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 THE MATTER OF	**THE CTV BUILDING COLLAPSE**

STATEMENT OF EVIDENCE OF MURRAY LIONEL JACOBS IN REPLY IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

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1. My Full name is Murray Lionel Jacobs. I have provided a previous statement of evidence dated 1 June 2012. This further evidence is in reply to the evidence of Dr Arthur O'Leary. (WIT.OLEARY.001)

Paragraph 22

2. I consider that NZS 4203: 1984 clause 3.2 DUCTILITY is a direct instruction that all the elements that resist seismic forces or movements, **or that in case of failure are a risk to life,** shall be designed to possess ductility. The words from the Commentary mentioned in Dr. O'Leary's paragraph 22 are most relevant:

   The general requirements for ductility must at present be qualitative rather than quantitative ***except for buildings designed to dissipate seismic energy by ductile flexural yielding.***

3. The CTV building was designed to dissipate energy by the ductile yielding of both the North Shear Core and the South Coupled Shear wall.

4. Further 1.1.3 Definitions section of NZS 4203:1984 notes:

   SECONDARY ELEMENTS means elements such as partition walls, panels, or veneers not necessary for the survival of the building as a whole but subject to stresses due to loadings applied directly to them or to stresses induced by the deformations of the primary elements.

   PRIMARY ELEMENTS means elements forming part of the basic load resisting structure, such as beams, columns, diaphragms, or shear walls ***necessary for the buildings survival when subjected to the specific loading.*** *(***Emphasis added***)*

5. These definitions make it clear to me that under NZS 4203:1984 the internal and external columns and beams, as well as the shear walls, are primary elements of the CTV building.

Paragraphs 23, 24, 25

6. Clause 3.2.3 does refer to the appropriate material standard for ductile detailing, as Dr. O'Leary points out. The Commentary C3.2.2, however, does give guidance on what is required.

7. The requirement of clause 3.2.2 is, in effect, a practical approximation for the assessment of section curvature ductility demand. A more rigorous analytical approach, which is

applicable only to reasonably regular frames without sudden changes in storey stiffness, is the method using the following approximate criteria:

> *The building as a whole should be capable of deflecting laterally through at least eight load reversals so that the total horizontal deflection at the top of the main portion of the building under the loadings of equations 4 and 5 and calculated on the assumption of appropriated plastic hinges, is at least four times that of first yield, without the horizontal load carrying capacity of the building being reduced by more than 20 percent.*

Where:	*U=1.0D+1.3 Lr +E*	equation (4)
	U=0.9 D+E	equation (5)

8. This requirement should have raised a signal that the internal columns were unlikely to remain elastic during this design demand. The deflection of the shear walls at 4 times the calculated elastic deflection would likely generate moments in the top and bottom of the load bearing columns that would have required ductile detailing for integrity. Also there would be a likely need to provide substantial shear reinforcement in the columns to prevent a sudden catastrophic shear failure. The assumptions that there is negligible shear in the columns, and hence that the concrete could carry the stress, would be hard to sustain.

Paragraph 26

9. I agree with Dr O'Leary that the shear wall on line (5) (the North Wall) did not show signs of plastic yielding. The shear wall on line (1) (the Coupled Shear Wall) may well have yielded at the base, but was brought down by the floors collapsing on the inside of that wall.

Paragraph 27

10. The building collapsed before the main North Shear Wall was required to carry its design load. There are many theories as to what collapsed first, but what is very evident is that the columns collapsed causing the catastrophic failures a short time after the earthquake struck.

Paragraph 32 and 33 General Comments

11. I consider that the NZS 4203: 1984 clause 3.4.7.1 (c) applies to this building. The building has a high degree of eccentricity in the East West direction and the floor plan is irregular. Hence clause 3.4.7.1(c) applies.

