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**

**STATEMENT OF EVIDENCE OF MICHAEL JOHN NIGEL PRIESTLEY
IN RELATION TO THE CTV BUILDING**

DATE OF HEARING: COMMENCING 25 JUNE 2012

STATEMENT OF EVIDENCE OF MICHAEL JOHN NIGEL PRIESTLEY

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1. My full name is Michael John Nigel Priestley
2. I have been involved in seismic performance of structures for more than 45 years, since completion of my PhD in 1966. My career has included periods as Head of the (then) Ministry of Works Central Laboratory (1966-1976), Faculty member at the University of Canterbury (1976-1986, first as Senior Lecturer, then as Reader), Professor of Structural Engineering at the University of California, San Diego (1986-2000), Co-Director of the European School for Graduate Studies in Reduction of Seismic Risk (The "Rose" School: 2001-2009), and as Structural Consultant to various organizations throughout my professional career. I hold emeritus status at UCSD and the "Rose" School.
3. I have been active in research, authoring or co-authoring more than 700 articles and reports, mainly related to seismic design or performance of structures, including three major reference books on seismic design of structures. I have received more than 30 awards, prizes or recognitions of my research.

EVIDENCE

4. I will address the following topics in my evidence:
    a. My involvement with the Department of Building and Housing (DBH) Expert Panel.
    b. My areas of agreement with the DBH consultants' report and the Expert Panel report on the CTV Building.
    c. My areas of disagreement with the reports.
    d. Whether the design of the CTV Building met best practice of the time.

e. The state of the CTV Building following the 4 September 2010 earthquake.

f. The causes and likely sequence of collapse of the CTV Building on 22 February 2011.

5. I have read the Code of Conduct for Expert Witnesses and agree to comply with it. I confirm that all of the matters to be addressed in my evidence are within my areas of expertise.

THE DBH EXPERT PANEL

6. I was appointed Deputy Chair of the Expert Panel assembled by DBH to investigate causes of damage or failure in the February 2011 earthquake of four buildings of major public interest. These buildings included the CTV building.

7. The Panel interacted with the consultants appointed by DBH, providing guidance and assessment of the work carried out by the consultants. This was a continual process through the development of the consultants' reports, and included additional analyses, where appropriate, and extensive comments and questions on the final report prepared by the consultants.

8. Aspects of the consultants' report were included in the summary report prepared by the Expert Panel. In some cases the emphasis in the Expert Panel report differed to a greater or lesser degree from that in the consultant's report. Comments from the Expert Panel were essentially the views of individual Panel members, whereas the Panel's report was more of a consensus document, which was required by the Panel's Terms of Reference to endorse the consultant's report. Inevitably, some of the comments or views of individual Panel members could not be included in the Panel's final report.

9. In the case of the consultants' report prepared by Dr Clark Hyland and Ashley Smith on the CTV building collapse (the CTV report), there was some difficulty in reconciling the views of the consultants and members of the Expert Panel. This is referred to in the Expert Panel Report at page 34, paragraph 4 [**BUI.VAR.0056.37**], which notes that although the Panel supported the general conclusions reached by the consultants on the reasons for the collapse, some members did not fully agree with the conclusions reached in the consultants' report on the identification of critical columns, the influence of

the spandrels and when in the collapse sequence the separation of the floor slabs and the North Core occurred. I was one of the Panel members who did not agree with all conclusions the consultants had reached on these issues.

10. Because of my known disagreement with the CTV report on several significant issues, and because my views also diverge from the Expert Panel report in some areas, I have been asked by the Royal Commission to give evidence to it in a personal capacity and not in my capacity as the Deputy Chair of the Expert Panel.

11. I will endeavour to present my evidence in a way that minimises the use of what are often the highly technical terms used by structural and earthquake engineers. However, this cannot be entirely avoided. I also understand that my evidence is expected to come after the evidence from the consultants and the Panel representatives and a number of these terms are likely to have been explained by them.

12. My evidence sets out and explains the differences in opinion between my personal view of issues leading to the collapse of the CTV building and the opinions expressed in the CTV report, and to a lesser extent, in the Expert Panel (Panel) Report. My views were all presented to the consultants, who have considered them when writing their final report. However, although I am pleased with changes in emphasis included in the Panel report I am not satisfied that all my concerns have been adequately resolved in either the CTV report or the Panel report.

13. The purpose of my evidence to the Royal Commission is to provide a clearer statement of my view on alternative collapse possibilities than was possible in the Panel report.

AREAS OF AGREEMENT

14. Before dealing with these issues, however, it is important that I note that there is much in the two reports with which I agree. This is particularly the case with the Panel report. Because my evidence deliberately concentrates on those areas where I disagree with either or both of the CTV report and the Panel report it could give a misleading impression of my view of the bigger picture conveyed in the reports, particularly in the Panel report.

