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**

4th BRIEF OF EVIDENCE OF CLARK WILLIAM KEITH HYLAND

DATE OF HEARING: COMMENCING 25 JUNE 2012

BRIEF OF EVIDENCE OF CLARK WILLIAM KEITH HYLAND
IN RELATION TO THE CTV BUILDING

1. My full name is Clark William Keith Hyland. I live in Manukau. I am Director of Hyland Fatigue + Earthquake Engineering a specialist consulting engineering company.

2. I prepared the report on the CTV Building Collapse Investigation (BUI.MAD249.0189) ("the Hyland-Smith Report") for the Department of Building and Housing jointly with Ashley Smith of StructureSmith Ltd. I have provided previous briefs of evidence to the Royal Commission dated 24 June 2012 (x2), 11 July 2012 and 3 August 2012.

3. I have read and agree to comply with the Code of Conduct for Expert Witnesses.

Evidence

4. This brief of evidence replies to the evidence of Mr Douglas Latham (WIT.LATHAM.0002 and WIT.LATHAM.0003).

Professional Practice

5. I have set out the details of my professional experience and areas of expertise in my earlier briefs of evidence. I am advised I do not need to repeat this here.

Comments on Principles of Interpretation of Design Standards Relevant to WIT.LATHAM.0002 and WIT.LATHAM .0003

6. In his two briefs of evidence I believe that Mr Latham has taken a contrary view to the ERSA Panel and to the Hyland-Smith report on a number of critical issues. For example in para 20 Mr Latham concluded that "it was appropriate for a static analysis to be run for the CTV Building" when the ERSA Panel had agreed that the relevant clause 3.4.7.1(b) in NZS 4203:1984 recommended an ERSA. An experienced engineer would recognise the wisdom of using the better ERSA whether it was mandatory or not to achieve a more reliable understanding of its behaviour.

7. In my opinion this indicates that Mr Latham has proceeded from an incorrect understanding of the principles of interpretation that need to be applied to design standards. It appears that he is trying to justify a less reliable design approach on the basis of a narrow interpretation of clause wording, when good sense and awareness of the intent of the provisions and the principles of seismic design for life safety would lead one readily to a better design approach.

8. In my opinion the correct approach to the interpretation of design standards expected of a Chartered Professional Engineer are as follows:

9. Structural engineering design standards and codes of practice such as the Loadings Standard NZS 4203:1984 and the Concrete Structures Code of Practice NZS 3101:1982 embody overarching design principles and prescriptive requirements derived from those principles.

10. Interpretation of these design standards and codes of practice is recognised to be difficult. Engineers gain the competency required by practice in their use, gained initially under the supervision and mentoring of experienced professional structural engineers and then in conjunction with continuing professional development.

11. IPENZ recognises that it takes in the order of 3 to 5 years of engineering practice after completion of academic training, and under appropriate supervision, for an engineer to gain sufficient competence to be recognised as a Chartered Professional Engineer in the design of structures. This level of competency is recognised as necessary for the preparation of design statements for structures submitted for Building Consents.

12. Where there is difficulty in interpreting a particular clause or requirement in codes and standards a Chartered Professional Engineer is expected to consider their interpretation in the light of the overarching design principles that those provisions have been derived from. These principles may be clarified through consideration of associated commentary clauses, design reference books written about the matter, discussions with other experienced engineers or university experts. For the reinforced concrete structures standard NZS 3101:1982 the book by Professors Park and Paulay "Reinforced Concrete Structures" published in 1975 was such a reference. It was particularly relevant as they were both members of the NZS 3101:1982 committee that prepared the standard.

Interpretation of Group 2 Frame Drift Compatibility Requirements

13. In WIT.LATHAM.0003 at paragraph 5 reference is made to the seismic analysis report Mr Latham deals with in WIT.LATHAM.0002, from which he incorrectly draws the conclusion that there was no requirement for the additional seismic requirements of NZS 3101:1982 to apply to the Group 2 frames in the CTV Building. These were the reinforced concrete beams and columns that supported the concrete floors along Lines 1,2,3 ,4 and F. In order for me to address this I need to first clarify what the overarching principles and requirements of drift compatibility were in the standards and the issues around them with respect to the CTV Building.

