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**

SECOND BRIEF OF EVIDENCE OF JOHN HENRY

DATE OF HEARING: COMMENCING 25 JUNE 2012

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INTRODUCTION

1. I have been asked by counsel assisting the Royal Commission to comment on the Elastic Response Spectrum Analysis (ERSA) described in the Seismic Analysis Report prepared by Douglas Latham dated 25 July 2012 and to further comment in relation to the Secondary Frame Design Review Report also prepared by Mr Latham, dated 31 July 2012.

2. I have read the Code of Conduct for Expert Witnesses and agree to comply with it. I confirm that the issues I have been asked to address are within my area of expertise and I have stated where my opinion is based upon information supplied by others.

3. In addressing these issues I have read and considered the following:

    a. The Seismic Analysis Report prepared by Douglas Latham dated 25 July 2012.

    b. The Secondary Frame Design Review Report prepared by Mr Latham dated 31 July 2012.

    c. Report to the National Science Foundation on the ETABS program, 1975.

    d. Information provided to me by Mr Ian McCahon on 5th August 2012, which is described below.

    e. Information provided to me by Dr. Clark Hyland on the 4th August 2012, which is described below.

    f. Information provided to me by Professor Carr on the 4th August 2012, which is described below.

    g. Calculations carried out on 6 August 2012 by Brendan O'Conner, structural engineer at Eliot Sinclair. Mr O'Conner has assisted me by carrying out an independent check of the inter-storey drift values that I have mentioned in my evidence.

h. Discussions with structural engineer John Brouard, senior structural engineer and Director of Eliot Sinclair. Mr Brouard has assisted me by providing a review of my evidence for technical consistency.

4. Counsel assisting asked me to consider these matters on 3 August 2012. Given the limited time, it has not been possible to consider them as thoroughly as I would have liked to. I have indicated below where this has affected the opinion I have formed.

My interpretation of what Mr Latham has done in his Seismic Analysis and Secondary Frame Design Review Reports

5. Mr Latham has re-modelled the CTV building using ETABS with the following significant modifications compared with the original David Harding model and/or the model used for the Hyland-Smith report:

a. Ground Stiffness

6. Ground stiffness properties have been included with some variation in stiffness over the building plan, such that the ground is softer under the North core, line F and line 4 than it is under the remainder of the building.

7. The soil properties used to model the soil stiffness are not from the original soil investigation report provided by Geotech Consulting (Ian McCahon) in 1986 but were recently requested of Mr McCahon by Dr Reay. Ian McCahon advised me of this by telephone on 5th August 2012. Ian McCahon said that Dr Reay asked him for soil properties for the site as he would have evaluated them in 1986. Ian McCahon advised me that the soil stiffness properties he provided to Dr Reay were for the Modulus of Subgrade Reaction, which may be used for various purposes such as calculating the bending forces in the foundation beams. He said it would be beyond his expertise to say that they were appropriate for dynamic earthquake analysis, but he was concerned that it may be the case that the soil properties were not appropriate, mainly because possible complex effects of pore water pressure would still need to be considered.

8. I phoned Clark Hyland on 4th August to discuss the foundation conditions for the Hyland-Smith model. He advised that the Hyland-Smith model included soil stiffness properties but of greater stiffness than that now used by Mr Latham. He also advised that a range of soil stiffness properties should be considered to test the sensitivity of the model.

b. Gravity load secondary elements

9. Gravity load secondary elements have been excluded from the analysis model, so that only the shear walls are included, as was done by David Harding but not Dr Hyland and Mr Smith. Excluding these secondary elements from the analysis was normal practice in the 1980's.

c. Building Weight

10. The weight of the building has been recalculated by Mr Latham using a detailed elemental breakdown, with the result that the weight used in the revised analysis is now 4% less than the original weight used by David Harding.

c. Ground Storey Height

11. The ground storey height has also been adjusted to 3.7m. I do not know what it was in the Hyland-Smith model, however David Harding's calculations show that he used 3.9m in his design.

Effect of Modifications

12. The effect of Mr Latham's modifications is that the building periods of vibration are now longer than those derived by David Harding and Dr Hyland/Mr Smith. The longer periods assessed have enabled Mr Latham to use reduced earthquake loads and a reduced torsional eccentricity on the building. I have not been able to verify that the longer periods are appropriate.