15. The areas where I have little or no disagreement with the reports are:

a. Deficiencies in the design of the CTV Building when considered against the code and best practice at the time (as described in Section 9 of the CTV Report [**BUI.MAD249.0189.139**] and page 50 of the Panel Report [**BUI.VAR.0056.52**]).

b. The presence of critical vulnerabilities in the design drawings in relation to:

   i. Columns (as referred to on page 35 of the Panel Report [**BUI.VAR.0056.37**]).

   ii. Irregularities/lack of symmetry (as referred to on page 37 of the Panel Report [**BUI.VAR.0056.39**]).

   iii. The diaphragm connection (as referred to on page 38 of the Panel Report [**BUI.VAR.0056.40**]).

c. The general conclusions as to the factors contributing to the collapse of the CTV Building as described under Section 5.13 of the Panel Report, although I disagree about the extent of the contribution of each factor and the likely failure sequence.

d. The recommendations (Section 11 of the CTV Report [**BUI.MAD249.0189.153**] and Section 5.14 of the Panel Report [**BUI.VAR.0056.55**]).

AREAS OF DISAGREEMENT

16. My concerns with the CTV report are: [**Figure 1 / BUI.MAD249.0402.1**]

a. Undue and inappropriate reliance on ERSA for assessing performance of the CTV building in the September 4, 2010 and February 22, 2011 earthquakes.

b. Misleading information about the design seismic intensity for the CTV building.

c. Excessive emphasis on the role of spandrel panels on the East and South Faces of the CTV building in initiating the failure. [**Figure 2 / BUI.MAD249.0402.2**]

d. The view that exterior columns were more likely than interior columns to initiate building failure.

e. Rejection of connection failure between the floor diaphragms and the North structural core as a high-probability failure initiator.

f. Excessive emphasis on torsional eccentricity, based on ERSA analysis.

g. The modeling of masonry infill panels on A-line and the assessment of the effect of the infill on building performance.

h. Reluctance to accept the results of the NTHA where these did not agree with the consultants' view of the collapse sequence.

i. The methodology for the "displacement compatibility" analysis used in the consultant's Appendix F to the CTV report.

17. My evidence now addresses these issues in more detail.

a. Reliance on Elastic Response Spectrum Analysis [Figure 3 / BUI.MAD249.0402.3]

18. Some of the significant areas of disagreement stem from the reliance the CTV report has placed on Elastic Response Spectrum Analysis (ERSA). The initial analyses and development of the consultants' view of factors leading to collapse of the CTV building, were based on ERSA analyses and eye-witness accounts. It is my firm opinion that ERSA was unsuitable for determining the causes and sequence of the CTV building collapse. It is a design method intended for determining the required strengths of structural members to satisfy code-specified seismic input. It is not suitable for determining the expected response when assessing a building.

19. It was because of the concerns I and other Panel members had about the reliance the consultants had placed on this analysis that the Panel required a Non Linear Time History Analysis (NTHA) was carried out. This produced some significantly different results that have not been adequately addressed in the CTV report or, to a lesser extent, in the Panel report.

20. The unsuitability of ERSA for determining the collapse issues is clearly stated in the Recommendations of the NZ Society for Earthquake Engineering (NZSEE) for "Assessment and Improvement of the Seismic Performance of Buildings in Earthquakes", 2006, Section 4.3.2(b) (ii) "Modal Response Spectrum Analysis (EMA)" **[ENG.NZSEE.0014A]** which recommends against use of ERSA for building

assessment where inelastic response is expected. EMA and ERSA are different terminologies for the same procedure.

21. Problems identified by the NZSEE Recommendations with the use of ERSA for building assessment include:

    a. Inadequacy in determining member inelastic deformation.

    b. Underestimation of higher-mode effects when inelastic response is expected.

    c. Overestimation of torsional response.

22. All of these considerations were present with the CTV building.

23. By way of a very brief explanation of what I am referring to:

    a. 'Member' refers to the various structural elements of a building, for example, the beams and columns.

    b. "Elastic deformation" is deformation, or displacement that will disappear if the forces causing the deformation are removed **[Figure 4 / BUI.MAD249.0402.4]**, while "inelastic deformation is the additional deformation or deflection that will remain after the forces are removed. Under low-level seismic excitation the building deformation is essentially elastic. Under higher levels of seismic excitation the response may be inelastic. Inelastic deformation implies damage to the structure. This is expected under high levels of seismic excitation.

    c. 'Mode effects' refer to the way a building vibrates in an earthquake. This can change during the course of an earthquake. Buildings respond differently depending on the size and shape of the building and the nature of the ground shaking. **[Figure 5 / BUI.MAD249.0402.5]** The building responds in a combination of the various possible modes of vibration, and the characteristics of the modes change if the building responds inelastically to the earthquake. The first or fundamental mode typically provides the largest part of the building response to an earthquake.

    d. 'Torsional response' refers to the twisting effect on a building and/or elements of a building. It is greater in buildings that are eccentric, in other words buildings where the strength and stiffness of the lateral-force resisting members (walls, beams and columns) are not symmetrically located on the building plan. In the case of the CTV building the principal lateral-force resisting element was the North Shear core and this was at the extreme north edge of the building and outside the building envelope.

24. In addition to the problems with ERSA identified by the NZSEE Recommendations, the important fact could be added that ERSA provides only an approximate estimate of the peak elastic response of structural actions (e.g. member forces or displacements) over the duration of the seismic input. It cannot predict inelastic deformations. It cannot determine when these peak response levels occur in the structural elements.

