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 STATEMENT OF EVIDENCE OF DEREK SCOTT BRADLEY IN RELATION TO THE NON-LINEAR SEISMIC ANALYSIS OF THE CTV BUILDING

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INTRODUCTION

1. My name is Derek Scott Bradley. I live in Auckland. I am currently employed with Compusoft Engineering Ltd as a senior structural engineer.

2. As the principal author of the 'CTV Non-linear Seismic Analysis Report' I have been asked to provide evidence to the Canterbury Earthquake Royal Commission (the Royal Commission) relating to specific aspects of Compusoft Engineering Limited's involvement in the CTV Building Non-Linear Seismic Analysis Report (the Compusoft Report) provided to StructureSmith Ltd, Hyland Consultants, and the Department of Building and Housing (DBH).

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

QUALIFICATIONS AND EXPERIENCE

4. I hold a Bachelor of Engineering with Honours Degree in the Civil Discipline (BE Civil (Hons)), am a Chartered Professional Engineer (CPEng), and am qualified as an International Professional Engineer (IntPE).

5. I am a chartered professional engineer with over fifteen years of extensive structural design experience both in New Zealand and abroad. My experience includes structural design, analysis and management of commercial, civil and industrial projects.

6. I have been employed by Compusoft Engineering Limited as a senior engineer for 8 years. During this time I have undertaken numerous structural analyses for both Compusoft's in-house designs, and for external structural design consultants. Compusoft Engineering is the New Zealand agent for the CSi suite of structural analysis packages and part of my role is to provide specialist technical support for the 3-D finite element packages SAP2000 and ETABS. Analyses experience includes Equivalent Static, Modal Response Spectrum, Non-Linear Pushover, and Non-Linear Time-History analyses techniques.

7. A selection of relevant projects I have been involved in are listed below:

- Seismic Assessment and design of strengthening work for the historic St James Theatre in Auckland, which included the use of time history analysis.

- Principal reviewer of the non-linear time history analysis undertaken of the Auckland Art Gallery seismic assessment and upgrade by Holmes Consulting Group.

- Non-linear pushover analyses of Mangatangi Dam Spillway Shaft.

- Te Uku Wind Turbine Foundation - design and analysis of a piled foundation for a 2.3MW wind turbine. Analyses included the effects of non-linear soil-structure interaction and fatigue considerations.

EVIDENCE

8. This brief of evidence will address the following topics:

    a. A description of Non-Linear Time History (NLTH) analysis;

    b. Interpretation of the outputs of the Compusoft NLTH;

    c. Comments on the Hyland/Smith and Expert Panel interpretation of the Compusoft NLTH.

9. I confirm that all of these matters are within my areas of expertise.

NON-LINEAR TIME HISTORY ANALYSIS

10. Non-linear Time History (NLTH) is an analysis technique which utilises computer modelling to simulate the performance of a structure when subject to recorded earthquake ground motions. This analysis is termed 'non-linear' as it can consider the degradation in stiffness and strength when the elastic capacity of an element (beam, column, wall etc.) of the building is exceeded.

11. The redistribution of forces from one element to another as a result of this degradation can be modelled, which allows for more realistic representation of the

actual building behaviour. For example, it is able to take into account steel reinforcement yielding as a concrete beam deflects during a building's movement during the passage of an earthquake.

12. The intent of a NLTH analysis is to predict (mimic) the response of a structure during an earthquake based upon the 'expected performance' of the detailing and construction present. This form of analysis is particularly useful to estimate the seismic demands of structures that are likely to exhibit non-linear behaviour. The results from this form of analysis are considered by the engineering profession as a better predictor of the seismic demands throughout the course of an earthquake than can be determined by linear analysis techniques.

13. NLTH analyses are useful in determining displacements and forces for a particular structure at each point in time during a seismic event. It can be used to determine the effects of structural weaknesses, trends in behaviour, and the likely sequence that elements become overstressed.

14. The steps involved in undertaking a NLTH analysis are as follows:

- Create an analysis model that has the same structural composition and geometry as the structure being assessed. The model should represent as closely as possible the mass distribution, strength, and stiffness of the as-built structure.

- The stiffness and strength of the structural components are required to be determined based on their sizes and composition along with consideration of the detailing and construction procedures employed.

- Acceleration time-history records of appropriate earthquake records are applied to the model. These are numerical descriptions of the ground motion which consist of three components of the acceleration of the ground (two horizontal and one vertical) at sequential steps in time.

- Extract and post-process the results so that they are in a presentable and useful format.