13. The significant details or results of the modifications to the ETABS analysis are discussed as follows.

14. The revised north-south period is now 2.07 seconds compared with 1.22 seconds for the Hyland/Smith model and 1.06 seconds for David Harding model. This revised period of 2.07 seconds is a most unusual result, given that in my

experience, the period of vibration for a 6 storey structure in Christchurch would be around 0.6 seconds, assuming the normal design condition of a fixed base structure.

15. The east-west period is now 1.19 seconds compared with 1.03 seconds for the Hyland/Smith model and 1.06 seconds for David Harding's model. This smaller change compared with the north-south direction indicates that the ground stiffness had less effect on the revised ETABS model in the east-west direction.

16. Based on the longer periods, together with the 4% reduction in building weight, Mr Latham has reduced the overall earthquake loading by 18% from David Harding's original design, at least for the Static load case, which has been reduced from 2350KN to about 1920KN.

17. The design force for the critical South Coupled Shear Wall has not changed significantly from David Harding's. i.e. 1439KN for Douglas Latham compared with 1424KN determined by David Harding's design before he applied modifications to it for the design of the South Coupled Shear Wall. This wall was critical to the performance of the CTV building for loading in the east west direction. The fact that the design load has not changed seems inconsistent with the overall 18% reduction in the total load on the building. There is not enough information in Mr Latham's evidence for me to understand how this has come about.

18. The modifications that David Harding applied to the loading on the South Coupled Shear wall were, (1) reduction in the S factor from 1.0 to 0.8 and (2) an additional 0.8 scaling factor to 0.8 of the Static load, which I believe was incorrectly used, as stated in my previous evidence. This resulted in an effective value for the S factor of 0.8 x 0.8 x 0.8 = 0.51 for the design of the South Coupled Shear Wall.

19. Hence, the design shear force used for the South Coupled Shear Wall by David Harding was 912 KN, i.e. 64% of 1424KN initially determined for the south-coupled shear wall for the period of 1.06 seconds.

20. By contrast, for the East-West direction, the earthquake loads calculated by Mr Latham for his analysis and subsequent derivation of loads and deflections on the Grid 2 gravity frame, adjacent to the South Coupled Shear Wall, are based on

SM=0.8 but in my view should have been based on the actual SM value used by David Harding in the final design of that wall, i.e. S = 0.51.

21. This would then result in a larger K/SM scaling factor i.e. 2/0.51 = 3.91 compared with 2/0.8 = 2.5 used by Mr Latham. On this basis, some of Mr Latham's deflections are under-reported by a factor of 0.8/0.51 = 1.56.

22. In the revised analysis, Mr Latham has determined the Centre of Rotation to be such that the torsional eccentricity from the centre of mass has been reduced by about 5 metres from that given in the Hyland-Smith report. Based on the assumption that the origin of coordinates used by Mr Latham is the same as that used by Mr Harding, i.e. in the south-east corner of the building, the revised Centre of Rotation is now in the middle of Bay 3-4, not the stairwell of the North Core as was determined from the Hyland-Smith report. I am surprised that the Centre of Rotation has changed to this extent in Mr Latham's revised analysis. I cannot understand how this has come about and I would not be able to comment on this without considerably more detail from Mr Latham.

23. This apparent reduction of about 5 metres in the eccentricity together with the 18% (1920KN versus 2350KN) reduction in earthquake loads has in turn reduced the torsional shear component in the shear walls, so that the calculated eccentricity now just meets the "moderate eccentricity" criterion given in the commentary to the code.

24. The new finding by Mr Latham that the building model is now of moderate eccentricity is then used to justify that an ERSA analysis was not required, and on that basis the higher ERSA drifts are disregarded.

25. Static and ERSA drifts are compared by Mr Latham for a fixed base shear wall structure. These deflections exclude the component caused by foundation rotation from the assumed ground stiffness.

26. The ERSA drifts are shown to be higher than Static, E.g. 0.53% versus 0.46% on line 1.

27. Static drifts for the fixed base structure are therefore adopted by Mr Latham to proceed with checking the secondary elements to show that they meet the code

except for the horizontal joint shear reinforcing, which Mr Latham has agreed did not comply with the code because there was not enough reinforcement.

Comments

28. The earthquake loads on the building have been reduced by 18%, mainly on the basis of the softer building response, which resulted from the inclusion of soil stiffness properties in the model.