25. In fact, different actions may peak at different times in the seismic response. This is particularly important when interpreting the response from the peak levels. For example, the peak floor diaphragm inertia forces are not normally provided in the computer output from an ERSA analysis. At any instant of the response, they can be determined from the storey shear forces in the members above and below the floor diaphragm being considered. However, the peak floor diaphragm inertia forces during the seismic response cannot be determined from the peak predicted storey shear forces above and below the floor under consideration, since it is unknown, and unlikely, that the peak storey shears above the floor and below the floor will occur at the same instant in the building response

26. Further, ERSA is particularly inappropriate when used with recorded response spectra rather than with smoothed design spectra. This is because actual response spectra are typically very irregular, with large changes in spectral amplitude for very small changes in effective period [**Figure 6/ BUI.MAD249.0402.6**]. The ERSA assumes that the periods associated with the various modes of response stay constant throughout the building response, whereas if inelastic deformation occurs, the periods lengthen. The effects may not be significant when related to the design spectrum, but can result in very large errors with “real” spectra, as the spectral ordinate may increase or decrease. Also, small errors in calculated periods have a much greater effect on predicted response of a building to a real spectrum than to a smoothed design spectrum.

27. As a consequence of these problems with ERSA, and the great importance of obtaining an analysis of the CTV building that could be relied on in determining the collapse

scenario, the Panel required the consultants to carry out a NTHA. This decision was made after the Panel had received the first draft of the CTV report. If I recall correctly, this was during a meeting of the Panel in August 2011.

28. As the Consultants did not have expertise in NTHA a decision was made to employ Compusoft, a consulting firm specialising in high-level structural analyses, including NTHA. Interaction between the consultants and Compusoft in preparing the necessary input data and in interpreting results was provided primarily by Ashley Smith of StructureSmith, though considerable input was also provided by Panel members, including myself.

29. The advantages of a NTHA over ERSA include: [**Figure 7/ BUI.MAD249.0402.7**]

    a. Direct computation of inelastic deformations.

    b. Output of time-dependent values of critical results, such as the forces applied to the members and the resulting deformation of those members.

    c. Direct determination of which member or members first reach failure, thus providing information of where failure might have initiated.

    d. Ability to impose orthogonal seismic accelerograms (i.e. recordings of the ground shaking in the two horizontal and the vertical directions) simultaneously, in other words to feed in the assumed actual effects of the ground shaking on the building, with that shaking occurring from various directions.

30. As with all analytical approaches there are limitations to what can be achieved with NTHA. These include: **[Figure 8 / BUI.MAD249.0402.8**]

    a. Limited ability to model strength degradation during a failure scenario.

    b. Limited ability to model shear failure in concrete members.

    c. Limited ability to model interaction between different actions in a member, such as moment and shear. 'Moment' refers to the demands which lead to bending. 'Shear' refers to the demands tending to cause diagonal cracking [**Figure 9 / BUI.MAD249.0402.9**]

    d. Problems with modeling elastic and hysteretic damping. 'Damping' refers to the absorption of energy in a building, which leads to a reduction in vibrations.

e. In some cases, excessive computational time, because of the complexity of analysis and the large number (many millions) of equations that must be solved. This can lead to instability in the computation, making solution impossible.

31. Results from the NTHA differed in many cases from the conclusions the consultants drew from the ERSA. However, 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.

32. The principal points of divergence between the results of the two analyses, and those where the consultants' decision not to revise their conclusions cause me the greatest concern, are:

    a. Importance of diaphragm disconnection from the North Core as a potential failure initiator.

    b. Conclusion that the exterior columns on line F were more critical than internal columns

    c. The significance of masonry infill on line A

    d. The potential for beam/column joint failure.

33. The validity of the ERSA analyses using the simplified response spectra adopted by the consultants is further placed in doubt by the modeling of member stiffness based on recommendations included in NZS4203:1984 and NZS3101:1982 [**BUI.MAD249.0189.267**], rather than state-of-the-art appropriate in 2011.

34. The consultants' choice of member stiffness was appropriate for estimating compliance in 1986 with code displacement and strength-distribution requirements. However it was inappropriate for estimating the response of the designed structure to the recorded seismic excitations of September 2010, December 2010 or February 2011. Here the aim of the investigation was to determine as accurately as possible what had actually happened and the best state of the art information about how to model member stiffness should have been used.

b. Design Seismic Intensity

35. The 1984 design spectrum used for the purpose of designing the CTV building, compared with the actual spectrum calculated from the recorded accelerograms referred to in Figure 11 of the CTV report (copied as Figure 5.9 in the Panel report) [**BUI.MAD249.0189.50**], has a number of inconsistencies with material presented elsewhere in the reports. If these figures are compared with Figure 4.10 of the Panel report, which purports to show the same information, significant differences are apparent [**Figure 10/ BUI.MAD249.0402.10**].

36. First, the 1984 design intensity in the CTV report Figure 11 has an initial plateau at 0.88g, which is 75% higher than that shown in the Panel report Figure 4.10. The latter is the value shown in NZS4203:1984. This scaling factor of 1.75 (corresponding to the increase of 75%) has been applied to the entire spectrum.