14. In terms of Group 2 frames on Lines 1, 2, 3, 4 and F of the CTV Building the interpretation of the relevant clauses in the Standards must recognise the critical consequence of failure of the columns and the dependable strength and ductility necessary to avoid their collapse prior to the primary structural elements of the South Wall and the North Core achieving their design ultimate limit state response and drifts. The primary purpose of the seismic design standards is to prevent the loss of life in the event of an earthquake.

15. Specific drift compatibility provisions were written in NZS 4203:1984 and NZS 3101:1982 for Group 2 frames. These show that specific attention to this matter had been identified by the code writers as being required by design engineers. A method was set out in NZS 3101:1982 that in my opinion required consideration of the demands imposed on the Group 2 frames along Lines 1, 2, 3, 4 and F of the CTV Building by the deformation imposed on them. This was set at 55% of the ultimate limit state deformations of the primary structure.

16. In Clause 3.5.14.3 of NZS 3101:1982 terms such as elastic behaviour, plastic behaviour, elastic theory and plastic theory are used but not specifically defined. Two interpretations of these terms are possible.

17. As I stated in the course of my earlier cross-examination [TRANS.20120709.17], the most lenient interpretation was used in the Hyland-Smith Report in Appendix F, in which elastic behaviour is limited by the two bounds of tensile yield of reinforcing steel and attainment of maximum strength of the concrete at a compression strain of ec=0.002. Application of this criteria identified that the additional seismic design requirements of the Concrete Structures Code were triggered. (Refer to p.255 to 257 of the Hyland-Smith Report).

18. The stricter interpretation of Clause 3.5.14.3 of NZS 3101:1982 is that elastic theory and elastic behaviour were defined by the limits of elastic theory set out in the prevailing elastic theory (working stress) design method prior to the introduction of NZS 3101:1982.

19. Over the years debate in the engineering profession has surrounded the interpretation of this clause. However the need to consider drift compatibility of secondary structural elements is not contentious.

20. Professors Park and Paulay were part of the Concrete Design Committee (31/12) responsible for the preparation of the standard (refer p.10 NZS 3101:1982). In their book, “Reinforced Concrete Structures” published in 1975 they discuss Elastic Theory design and contrast it with the new ultimate strength design method that was then emerging. Their book became the companion reference to the NZS 3101:1982 standards In my view it is therefore reasonable to interpret the terms “elastic theory” and “elastic behaviour” in NZS 3101:1982 cl 3.5.14.3, in the way these terms are described in their book. In NZS 3101:1982 Appendix B sets out limiting stresses for working stress (elastic theory) design of 0.45 f’c for concrete and 0.55fy or 200 MPa for Grade 275 reinforcing steel.

21. This interpretation also appears consistent with the application of $\nu\Delta$ drifts equivalent to 55% of the ultimate limit state drifts as set by NZS 4203:1984 and similar amounts in the precursor provision in NZS 4203:1976. This would mean that the ideal capacity may be achieved at the ultimate limit state drift if tensile yield of the reinforcing steel governed capacity.

22. Irrespective of whether the lenient or stricter interpretation is taken, the requirement to consider the effect of primary structural frame drift on the Group 2 frames could not be ignored, as appears to have been done in the ARCL calculations (refer p. G9) for the CTV Building. These show that the Group 2 frames were designed as continuous gravity beams on pinned supports assuming the columns had no stiffness.

Specific Comments on Mr Latham's Evidence on the Group 2 Frames on Line 2 and F

23. I have undertaken a review of the compliance of the Line 2 and F Group 2 frames in response to the analyses reported in WIT.LATHAM.0002 and WIT.LATHAM.0003. This review and the calculations for it are included in the Appendix to my brief of evidence.

