Christchurch City Council

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 NGĀ WHARE I HORO I NGĀ RŪWHENUA O WAITAHA

FIRST STATEMENT OF EVIDENCE OF ARTHUR JOSEPH O'LEARY IN RELATION TO THE CTV BUILDING (AMENDED)

HEARING BEGINNING: 25 JUNE 2012

Legal Services Unit, 53 Hereford Street, Christchurch 8013
P O Box 73013, Christchurch 8154
Telephone (03) 941 8999

INTRODUCTION

1. My full name is Arthur Joseph O'Leary. I am a retired structural engineer. I have had extensive design and design management experience of commercial buildings with emphasis on earthquake engineering during my professional career. That career has spanned some 40 years.

2. I graduated from the University of Canterbury with a Bachelor of Engineering (Civil) with first class honours in 1966, and was awarded the degree of Doctor of Philosophy in Civil Engineering from the University of Canterbury in 1970. My doctorial thesis was entitled "Shear, Flexure and Axial Tension in Reinforced Concrete Members".

3. Upon completion of my thesis, I was employed by Morrison Cooper and Partners, a Wellington based consulting engineering practice for two years. I then travelled to the United Kingdom for two years and worked for two consulting practices, one being Mott Hay and Anderson (subsequently becoming Mott McDonald), and the other Ove Arup and Partners.

4. Returning to New Zealand in 1974, I was reemployed by Morrison Cooper and Partners, staying with them through various practice mergers until retirement at the end of 2010. The practice was known as Sinclair Knight Merz Limited at the time of my retirement.

5. During my employment in New Zealand I held various technical and management positions including:

   - Structural Engineer and Senior Structural Engineer in Morrison Cooper and Partners, and Morrison Cooper Limited.
   - Chief Structural Engineer in Morrison Cooper Limited.
   - Shareholder and Board Member of Morrison Cooper Limited.
   - Shareholder of Kingston Morrison Limited.
   - Shareholder and Principle of Sinclair Knight Merz.
   - Wellington Structural Engineering Manager of Sinclair Knight Merz.
   - Earthquake Engineering Practice Leader for the world wide practice of Sinclair Knight Merz until retirement.
   - Senior Consultant within Sinclair Knight Merz until retirement.

6. I was a Chartered Professional Engineer, and on the New Zealand register of the International Professional Engineers Register up to my retirement. I am a fellow of the Institution of Professional Engineers, New Zealand, a fellow of the New Zealand Society for Earthquake Engineering, and a member of the Structural Engineers Society, New Zealand.

7. I served on the "Relative Seismic Risk" subcommittee for the preparation of NZS 4203:1992, Code of Practice for General Structural Design and Design Loadings for Buildings.

8. I was on the technical committee that produced NZS 1170.5:2004, Structural Design Actions Part 5: Earthquake Actions – New Zealand.

9. I was part of a Federation International du Beton (CEB-FIB) study group producing the state-of-art report "Seismic Design of Precast Concrete Building Structures", October 2003.

10. In 1997 I received the American Concrete Institute, Maurice P Van Buren Award for Structural Engineering, 'for authoring a paper describing the application of the latest innovations in precast technology to the construction of a moment resisting frame in a high seismic zone'.

11. I have received an Association of Consulting Engineers New Zealand, Silver Award of Merit for the "Queenstown Bay Reservoir', and also the New Zealand Concrete Society commendation for the same project.

12. I have authored and published 10 papers on concrete and earthquake resistant design.

13. I have designed and been structural design manager for a number of medium and high rise buildings in Wellington and Lower Hutt. The highest of these was 30 storeys, but the building was never built because of the construction down turn in the late 1980's. The detailed design and detailing for the building was however completed. I was involved with at least six buildings over 10 storeys.

14. In 2002 I was structural design verification team leader, and for a period site design engineer, for the new 65,000 seat stadium to stage the final of the European Cup in Lisbon, Portugal. Lisbon is one of the more seismically

active areas of Europe, having been devastated by a very large earthquake in the 1700's. I also gave specialist seismic advice on the 2004 Olympic Games main stadium roof in Athens, and the basketball and fencing arenas.

15. My evidence:

(a) outlines my understanding of the loading and concrete standards forming part of Christchurch City Council Bylaw No, 105 (1985) in force when the building permit for the CTV building was issued;

(b) considers the Design, Construction, and Standards Issues set out in section 9 of the CTV Building Collapse Investigation for Department of Building and Housing, January 2012, Hyland and Smith (**"the Hyland Smith report"**);

(c) provides some further observations on aspects of the Hyland Smith report and the Expert Panel Report; and

(d) discusses aspects of the William T Holmes Peer Review (**Holmes Peer Review**) of the Hyland Smith report.

16. 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.

REVIEW OF RELEVANT MATERIAL

17. I have reviewed the original calculations for the structural design of the CTV building undertaken by Alan Reay, Consulting Engineer. These calculations are pages numbered G1 to G79, S1 to S57, and F1 to F52. (The last page is presumably F52 although unnumbered) (**BUI.MAD249.0008**). I have focused my attention on the pages in the 'G' and 'S' numbered series, as these are the relevant pages associated with the main seismic resistant design of the structure of the CTV building. The 'F' numbered series appear to be specifically related to foundation design.

