Under	**THE COMMISSIONS OF INQUIRY ACT 1908**
In the matter of the	**CANTERBURY EARTHQUAKES ROYAL COMMISSION OF INQUIRY INTO THE COLLAPSE OF THE CTV BUILDING**

THIRD STATEMENT OF EVIDENCE OF JOHN BARRIE MANDER

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THIRD STATEMENT OF EVIDENCE OF JOHN BARRIE MANDER

1. My full name is John Barrie Mander. I reside in College Station, Texas. I hold the position of the Zachry Professor of Design and Construction Integration 1, within the Zachry Department of Civil Engineering at Texas A&M University.

2. I refer to my first statement of evidence dated 10 June 2012 for details of my qualifications and experience. I again confirm that I have read the Code of Conduct for expert witnesses and that my evidence complies with the Code's requirements.

Background

3. In my first statement of evidence it was mentioned that ARCL had retrieved portions of columns from the CTV landfill site for testing in the DARTEC universal testing machine at the University of Canterbury's structures laboratory.

4. Arrangements for the retrieval of the test specimens were carried out by Mr Douglas Latham and Mr Christopher Urmson of ARCL. The specimens were transported to the University of Canterbury labs, and the testing was conducted under the direction of Dr Rajesh Dhakal.

5. The three retrieved specimens that were tested were marked as follows:

   (a) C5 upper (this specimen showed only minor signs of damage);

   (b) C5 lower (this specimen showed only minor signs of damage); and

   (c) C13 (this specimen showed signs of a reasonable degree of damage).

Methods

6. I had instructed Dr Dhakal to conduct the column tests in a similar fashion to the earlier experiments carried out described in Mander (1983) and Mander et al (1988 a,b). This required ensuring the specimen ends were square. Thus for specimen C13 special end-caps were cast due to serious damage evident at the end regions.

7. The specimens were instrumented over a central 400 mm gauge length using linear potentiometers for the purpose of inferring axial strain.

8. Each specimen was tested at the full test velocity (16 mm/s) of the Dartec machine. Axial load and displacements measurements were sampled at a rate of 1024 Hz.

Results

9. The raw load and displacement results were forwarded to me by Dr Dhakal for further analysis. For each specimen I reduced the strain results in an identical fashion to that described in Mander (1983). The left hand column of graphs depicted in Figure 1 shows the overall axial load versus axial strain results for each of the three specimens.

10. In Figure 1, the right hand column shows the analytically modeled results for each specimen. Satisfactory agreement between the modeled outcome and the experimental observations is evident when comparing the left and right hand graphical pairs in Figure 1.

11. It should be noted that the total load, as modeled in the right hand graphs in Figure 1, is made up of three components as follows:

    (a) Longitudinal steel. This has been modeled to include the effects of buckling, as observed in the final damaged condition of the specimens.

    (b) Cover concrete. This has been modeled using the stress-strain relations in Karthik and Mander (2011) with the three main concrete parameters: f'_c = concrete compression strength; E_c = Young's modulus of elasticity; and ε_{co} = strain at the peak strength (f'_c).

    (c) Confined core concrete. The three parameters used were calculated as per Mander et al (1988a) are: f'_{cc} = concrete compression strength; E_c = Young's modulus of elasticity which is the same as for unconfined concrete; and ε_{cc} = strain at the peak strength (f'_{cc}). These are then used in the model prescribed in Karthik and Mander (2011).

12. The values of the parameters adopted in the modeled results are listed in Table 1, and the complete stress-strain results plotted in Figure 2.