12. There are also re-entrant angles between the major seismic resisting element (the North Shear Wall line (5)) and the rectangular floor plate which made it difficult to structurally connect that shear wall to the main part of the building. The main resisting element of the building is outside the floor plate. Only a small length of slab approximately 3.75 meters long, reducing to 2.75 meters at the wall interface, is available to transfer the horizontal shear to the main seismic resisting element in the East West direction. The building appears regular in the vertical direction between floors.

13. The two shear walls that resist the seismic shear in the East West direction are very unequal in stiffness and strength. The computer analysis carried out by Compusoft found that the North Shear Wall on line (5) had much greater stiffness than the South Coupled Shear Wall on line (1).

14. Even though NZS 4203:1984 clause 3.4.7.1 (b) recommends rather than states as mandatory that a 3 dimensional analysis be used for such structures, a design engineer should carry out a 3 dimensional analysis which took into account eccentricity for a structure such as this. Alan Reay's firm (ARCE) undertook a 3 dimensional analysis, although I have not seen the results.

15. The North Shear Wall on line (5) is in the shape of a C section 11.65 meters deep with the centre of stiffness outside the building. The South Coupled Shear Wall on line (1) consists of two walls 2 metres deep joined at each floor with coupling beams.

16. Dr. O'Leary alludes to the problem of two unequal walls in the same directions when he states in paragraph 32:

> *... 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 modes.*

17. This problem was well known at the time of the design of the CTV building. A paper published in the Bulletin of the NZ National Society for Earthquake Engineering Vol. 13 No 2 June 1980 by T. Paulay and R.L. Williams addresses this. Professor T. Paulay, Professor of Civil Engineering, University of Canterbury, is a well known and internationally respected expert on seismic performance of buildings with concrete shear walls. The paper states on page 118 under the heading "Torsion":

> *...as in all structures in seismic areas, symmetry in structural layout should be aimed at...Deliberate eccentricities should be avoided, if possible, because uneven excitations may aggravate eccentricity and this in turn may lead to **excessive ductility demand** in lateral load resisting elements situated far away from the centre of rotation.*

An example is then given of a shear wall arrangement very similar to the CTV building where a shear wall of unequal stiffness is placed at each side of the building:

> *An example of the unintended inelastic response of two ductile shear walls is illustrated in figure 13a. Because the centre of mass CG is approximately at the centre of the plan, approximately one half of the earthquake induced load E will have to be resisted by each of the end walls at A and B. It may be difficult to prevent Wall A from having a lateral load carrying capacity considerably in excess of that on wall B. Hence energy dissipation due to inelastic deformation may well be restricted to wall B only which, as a result of this, could be subject to a displacement Δ much larger than expected. Irrespective of the relative stiffness or strength of the two shear walls, structures in which only two principal planes of lateral resistance exist parallel to either major axes, are likely to be torsionally unstable during large inelastic seismic excitations.*