Clause 3.4.7, NZS 4203:1984:

24. My opinion is that cl 3.4.7 NZS 4203:1984 required an ETABS (ERSA) analysis to be carried out by the designer of the CTV Building. This was in fact done by Mr David Harding. Mr Latham has taken a contrary view. However, in my opinion his assessment that the CTV Building had only moderate eccentricity is incorrect and does not correspond with the assessment using the method set out by Park and Paulay in their 1975 book "Reinforced Concrete Structures (Refer p. 631).

25. Use of this method, which is described at pages SW1 to SW6 in my response to the Items 2e to 2j of the Royal Commission Minute Concerning the Performance of the CTV Building dated 27th June 2012, showed that the building was highly eccentric. This required clause 3.4.7.1(b) to be applied, which recommended use of ERSA.

26. Paragraph 4 of C3.4.7.1 of NZS 4203:1984 can also be interpreted to indicate that the CTV Building was irregular due to the North Core forming re-entrant angles to the main body of the structure. On this basis cl 3.4.7.1(c) made ERSA mandatory.

27. In summary in my opinion it would have been either inappropriate or non-compliant to not undertake an ERSA for the CTV Building.

Soil properties

28. Use of the "soft" soil properties recommended by Tonkin and Taylor with the existing structural weights, as used in the Hyland-Smith Report, resulted in slightly greater drift demands than using the "stiff" soil properties that were used in the original CTV Building ERSA analysis in the Hyland-Smith report.

29. To show the effect of different soil properties on the outcomes I have re-run the analyses using the "soft" and "stiff" soil stiffnesses as well as the condition with rigid subgrade. The results are shown in the attached calculations of the Appendix. In summary on page DA-6 these showed that a fixed base or rigid subgrade condition resulted in the most severe demands in terms of base shear and therefore demands on the South Wall, but also had the lowest inter-storey drifts affecting the Group 2 frames. The greatest inter-storey drifts occurred when the "soft" subgrade was used but also had lower base shear. Good design practice would be to envelope these two extremes of behaviour where doubt existed as to the subgrade effects and design the primary structure for the higher actions from the stiffer subgrade conditions and the Group 2 frames for the higher drifts from the softer subgrade conditions.

30. Page S11 ARCL calculations show that a structural period of 1.05 seconds was assumed for the design. This is similar to the period of 1.04 seconds found from the analysis in the Appendix to this statement assuming "stiff" soil properties, but is greater than the 0.77 seconds in the North–South direction and 0.92 seconds in the East-West direction found for the fixed base condition. Using a model with a natural period of 1.05 seconds would result in lower design actions on the South Wall than for a model with natural period of 0.90 seconds. The ARCL calculations do not state how the higher structural period was derived or if some sort of subgrade assumptions had been used in its ERSA modelling.

Foundation rotations

31. In WIT.LATHAM.0002.22 Mr Latham has incorrectly neglected the effect of foundation rotations in the primary elements when assessing his drift demands on the Group 2 frames. The effect of foundation rotations in the South Wall and North Core must be accounted for when considering the drift effects on the Group 2 frames. For the wall elements themselves the effect of foundation rotation can be neglected, as allowed by cl 3.8.1.2 of NZS 4203: 1984, as no change to inter-storey deformations in the wall elements will occur if that is done. However for the effect on Group 2 frames the overall drifts resulting from the combined foundation rotation effects and superstructure deformation of the primary wall elements is required to satisfy the principle of safe drift compatibility design of the Group 2 frames.

Seismic Design requirements apply

32. Using the elastic behaviour limits of the elastic theory (working stress) design method of reinforcing tensile stress of 0.55fy and concrete compression strain of ec=0.001, in my opinion columns *C/1* on Line 1, *D/2* on Line 2 and *F/2* on Line F all were all required to be designed at all levels using the additional seismic design requirements of NZS 3101:1982.

33. Using the more lenient elastic deformation limits of tensile stress of fy and concrete compression strain of ec=0.002 and stiff soil subgrade properties, in conjunction with moment -curvature analysis as reported in Appendix F of the Hyland-Smith Report, in my opinion the upper level columns on Lines 1 and F also triggered the additional seismic design requirements of NZS 3101:1982.

