results. Strictly, there should be shaking table reduced-scale physical model experiments on a 6 degree-of-freedom shake table to investigate the overall behaviour and to recreate the structural failure. Instruments can be used to assess the effects of lateral-torsional coupling, wall-frame interaction and vertical motion effects. From these results, it is inevitable that the underlying assumptions in the computational models will lead to some modifications in order to more accurately capture overall effects. It is conceded that to do this, facilities would either have to be

developed in New Zealand, or else the study would need to be done abroad in either the United States or Japan.

23. A dual shaking table-computational modelling study will no doubt highlight several key components and sub-assemblages that were instrumental in triggering the collapse. In order to gain additional confidence in the results and to remove the uncertainties in the modelling process, further full-scale experimentation of these key components should be tested under simulated earthquake loads and displacements. It is likely that this would include beam-column joint tests, vertical floor-slab dynamic behaviour, columns buckling tests over several storeys and the like. Again, following the results of such an experimental testing investigation, the computational models should be enhanced to properly capture observed behaviour, and then the entire NLTHA rerun for all known earthquakes in the vicinity of the CTV Building to gauge the effects of cumulative damage. Only in this way can the true reasons for the CTV Building collapse be known.

24. Completing these analyses will take considerably longer than the time that was available to the authors of the DBH Report, but in my view, in the absence of these analyses, the modelling to date is inadequate and the Royal Commission does not have access to the best available information to assist with understanding the causes of the collapse.

Concrete

25. The DBH Report refers to concrete strengths, at the time of construction, being of a range between 16 MPa and 43.8 MPa.

26. The DBH Report recommended that an average of 20 MPa (increased from 17.5 MPa in the draft DBH Report) 28 day strength would be appropriate for utilisation in further analyses of the CTV Building as compared to the 35, 30 and 25 MPa strengths for the columns specified in the original design documents.

27. It was my opinion that the probability of concrete strengths as low as this was negligible unless the contractor deliberately set out to order substantially under strength concrete and mishandled the concrete workmanship on site. Alternatively, the low strength results may have been taken from columns which were affected by the fire that broke out at the site. ARCL raised this issue in its comments to the draft DBH report, but this possibility remains inadequately accounted for.

28. The ARCL Report to the DBH on 22 December 2011, in response to the draft DBH Report, recommended further testing by the DBH but this was not undertaken.

29. I now have been advised that the samples, which the draft DBH Report stated were kept for further testing (refer **[BUI.MAD249.0126.79]**), were not in fact kept **[BUI.MAD249.0459.5 and BUI.MAD249.0459A.8].** The final DBH Report did not contain the reference to the samples being retained.

30. Following the release of the final DBH Report which indicates that no further testing had been undertaken by the DBH, ARCL obtained approval to extract samples for further testing and the results are presented in the evidence of Douglas Haavik **[WIT.HAAVIK.0001]**.

31. The DBH testing was limited in scope and did not comply with testing codes of practice, as detailed in others' evidence **[BUI.MAD249.0373.1, BUI.MAD249.0362.1]**. The testing undertaken by ARCL was fully compliant and demonstrated that based on the testing of samples of the columns remaining, the concrete complied with the standards of manufacture and workmanship of the time. I refer to the evidence of Douglas Haavik **[WIT.HAAVIK.0001]**.

Geotechnical Report

32. The Geotechnical Report utilised in the DBH report was provided by Tonkin & Taylor. There appears to have been no advice sought from the Geotechnical Engineer who prepared the original site report in 1986, in particular with respect to the likely soil stiffness properties that would have been recommended at the time of the design. I have sought this advice from the author of the original Soils & Foundations (1973) Limited report, Ian McCahon and it is now produced **[BUI.MAD249.0460.1]**.

33. Tonkin & Taylor has provided recommendations with regard to interpretation of results of the 22 February earthquake from various seismic recording devices. ARCL did not agree in general with the basis of the recommendations regarding probable seismic activity at the CTV site for the February earthquake. An expert report on seismic predictions has been provided to the Commission by Dr Brendan Bradley **[WIT.BRADLEY.0003]**.

34. Dr Bradley's evidence includes reference to seismic recording results from the CTV site which were obtained from equipment installed on the site by ARCL. The decision by ARCL to procure and install this equipment was

made based on a recommendation from Mr William Holmes (now an expert reporting to the Commission) that it was essential to record aftershocks at the specific CTV site for future analysis. I refer to Dr Bradley's evidence **[WIT.BRADLEY.003]**.

Spandrels Tolerance

35. The DBH Report refers to construction tolerances being utilised to enable the installation of spandrel panels with either limited or no gap between the end of the panel and the concrete column. In ARCL's report to the DBH on the draft DBH report, we stated that we did not consider that the construction would have been completed in this manner and that the specific gap would have been maintained. Our comment was not reflected in the final DBH report and this remains a concern.