INTERPRETATION OF COMPUSOFT NLTH OUTPUTS

Non Linear Pushover Analyses

15. Non-linear pushover analyses are analyses where a structure is pushed to a prescribed displacement using an assumed force distribution. They are useful for identifying trends in building behaviour and for model troubleshooting. The Compusoft pushover analyses indicate that the seismic resistance and response of the CTV structure would have been dominated by the shear walls in both the east/west and north/south directions. This is evident by the proportion of seismic shear force resisted by the walls relative to the frames. The walls resist approximately 80-90% of the total seismic shear force in the north/south and east/west directions, and thus contribute more to the overall stiffness of the building than the frames.

16. Pushover force-displacement plots indicate that the building is highly irregular for east/west response due to the south wall being less stiff than the north core walls i.e. the centre of building stiffness was eccentric to the centre of building mass. As a consequence of this, any earthquake actions in the east/west direction would have resulted in a twist as well as a translation of the building. Whilst the pushover plots exhibited very little irregularity in the north/south direction, they did show that the building was less stiff in the southerly direction than for the northerly direction. This can be attributed to the north core walls having a northward rotation resisted by more building weight than for a southerly rotation. Stiffness in the east and west directions was shown to be similar.

17. Inter-storey drifts that initiate column hinging were predicted (via Pushover analyses) to be typically less than 1%.

18. A pushover analysis was undertaken for the case where spandrels were assumed to interact with the columns. The result of this pushover indicated that spandrel interaction stiffened the structure, although the effect was nominal in the north direction.

19. Pushover plots indicated that the inclusion of masonry stiffened the structure for displacements less than 120 mm (at the level 6 centre of mass). For displacements

in excess of this the contribution of the masonry lessened as the masonry degraded in strength and stiffness until no contribution was present at 160 mm.

20. From the pushover plots it was apparent that the building was much stiffer and had more strength in the east/west direction than in the north/south. In addition the stiffness of the structure was shown to be less in the southerly direction than for the northerly direction. This can be attributed to the north core walls having a northward rotation resisted by more building weight than for a southerly rotation. Stiffness in the east and west directions was shown to be similar.

21. The result of pushover analyses including spandrel interaction indicated that spandrel interaction stiffened the structure, although the effect was nominal in the north direction.

4 September Earthquake NLTH Analysis

22. The Darfield NLTH analysis results predicted cracking of concrete and yielding of reinforcement that was not inconsistent with the damage observed after the Darfield event.

23. Tensile connections between the floor diaphragm and the lift core walls were included in the analysis model at levels 4, 5, and 6. These connections represented the 'drag bars' that were added to the building as part of a retrofit strengthening post construction. Analyses indicated that some of the diaphragm connections to the lift core walls could have been subject to distress during the earthquake. Disconnection of the 'drag bars' at levels four and five of the eastern most wall was predicted. It was not possible to correlate this against observed damage as no notes were made on this area by the engineer reviewing the structure. It is possible that disconnection damage may have manifested itself as cracking of the concrete floor slab, or as localised crushing of the concrete, both of which could have easily been obscured by a floor lining.

24. Analyses indicated that minor cracking would have been present only in the lowest storey of the north core and southern walls. It was estimated that residual displacements at the end of the earthquake would have been between 0.4mm and 1.5mm per meter. This correlates well with the hairline to 0.3mm wide cracks observed after the Darfield event.

25. Up to 10 column hinges were predicted to form by the analysis (hinging is where the elastic capacity of the column is exceeded locally, causing inelastic deformations to occur). All hinges were predicted to form in the gridline F frame with hinges forming at the top level and progressing to lower levels as drifts increased. Hinges were primarily located at the column bases. Only columns F2 at level 4 and 5, and column F3 at level 5 were predicted to have hinges forming at the top and bottom of the column. With the exception of the bottom of column F2 at level 5, hinge rotations predicted on frame F were below that which would have exceeded the limiting concrete strain, indicating that cracking would be present, but no concrete spalling would have occurred. At level 5 the analysis predicted that a single hinge may have reached a concrete strain of 0.0045, and could have been subject to concrete spalling. Little column cracking was observed after the Darfield earthquake, although it is not known if the column bases were exposed at columns F2 and F3 on level 5 where cracking is predicted to be most obvious. Axial compression due to gravity would have partially closed any column cracks post-earthquake. Consequently the level of cracking predicted by the analysis is minimal, with the possible exception of the base of column F2 at level 5 and 4, and the base of column F3 at level 5. It is believed that the analysis predictions are not inconsistent with the damage observed on site.

26. Analyses indicated that if the masonry walls resisted forces at the start of the earthquake then they would have been subject to significant distress during the earthquake. Any building interaction with the masonry walls i.e. masonry coming into contact with the concrete frames, would have increased the probability of disconnections between the diaphragm and lift core walls with the analysis indicating more disconnections had occurred.