18. The relevant New Zealand concrete and loading standards current at the time of the design of the CTV building were NZS 3101 Part 1:1982, Code of

Practice for the Design of Concrete Structures (**NZS 3101**), and NZS 4203:1984, Code of Practice for General Structural Design and Design Loadings for Buildings (**NZS 4203**). I understand that these two documents were the standards that applied to the design of reinforced concrete buildings under the Council's Bylaw 105 at the time of the issue of the building permit for the CTV building (1986). I have reviewed these documents, as well as NZS 3109:1980 and NZS 3101 Part 2:1982 Commentary as they relate to the CTV building design.

19. In addition, I have read the following documents for the purpose of preparing this Statement of Evidence:

(a) The Hyland Smith report. (**BUI.MAD249.0189**)

(b) CTV Building Site Examination and Materials Tests, for Department of Building and Housing, January 2012, Hyland. (**BUI.MAD249.0126**)

(c) Structural Performance of Christchurch CBD Buildings in the 22 February 2011 Aftershock, February 2012, Expert Panel Report (**Expert Panel Report**). **(BUI.VAR.0056)**

(d) William T. Holmes Peer Review of the Hyland Smith report dated April 30 2012 (**BUI.MAD249.0372**).

(e) The Cement and Concrete Association of New Zealand submission to the Royal Commission (**BUI.MAD249.0373**).

(f) The James Mackechnie submission to the Royal Commission (**BUI.MAD249.0362**).

(g) Compusoft CTV Building Non-Linear Seismic Analysis Report, Revision 0 February 2012, Derek Bradley and Tony Stuart, reviewed by Dr Barry Davidson (**ENG.COM.0001**).

(h) The structural drawings for the CTV building as stamped and dated 30 September 1986 by the Christchurch City Council. These drawings are on Alan M. Reay Consulting Engineer drawing sheets and are identified as File 2503, numbers S1 to S39 inclusive. There are no amendment identifiers on the drawings.

(i) Sections of the Specification issued as part of the permit documentation (**BUI.MAD249.0199**). These are the sections relating to:

(i) Excavation and Hardfill;
(ii) Concrete and Reinforcing Steelwork;
(iii) Precast Concrete; and
(iv) Structural Steelwork.

(j) The evidence of Nigel Priestley (**WIT.PRIESTLEY.0001**), Peter Nichols (**WIT.NICHOLS.0001** and **0002**) and Stephen McCarthy (**WIT.MCCARTHY.0001**).

20. I am aware that there was remedial work undertaken in 1991 to bring an identified deficiency in the connection of levels 4, 5 and 6 floor slabs to the north shear walls up to compliance with the requirements of NZS 3101 and NZS 4203. I have been unable to check the calculations undertaken at the time of the discovery of the need for remedial work as the reproduction of the calculations supplied to me was not of a quality to be readable. However, I have checked the adequacy of the agreed loads to be transferred from the slab to the walls and, as I later note, <u>except for in the east west direction</u> they were in my view appropriate to comply with NZS 4203 after the remedial work had been carried out.

THE LOADING AND CONCRETE STANDARDS IN THE BYLAW

21. To address these issues, I will firstly comment on the overall approach to both the loading and concrete standard requirements of the time. Both the loading (NZS 4203) and the concrete (NZS 3101) standards apply to the CTV building design. The loading standard is independent of the construction materials used in a building. The concrete standard, as a material standard, uses the loading criteria derived from the loading standard and applies those criteria for the use of reinforced concrete as the construction material.

22. The introduction to NZS 4203 contains a general statement in clause 3.2.1 that "*the building as a whole, and all its elements that resist seismic forces or movements, or that in case of failure are a risk to life, shall be designed to possess ductility...*". This requirement, is in my view, put into context by the commentary clause C3.2 which states in part, "*the general requirement for ductility must at present be qualitative rather than quantitative except for buildings designed to dissipate seismic energy by ductile flexural yielding ...*".

23. Clause 3.2.2 of NZS 4203 requires that "*Structural systems intended to dissipate seismic energy by ductile flexural yielding shall have 'adequate ductility*'". Clause 3.2.3 then states "'*Adequate ductility' in terms of clause 3.2.2 shall be considered to have been provided if all primary elements resisting seismic forces are detailed ...*".

24. There is no quantitative guidance in NZS 4203 as to what is "adequate ductility", and how to provide for it. However, clause 3.2.3 refers the designer to the "*appropriate material code*" which in the case of the CTV building is NZS 3101 and which provides quantitative guidance.

25. The requirements of clause 3.3.3 of NZS 4203 are related to ductile frames and not secondary elements. Thus if it can be shown that the frames on grids 1, 2, and 3, and f, of the CTV building were secondary elements, then clause 3.3.3 would not apply.

26. The CTV structure may have been designed to dissipate seismic energy in ductile flexural yielding of the shear walls, but from the observations in the Hyland Smith report, and from reviewing photographs of the shear walls after the collapse, it would appear that it did not. Rather, it would appear that the onset of collapse occurred before any of the shear walls yielded significantly, except with the possible exception of the wall on grid 1, and therefore the shear walls were not called upon to dissipate energy by flexural yielding.