36. I produce photographs of a building at 58 Kilmore Street, constructed by Williams Construction Canterbury Limited ("**Williams**") where there is good alignment of spandrel panels **[BUI.MAD249.0461.1, BUI.MAD249.0461.2]**. The photos illustrate the high standard of construction achieved by Williams on this project which includes the precast and insitu concrete. The concrete columns of the Kilmore Street building were tested by ARCL with a Schmidt hammer and the indicative concrete strengths were between 34.5 MPa and 41.4 MPa.

Destruction of Evidence

37. I have referred above to the destruction of the samples which were to be retained for further testing. Destruction of evidence also occurred when the remaining structures on site, following completion of the onsite investigation for the DBH Report, were demolished and taken to the Burwood site. ARCL has established the general location where this material is at the Burwood site.

38. I have particularly noted that no attempt appears to have been made to retain the sections of the remaining shear wall and floor elements that were intact after the collapse. Those elements might have been saw cut and transported to the Burwood site. Instead the shear wall and floor were demolished into small pieces for transportation off site and now most of the building is not specifically identifiable.

Change of Use

39. The CTV Building was designed as an office building with a live load of 2.5 KPa, with a seismic design live load of 0.83KPa and for a risk factor for buildings with normal occupancy of 1.0.

40. In 2001 a change of use application was made to the Council for a school to occupy level 2 of the building **[BUI.MAD249.0151C.29]**.

41. The live load requirement for a school under the relevant 1992 loading code was 3.0 KPa with a reduced seismic design live load of 1.8KPa. The seismic risk factor for the structure, based on Category 2, which includes school classroom buildings, was 1.2.

42. The change of use, together with the basic increase in the design lateral load coefficient for the building, resulted in a substantial change to the seismic and gravity loads for the building. It does not appear that there was the expected engineering review and reporting associated with the 2001 change of use.

43. The drawings indicate a possible occupancy of over 150 on the floor level **[BUI.MAD249.0151C.41 and BUI.MAD249.0151C.42]**, although it appears that actual occupancy of 126 was anticipated **[BUI.MAD249.0151C.40]**.

Collapse Considerations

45. There are at least five scenarios which have not been, in my opinion adequately considered in relation to potential collapse scenarios for the building.

Reinforcing Strain Hardening

46. The effect of strain hardening on the reinforcing steel has not been considered in the DBH Report. The issue arises from the impact of the 4 September 2010 earthquake, the 22 February 2011 earthquake and possibly the intervening aftershocks.

47. This significant structural issue was first noted at a seminar at the Art Gallery on Friday 1 April 2011 where comment was made that this issue would probably result in damaged reinforced concrete structures being significantly affected in terms of future seismic performance.

48. ARCL has subsequently found in several shear wall buildings in particular that the reinforcing steel has been subject to strain hardening, with the strain hardening being limited to a very short length of the reinforcing steel frequently in the order of 1 to 2 bar diameter. This is a significant reduction in the elongation necessary for the required performance of reinforced concrete to achieve code level assumptions. The degree of strain hardening varies but loss of capacity is of the order in some significant instances of over 40 – 50%.

49. I particularly refer to the impact of the strain hardening in the shear walls and floor diaphragms of the IRD Building, the building on the other side of Cashel Street from the CTV Building, where the strain hardening has resulted in the building having an assessed strength of between 30 and 40% of NBS. This building, if undamaged, would have a design code level strength of 100% of NBS (current code).

50. I note that the IRD Building complies with the strength requirements of the latest Building Code. It also complies with the requirements to use ductile reinforcing of the floor diaphragms. This has not prevented significant strain hardening damage to the floor diaphragm reinforcing.

51. These issues are unlikely now to be able to be investigated for the CTV Building due to the level of destruction of the original building structure.

52. The potential significant impact of this strain hardening on the CTV Building, where floor diaphragms may have been subject to reinforcing fracture and the shear walls could have been subject to a similar effect, could have potentially caused a materially different response of this structure to earthquake loading than that predicted by the analysis.

Vertical Acceleration

53. The vertical acceleration has been considered in the Tonkin & Taylor Geotechnical Report. It is probable that the vertical accelerations were very high particularly at this site, based on eye witness reports.

54. The effect of the high vertical accelerations is to result in significantly increased gravity loading on structural elements such as the beams supporting the floorslabs. I refer further to the evidence of John Mander **[WIT.MANDER.0001]** and Brendon Bradley **[WIT.BRADLEY.0003]**.

55. In my view the effects of high vertical accelerations have not been adequately accounted for the in the collapse analysis to date.

South Wall Lateral Load Resistance

56. The lateral load resistance of the southern wall is dependent on the gravity restoring force provided by the gravity loading of the floor. The effect of the vertical accelerations is to potentially increase or decrease this force. Should this force be substantially diminished, as could occur, at the same time as there is a significant seismic lateral load on the wall then the wall will tend to commence overturning and allow a significant rotation to occur in the south side of the building.