29. The reduced loads were then used to calculate the inter-story drifts at the critical positions. These drifts were calculated on the basis of a fixed base structure, without including the implied foundation rotations which were the basis for the reduced loading. Mr Latham states that deflections were calculated in this way on the basis of the code clause 3.8.1.2, which says that computed deformations shall be calculated neglecting foundation rotations.

30. I believe that this would be reasonable if the foundation rotations were small, or if all parts of the structure were to undergo the same foundation rotation, but I cannot see that this would have been the case with the CTV building because:

    a. Firstly, the CTV building had differing types of foundations, for example, the shear walls were founded on relatively stiff beams compared with the isolated pad footings under the internal columns on line 2 adjacent to the South Coupled Shear wall.

    b. Secondly, the gravity frames were relatively flexible compared with the shear walls and their foundations, so the gravity frames were not subject to earthquake loads that would cause their foundations to rotate.

    c. Thirdly, the building deflections caused by the foundation rotations calculated by Mr Latham were relatively large compared with the fixed base deflections, particularly in the north-south direction.

31. In the north-south direction along Grid A, the elastic deflection due to rotation was about 3.3 times the fixed base deflection, i.e. 102 versus 31mm. In the east-west direction on line 1, the elastic deflection due to rotation was about 1.3 times the fixed base deflection, i.e. 37 versus 28mm.

32. The 1984 code allows for foundation rotations to be neglected in the calculation of the building deflections, as given in clause 3.8.1.2 of NZS 4203:1984.

33. However, I believe this is on the basis that the foundation rotations do not affect the relative inter-storey drifts because all storeys rotate together. This would be correct if the building were founded on one large foundation, but not if there were differing foundations and differing seismic loadings on the elements supported by those foundations as it was for the CTV building.

34. The significance of this is that the foundations under the interior gravity frames of the CTV building would not rotate compatibly with the stiff shear walls which would tend to rotate as rigid bodies. However, because the interior columns are connected to the shear walls by the floor diaphragms, the increased deformation caused by the rotation of the shear wall foundation would be imposed on the interior columns over the height of the ground floor. This means that the ground floor columns, especially those on line 2, would be subject to increased inter-storey deformation caused by foundation rotation.

35. In the north-south direction on Grid A, the ground storey total elastic deflection is calculated by Mr Latham as 23.3mm, of which 19.6mm is caused by foundation rotation, if I understand his figures correctly. Therefore, if the total deflection including foundation rotation is considered, with the fixed base inter-storey deformation component scaled by K/SM = 2.5, then the ground to first floor drift would be 19.6 + ((23.3-19.6) x 2.5) = 29.0mm, compared with 9.4mm given in Mr Latham's Seismic Analysis Report. This would increase the drift from 0.25% to 0.77%.

36. In the east-west direction on Grid 2, the total elastic deflection including foundation rotation is 8.0mm of which 6.1mm is due to rotation. On the same basis as outlined for the north-south direction including foundation rotation and using K/SM = 2.5 the drift would increase to 10.8mm. However, if the value of K/SM = 3.91 that was effectively used by David Harding in the design of the South Coupled Shear Wall is applied, then the ground to first floor inter-storey deformation would be 6.1mm + ((8.0-6.1) x 3.91) = 13.5mm, compared with 4.5mm given in Mr Latham's Seismic Analysis Report. This would increase the drift from 0.12% to 0.36%.

37. In summary, the approach taken by Mr Latham in the revised analysis has not taken into account that the secondary frames would be forced to undergo the same

amount of lateral deflection as the primary shear walls, including rotation of foundations over the height of the ground storey.

38. To clarify this I make the following explanation regarding the basic effects of ground stiffness on the structure.

Effects of ground stiffness on the structure deflection

39. As a structural engineer, I have a basic knowledge of soil-structure interaction and I would rely upon advice from a Geotechnical engineer for advice on the appropriate parameters to use if undertaking an analysis that included the soil stiffness properties. However, for the purpose of making my evidence clear, I give the following brief explanation of my understanding about how the nature of the supporting ground can affect the deflection of the structure.

40. Depending on how stiff the ground is, the compression of the ground under foundations resisting earthquake loads can allow the foundations to rotate. If the ground is relatively soft and the foundation beam stiff, the foundations and the superstructure that they support may rotate as a rigid body. The rotation increases the lateral deflection of the building but does not necessarily increase the inter-story drifts of one floor relative to the other.

41. If the entire structure of the building was to be supported on one rigid foundation then the entire building would rotate as a single rigid body with all of the structural elements undergoing the same amount of increased lateral deflection due to rotation of the foundations.