27. Although the shear walls were designed as ductile, because of the likely sequence of failure of the CTV building they were (except with the possible exception of the wall on grid 1) not called upon to dissipate energy in a ductile mode of failure. The likelihood that there was an early diaphragm disconnection at probably level 2 and/or 3 meant that instability of some columns and their failure precipitated collapse before the shear walls were

fully loaded. In other words, the collapse sequence was somewhat independent of fully loading the shear walls.

THE HYLAND SMITH REPORT – DESIGN, CONSTRUCTION AND STANDARDS ISSUES

28. Section 9 of the Hyland Smith report (pages 109 to 120) identifies *Design, Construction and Standards Issues* in relation to the possible causes of the collapse of the CTV building in the 22 February 2011 aftershock. In the following parts of my evidence, I address the issues raised in this section of the Hyland Smith report. As a starting point however, it is necessary to review relevant aspects of earlier sections of the report, and then I will make some general comments about structural design of the CTV building.

Introduction

29. I have reviewed tables 1 and 2 on page 26 of the Hyland Smith report. I have found no clear indication in the report as to how the *1986 non-ductile detailing* and *1986 ultimate* drift figures have been determined. Reference to Appendix F of the report has not assisted me, as one of the columns to which table 13 and 14 apply is different from those in tables 1 and 2. The common column is F2. At level 3 (chosen level in table 1), the "1986 non-ductile detailing" limit in table 14 is 0.69% which is inconsistent with the 0.6% in table 1. The *1986 ultimate* drift at 1.1% in table 1 is consistent with table 14 in one direction earthquake (N-S shows 1.12% drift), but very different for the other direction earthquake (E-W shows 0.46%).

30. Further, the last paragraph on page 27, continuing on to page 28, contains a statement "*... Under this interpretation, elastic performance of the secondary members in the CTV Building was required to be demonstrated at 55% of the design maximum or ultimate earthquake drifts.*" For the CTV indicator columns, the applicable drifts for this check are the 1986 non-ductile detailing figures (refer Table 1 and Table 2). These references are not clear. The origin of the 55% figure and what it means are not adequately explained either in the body of the report or in Appendix F. In this respect, I also refer to the Holmes Peer Review **(BUI.MAD249.0372)** at page 3.

31. In addition, the discussion of "*Drift Capacity of Columns*" on pages 27 and 28 of the Hyland Smith report *seems to assume that the commentary to NZS 4203 was mandatory (which it is not)*. The analysis techniques discussed in the commentary clause C3.2 were not available to designers in 1986. They were only available to researchers at the time.

General Comments

32. The CTV building horizontal load resistance was provided by shear walls. The shear wall configuration of the CTV building would have placed it in the category of a '*moderately eccentric structure*' according to clause 3.4.7.1 of NZS 4203 and the commentary to that clause. I accept that there is room for differing interpretations of the definition of moderate eccentricity in the commentary to the clause, but I believe that the intent of the definition was to limit the amount of torsional resistance provided by elements required to resist predominantly translational loads.

33. Reasonably regular structures more than 4 storeys high even with a high degree of eccentricity, were allowed by clause 3.4.7.1(b) of NZS 4203 to be analysed by quite unsophisticated methods of analysis (the static method or two dimensional modal analysis), although the standard recommends (but does not require) a more sophisticated three dimensional modal analysis. If the two dimensional methods of analysis were used, calculating torsionally induced interstorey drifts would be precluded. I discuss this further later in my evidence.

34. The CTV building structural configuration meant that the torsional resistance to horizontal earthquake generated effects was shared by two separate combinations of shear walls. The east-west wall on grid 5 and the wall on grid 1 provided efficient torsional resistance. The walls running north-south between grids 4 and 5 provided a second independent structure to provide torsional resistance.

35. The understanding of torsional resistance in structures of this type at the time of the CTV building design was that the torsion would be shared by both sets of torsional restraining elements, but if one was fully loaded by the translational effects of an earthquake, then the other could be designed to

resist all the torsional effects. This was, in my view, a widely held understanding amongst structural engineers at the time, and until recently.

36. The corollary of the ability to share the torsional resistance was that if a wall (in the case of the CTV building, the wall on grid 1), was highly loaded by translational effects of the earthquake (to the extent of significant yielding), then the torsional load that it may have been resisting would be "shed" to the other torsional resisting set of walls. This in effect should guard the structure from a wall failure resulting from a combination of torsional and translational loading.

37. NZS 3101 defined primary and secondary elements and had different seismic resisting requirements for each class of element. The relevant definitions were contained in clause 3.5.14, but the differing design requirements were covered in other parts of the standard.

38. An important issue in considering the compliance of any design with the standards of the day is the interpretation of clause 3.5.14 of NZS 3101. My interpretation of clause 3.5.14.1 is that the beams and columns of the CTV building were *group 2 secondary elements*.

39. This is supported by the first sentence in the commentary to clause 3.5.14.1 of NZS 3101 which states in part "*The definition of a secondary element is more particular than that in NZS 4203, and includes such primary gravity-load resisting elements as frames which are in parallel with stiff shear walls and do not therefore participate greatly in resistance to lateral loads ...*". The clause further states that "*... Frames in parallel with slender shear walls should be designed and detailed as fully participating primary members ...*".