57. This issue does not appear to have been considered by other experts but in my opinion, collapse initiated in this manner is a highly feasible scenario.

Building modifications

58. My concerns in this respect relate to two issues:

(a) Beam Damage; and

(b) An internal staircase.

59. Evidence of drilling carried out on the concrete beams during the 1990's has been produced **[WIT.MORRIS.0001]**.

60. It appears that extensive drilling was carried out, including through beam reinforcing. The effect of 200mm dia holes near the column supports would be to cut through beam reinforcing and concrete which, together with the seismic vertical accelerations, could have resulted in beam shear failure.

61. Holes which cut the bottom beam reinforcing in the central region of the beam could have significantly reduced the load capacities of the beam, which could then have collapsed under the high vertical accelerations.

62. I was very concerned to hear about this practice, particularly the fact that the contractors were told to drill through the reinforcing bars.

63. In an earthquake with high vertical acceleration, such as the 22 February 2011 aftershock, the integrity of elements such as the beams becomes critical. If the main reinforcing fails, it could cause a catastrophic failure of the building, such as occurred on 22 February 2011.

64. In my opinion the possibility that the holes drilled in the concrete beams could have contributed to the collapse of the CTV Building ought to have been given considerable attention by the DBH and I am surprised that it appears to have been disregarded without investigation. I cannot rule out the possibility that the damage caused to the beams as a result of these holes contributed to or even caused the collapse.

65. I have noted from evidence presented by staff members of CTV **[WIT.JACKSON.0001]** and council files **[BUI.MAD249.0009]** that an internal stairwell was added between levels 1 and 2 of the CTV Building in 2000. A Building Consent application for these works and an associated fit-out was made in April 2000 **[BUI.MAD249.0009.57]** and a final Code Compliance Certificate was issued on 11 December 2000 **[BUI.MAD249.0009.1]**.

66. I have reviewed the Council file in relation to this Building Consent. I note that David Falloon of Falloon and Wilson Limited was engaged as structural engineer and Mr Falloon provided Producer Statements for the Design and for Construction Review **[BUI.MAD249.0009.71, BUI.MAD249.0009.37]**. Mr Fallon's Design Producer Statement is dated 26 April 2000. I note from correspondence on the Council file that this appears to predate preparation of of the structural drawings **[BUI.MAD249.0009.70, BUI.MAD249.0009.107, BUI.MAD249.0009.96]**. No structural drawings are referenced on the Producer Statement for Design as would usually be expected. The structural drawings dated May 2000 are on the Council file **[BUI.MAD249.0151B.2- BUI.MAD249.0151B.3]** but there is no structural assessment report. In the absence of the expected seismic structural review I am unable to assess the Engineer's opinion as to the impact of the installation on the CTV Building.

67. The DBH Report makes passing reference to the installation of the internal staircase. In my view, the authors ought to have assessed this issue further. According to the drawings, the staircase was installed by cutting through floors and I would be concerned about the potential effects of these works on the overall structure.

Cumulative Damage Resulting from Aftershocks

68. I have carried out numerous post-aftershock building inspections across Christchurch to assess for further damage to support occupancy or insurance assessments. I have noted on buildings such as the IRD

Building that the crack widths in structural elements such as shear walls have increased following ongoing aftershocks. The inspections have generally been carried out after aftershocks of greater than 5.0 magnitude. I have observed that cracks that were originally limited in extent and crack width have over time increased gradually in length, number of cracks and crack widths.

69. This change has occurred progressively as the aftershocks have occurred.

70. A similar effect has been noted on the beam column joints and it has also been noted at times that debris falls from the joint following the aftershocks.

71. I produce a schedule listing all major aftershocks (magnitude 4.9 or above) between the first earthquake at 4.35am on 4 September and the earthquake at 12.51 on 22 February 2011 **[BUI.MAD249.0462.1]**.

72. In my opinion, the ongoing sequence of aftershocks continues to cause cumulative damage to concrete reinforced buildings, each time reducing the capacity of the building to some extent. I believe that by the time of the 22 February earthquake, the CTV Building had lost part of its capacity as a result of not only the 4 September 2010 earthquake but all of these large ongoing aftershocks.

THIRD STATEMENT OF EVIDENCE OF ALAN MICHAEL REAY – EVIDENCE IN REPLY

Timothy Sinclair

64. In his initial report **[BUI.MAD249.0083.1]**, Mr Sinclair recommended that the REHS site should be disregarded for the purposes of analysing the CTV site response. In his latest report, Mr Sinclair now accepts that the REHS site is suitable for inclusion for the assessment of the CTV Building **[BUI.MAD249.0470.1].**

65. The ground accelerations recorded and the calculated acceleration response spectra at the REHS site are different to those used in the DBH Report on collapse scenario evaluation.

66. The collapse assessment of the CTV Building should now be reconsidered on the basis of these ground motion records.