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**

THIRD STATEMENT OF EVIDENCE OF ARTHUR JOSEPH O'LEARY IN RELATION TO THE CTV BUILDING

HEARING BEGINNING: 25 JUNE 2012

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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. This is my Third Statement of Evidence. My relevant qualifications and experience were provided in my First Statement of Evidence (**WIT.OLEARY.0001**).

3. This evidence provides comments on:

   (a) floor loadings relating to the change of use associated with the going places tenancy in response to paragraphs 39-43 of the evidence of Dr Reay (WIT.REAY.0004.8);

   (b) the second and third statements of evidence (and related reports) of Mr Douglas Latham, in relation to his ERSA analysis of the CTV building structure; and

   (c) Mr Henry's comments in relation to the evidence and related reports of Mr Douglas Latham, made in cross examination on August 2.

4. 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.

Floor Loadings

5. Dr Reay in his evidence (WIT.REAY.0004.8) at paragraphs 39-43 comments about the loads following a change of use involving the Going Places tenancy.

6. Dr Reay is correct when he says at paragraph 41 that the live load for a classroom was 3.0 kPa under Table 3.4.1 of NZS 4203:1992. This does not alter the overall safety of the building except for the necessity to check that the floor slab can support the extra 0.5 kPa. In effect, this increases the total load

on the floor slab by 7.1% (0.5/7.0). This is only applicable to the serviceability of the floor in normal operation.

7. Serviceability is related to how the structure performs its functions from day to day. For floor slabs, it governs doors not closing, gaps under partitions and similar matters. It does not relate to the safety or strength of a building structure.

8. In fact, the load factors governing strength for dead load and live load in NZS 4203:1992 (Table 3.4.1) were decreased to 1.2 and 1.6 respectively from the NZS 4203:1984 values which were 1.4 and 1.7 respectively. Accordingly, adjusting for the changed 1992 load factors meant an actual reduction in the effective ultimate limit state design load on the floors from 10.55 kPa to 10.2 kPa. The ultimate limit state is the limit state used to design for strength.

9. Because of reduced factored load allowances over the whole of the building, increasing the floor live load from 2.5 to 3.0 kPa would not in my view have had any negative effect on the overall strength requirements for columns, beams and foundations.

10. There are however two issues raised in paragraph 41 on which I want to comment. These are:

    (a) the suggestion that the change of use involved a change in the seismic risk factor;

    (b) Dr Reay's calculation of reduced seismic design live load as 1.8 kPa.

The Risk Factor

11. My interpretation of NZS 4203:1992 is that one floor of classrooms in a six storey building would not increase the seismic risk factor for the whole building. Table 2.3.1 (page 17 of NZS 4203) – 'Classification of Buildings' states that Category II buildings are 'Buildings which *as a whole* contain people in crowds' (my emphasis).

12. In my opinion, the seismic risk category would be Category IV - 'Buildings not included in any other category' so the seismic risk factor at Table 4.6.2

(page 45) would have remained the same as when the CTV building was originally designed in 1986. I consider that clause C2.3.1 of the commentary to NZS 4203:1992 relating to Category II, supports this view. In the examples listed for Category II buildings, "school classroom buildings" are included.

Calculation of seismic load

13. Dr Reay's calculation of "reduced seismic design live load" is in my view incorrect as I interpret clause 2.4.3.2 of NZS 4203:1992. He states the reduced live load is 1.8 kPa. I calculate it to be 0.90 kPa. He seems to have missed a factor out of his calculation of the reduced live load.

14. There are two matters to consider. Clause 2.4.3.2 gives a factor of 0.6 in table 2.4.2. There is also a factor to be used in accordance with clause 3.4.2.1(b) that calculates to be 0.50. The effect of these in combination is the reduced live load is 0.6x0.5x3.0=0.90. Dr Reay appears to have left out the 0.5 factor. It is referred to in clause 2.4.3.2 but I admit somewhat obtusely.

15. The overall effect of the correct reduced live load for the classroom under NZS 4203:1992 is to increase the classroom reduced live loading under seismic loading by approximately 8.4%, but the overall effect on the required seismic design strength of the building would be less than 0.3%.