40. There is no applicable definition in NZS 3101 as to what is a stiff or slender shear wall in terms of clause 3.5.14.1, but I consider that the widely held interpretation at the time would have been whether the frame would provide a significant contribution to the lateral load resistance of the structure. On that basis, the shear walls of the CTV building are likely to have been regarded as stiff by both the designer and the Council reviewing engineer.

41. The frames running east-west on grids 1, 2, and 3 were not designed to resist any lateral load. There ~~was~~**were** only ~~one frame~~**two frames** running north-south (**one** on grid (f) and **the other a part frame on grid (a)) and** again ~~that was~~**those were** not designed to carry any lateral load. All the lateral load resistance was provided by the shear walls as can be seen from a review of the structural calculations. Accordingly, the shear walls are likely to have been regarded by the designer as stiff, and the Council reviewer could in my view have reasonably come to a similar conclusion.

42. On the basis that the frames (including columns) were group 2 secondary members, then clause 3.5.14.3 of NZS 3101 becomes applicable:

(a) The intent of sub-clause (a) is that when secondary elements have deformations less than a defined value (v times delta) and remain elastic, they do not to have to comply with the '*additional seismic requirements*' of the standard.

(b) Sub-clause (b) requires the additional seismic standards to be met when the element starts to yield at deformation less than the defined value.

(c) Sub-clauses (c) and (d) do not influence the discussion in this section of my evidence.

(d) Sub clause (e) requires that elastic theory shall be used to at least the deformation level compatible with ¼ of the defined value. This is a requirement to avoid excessively large post elastic deformations under any circumstances. It is not directly applicable to the CTV building design as it does not govern any relevant compliance requirements.

(e) I interpret sub clause (f) to mean that even if elastic theory is applied at deformations greater than ½ the defined value, then '*limited ductility requirements*' may be applied which are covered in chapter 14 of NZS 3101. There is no upper limit of deformation identified in this requirement, but it would not be logical to extend the upper limit of the requirement to the defined value as that situation is already adequately covered in sub-clause (a). I believe that sub clause (f) limits were to be applied when the limit of elastic theory lay between ½ and the full defined value of deflection, but not at the full defined value.

43. In summary, the design and detailing of the secondary members of the CTV building should have satisfied the applicable sub clauses (a) or (b) of NZS 3101 clause 3.5.14.3. The beams and columns on grids 1, 2, 3, a, and f were, in my view, group 2 secondary elements.

44. Returning to the Hyland Smith report, I do not consider that it is appropriate for methods of analysis not available except as research tools, or excluded by the standards of the day, to be used to make assessments as to whether the analysis, design, and detailing of the CTV building complied with the standards of the day. I provide two examples to illustrate my point:

(a) Firstly, I refer to Appendix F of the Hyland Smith report – Method - first paragraph at page 253. The displacement compatibility analysis referred to in this paragraph used "Cumbia" software from a paper published in 2007.

(b) Secondly, the ERSA modelling used to perform various analyses included the effect of flexible foundations. This is referred to at page 236 of the Hyland Smith report. Clause 3.8.1.2 of NZS 4203 specifically stated that for the purpose of computing deformations, foundation rotations were to be neglected. This means that the computed interstorey drifts used to investigate compliance in the Hyland Smith report are in conflict with the provisions of NZS 4203 and almost certainly give drifts that are larger than those where foundation rotation was not allowed for in the model.

45. I accept that the use of the most up to date structural modelling and computer analysis techniques to ascertain the reasons for the collapse of the CTV building is appropriate. However, I have a different view in relation to using these techniques to identify compliance/non-compliance with the standards current at the time of design. In my view, only analysis techniques available to the practising structural designer at the time the design was carried out should be used for the identification of compliance. Even the use of research tools available to the research community, but not the practising engineer, is not appropriate when considering compliance issues.

46. Much of the evidence below is based on my interpretation of clause 3.5.14.3 of NZS 3101 as discussed earlier. I now discuss the design issues at pages 109 to 115 of the Hyland Smith report.

Section 9 of the Hyland Smith Report – Design Issues

Building Inter-storey Drift Limits

47. The Hyland Smith report concludes at page 109 that "*...It is therefore debatable whether the drift limit was satisfied.*" I believe that this conclusion is open to question for at least three reasons: -

(a) Clause 3.8.1.2 of NZS 4203:1984 expressly states that "*Computed deformations shall be calculated neglecting foundation rotations.*" The analysis from which the report draws its conclusions allowed for some level of foundation rotation, although the report acknowledges that allowing foundation rotation was contrary to what was required in NZS 4203.

(b) In addition, the method of analysis in Appendix F was undertaken using techniques unavailable to designers in 1986. The factor of 0.85 for 'frame effects' referred to at page 253 of the report is based on a paper published in 2007.

(c) The report (page 109) also notes that the drift requirements were satisfied "... *if no account was made of the effect of inelastic deformation initiating in the South Wall at the K/SM deformation levels*". Recognition has not been given to the fact that drift levels in NZS 4203 were set making allowance for inelastic deformation. This is explained further in commentary to clause C3.8.1.2 of NZS 4203.

Drift Capacity of Columns (and Column Confinement)

48. The drift capacity of columns and the column confinement are intimately linked. As two adjacent floors of a building move horizontally relative to each other (interstorey drift), the columns between the floors develop vertical curvature which produces bending moments (flexure) in the columns. The bending moment is a function, among other things, of floor element strength and stiffness at each end of the column.

