WIT.REAY.0002.1

Under	**THE COMMISSIONS OF INQUIRY ACT 1908**
In the matter of the	**CANTERBURY EARTHQUAKES ROYAL COMMISSION OF INQUIRY INTO THE COLLAPSE OF THE CTV BUILDING**

SECOND STATEMENT OF EVIDENCE OF ALAN MICHAEL REAY

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SECOND 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.

2. I refer to my first statement of evidence dated 7 June 2012 for details of my qualifications and experience. I again confirm that I have read the Code of Conduct for expert witnesses and that my evidence complies with the Code's requirements.

Scope of evidence

3. My first statement of evidence covered factual issues relevant to the involvement of my firms (Alan M Reay, Consulting Engineer ("**ARCE**") and Alan Reay Consultants Limited ("**ARCL**")) in the design of the CTV Building and subsequent events.

4. In my second statement of evidence, I will cover the following issues:

   (a) Landsborough House and the similarities and differences between that building and the CTV Building;

   (b) The report prepared for the Department of Building and Housing by Hyland Consulting Limited and Structure Smith ("**DBH Report**");

   (c) Compliance of the CTV Building with the Code of the day;

   (d) Change of use;

   (e) Collapse considerations; and

   (f) Additional factual evidence.

Landsborough House

5. This building on the corner of Gloucester and Durham Streets was designed in 1985 by John Henry when he was with ARCE. Mr Henry discusses his design work on Landsborough House in his evidence **[WIT.HENRY.0001]**. He makes observations about the design of Landsborough House and states that parts of its design were utilised in the work Mr Harding did in designing the CTV Building, including that Mr Harding adopted many of Mr Henry's calculations for Landsborough House when designing the CTV Building.

6. I have reviewed the structural drawings and calculations for the Landsborough House Building. The design was generally similar to the CTV Building. I comment on some particular features, and how they compare to the CTV Building below.

Height

7. Landsborough House was eight storeys and the CTV Building six storeys. The CTV Building had 40% larger floor plates.

Asymmetry

8. The design asymmetry of the buildings is similar, with the Landsborough House Building being slightly more asymmetric.

9. The beam column structures of both buildings are designed as secondary structural elements in terms of NZS3101:1982.

Construction

10. Both buildings used pre cast concrete beams, insitu concrete columns and shear walls for lateral earthquake loads including coupled shear walls with diagonal reinforcing.

11. The buildings differed in the location of the coupled shear walls, or wall in the case of the Landsborough House Building. The coupled shear wall was an integral part of the core shear wall system to Landsborough House, but the coupled wall was a separate wall on the south side opposite the side of the northern shear wall system for the CTV Building.

Suspended Floors

12. The suspended floors differ in that the CTV Building has a metal Bondeck floor with 664 mesh and the Landsborough House Building has a concrete rib and infill floor with the topping slab reinforced with 665 mesh. The CTV Building has 28% more mesh per square metre within the mesh reinforced floor than the Landsborough House Building.

Columns

13. The reinforcement of the columns varied. Neither was designed for additional requirements of ductile seismic detailing under NZS 3101:1982. The columns in the Landsborough House Building were generally 400mm square with ties of 10mm at 150 or 250mm centres generally. The CTV

columns were 400mm circular columns with 6mm ties at 250mm centres. The vertical reinforcing laps were 900mm for the Landsborough House Building and 1200mm for the CTV Building.

Foundations

14. The foundations differed in that the Landsborough House Building was supported on piles, and the CTV Building was based on shallow footing foundations.

Landsborough House Earthquake Damage

15. I inspected Landsborough House on 16 May 2012. Damage to the structure that was evident on this date was as follows:

(a) **Shear Walls**: There was minor cracking to the shear walls surrounding the shear core.

(b) **Coupling Beams**: The coupling beams, particularly at the lower levels, had substantial damage and would have introduced a significant degree of flexibility on the coupling beam shear wall line of the building.

(c) **Stairs**: There was no significant damage to the stairs or supporting system.

(d) **Columns**: There was some limited movement between the column and the beam soffit interface at the top of the column with some limited initiation of column cover concrete spalling.

(e) **Beam Column Joints**: There was no evidence of cracking or joint failure.

(f) **Floor Diagram**: There was no cracking to the floor diagram in the immediate area where the floor was adjacent to the shear wall system.

(g) **Foundation Levels**: I understand that the differential settlement of the foundations is approximately 100mm.

16. Based on my inspection, the Landsborough House building has performed satisfactorily with no significant damage to the columns, beam column joints or stairs.

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]**.

CTV Building Compliance

44. I have been asked by the Royal Commission to express an opinion on the compliance of the CTV Building with the Code of the day. My opinion on the compliance of the CTV Building is as follows:

(a) **At time of Building Permit application and issue:**

With the passage of time there is no certainty as to the documentation used for the permit application. Equally there is also no certainty as to the documentation issued to the building contractor with the Building Permit. It is therefore not possible to definitively state whether the building documentation complied with the Building Code/Christchurch City Council ("**CCC**") bylaws at that time.

Based on the fact that the CCC issued the Building Permit for the building, the CCC must have considered that the building complied with the relevant codes/bylaws at that time.

(b) **When constructed:**

When constructed by Williams the building did not include the additional ties installed between the shear wall and the floor diaphragms in 1991. The level of existing compliance was not reviewed by ARCL prior to the design and installation of the additional ties.

(c) Following the 1991 additional work.

Holmes Consulting Group Limited ("**Holmes**") noted in its January 1990 report:

"*The layout and design of the building is quite simple and straight forward and generally complies with current design loading and materials codes.*" (clause 3.0 (2))

Based on the Holmes report, together with the work carried out as designed by ARCL, it was my opinion that the building complied with the code and bylaws following completion of the 1991 retrofit works.

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.

Additional factual evidence

73. As a result of locating additional records in historic files held by ARCL, I wish to give some supplementary factual evidence.

74. I have located the ARCE time records from the time of the CTV Building project. I produce these records **[BUI.MAD249.0463.1]**. The CTV Building was job number 2503. Other job numbers, names of staff that are not involved in this hearing and totals have been redacted. The schedule **[BUI.MAD249.0463A.1]** summarises the time spent by various staff members on the project.

Dated this 19 day of June 2012

A M Reay