27. Spandrel interaction with columns may have occurred along gridline F. Analyses showed that the gap between the top of the spandrel and the adjacent columns reduced by as much as 8.6mm during the Darfield event.

22 February 2011 Earthquake NLTH Analysis

28. NLTH analyses were performed for three different ground motions, all recorded during the event from locations close to the CTV building i.e. Christchurch Botanical Gardens (CBGS), Christchurch Cathedral College (CCCC), and Christchurch Hospital (CHHC). Whilst there was considerable variation in inter-storey

displacements exhibited in the results of the three earthquake records, all earthquake records considered show a similar trend with regard to inter-storey displacements.

29. The inter-storey drifts were less than 1.3% for 2 cycles before being subject to a large spike in demand where they increase to between 2.8% and 3.5% in the N/S direction and 1.9% and 3.4% in the E/W direction. This 'spike in demand' corresponded with a large increase in the number of columns hinging (where the elastic capacity of the column is exceeded locally, causing inelastic deformations to occur), and an increase in the number of beam column joints exceeding capacity. Prior to the spike only 11-16 columns had initiated hinging. This number grew to in excess of 75 hinges immediately after the first major peak in displacement demand, within 0.5 seconds of shaking. A similar increase in overstressed beam column joints were predicted due to this spike in demand.

30. Analyses indicated that beam column joint failures may have occurred prior to column failure (through excessive hinge rotations).

31. All records indicate that disconnection of the diaphragm 'drag bars' to the lift core walls would have been initiated shortly after the commencement of significant shaking. Disconnection is shown to have occurred prior to, or at a similar time as column hinging was predicted to have initiated. Once disconnection occurred, in-plane diaphragm actions on the remaining slab attached to the north core at the same floor level would have increased, and by a considerable margin at the level 6 floor slab. In addition, the remaining slab at the interface with the north core was subject to significant out-of-plane moments as the core rotated in a northerly direction, which would have placed additional demands on the diaphragm in this location. It should be noted that diaphragm connections to the lift core have been modelled to represent the performance of the 'drag bars', however it is possible that failure of the floor reinforcement could have resulted in diaphragm connection failure at loads smaller than the 'drag bar' capacity, and at locations away from the wall face.

32. It was shown that the drifts were not significantly affected by the disconnection of the diaphragm to the lift core walls, although disconnection noticeably increased the forces acting on the remaining section of diaphragm connected to the toilet area of the north core.

33. Analyses indicate that the displacement demand under the Lyttelton aftershock quickly exceeded the likely capacity of the masonry early in the passage of the earthquake (assuming the masonry was resisting forces at the commencement of the earthquake), and that building performance converged to that of the non-masonry case once this had occurred Building trends with regard to column hinging and beam-column joints were similar to the non-masonry analysis.

34. Examination of the vertical accelerations has shown that axial load varied on the columns and walls. The frequency of vertical actions was in the order of 4 to 5 times that for horizontal actions. Peak vertical actions did not correspond with peak horizontal actions, which would mitigate the influence of axial load variations on structural capacity.

COMMENTS ON HYLAND/SMITH AND EXPERT PANEL INTERPRETATION OF COMPUSOFT REPORT

35. All comments refer to the interpretation of the Compusoft analysis results detailed within the report 'CTV Non-Linear Seismic Analysis Report' dated 9th February 2012. Relevant documents reviewed include.

- CTV Building Collapse Investigation' for Department of Building and Housing 25th January 2012 Part 2 of 3, by Hyland Fatigue + Earthquake Engineering and StructureSmith Consulting Engineers.

- CTV Building Collapse Investigation' for Department of Building and Housing 25th January 2012 Part 3 of 3, Appendix D, by Hyland Fatigue + Earthquake Engineering and StructureSmith Consulting Engineers.

- Structural Performance of Christchurch CBD Buildings in the 22 February 2011 Aftershock, Report of an Expert Panel appointed by the New Zealand Department of Building and Housing dated February 2012.

4 September Earthquake NLTH Analysis

Masonry Interaction

36. The NLTH analysis of the Darfield earthquake indicates that there would be considerable interaction between the structure and the masonry walls on line A if the masonry was active in resisting seismic actions at the commencement of the earthquake. In the 'CTV Building Collapse Investigation' report it was concluded that as there was no masonry damage or spalling observed after the Darfield event, then the NLTH analysis may have over predicted the structural response to the ground motion, or the masonry stiffness may have been under-represented. Specifically *'This suggests that the masonry walls, at least for the September Earthquake level of shaking, may have been stiffer than assumed in the NTHA analysis and that the response of the structure to the ground motion may have been less than that indicated by the ERSA and NTHA using full ground motion and spectral acceleration records'.* I do not agree with this interpretation. It is possible that the masonry behaved differently to that assumed in the analysis. However, I believe that the analysis model findings would be more consistent with the masonry being seismically separated, with interaction occurring when the inter-storey displacements exceed the seismic separation. The Darfield analysis inter-storey drifts (when no masonry interaction was modelled) are reported as being 30mm, 27, & 26 mm for the floors that support masonry (from highest to lowest). With a seismic separation of 25mm the analysis would indicate only minor interaction with the masonry.