49. As the interstorey drift increases, the bending moment in the columns may start to yield the concrete and/or the reinforcing. This is the onset of plasticity and to sustain this plasticity, the column needs to be ductile. How ductile the column is, depends on how much confining reinforcing (spirals or stirrups) is provided in the column.

50. The drift limits discussed in paragraph 47 of my evidence equally apply to the drift capacity of columns. Accordingly, I consider that the conclusion in the Hyland Smith report at page 110 that the columns "... *were required to be designed using the additional seismic design provisions of NZS 3101:1982*" is also in question and possibly incorrect. In my opinion, the more detailed analysis in Appendix F of the report does not clarify the issue for various reasons. The most critical columns in the centre of the building were not included as sample columns in Tables 13 and 14.

51. I have concluded that the internal circular columns (located at grids B2, B3, B4, B5, C2, C3, C4, and C5) and including the circular columns at A/B1 and B/C1, comply with the requirements of clause 3.5.14.3(a) of NZS 3101. That is, it was appropriate to detail these columns as "*members not designed for seismic loading*". This conclusion is based on the interstorey drifts given in Alan Reay calculations at pages S15 and S16. It is not known how these drifts have been calculated, but it is likely they were from an output from the ETABS analysis that I understand was carried out at the University of Canterbury. The output from that analysis has not however been able to be found at this time.

52. The actual drifts do however seem reasonable to me. The drifts in the N-S direction are approximately 5 times those in the E-W direction. This may have been influenced by the drifts on grid F being magnified by torsional effects. The drifts may be the maximum drifts from any of the relevant load cases. Torsional effects would have a much lower influence on the E-W drifts.

53. The circular columns on grid F did not in my view meet the criteria of clause 3.5.14.3(a) of NZS 3101, as they did not remain elastic under the factored drifts (K/SM). They should, in my view, have been designed to clause 3.5.14.3(b) of NZS 3101 (Additional seismic requirements of the standard).

54. The confinement reinforcement in the columns is governed by clause 6.4.7.1(b) of NZS 3101 for non seismic columns. The confinement spiral for the non-seismic columns, shown on the drawing S14, complied with that clause for the non seismic columns. The confinement spiral for the columns on grid F should in my view have been designed in accordance with clause 6.4.7.1(a), and thus did not comply with respect to either the spacing or the size.

55. In conclusion, I consider that the decision to design the columns as "*members not designed for seismic loading*" may have been justified according to the standards of the time, except for the columns on grid F, which should in my view have been designed for seismic loading.

Minimum Shear Reinforcing of Columns

56. If the internal columns did not need to be designed as seismic elements, the test as to whether they required shear reinforcement would have been covered in clause 7.3.4.1 of NZS 3101. The relevant test was if the shear stress across the columns was less than ½ the shear strength allowed to be provided by the concrete, then no shear reinforcement was required. I have considered a typical circular column over several floors and have concluded that the shear stress was low enough to not require shear reinforcement for a column designed as non seismic.

57. There is nothing in NZS 3101 relating to columns designed for seismic loading that alters that test, except in certain circumstances that do not apply to the circular columns. Clause 7.5 allows shear to be carried by the concrete (I refer to equation 7.41 and clause 7.5.2.2). The restriction in clause 7.5.2.2 is met and so some shear is allowed to be carried by concrete. The two back references in clause 7.5.3.1 are also satisfied and support the analysis above. I note that the requirements of clause 7.3.16.1 appear to apply only to slab/column joints. This is clarified by clause 9.2.1.

58. In summary, I consider that the internal columns on grids 2 and 3 did not require shear reinforcement if seismic detailing was not required. It is likely however, that the circular columns on grid f were required to be reinforced for shear. Therefore, in my view, neither the size nor the spacing of the spiral provided would have complied with NZS 3101.

Spandrel Panel Separation

59. The issue raised in the Hyland Smith report (at pages 110 – 111) about the spandrel panel separation is, in my view, essentially a construction matter. A note on the drawings or in the specification related to minimum separation would have provided assistance to the building contractor, but it is not, in my view, a design error or a standards compliance issue as such. My calculations indicate that a clearance at either end of each panel of 10mm would have been sufficient.

60. I do note in the CTV Building, Non-Linear Seismic Analysis Report (Ref: 11033-00 Revision 0 February 2012) that the spandrel panels were modelled as "*planar linear elastic shell elements located along the column centre lines*" (page 20). I do not believe that this is correct ,because the panels were quite flexible longitudinally as the vertical part of the panel was torsionally flexible and it was offset from the columns. Therefore, the influence of the column being restrained by the panels has been given undue weight in the modelling outlined in that report that included the spandrel panels. I consider that any influence which the spandrels had in the premature onset of failure of the columns on line F would be relatively minor.

61. I believe that my views as to the spandrel panel influence on the seismic performance of the CTV structure are supported in the Statement of Evidence of Michael John Nigel Priestley at paragraphs 43 and 44 (**WIT.PRIESTLEY.0001.13**).