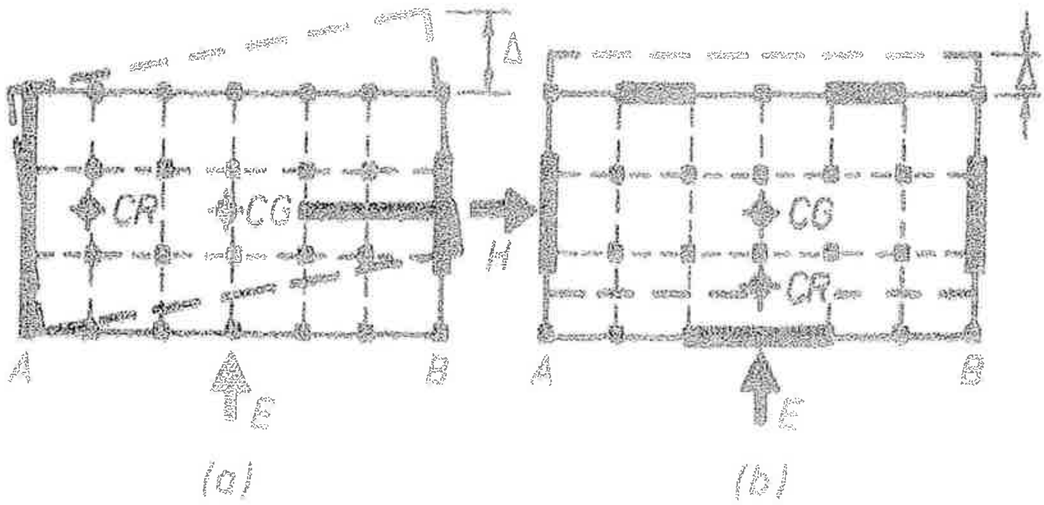


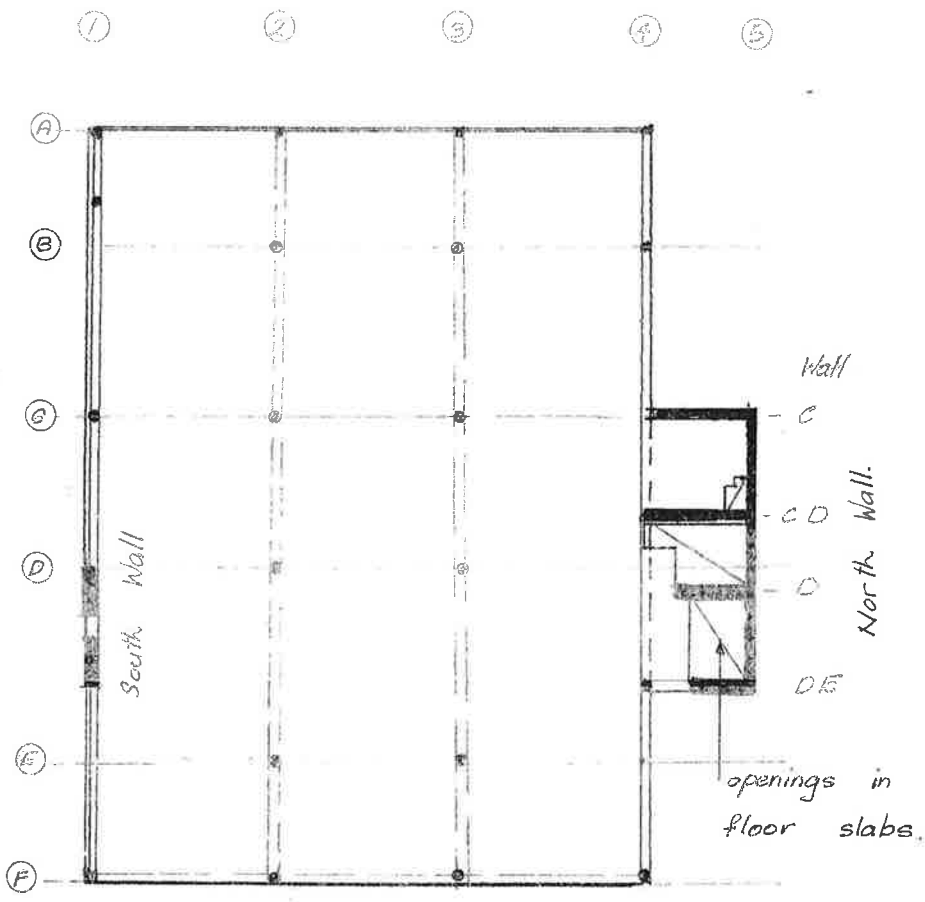
Fig. 13 - The layout of Shear Walls affects the Torsional Stability of the Lateral Load Resisting System.

Fig 13 b above shows a more balanced configuration of shear walls with more equal shear walls placed around the periphery of the building and resulting limited torsional loads. This paper reinforces the perils of designing a building with unsymmetrical resisting elements

18. The description of wall A and B in the paper is very similar to the situation of the North and South Shear Walls on the CTV building. Wall line 1 (the South Wall) is as for wall B in figure 13a and wall line 5 (the North Wall) is as for wall A in figure 13a, except wall line 5in the CTV building is outside the main building floor plate – which is worse.