Second Report of Douglas Latham

16. I have read Douglas Latham's second and third statements of evidence and accompanying reports (WIT.LATHAM.0002 and WIT.LATHAM.0003), which cover the analysis of the CTV building structure using the ETABS analysis procedure as would have been undertaken as part of the analysis of the CTV building structure in 1986. I refer to the reports as the first and second reports respectively, and comment on them below.

17. The ERSA analysis option reported in Mr Latham's first report is a 3 dimensional analysis with flexible foundations. As allowed by clause 3.4.7.1(b) of NZS 4203:1984, a 2 dimensional analysis would in my view have complied, but Mr Latham has chosen to use the more sophisticated 3 dimensional analysis.

18. Although Mr Latham has modelled flexible foundations, he has corrected the results that he obtained for interstorey drifts reported from the analysis so that

the analysis neglects foundation rotation altogether. In my view this approach is permissible, indeed required under a literate reading of clause 3.8.1.2 of NZS 4203: 1984. I do however have some reservations with an approach which removes from consideration all foundation rotations where ever they occur in the building.

19. The alternative approach is that clause 3.8.1.2 applies only to the rotation of a continuous foundation under the entire building. Under this approach clause 3.8.1.2 does not apply to the rotations associated with the footing of individual members located variously throughout the base of the building. I have considerable sympathy for this approach, albeit that it offends the express words of the clause. If the clause is interpreted literally the resulting drifts for some members become quite artificial which I do not think would have been the intention of the drafters of the code.

20. The problems with the application of clause 3.8.1.2 highlight, in my view, how there can be more than one defensible interpretation of aspects of the codes.

21. Subject to these reservations I am in general agreement with how Mr Latham has modelled the CTV building structure. I make the additional qualification that I have not studied the input data he has used,. Mr Latham has used cracked stiffnesses for various structural components (Table 3 of his first report) which I consider realistic.

22. I was initially surprised that the first mode natural period of the structure in the North/South direction is so long (2.07 seconds), but I understand that it is significantly influenced by the soil stiffness chosen for the foundation flexibility part of the model. If the natural period in the North/South direction is reduced to 1.20 seconds (a period consistent with that suggested by Mr Smith in his Seventh Statement of Evidence attachment B), there would be noeffect on the interstorey shears reported for the design of the North – South shear walls.(see Fig 3 of NZS 4203:1984). The interstory drift would however by significantly decreased under this approach.

John Henry Cross Examination on Thursday 2 August 2012

23. I have reviewed the transcript of Mr. Henry's evidence and cross examination, particularly in regards to what he said about the original Alan Reay Consulting Engineer seismic calculations for the CTV building. I find Mr Henry's arguments relating to the various factors associated with the seismic coefficients difficult to

follow. I cannot reconcile his multiple factors of 0.8 with the original calculations. I have looked at the base shears shown on page S18 of the calculations and find that an extra factor of 0.9 may be included in the conversion of 3300kN base shear to 1881kN (the shear at the bottom of page S18). My calculations show that the base shear should probably be 2114kN. I derive this figure from 3300kN multiplied by 0.712 (see S15 for the factor given towards the top of the page) multiplied by 0.9, allowed because a modal analysis has been done to derive the base shear. The 0.712 is to compensate for the adjustment of the originally chosen period of the building from 0.7 seconds to 1.06 seconds, the period presumably resulting from the ETABS analysis.

24. I have checked the first report of Mr Latham and I am satisfied that the base shear values he derives in Table 10 are correct and consistent with NZS4203:1984. Again, I cannot follow the line of reasoning that is followed by Mr Henry in his cross examination on Day 68, pages 95 to 98 of the draft transcript (TRANS.2012.0802.99-103). I point out that any period over 1.2 seconds attracts the same 'seismic coefficient' 'C' of 0.075 because Table 3 of NZS4203 gives the basic design coefficient as constant for periods beyond 1.2 seconds.

Dated: 8 Aug 2012

Signed by:

Name: Arthur Joseph O'Leary