62. The Compusoft report on modelling the CTV structure states at section 5.6 "*...spandrels have been modelled as planar linear elastic shell elements located along the column centre lines as shown in fig. 18...*". As discussed above, this is not, in my opinion, an appropriate approach to modelling the spandrel panel effect on the response of the structure. I consider therefore that any part of the Compusoft analysis that contains the spandrel panel modelling is not correct and conclusions drawn from it cannot be supported.

Beam-Column Joints

63. This issue is referred to at page 112 of the Hyland Smith report. I have considered the requirements in NZS 3101 for shear reinforcement in the beam column joints of the frames particularly on grids A, 2, and 3, but my comments also apply more generally to grids 1, 4, and F. The wording of the standard is not entirely clear given that there is some scope for conflict between the requirements of different sections of the standard. I will, however, identify each clause that may be relevant, and then comment on how I believe it should be interpreted.

64. Clause 7.3.16.1 of NZS 3101 states "*When gravity load, wind, earthquake or other lateral forces cause transfer of moment at connections of framing elements to columns, shear resulting from moment transfer shall be considered in design of shear reinforcement in the joint*". Although the clause is in the chapter on "*Shear and Torsion*", it is a general requirement related to beam/column joints which is covered in detail in Chapter 9 of the standard.

65. Clause 7.3.16.1 is a general statement and is expanded on in clause 7.4.1. Clause 7.4.1 requires a minimum level of shear reinforcing in a beam column joint not designed for seismic loading. The spiral shear reinforcement in the beam column joint did not, in my view, comply. It provides about half the minimum requirement. But the question then arises as to what is required for a joint that is designed for seismic loading. There is no requirement as such for transfer of moments to columns in clause 7.5 which is the relevant clause for the design of shear resistance in members designed for seismic loading. The relevant clauses for beam/column joint shear are in chapter 9 of the standard but it does show an inconsistency in the standard.

66. There is a further issue about how clause 7.3.16.1 should be applied. It is one of two clauses that contain requirements for slab connections to columns. It appears to stand on its own, but its location in the standard is not logical and there is a general reference later (in clause 9.2.1) that seems to infer that all requirements of clauses 7.3.15 and 7.3.16 relate to slab connections to columns.

67. I believe clause 9.4.1 of NZS3101 is inconsistent with clause 3.5.14.1(a). I consider that clause 9.4.1 should be part of general clause 9.3 and is

misplaced as part of general clause 9.4. Even structural members such as columns in very stiff buildings are subjected to seismic load reversals, and on that basis the inference is that all such columns should be subjected to clause 9.5.

68. However, clause 9.5.1 "*General*" states that the clause only applies to "*...beam/column joints that are subjected to forces arising as a result of inelastic lateral displacements of ductile frames.*" This is very much more specific than what may be inferred from clause 9.4.1. This leads me to conclude that clause 3.5.14.3(a) may well govern the beam/column joints of grids 2 and 3 of the CTV building if the analysis of the structure showed that the building complied with the requirements of that clause. However, it is unlikely that the beam column joints on Grid F complied with NZS 3101.

69. I conclude that the beam/column joints for all circular columns were unlikely to comply with NZS3101.~~ on grid f were unlikely to comply with NZS3101, and that those on grids 2 and 3 may have complied but I have not performed calculations to confirm this.~~

Plan Asymmetry and Vertical Irregularity

70. The Hyland Smith report at page 112 discusses the CTV building *Plan Asymmetry and Vertical Irregularity*. My interpretation of clause 3.4.7.1 of NZS 4203 is that the plan of the building was not asymmetrical. NZS 4203 would, in my view, class the CTV building as being a "*reasonably regular structure*", of "*moderate eccentricity*". I categorise the structure as reasonably regular on the basis that all floors are the same shape and size, and, apart from the bottom and top floors, the interstorey height is constant. The height differences in the bottom and top storeys are not in my opinion excessive.

71. Commentary clause C3.4.7.1 of NZS 4203 has a definition of moderate eccentricity with which I believe the structure complies, although I accept that the definition is not entirely clear. The NZS 4203 commentary has, as an aim, achieving symmetrical structures, but it is certainly not a requirement of the standard. Methods of analysis of unsymmetrical structures are incorporated in the standard.

72. The comments made in the Hyland Smith report at page 112 do not, in my view, reflect a correct interpretation of the standard, especially as understood at the time by practising structural engineers.

Wall on Line A

73. This issue is raised at pages 112 to 113 of the Hyland Smith report. It is clear, from both the drawings and from the calculations related to the masonry wall on line A, that the wall was to be seismically separated from the seismic load resisting elements of the structure. The design was, in my view, consistent with this position.

74. It is far from clear whether the drawings "... *showed the top course to be grout filled...*" as indicated by the Hyland Smith report at page 113. I understand that when work was being carried out on the wall after the September 2010 earthquake, the top course of the block work was observed not to be filled to the top (**WIT.FORTUNE.001.3**). The design calculations have a very clear detail as to how to construct that joint and show the top course as not being fully grout filled (see page G53 of the CTV building calculations).

75. Further, greased vertical bars were shown on the drawings between the top of the block work and the beam above (see drawing S9). Even a detail in the calculations for the design of the vertical greased bar to resist the out of plane loads from earthquake generated forces, was included. This shows a clear intention for the block work to be separated from the beam above, and I believe that an experienced contractor would have understood that intention. The site foreman confirmed in an interview with the Department of Building and Housing that this was his interpretation (**BUI.MAD249.0042.21**).