19. The smaller shear wall (South Wall) could have been subjected to a displacement Δ much larger than expected, as is outlined in this paper. This would mean that the columns in

lines (1) and (2) would have also been subject to greater displacements than expected. These columns were brittle. They have not been detailed for ductility. They could only accept a limited amount of deflection before failure. The code has warned many times about the limitations of assumptions, analysis and the uncertainty of the behaviour of unsymmetrical structures. These are the reasons that the warnings and requirements have been outlined. This warning was given in his paper by the Professor of Engineering in Christchurch University many years before the CTV building was designed. It was available in the widely read Bulletin of the NZ National Society for Earthquake Engineering and is reflected in the many instructions in the code about eccentric buildings.



Plan of typical floor showing arrangements of shear walls.

Paragraph 34

20. I do not agree with the statement of Dr. O'Leary that the East West wall on line 5 and the wall on line 1 provide efficient torsional resistance, for the reasons outlined in paragraphs 17 to 20 above and in the paper by Professor Paulay.

21. The smaller wall on line 1 will cause excessive deflection and aggravate the eccentricity. The return wall lines C and DE in the North Shear Core are slender and relatively close together. They were not effective in resisting torsional loads from earthquake forces in the East West direction. The walls at the end of the C shaped North Shear Wall line on (5) were also not connected to the slab with sufficient capacity to take the tensile forces from torsion induced seismic loading in the East West direction. This especially applies to the return wall line (DE). I am aware that retrofitted "drag bars" were installed at a later date. However the tensile strength of the slab with only 664 mesh would have been suspect at a position in the slab outside the saddle bars' line.

22. Retrofitting effective drag bars under the slab would have been a difficult task to achieve practically once the building had been completed. The drag bars would have needed to connect to the slab back to line 3 to be effective, in my opinion.

Paragraphs 35, 36

23. The design for torsion at the time was based on the predicted behaviour of the walls. If a wall was overstressed by torsional effects then it may become torsionally unstable, as described in the paper by Professor Paulay in 1980 to which I have referred. The return walls on the North Core Wall line 5 were not likely to have provided significant torsional rigidity and ability to carry the torsional loads. They are slender walls relatively close together and poorly connected to the floor.

Paragraphs 37, 38, 39, 40, 41

24. In these paragraphs of his evidence Dr. O'Leary sets out his reasons for concluding the non ductile detailing of the internal columns in the CTV building complied with the code. I do not agree with him. I understand from the code that the columns should have been designed for ductility. There are many instructions that this should be so. They are mentioned in the Hyland- Smith report, the third statement of evidence of Ashley Henry Smith, and my first statement of evidence (WIT.JACOBS.0001) at paragraphs 27 to 37. The need for ductility in all elements is specified in 4203:1984 at the start of Part 3 EARTHQUAKE PROVISIONS, to which I have referred previously:

> 3.2 Ductility: *The building as a whole, and all of its elements that resist seismic forces or movements, or that in the case of failure are a risk to life, shall be designed to possess ductility.*

25. I have referred previously to the point that secondary elements are defined in NZS 4203:1984 as: *elements such as partition walls, panels, or veneers not necessary for the survival of the building as a whole …*

NZS 3101: 1982 is the code that relates to the design of reinforced concrete.

26. I consider that the North Shear Wall in the North-South direction was slender. The walls on lines C and CD had a severe notch out of them between levels 1 and 2. The other two walls were narrower all the way up the building. The frame line F would have been found to be subject to significant seismic loads had it been included in the analysis.

Paragraph 42

27. *NZS 3101:1982 Clause 3.5.14 states:*

> *3.5.14.1 Secondary structural elements*
>
> *Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore* ***not*** *necessary* ***for the survival of the building as a whole under seismically induced lateral loads,*** *but are subjected to loads due to accelerations transmitted to them, or due to deformations of the structure as a whole…***(emphasis added)**

28. I have stated earlier in my evidence that in my opinion the columns in this building were necessary for the survival of the building as a whole under seismically induced lateral loads.