37. The reason for this is that NZS4203:1984 required that when ERSA was used for design, the base shear (which refers to the design lateral force from the earthquake ground motion at the base of the structure) should not be less than 90% of the value resulting from an equivalent lateral force (single mode) analysis. To briefly clarify this, a single mode analysis is a simplified approach, which assumes that 100% of the building mass is associated with the fundamental mode of vibration. In an ERSA, the response is found by calculating the response of the various different modes, with an appropriate proportion of the total building mass, and combining these modal responses in accordance with procedures specified by the design code. This generally meant that it was necessary to increase the base shear that had been calculated from the ERSA because ERSA usually produced base shear results that were less than this 90% figure.

38. In the case of CTV, the “scaling factor” calculated by the consultants was about 1.75, presumably for the EW direction of response. I would expect a different, and much lower, scaling factor to apply for NS response as a consequence of the greater regularity, and the reduced torsional response but this is not discussed in the CTV report. A scaling factor of 1.75 is unusually high, implying that the base shear calculated from the lateral force approach, using the same member stiffness as used in the ERSA analysis, was about 95% higher than that from the ERSA analysis.

39. Compounding my concerns about the correctness of the scaling factor used by the consultants, the CTV report then applies this same scaling factor to the design response spectrum (Figure 11 in the CTV report [**BUI.MAD249.0189.50**]). This is incorrect. It should only have been applied to the base shear and not to the design spectrum. The scaling of the spectrum implies that the design spectrum is a function of the building's characteristics, which it is not. At best I find this confusing.

40. What in my view is an inappropriate scaling of the design spectrum also results in a false comparison with the recorded earthquake intensity, implying that the recorded intensity was not much higher than the design intensity.

41. Second, the spectrum representing the average Feb 22, 2011 response in the CTV report (Figure11 [**BUI.MAD249.0189.50**]) differs from that in the Panel report (Figure 4.10 [**BUI.VAR.0056.25**]). The reason appears to be that a different set of accelerograms was used for the consultants' ERSA studies than the generally accepted four accelerogram sets described in the Panel's report. The accelerograms are the recordings made by accelerographs, the instruments that were used to measure the intensity of the ground shaking. Four sites were initially selected as those most likely to represent the ground shaking Christchurch CBD and agreed by the Expert Panel.

42. The spectra used for the consultants' ERSA studies also differ from the spectra set used in the consultants' NTHA studies. In Appendix E, (ERSA) p233 of the CTV Report [**BUI.MAD249.0189.263**], it is noted that the response spectrum used for ERSA was the average of the Westpac Building, the CCCC, the CHHC and the Police Station accelerograms. In Appendix D (NLTHA), Table 8, p207 [**BUI.MAD249.0189.237**], it is identified that the NTHA used the CCCC, CHHC and CBGS records. Note that the Westpac Building and Police Station accelerograms are not included in the Context report, summarized in Section 4 of the Expert Panel Report [**BUI.VAR.0056.17**]. Finally, based on correspondence and conversation with Dr. Hyland, I understand that the consultant's ERSA spectrum (Figure 11 [**BUI.MAD249.0189.50**]) is the average, for a given period, of the maximum ground shaking from both components of the four accelerogram records, regardless of the direction of the ground shaking. The spectrum, so calculated, was then applied independently, to both the NS and EW directions. It appears that no combination of EW and NS response was attempted, despite the

evidence from ERSA and to lesser extent NHTA showing that the torsional eccentricity of the CTV building resulted in significant NS response from EW excitation.

c. Role of the Spandrel Panels on East and South faces on the CTV Collapse

43. In an earlier draft of the CTV report interaction between the spandrels and the columns was stated as the definitive cause of column failure, in other words the initiating action in the collapse. This is still the only mechanism discussed in detail in both the CTV report and the Panel report, with a series of figures describing the sequence of failure (Panel report Figures 5.14 to 5.16 [**BUI.VAR.0056.50-51**]; CTV report Figures 17 to 19 [**BUI.MAD249.0189.65-67**]). These figures show column hinging at or above the top of the spandrel panels. [**Figure 11 / BUI.MAD249.0402.11**]; **Figure 12 / BUI.MAD249.0402.12**]

44. I do not agree with this conclusion. My calculations indicate that it is unlikely for a number of reasons:

  a. The permit drawings show an intended gap of 10mm between the columns and spandrels. If as-built this gap existed, the interstorey drift prior to contact with the spandrels would have been sufficient to cause the columns to fail without any influence of spandrel/column contact.

  b. The analyses in the CTV report supporting this mechanism of collapse were based on a zero gap between columns and spandrel.

  c. The strength and stiffness of the spandrel/column contact were assumed in the CTV report to be infinite, meaning that if contact between a column and a spandrel occurred there would be no deformation of the spandrels or of the connections of the spandrels to the beams, and that there would be no failure of the spandrels, or the connections prior to the column failure.

  d. My calculations on the strength of the connections between the spandrels and the supporting beams indicate they would fail at a level of column/spandrel contact force about 20% of that required to induce hinging at the top of the spandrel.