76. In my opinion the block walls on grid a complied with the seismic separation principles of NZS4203.

Diaphragm Connection

77. This matter is raised at page 113 of the Hyland Smith report.

78. Disconnection of the floor slabs (diaphragms) from the northern shear core walls has been suggested as a cause, at least in part, of the collapse of the CTV building, in several of the collapse scenarios of the Hyland Smith report.

79. Most of the horizontal load generated by an earthquake is attributable to the floor slabs of a structure. The load from the slabs has to be transferred into the horizontal load resisting elements of the structure (the shear walls in the case of the CTV building), and from the shear walls, the load is transferred into the surrounding ground through the foundations. The question whether the load path from the floor slabs to the shear walls complied with NZS 4203 and NZS 3101 has been discussed in the Hyland Smith report at page 113 and in more detail in Appendix G.

80. I have reviewed the calculations related to the diaphragm connection to the north core walls and have concluded that the connection did not comply with NZS 4203.

81. I have also reviewed the loads agreed by Holmes Consulting Group and Alan Reay Consultants for the remedial work undertaken in 1991. There is a letter from Alan Reay Consultants Limited to Holmes Consulting Group dated 2 February 1990, received by Holmes Consulting Group on 7 February 1990 noting the agreement of the loads required to be resisted by the remedial work. I have concluded that, with that work completed, the connection at all floors complied with NZS ~~4263~~4203 in the North South direction. I do not believe that the slab connection to the North core shear walls complied in the East West direction.

82. The Hyland Smith report appears to indicate that there was no diaphragm/wall connection failure until after other structural failures that started the sequence of events leading to total collapse. However, this seems to be a controversial point. I hold the view that an early disconnection at possibly level 2 or 3 could have occurred, leading to instability of internal columns between levels 1 to 2 or 2 to 3. This is supported by the Mr Holmes peer review of the Hyland Smith report (**BUI.MAD249.0372**) at page 15, first and second paragraphs.

Robustness

83. I now refer to the question of robustness raised in chapter 9 of the Hyland Smith report (pages 113-114). Robustness as a concept was understood by structural engineers at the time of the design, but my understanding was that if the design complied with the standards of the day (in this case NZS 4203 and 3101), then the required robustness was regarded as being incorporated in the design.

Documentation

84. The issues raised at page 114 of the Hyland Smith report under "*documentation*" are covered in various paragraphs of this evidence - construction joints in paragraphs 89 and 90, masonry infill in paragraphs 73 to 75 and spandrel panel gaps in paragraphs 59 to 62. The issue of starters from precast beams on grids 1 and 4 is, I believe, shown on drawing S15.

Construction Issues

85. The Hyland Smith report refers to a number of construction issues relating to the CTV building. Item 2 on page 4 of the letter of the Royal Commission to the Christchurch City Council, dated 27 April 2012, refers to these construction issues. My headings below relate to the list of issues in the Royal Commission's letter.

The concrete strength distribution

86. Concrete strength distribution, and other issues related to the concrete testing, have been the subject of two submissions made to the Royal Commission. These are **BUI.MAD249.0373** and **BUI.MAD249.0362**. It is inappropriate for me to add any further comment on the concrete strengths as reported in the Hyland Smith report and the Hyland materials report, given that it is not my area of specialist expertise.

Lower than expected concrete strengths

87. I refer to paragraph 86 above.

Cores taken from a line 4-D/E column

88. I again refer to paragraph 86 above.

Construction joints

89. Surface finishes of construction joints in the precast concrete were specified in clauses 3.6 and 3.12 of the precast concrete specification for the CTV building. Section 2.3 of the concrete specification required compliance with NZS 3109:1980 "*The Contractor shall comply with all requirements of NZS 3109:1980 except where specified otherwise herein...*". Preparation of construction joints is covered in that standard.

90. It would be unusual, in my view, for a Council building inspector to give detailed scrutiny to how construction joints were prepared. Very frequent presence would be required on site and even then many construction joints would already be covered up, (although they may have been prepared only a few hours earlier), and others would be inaccessible. I would not therefore realistically expect a Council building inspector to pick up poor preparation of construction joints.

Smooth precast to cast in situ interfaces on precast beams

91. I refer to paragraph 90 above.

Shell beam reinforcing steel poorly developed in line C wall

92. The fact that reinforcing steel details at the interface of precast members to in situ concrete were not as detailed on the drawings, would be very difficult to pick up by a building inspector doing limited inspections. I would be surprised if a Council building inspector were to pick up such an error.

Connection at line 4-D/E column missing a bar

93. I refer to paragraph 92 above.

North core being significantly out of plumb

94. I am not surprised that the north core was out of plumb after the collapse of the building. Before the building collapsed, the north core carried the load of the floors spanning into it, particularly along grid line 4. After the collapse, that load was distributed far more uniformly over the adjacent site. Thus there would have been 'elastic rebound' of the foundation soils under the north core when it was unloaded by the collapse. The unloading on grid 4 would have been much greater than on grid 5 and so the north core would have tilted towards the north. I have not done any calculations to estimate the tilt as the detail is a geotechnical issue.