37. The DBH has stated in its findings of the review into the CTV Building collapse that *'Overall, the output of the NTHA analyses was not inconsistent with the reported condition of the building after 4 September 2010'.* I believe that the damage predicted for the walls and columns is consistent with that observed after the Darfield event, but note that the analysis predicted that diaphragm disconnection to some of the lift core walls may have occurred. As the diaphragm connections were not inspected after the Darfield earthquake it is not possible to correlate damage in these locations.

22 February 2011 Earthquake NLTH Analysis

38. The DBH has stated in its findings into the CTV Building collapse that with regard to determining the sequence of failure *'Many reasonable possibilities existed. In these circumstances it has been difficult to identify a specific scenario with confidence'*. I agree with this as the NLTH analyses indicated that the CTV building had a large

number of components exhibiting non-linear behaviour at an early stage during the Lyttelton aftershock. A lack of confinement in the column hinge zones, and relatively high gravity actions meant that some columns (particularly in the lower levels of the building) had a very small plastic rotation capacity. Consequently small increases in building displacements had the potential to cause column hinge rotation capacities to be exceeded. It was found that the limiting hinge rotations, beam column joint capacities, and diaphragm disconnections occurred at similar times. Combined with the possibility of masonry and spandrel interaction occurring with the primary structure would made the isolation of a failure hierarchy extremely difficult to determine.

39. Collapse initiation scenarios outlined in the Hyland/Smith and Expert Panel reports do not consider the possibility that beam column joint failures may have occurred prior to column failure (through excessive hinge rotations). The NLTH analyses indicate that this is a possibility, with some beam column joints predicted to become overstressed prior to the limiting concrete strain in the columns occurring.

Collapse Initiation Scenario 1

40. This scenario focused on column failure on line F or line 1 at a mid to upper level of the building. Frame lines 1 and F are subject to the largest inter-storey drifts during the first 4.5 seconds of significant EQ shaking. Frame F is consistently shown to be where column hinging is initiated and is subject to considerably higher bi-axial bending demands than other frame lines. Column hinging is initiated at the highest level in these frames, although column hinge rotation capacities decrease as you progress down the building. Both of these column lines support precast spandrels and interaction with the spandrel panels would have had an adverse effect on the column capacity. I believe that this scenario is plausible.

Collapse Initiation Scenario 2

41. This scenario centred on building failure being initiated by a gridline 2 or 3 column failure. Whilst the columns on these frame lines hinged after the gridline F columns, the rotational capacity of the gridline 2 and 3 columns was typically lower due to the higher gravity loads acting on the columns. I believe this scenario to be plausible, although I note that column failure would be more likely to occur on gridline 2 than on gridline 3 as the demands are predicted to be greater along this line.

Collapse Initiation Scenario 3

42. This scenario centred on a diaphragm disconnection from the north core walls at level 2 and 3 initiating large storey drifts, resulting in column failure. This scenario is dependant on the relative demands acting on the each of the potential failure planes in the floor diaphragm. All analyses predict greater diaphragm in-plane moments above levels 2 and 3 (for the section of slab adjacent the toilet area of the north core), however larger tensile forces are predicted for the level 2 and 3 diaphragm connections. Actual failure mechanisms will be sensitive to moment/axial relationship, as well as the performance of the floor reinforcing mesh. Based upon the analysis results alone this could be a plausible collapse scenario.

Collapse Initiation Scenario 4

43. This scenario focuses on diaphragm disconnection between the floor slab and the north core walls leading to large storey displacements and ultimately column failure. From the results of the NLTH this scenario is plausible as the analyses indicate that the diaphragms disconnect from the lift core walls prior to hinge formation, or hinge rotation capacities being reached. Once disconnected the diaphragm actions increase on the remaining slab connected to the north core, particularly at level 6. Complete disconnection of the diaphragm from the north core at any level would have resulted in increased inter-storey drifts and diaphragm actions on the remaining floors. It is plausible that total diaphragm disconnection at the north core occurred prior to column failure and contributed to the story drifts which in turn, would have resulted in column rotation capacities being exceeded.

Signed: D. Bradley

DEREK BRADLEY

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