  e. My calculations for the capacity of the end diaphragm of the spandrels to transmit the column/spandrel contact force, using yield-line theory, indicated an even lower capacity than the value corresponding to the connection capacity. [**Figure 13 / BUI.MAD249.0402.13**]) This means that the end diaphragm of the spandrel would fail before significant force could be transmitted at the point of contact between the column and spandrel, if this contact occurred.

f. Photos of spandrels on the ground after the collapse indicated failure to the end diaphragm, consistent with what the yield-line theory would predict. [**Figure 14 / BUI.MAD249.0402.14**] In this figure the end diaphragm has been completely broken off from the spandrel, and the small portion remaining is consistent with the predicted failure pattern.

g. I have seen no evidence of column failure occurring at the level corresponding to the top of diaphragm.

h. Photos of column failures tend to indicate failure at the top of splices, probably due to high vertical compression force. The moment at the top of the column bar splice (the location implied by CTV Figure18 [**BUI.MAD249.0189.66**]) is much lower than at the top of the spandrel and as a result is not a critical location for formation of a plastic hinge. However, it is a weak location for a column compression failure. [**Figure 15 / BUI.MAD249.0402.15**]

i. The CTV report Figure 17 [**BUI.MAD249.0189.65**] shows a plastic hinge forming at the top of the spandrel. However, Figure 18 [**BUI.MAD249.0189.66**] shows the hinge forming some distance above the top of the spandrel. The Panel report does not refer to, or include CTV Figure 18.

j. The CTV report refers to a Line F column failure, probably induced by spandrel contact, as the preferred collapse initiator. However:

i. My calculations indicate that failure of the F line columns would not result in sufficient load transfer to overload the E line columns to the extent necessary to cause compression failure. These calculations show that the beams connecting lines E and F (including full contribution from the slabs between columns) would fail as cantilevers at less than 60% of the full axial force on line F columns. As a result the full axial force on Line F could not be transmitted to line E.

ii. If the line F columns had failed this would, in my view, have resulted in a different pattern of failure to that which is known to have occurred – possibly protecting the interior of the building from failure and leaving more cavities and voids in the building that might have provided safe areas for occupants of the building. It would not have led to the almost complete concertina effect that occurred, without other collapse initiators occurring.

iii. I also note that the columns on line E have lower axial load due to gravity effects than columns on line D, which, as I note later, are in my view more likely to have acted as a collapse initiator than the line F columns. The comparatively low gravity load on the E line columns made them less susceptible to failure due to the limited possible load transfer from the F-line columns.

45. The Hyland thesis involves collapse as a result of an EW lurch and identified the spandrel–column interaction as the initiating event. Based on the ERSA, this EW lurch involved high torsional response, with high displacements of F-line columns in the NS direction as a consequence. The NTHA did not predict such high torsional response. However, a combination of EW and NS response is predicted by the NTHA to induce significant NS response of line F.

d. Exterior Columns vs Interior Columns as Collapse Initiator [**Figure 16 / BUI.MAD249.0402.13**]

46. Analysis results (CTV report Appendix D; Panel Report, Figure 5.13 [**BUI.VAR.0056.47**]) indicate that drift demand/capacity ratios at level 3 were higher for internal columns D2 than for the column F2 that the CTV report identifies as the initiator column. The estimated demand drift is listed in CTV Tables 1 and 2 as 1.9% for both line F (exterior) and Line D (interior) columns. Under gravity loads the drift capacity from Panel Figure 5.13 is 1.30% for Column F2 and 1.09% for Column D2. These drift capacities are taken from Figure 5.13, and differ from those listed in CTV Tables 1 and 2.

47. The resulting demand/capacity ratios are thus 1.46 and 1.74 for F2 and D2 respectively. If maximum vertical acceleration effects are included, the difference between the F2 and D2 demand/capacity ratios increases. For F2 the demand/capacity ratio increases to

1.65, but the D2 value increases to 2.44. Because of the high gravity loads on the interior columns they were also more vulnerable to low concrete compression strengths. I am aware that some of the evidence the Royal Commission will hear is critical of findings made in the CTV Report about concrete strengths. The point I am making here is that, if concrete compression strength was low, it would have made the interior columns more vulnerable to failure.

48. The demand drift for D2 is based on the CHHC record, whereas the demand drift for F2 is based on the CBGS record. The NTHA results indicate that the CCCC record is more critical for EW response [**Figure 17 / BUI.MAD249.0402.17**], and that Line 1 would be more critical than line F for exterior column response. These points reduce the probability that failure of the F line columns was the critical collapse initiator.

49. Both the CTV report and the Panel report imply that line 1 was less critical than line F (CTV report p95 "Critical Column Identification, first paragraph, final sentence [**BUI.MAD249.0189.125**]), though the reasoning is not clear to me and I do not agree with it.