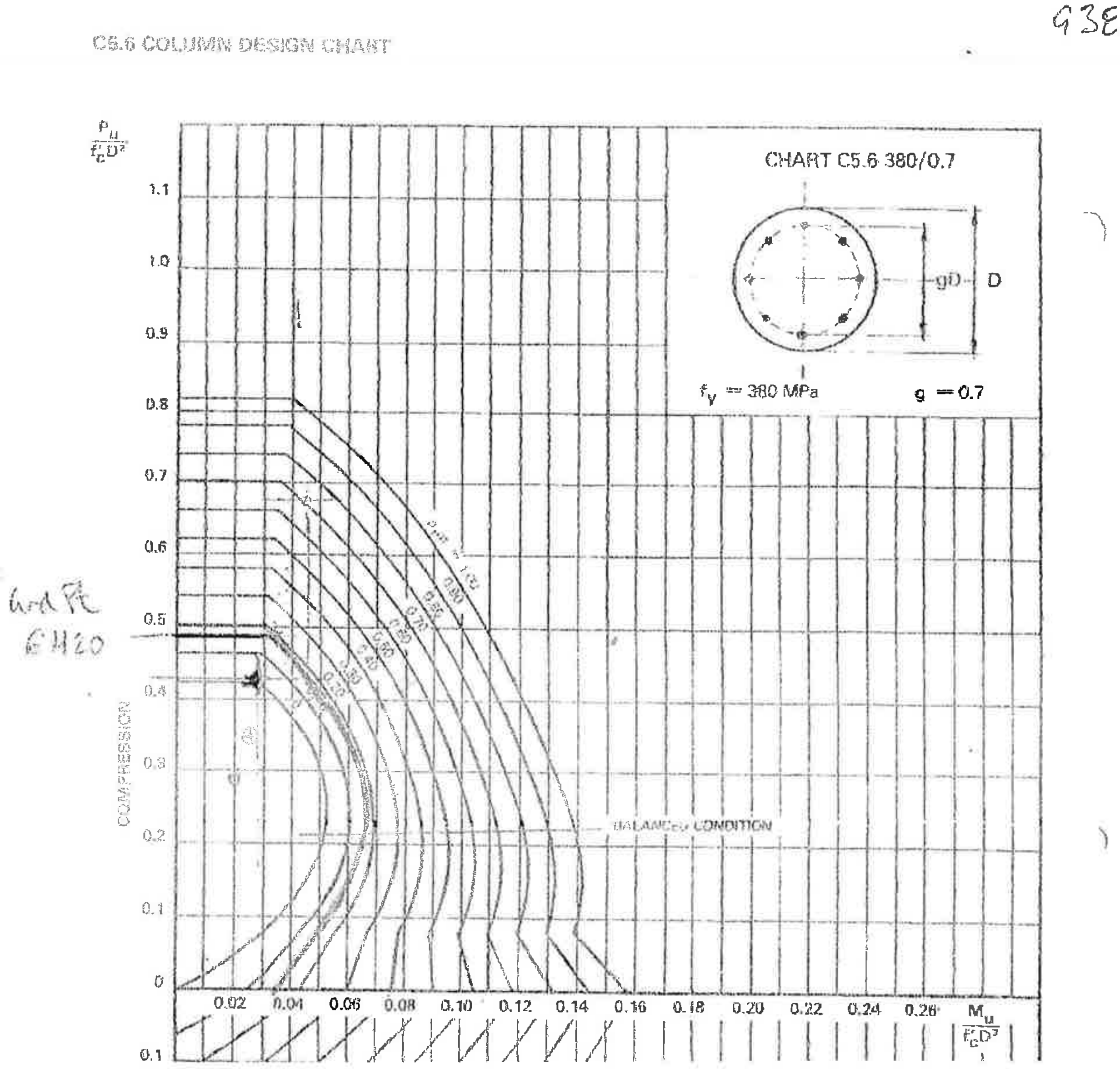
29. NZS 4203:1984 Clause 3.5.14.3 states:

> *Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis.*

A 3 dimensional modal analysis carried out by Compusoft as required by the code 4203:1984 gave a set of deflections. When these deflections were imposed on the columns of the CTV building they indicated that the columns did not remain elastic, especially in the levels 5-6 and 4-5. The shear stress generated by the moments was also greater than that allowed on the concrete without the required minimum shear reinforcement. The modal analysis was carried out for the Hyland-Smith report, Part 3 Appendix E, referred to as the ERSA MODELLING and mentioned on page 236 of the report.

30. The column design chart reproduced below from the design calculations of Alan M. Reay Consultants at page G38 (BUI.MAD249.0273.40) show the design point as a circle at

around 0.43 on the vertical axis and 0.028 on the horizontal axis. This position on the design chart indicates a heavily loaded column with a small capacity for moment. If the moment is plotted on this graph for the design moment calculated from the 3D analysis referred to above, the design point falls well to the right on the horizontal line between 0.4 and 0.5 ordinates, indicating yielding of the column by plastic action.



Column design graph from Alan M. Reay consulting engineer calculations page G38

Paragraph 43

31. I am not sure what Dr. O'Leary means by the statement in paragraph 42:

> *...the design and detailing of the secondary members <u>should</u> have satisfied the applicable sub clauses (a) and (b).* It is not clear whether he means that they <u>*would*</u> *have* or <u>*should*</u> *have* satisfied those sub clauses.

Dr O'Leary comments on Evidence Murray Lionel Jacobs (WIT.OLEARY.002)

32. I reply to Dr O'Leary's specific comments on my evidence as follows.

Paragraph 9

33. Dr O'Leary states in paragraph 9:

> *The question of "irregularities/lack of symmetry" (paragraph 15(b) (ii) needs, in my opinion, to be put in context of the understandings of the time and the understandable desire to have attractive and functional buildings.*

34. In my experience, at the time, the negative effects of eccentricity were well known. A professional engineer's role was to work closely with the architects to develop a building that complied with the code and at the same time was acceptable from a planning and aesthetic point of view. I do not believe that design engineers were forced into designing eccentric buildings, irregular in shape, because of client or architectural requirements. All the New Zealand architects I have worked with have been amendable to the requirements of sound seismic design once the requirements are explained to them. I have worked for a range of developers from smaller developers through to large developers.

35. I have found the same situation in the requirements of the code in relation to seismic detailing. A client will usually accept that a building has to be designed with ductile detailing to conform to the applicable code, i.e. to resist seismic movements and to avoid a risk to life, once the issues have been explained. In my experience to aim to minimize the amount of reinforcing steel in the critical structural elements of buildings is not the most important design criteria.

Paragraph 28

36. Dr O'Leary only selects a small part of the Commentary on clause C3.1.1. The full Commentary on clause 3.1 "Symmetry" in NZS 4203:1984 is very specific about the perils of designing non symmetrical buildings and is reinforced by the paper of Paulay & Williams page 118 I have previously referred to:

> *For high buildings, symmetry is one of the most basic requirements in achieving a structure of predictable performance. Simple geometry is essential for obtaining symmetry in practice. Notwithstanding the availability of modern computers, considerable uncertainty exists in selecting a mathematical model representing the true behaviour of complex arrangements such as combinations of geometrically dissimilar shear walls and*

frames shear walls and frames and unsymmetrical combinations of shear walls and frames.

Geometrically dissimilar resisting elements are unlikely to develop their plastic hinges simultaneously, and ductility demand may also be increased by torsion effects.

Paragraph 29

37. I do not believe that you can dismiss this clause. It is very specific and it is based upon the accumulated knowledge that buildings do not perform if they are unsymmetrical. As the Commentary C3.1.1 outlines, the analytical tools cannot predict accurately the complex action of a real building. The ductility demand on dissimilar shear walls cannot be predicted and can lead to the overstressing and failure of the walls. This was outlined in 1980 in the paper by Professor Paulay, to which I have already referred.