34. I recognise that engineers practicing at the time of the design of the CTV Building did not typically undertake moment-curvature analysis and would have resorted to more conservative and simpler approaches to assessing elastic behaviour. This would normally have included assuming the column properties were uncracked for analysis purposes. This makes the Appendix F approach in the Hyland–Smith Report an upper bound method.

35. On the basis of the application of the two bounds on interpretation of elastic behaviour described above it is my opinion that the additional seismic design requirements were triggered and it would be wrong to consider that it was not necessary or appropriate to design the Group 2 frames using those requirements. In my opinion Mr Latham is not correct when he says in WIT.LATHAM.0003 paragraph 5 that the additional seismic design requirements were not necessary for the design of the CTV Building.

36. The analysis I have done which is described in the Appendix to this brief shows that even if these additional seismic requirements were *not* applied to the Line 2 and F frames they were not adequately reinforced to comply with the design level drifts for the Group 2 frame design of 55% of the ultimate limit state drifts.

37. In WIT.LATHAM.0003.23 to 25 Mr Latham attempts to justify the amount of shear and flexural reinforcing in the CTV Building columns based on his analysis. However the analysis and calculations shown in the Appendix to my evidence show that on Line 2

the D/2 column triggered the minimum shear reinforcing limits that would have required spiral wrapping of R6 at 55 mm centres for columns from Level 4 and above if cracked column properties were used in the analysis. If uncracked column properties were used in the analysis as was the recommended practice at the time then R6 at 55 mm centres would have been required at all levels on the column D/2.

38. My analysis and calculations also show that the Line 2 column D/2 had insufficient dependable flexural compression strength to cope with the 55% ULS drift demands, whether cracked or uncracked properties were used for the columns in the analysis.

39. The Line F columns F/2 and F/3 were found to require R6 spirals at 55 mm centres to satisfy the minimum shear reinforcing requirements at Level 1, 3, 4 and 5 if cracked column properties were used in the analysis.. If uncracked column properties were used then R6 at 55 mm centres would have been required from Level 3 and above and R6 at 55 mm at Level 1 and 2.

40. In my opinion, both in 1986 and now experienced structural engineers would not reduce the level of column spiralling down a column if was required at a higher level. Similarly once the additional seismic design requirements were triggered somewhere in a frame the expectation would be that the whole frame would be designed using the additional requirements to achieve consistency of safe performance.

Conclusion

41. Interpretation of seismic design codes and standards requires experience and the application of the overarching principles of seismic design embodied in them.

42. Use of ERSA using ETABS software or similar to analyse the CTV Building was either recommended or mandatory in 1986, depending on the interpretation of what “regularity” meant. A sound engineering approach would be to use the better analysis technique provided by ETABS whichever interpretation was favoured and Mr Harding and Alan Reay Consulting Engineer made the appropriate decision in deciding to use ETABS

43. The soft soil subgrade conditions using the existing building weights used in the Hyland-Smith report resulted in the worst combination of design drifts and so was used for the design checks.

44. Subsoil rotation effects of primary wall elements must of necessity be included when considering inter-storey drift demands on Group 2 frames. However they do not need to be considered when assessing inter-storey drift effects on the primary wall elements themselves.

45. The columns trigger the additional seismic design requirements of NZS 3101: 1982 when either the definitions of elastic behaviour used in Appendix F of the Hyland-Smith report or the more conservative Elastic Theory limits, consistent with Park and Paulay’s definition, were used. Once the additional seismic design requirements are triggered somewhere in a frame, it is my opinion that the whole frame should be designed using the additional requirements to achieve consistency of performance.

46. The columns checked on Line 2 and Line F were insufficiently reinforced in shear and flexure to satisfy the dependable strength and minimum requirements of the standards when the 55% ULS drifts were applied, even if the additional seismic design requirements of NZS 3101:1982 were not applied.

47. Not all aspects of the design compliance of the Line 2 and F frames have been considered in this brief of evidence. I have addressed only those required to address particular matters in the two briefs of evidence from Mr Latham, WIT.LATHAM.0002 and WIT. LATHAM. 0003.

Signed:____________________________________

Dated: 8th August 2012