50. If an interior column failed it would tend to induce lateral catenary action, as a result pulling other columns towards it. Very little displacement would be required (only 30-50mm) to cause failure of the adjacent columns. Failure of the F line columns would not result in lateral catenary forces to other columns. This is because the F line columns are exterior columns and the horizontal forces required for catenary action cannot be sustained in the EW direction,

51. Catenary action is like a chain.[**Figure 18 / BUI. BUI.MAD249.0402.17**]. To support a chain vertical forces and horizontal forces are needed at both ends of the chain. If one of the end supports is not available, the chain collapses. If (say) column D2 failed, the necessary supports for catenary action would be provided by the columns at E2 and C2, inducing horizontal forces to these columns pulling them towards D2. If an F line column failed, the horizontal forces necessary for catenary action cannot develop, since there is no column outside the building to support the necessary horizontal force. Hence failure of an F line column does not provide horizontal forces on columns on line E.

52. These results indicate that the interior columns were significantly more vulnerable to failure than the exterior columns, particularly when the effects of the recorded high vertical

accelerations are considered. This conclusion differs from that in the CTV report, but agrees with the conclusions reached in the statement of evidence of Graham Frost, which I have read. Mr Frost is a CPEng who acted as risk manager for the USAR and Police teams during rescue and recovery work, immediately following the CTV collapse. His interpretation of the collapsed state of the building was that collapse of the floor and beam elements started near interior columns.

e. Connection Failure Between Floor Diaphragm and North Core [Figure 19 / BUI.MAD249.0402.19].

53. The connection between the floor slabs and the North core as designed and permitted was clearly inadequate to achieve a sufficient connection. It should be noted that most of the weight of a building comes from the floor slabs. As a consequence, most of the forces induced by an earthquake on a building result from the floor slabs. For these forces to be resisted by a building, they have to be transferred to the lateral force-resisting members – in the case of the CTV building this is primarily the North core. This force transfer involves high tension and compression forces. If the connection between the floor and the lateral-force resisting members does not have sufficient strength (particularly tension strength), the connection will fail, and the earthquake forces cannot be resisted. The consequence is greatly increased displacements of the floor, including the columns supporting the floor, making structural failure more probable.

54. In the case of a building supported by frames designed to provide the lateral resistance of the building, the connection between the floors and the columns is rather straight forward as it occurs at many locations. In the case of the CTV building, the connection is primarily to the webs of the North core in very localized positions. These positions where connections were made thus needed to be designed for very high tension forces. This high tension force capacity was not provided. In particular, the lack of designed connection between the floor slabs and the North core webs on Lines D and D/E was remarkable. These locations had essentially no tension capacity, and would fail under comparatively minor seismic loading, placing additional demand on the connection between the floor slabs and the North Core between lines C and C/D. The eccentricity of the connection as designed was particularly susceptible to failure under EW response, due to the eccentricity of the lateral forces from the North core. However, eccentricity

was also present in the NS direction, and would have been exacerbated by failure of the connections of the floor slab to the North core at lines D and D/E

55. This serious inadequacy was noted some time after construction by Holmes Consulting Group and partially remedied by Alan Reay Consultants installation of drag bars, but for reasons that are not clear to me these were only installed on the higher floors. They were not installed on levels 2 and 3. This design inadequacy has been fully discussed in the CTV report.

56. Without drag bars installed at levels 2 and 3, the eccentricity discussed earlier would have been present from the start of the earthquake response, placing additional tension forces on the slab connection at lines C and C/D as previously discussed.

57. In my view, drag bars, as designed for the CTV connection retrofit, were a poor alternative to a properly designed connection involving a greater contact area between the floor and the webs of the North Core. The method of connection, requiring overhead grouting, would have been very susceptible to instalment problems.

58. NTHA results indicate that failure of the drag bars between the floor diaphragm and the North Core on Lines D and D/E would occur before column failure [**Figure 20 / BUI.MAD249.0402.20**].

59. The calculated capacity of the drag bars that the CTV report assigns to them is in my view unrealistically high. It is based on the shear capacity of the bolts connecting the drag bars to the floor. This implies perfect placement of the epoxy in the pre-drilled bolt holes, and infinitely strong concrete surrounding the bolts. Calculations based on a concrete compression strength of twice the actual recorded concrete strength in the building indicate the bolts would fail in flexure, not shear, at about 50-60% of the strength used in these figures, implying still earlier failure in fact.

60. Similarly, the NTHA results predict that the floor diaphragm failure adjacent to walls on lines C and C/D would occur prior to column failure. The critical section is about 1.2m away from the end of the walls, where the H12 bars terminate and all of the strength is provided by the mesh. The floor would almost certainly have been cracked at this location by shrinkage effects, but, if not, based on the NTHA results the uplift of the wall

ends due to flexure during response displacements to the North would have been sufficient to crack the concrete at the termination of the H12 saddle bars.

61. My own calculations indicate that the resistance to slab fracture adjacent to the North core flange on line C, up to midway between lines C and C/D, and including dowel action of the anchorage bars of the beams on grid 4 into Wall C, would be about 800kN. Figure 21 [**BUI.MAD249.0402.21**] shows the Compusoft NTHA results for the connection force between the North core and Wall C. Maximum tension forces exceeding 2000kN (2.5 times the calculated slab capacity) are shown to have occured frequently.