42. However, if the various vertical elements are supported on separate foundations then the degree of rotation may vary depending on the nature of the individual foundations. For example, stiff shear walls that are supported on rigid foundation structure such as basement walls would tend to rotate as a rigid body, but isolated pad foundations under flexible secondary columns which are not subject to seismic forces would not rotate to the same degree due to the absence of a stiff superstructure element connected to the foundation that would cause it to rotate.

43. This means that secondary elements or frames connected to the main shear walls by the stiff floor diaphragms would be dragged through much the same amount of

lateral displacement as the shear walls over the height of the ground storey, and would undergo deflections caused by interstorey deformation as well as the deflections caused by rotation of the foundations of the shear walls in the ground storey.

44. Therefore, for a shear wall protected gravity load system, if significant foundation rotations are calculated for the main shear wall foundations, as Mr Latham has calculated, then in my opinion those deflections need to be taken into account in the design of the secondary frame elements of the ground to first floor storey in particular.

Design Procedures used in the early 1980s with regard to Ground Stiffness

45. In my experience of using the ETABS programme and designing multilevel buildings in the 1980s, the effects of ground stiffness were not included in the analysis and design process. At that stage, the effects of ground stiffness on the structure were not well understood and I believe were not normally considered to be significant for reasonably uniform ground conditions.

46. The only exception to this that I experienced was in the design of the 12 storey base isolated Union House building in Auckland. The base isolation system for this building included deep piles which were free to displace within steel tubes bored into the ground. The ground therefore became part of the foundation system and it was essential to determine reasonably accurately what the interaction of the ground was with the piles. This was an exceptional case requiring a sophisticated time history analysis which modelled the ground properties using springs and dash pot parameters. The analysis was carried out for a two-dimensional frame using the program "Ruamoko" at the University of Canterbury, with the assistance of Professor Carr.

47. The ETABS programme had no facility to model the ground stiffness, other than with a "dummy storey" at the base of the building, which would have been an approximation requiring comprehensive investigation and correlation with actual site data. I do not know of any example where this approach was used. I discussed this aspect with Professor Carr to check if he knew of any other way that the ground stiffness could have been accounted for using the 1986 ETABS programme. He confirmed that he did not know any other way using ETABS at

that time, although two dimensional soil structure interaction modelling was being done at the University of Canterbury at that time using the programme Ruamoko.

48. I also discussed the effect of ground stiffness and foundation rotations on the structure of the CTV building with Professor Carr before forming my opinion expressed in this evidence. The key points covered in the discussion were:

   a. That the building rotations would need to be taken into account in assessing the drifts for the gravity frames because their foundations were independent of the main shear walls and would not undergo “rigid body” rotations along with the shear walls.

   b. That the soil stiffness properties used in analysis would need to be appropriate for dynamic earthquake loadings, not static loadings.

Conclusion

49. I am unable to comment on whether or not the revised analysis and subsequent calculations carried out by Mr Latham are appropriate to show that the CTV building was code compliant. However, there are several aspects of the calculations that I do not agree with.

50. The key factors are:

   a. The soil stiffness properties assumed in the analysis have not been confirmed as suitable for dynamic earthquake analysis and appear to have altered the response of the building in the north-south direction to an unrealistic degree compared with other building designs of the time. i.e. period increase from 1.22 to 2.07 seconds in the north-south direction.

   b. If the approach proposed by Mr Latham is to be pursued, then I believe that the foundation rotations should be included for calculating the ground floor drifts and member forces in the gravity frames. If the rotation is included, then the resultant drifts are three times greater on Grid A, i.e. increased from 0.25% to 0.77%, which would significantly affect the results.

   c. The SM factor used for the East-West direction to calculate the drifts and member forces applied to the gravity frame on Grids 2 is not consistent with that used by David Harding in the actual design. If the actual design SM factor was applied, it would result in greater drifts in the ground storey than those used by Mr Latham to assess the secondary element capacities on Grid 2. i.e. the drifts would increase from 0.12 to 0.36%.

51. I have not had time or resources to follow through with the full implications of the above conclusions with regard to the capacities of the secondary members calculated by Mr Latham. However, my preliminary view is that I expect that the north-south drifts at 0.77% will make a significant difference to the assessed capacity of the gravity frame elements.

Signed: ………………………………

JOHN HENRY

Date: 7th August 2012