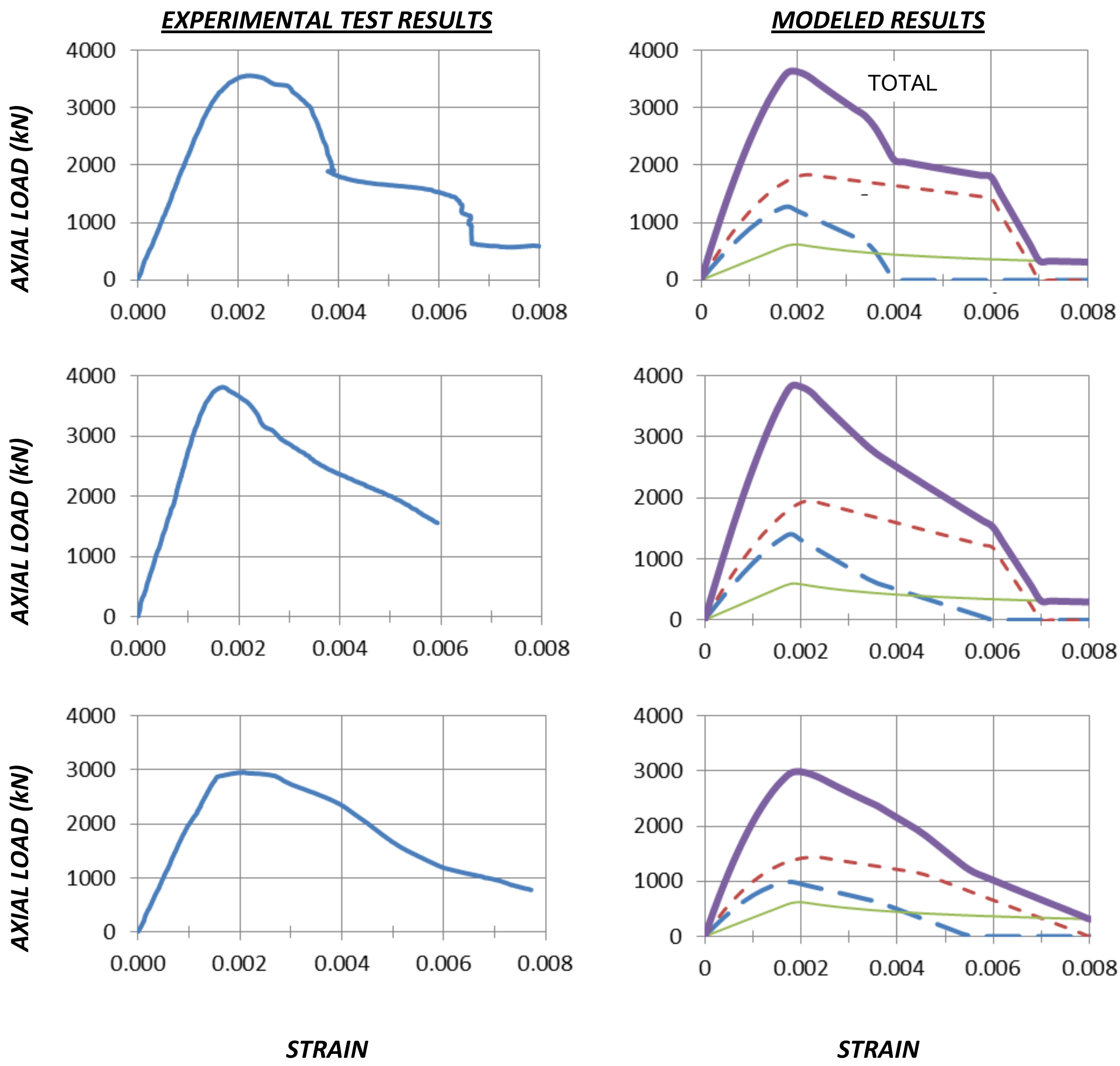

Figure 1. Experimental test results and the modeling of those results to determine values of the parameters for the stress-strain relations of the in situ concrete.

Note: The left had column of graphs shows results for: the upper portion of column C5; the lower portion of column C5; and a specimen extracted from column C13 that showed a damaged condition. The right hand column of graphs show the results modeled for the experiments presented in the graph to the left. For each graph shows four curves as follows: the calculated contribution of resistance provided by the longitudinal reinforcing steel in the lower green curve; the contribution from the unconfined cover concrete shown by the blue dashed curve; the contribution of the confined core concrete as shown by the red dashed curve; and the upper curve which is the total capacity found from the summation of the previous three curves.