62. I have not included any contribution of the Hi-bond steel trays to the tensile capacity of the slab connection, as the Hi-bond was discontinuous at the beam support on line 4 with very short seating. In addition, the statement of evidence of Graham Frost reports that he observed total bond failure between the slab and the Hi-bond. He statement says that he found “Not a single section of slab to which the tray deck was still attached”. If either the drag bars at the upper levels, or the concrete slab, failed, the displacements of the floor under inertia effects would have been greatly increased as discussed earlier. I note that the Hi-Bond trays, and the supporting beams, would have provided adequate vertical support for the floor until the columns failed due to excessive displacement. Thus under a moderate earthquake such as the Sept 4, 2010 earthquake, the displacement might not fail, unless the maximum response displacement exceeded the available seating length of about 400mm.

f. Torsional Eccentricity

63. The CTV report, based on ERSA studies, emphasizes the high torsional eccentricity of the building and the exaggeration of this eccentricity by the concrete infill wall at the lower levels of the West frame (Frame A) of the building.

64. The prediction made in the CTV report, based on ERSA, was that displacements on Line F would be increased by the effect of infill on line A. The NTHA results indicated the opposite trend. I again note the advice in the NZSEE Guidelines that ERSA is inappropriate for assessing the response of buildings, particularly when torsional eccentricity exists.

g. Modelling of Masonry Infill on Line A [BUI.MAD249.0222.2]

65. I also believe that the influence of the western masonry infill wall on Line A has been exaggerated in the ERSA. The design drawings for the building specified a gap between the panels, and between panels and columns, and the use of flexible sealant in these gaps. However, on the basis of a recollection by a Mr Fortune, one of the two men working on a scissor lift on the west side of the CTV building on 22 February, it is claimed in the CTV report that the infill panels (three per bay) were constructed with full contact between the panels and the columns and with full contact between the panels. This is despite the fact that it would have been very difficult to place competent mortar to the outside header joints adjacent to the columns from the inside of the building, particularly for the upper courses of blockwork. At the time the CTV building was constructed there was an adjacent building that made access to the outside of the wall impossible.

66. The consultants and the Panel also had available to them a statement by Mr Coatsworth, the engineer who carried out a post September inspection of the building, that the required flexible sealant was correctly placed on the inside header joints. In the course of preparing my evidence I have also been advised by Counsel Assisting that there is now another witness who says there were gaps at the top of the concrete sections, under each of the beams.

67. My view is that it is more likely that there was intermittent mortar in the outside header joints between panels and columns and beams, but this would not have been sufficient to “lock up” the panels.

68. I also note that the top course of blocks, under the beams, was not grouted and in fact could not have been. This was confirmed by Mr Fortune who said that they found that the top course in each section was hollow.

69. Despite these points, and the fact that even if grouted the vertical joints between panels would not have been able to transmit significant vertical shear stress, the frame on Line A was modeled in the ERSA as a fully competent monolithic wall for the lower three

stories, with a modulus of elasticity of 15GPa. Information provided in the CTV report on the torsional response of the building is based on this assumption.

70. In the NTHA carried out by Compusoft and StructureSmith the panels were modeled as individual flexural elements as an upper bound on stiffening effect, and compared with analyses where the infill panels were ignored, as intended by the designer. Analyses separately based on flexural action and on sliding shear failure produced similar additions to the lateral strength of the Line A structure. My view is that the flexural model adopted by the NTHA was an upper bound on strength. Displacement capacity would have been underestimated as it was assumed that the upper course was fully grouted, with no slip between the top of the panel and the beam above. It seems clear on the evidence that this was not the case.

71. The strength and stiffness of the line A wall assumed by the consultants for the purposes of the ERSA analysis was, in my view, much too high.

h. Consultants' Reluctance to accept NTHA Results

72. I have already discussed several areas where the Consultants have preferred to accept the results of the ERSA analyses over the NTHA results. These include: (i) potential for failure of the floor diaphragm/North core connection to act as a failure initiator; (ii) relative importance of columns on lines 1, D and F as potential failure initiators; (iii) extent of torsional response in the Feb 22, 2011 earthquake; (iv) influence of line A infill on CTV response.

73. In addition to these should be added the importance of beam/column joint capacity to the performance of the CTV building. This is only briefly discussed in the CTV report. The lack of ductile detailing in beam-column connections, principally as a consequence of there being no transverse reinforcement in the joints, is identified on p14 of the CTV as a critical design weakness, without further comment. On p93, the potential weakness is repeated, but claims that there are uncertainties in the assessment method and that limiting the assessment to column capacity (i.e eliminating the beam-column joints as a potential failure initiator) "would be sufficient for the purposes of this investigation". This explanation is repeated in Appendix D NTHA on p228. The Panel report does not

include beam-column joint failure as a possible contributor to the CTV collapse (p 53, 54).