Vertical seismic separation joints in the masonry infill

95. The vertical seismic separation joints in the masonry infill may have been compromised by mortar on the outer face. This would have been very difficult to observe as the wall was built adjacent to an existing wall. Intermittent inspections of the gap are unlikely to have picked up the problem if there were only isolated instances of mortar in the gap. If the problem was more widespread, it could possibly have been expected to be discovered by some building inspections. However, if it was isolated, then I would not expect that a building inspector would be likely to pick up on this issue.

96. I do not know what level of inspection was stipulated in the consultant's terms of engagement. The ultimate responsibility for construction lies with the contractor. It is up to the contractor to construct the building according to the drawings and specification. The contractor is the entity that is in control of the site, particularly with a new building project. What level of construction observation is to be provided by the consultant structural engineer is a matter for the owner. The consultant will advise the owner on the level of observation that would be appropriate, but the final decision is for the owner.

A REVIEWING ENGINEER'S ASSESSMENT OF COMPLIANCE WITH NZS 3101 CLAUSES 3.5.14.3(A) AND (B)

97. I have been asked to comment from my own perspective as to how a reviewing Council engineer could reasonably have interpreted the CTV building plans in the light of the standards of the time. I discuss this in the following paragraphs of my evidence.

98. Clause 3.5.14.1 of NZS 3101 defines a secondary structural element which is then addressed in the rest of the clause. Paragraphs 38-40 of my evidence discuss the interpretation of this clause, but I draw attention to it again as it has a significant influence on the discussion which follows.

99. The columns ~~on grids 2 and 3~~ appear to have been secondary elements as they are, as stated in clause 3.5.14.1 of the Standard, "*...not necessary for the survival of the building as a whole under seismically induced lateral loading*".

100. A reasonable interpretation of this requirement is that a column could fail resulting in a localised (part) failure of the structure, but not collapse. The consequence of such an interpretation is that the requirements of clause 3.5.14.3 then become relevant. Of particular interest is clause 3.5.14.3(a). If the imposed deformations, (v times delta), were not exceeded under the criteria in this clause, then the seismic requirements need not be satisfied.

101. This raises essentially the same debate as to whether the columns complied with NZS 3101.

102. Related to these issues, is the point that a Council reviewing engineer is likely to have looked at the overall design and noted that it was a shear wall structure. The reviewing engineer would know that shear wall structures are relatively stiff and therefore probably fall into the category of a structure covered by clause 3.5.14.3(a) of NZS 3101. The conclusion flowing from this would have been that the gravity load columns (i.e. all those in the CTV building) did not need to comply with the "*additional seismic requirements of that code...*". On this basis, the reviewing engineer could in my view have been justified in assuming the columns complied.

103. For Christchurch, which was in an area of only moderate seismicity, an assessment that went along the lines of "gravity only columns in a building with adequate shear walls should not need to be designed for seismic loading", had some reasonable basis.

FURTHER OBSERVATIONS ON THE HYLAND SMITH AND EXPERT PANEL REPORTS

104. I refer to section 11 of the Hyland Smith report, and clause 5.14 of the Expert Panel report. With minor variations, the recommendations of both reports identify five "potential vulnerabilities in large earthquakes', derived from the performance of the CTV building in the February 22 aftershock.

105. There is an inference that all five potential vulnerabilities were not particular to the CTV building but were rather a generic issue related to the standards of the time. This is particularly true for non-ductile columns and diaphragm connections where there is a direct reference to the problem not being addressed in standards prior to NZS 3101:1995 and NZS 4203:1992.

106. Two dimensional Elastic Response Spectrum Analysis (ERSA) was allowed for buildings such as the CTV building. I refer again to paragraph 33 of my evidence. The two dimensional modal analysis noted in clause 3.4.7.1(b) of NZS 4203 is what a two dimensional ERSA would produce. How the two dimensional ERSA analysis was carried out was that the structure was modelled in three dimensions and then the earthquake input was applied along the two principle axis of the model as two separate analysis.

107. In ETABS at that time, the diaphragms were modelled as rigid and the model of the structure was able to be constrained to respond in the direction of the earthquake input only. This meant that 'out of plane' torsional effects were not reported in the output from the analysis and therefore torsional effects were not readily available from the model. Applying the NZS 4203 prescribed accidental and actual eccentricities to the model~~—required accidental eccentricities and actual eccentricities~~ in combination~~, took into~~ was intended to take account of torsional effects in ~~this~~ a quasi ~~two~~ three dimensional way ~~analysis~~.

*Holmes Peer Review of the Hyland Smith report and the Hyland materials report **(BUI.MAD249.0372)***

108. On page 3 of the Mr. Holmes Peer Review report he discusses "Gravity Frame Ductility as Required by Code". I agree with the first sentence – "*this issue is*

discussed (in) several places in the overall documentation, but in my opinion is never clear".

109. I agree in general terms with the Mr. Holmes Peer review, but I am of the opinion that he places undue emphasis on where design for limited ductility (NZS 3101 chapter 14) should be used. See paragraph 42(e) above for my understanding of when limited ductility would be used in the design of the CTV columns.

Dated: 8 August 2012

Signed by:

A J O'Leary

Name: Arthur Joseph O'Leary