Paragraph 30

38. This is covered by my comments on the first statement of Dr. O'Leary.

Paragraph 32

39. The columns are critical as failure would and did cause a risk to life. They support 90% of the building. Clause 3.2.1 specifically states that *all elements that in the case of failure are a risk to life* ***shall be designed to possess ductility.***

Paragraph 33

40. I was referring to the placement of the spirals as part of the construction sequence as they had to stop below the bottom steel. The anchorage of the spiral, usually by welding to make them continuous or by bending into the core of the column, would be difficult. The bond length of a plain 6 round bar is around 48 d or 280 mm. This means that length of column is without effective spiral reinforcement. *Practice Advisory 8, Department of Building and Housing: Don't be undone – anchor your spiral*, points out:

inadequate anchorage or splicing can cause the spiral reinforcing to unravel under seismic condition resulting in:

- *loss of confinement to principle reinforcement*
- *premature and brittle failure*
- *failure of the whole structure.*

Paragraph 34

41. My point in quoting this clause was that a designer should be cautious about aiming to look for ways of minimising the rules of this code as it is a minimum standard not a **maximum.**

Paragraph 35

42. The North Shear Core Walls are either notched at the base from level 1 to just under level 2, or they are narrower than the wall mentioned on line D. I do not understand what Dr. O'Leary means by the statement: *...it would be inappropriate in my view to class the group as slender on the basis that only one of them could have that characteristic.* In fact two have that characteristic (lines D and DE) and the other two are slender all the way up the building with an aspect ratio of over 5 to 1

Paragraph 36

43. The critically important columns do not meet the requirement of elements that can be designed without ductile detailing as they are elements that are a risk to life: clause 3.2.1 NZS 4203:1984 to which I have referred previously.

Paragraph 37

44. I stated that the 664 mesh did not meet the code requirements for shrinkage reinforcement, according to my calculations. The calculation for topping steel required for diaphragm action is another separate design requirement. My point was that this reinforcement did not comply with the very minimum required for temperature movement. The requirements for diaphragm action, if they had been adequately provided for, would have ensured that the return walls on the North Shear Core were connected to the rest of the building by adequate reinforcing in the slab. Special reinforcing drag bars could have been provided within the slab. However, as designed the 664 mesh was the only reinforcing in the slab to resist the tension forces from the return walls of the North Shear Wall at the position of the slab where the short saddle bars finished. These walls were the only seismic resisting elements in the North South direction. D12 'saddle bars' were placed over the beam lines, but these were relatively short.

45. The 664 mesh was draped down to form support for the slab in the case of a fire. I had not come across this method of providing fire steel before as it is very difficult in practice to accurately place mesh reinforcing at different depths in a slab. Usually in my experience extra bottom steel reinforcing bars are placed in the bottom of the metal deck profile to support the slab in case the exposed metal deck strength is affected by the heat

of the fire. Draping of the mesh also means that a large area of slab in centre span has no reinforcing in the top portion of the slab.

Paragraph 38

46. The columns were required to be designed for ductility as I have explained in previous paragraphs, as they were a risk to life. They were also subjected to reverse moments top and bottom of the columns due to the movement of the building during the earthquake motion. The calculated moments generated in the interior columns as a result of the 3D analysis, referred to in paragraph 30 of my evidence, were above the yield strength of the columns. The resultant shears were also well above that allowed by NZS 3101:1982 clause 7.3.4.3. To assume that columns in a 6 storey building would not have significant deflection induced moments and shears, and therefore not to reinforce them with the minimum reinforcing, is difficult to understand. In my 37 years of structural engineering I have never seen a column with such little lateral reinforcing in New Zealand.

Paragraph 40

47. I think these objections to my conclusions by Dr. O'Leary have been covered already.

M.L. Jacobs.

Murray Lionel Jacobs