74. However, the Compusoft analyses described in the Compusoft report p65 and Appendix B (pp85-89) examine the predicted performance of beam-column joints in considerable detail, based on a principal tension stress model. This assessment shows that 18 joints were at capacity between 2.3 and 3.6 seconds after shaking began, and that a further 26 joints (a total of 44) became overstressed between 4.5 and 5.7 seconds. The assessment draws attention to the detail of the beam bottom reinforcement which is hooked up into the joint rather than being continuous through the joint. Transfer of the tension in these bars under positive moment would have created additional tension outside the hook, increasing the probability of joint failure. These points are not included in either the CTV or the Panel report, despite being listed in the Compusoft report.

i. Displacement Capacity ERSA analysis [**Figure 1 / BUI.MAD249.0402.2**]

75. The methodology described in the CTV report for estimating the drift on line F (CTV report Appendix F [**BUI.MAD249.0189.283**]) is inappropriate for response estimation for the following reasons:

    a. The analysis is based on ERSA. This is inappropriate for estimating the response to the accelerogram of a real earthquake as effects of period shift due to inelastic response, which can be very substantial for recorded accelerograms, cannot be considered. I have explained the limitations on the use of ERSA and I will not repeat it here.

    b. The interstorey drifts were calculated as the difference between maximum storey displacements although the drifts in floors do not necessarily occur simultaneously.

    c. The maximum displacements are based on member stiffness values recommended in codes current at the time the building was designed, rather than 2011 state-of-the–knowledge values.

    d. The stiffness of frames on Lines 1, 2, 3, 4 and F were not included in the analyses.

e. The results of the analyses, incorporating these errors, were taken to be the true displacements and applied to an inelastic model of frame F.

WHETHER THE DESIGN OF THE CTV BUILDING MET BEST PRACTICE

76. The CTV Report refers to a number of code compliance issues with the design.

77. I have not directly compared the detailing of reinforced concrete members, nor their connectivity, with code design requirements in place in 1986. However, in my view it is clear that in 1986, the date on which the structural drawing were submitted for permit, many of the details used in the building would fail a test of “best practice to current state of knowledge”. These include:

    a. The lack of ductile detailing for the columns.

    b. The excessive spacing of transverse reinforcement in the columns.

    c. The excessive cover to reinforcement of columns, resulting in inadequate compression strength of the concrete core in the event of spalling of the cover concrete.

    d. Very high levels of axial compression in the columns.

    e. The lack of transverse reinforcement in the beam/column joints.

    f. Poor connectivity between precast beams and columns.

    g. The lack of adequate connection between floor diaphragms and the North core on Lines D and D/E. Even when the Alan Reay Consultants was informed of this deficiency by Holmes Consulting Group in 1990, when that firm carried out due diligence for a prospective purchaser of the building, this serious problem was only partly rectified by Alan Reay Consultants.

78. Of particular concern to me is the poor detailing of the columns, combined with the high axial load levels. Park and Paulay “Reinforced Concrete Structures” Wiley, 1975, clearly identifies this as dangerous (Section 6.4, pp217-221). This book was published some 10 years before the CTV building was designed and was widely referred to by NZ designers using reinforced concrete as “the bible”. It is inconceivable, in my view, that Alan Reay

Consultants was unaware of this information. Designers have a duty to design not only to the code, but also to the state of accepted knowledge applicable at the time of design.

The state of the CTV Building following the 4 September 2010 earthquake

79. It is entirely possible that a partial floor diaphragm-North Core connection failure could have occurred in September 2010. The NTHA results indicate the possibility of drag bar/ floor diaphragm failure under the 4 September earthquake. The displacements the building experienced in the September 2010 earthquake would not have been sufficient to cause complete failure of the building. Because the Hi-Bond trays and the EW supporting beams would have continued to support the floor, it is conceivable that separation did occur but it was not picked up in the post-earthquake inspections and it was the reason for the increased flexibility of the building that was noted by many of the occupants.

80. This separation might have been difficult to observe during the post-Sept 4 inspections. The investigators would probably not have known about the drag bar installation, and hence would not have paid them attention. If fracture of HRC mesh in the floor had occurred, this might not have been visible because of floor coverings, or may have been construed as shrinkage cracking, as crack widths of only 2mm are required to induce mesh fracture.

81. It should also be noted that low crack widths in columns after Sept 22 noted in the CTV report as an indicator of near elastic response can be misleading. During the earthquake, the crack widths may have been very much larger, but due to the high vertical loads on the columns these cracks could almost completely close up when the shaking associated with the earthquake ceased.

The causes and likely sequence of collapse of the CTV Building on 22 February 2011

82. It will be apparent from my evidence that my views on the critical weaknesses of the CTV building in general align closely with the Panel Report, and, albeit to a somewhat lesser extent with the CTV report. My areas of disagreement relate primarily to the relative importance given to the many weaknesses in terms of their potential to act as

failure initiators. My view is that too much emphasis is given to failure of line F columns as the failure initiator, and too little on other possibilities.

83. My view is that the columns on line F are unlikely to have acted as the failure initiator. A more likely sequence is that failure of the floor diaphragm/North core connection would have occurred early in the response to the Feb 22 2011 shaking, followed closely by distress to a number of beam-column joints. This beam-column joint damage would be initially concentrated in the bottom region of the joint, and the consequent spalling of concrete would reduce the column capacity to support the vertical loads, and the lateral displacements. The diaphragm/ North core failure would increase the displacement demands on the columns, and failure of internal columns due to the combination of large displacement, spalling of concrete and high vertical loads (including vertical acceleration effects) would result in explosive failures of the columns and the joints.

Dated: 18 May, 2012

M J N Priestley

MJN PRIESTLEY