Table 1. Parameters used to model the performance of the experimental test specimens

Parameter	f'_{co} (MPa)	E_c (MPa)	ε_{co}	ε_{spall}	f_y (MPa)
C5 Upper Column	25	20000	0.0018	0.004	380
C5 Lower Column	27	20000	0.0018	0.006	380
C13 Damaged Column	19	18000	0.0018	0.0055	380

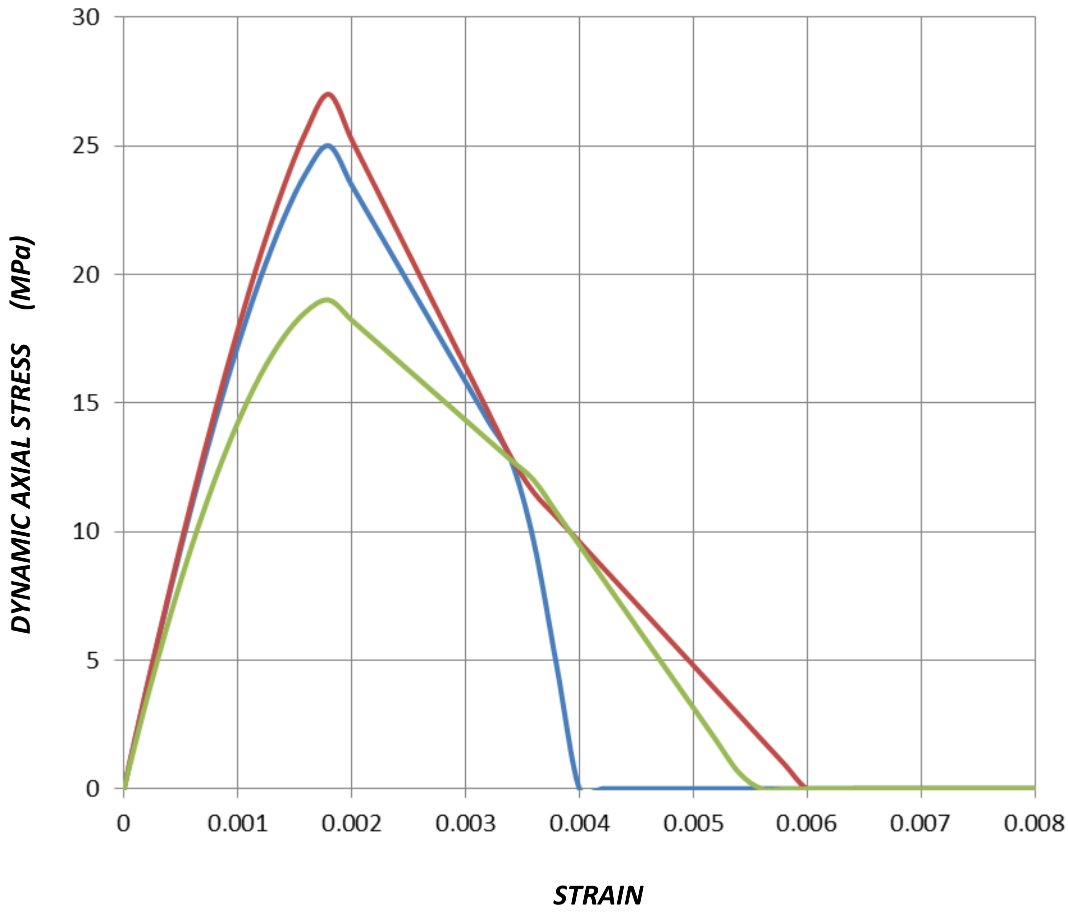

Figure 2. Inferred stress-strain relations derived from the experimental high-speed test results on the full-scale columns retrieved from the CTV Building.

Discussion

13. The specified strength of the concrete in the columns in the upper three storeys was f'_c = 25 MPa. It is of interest to compare the modeled results with the companion strength tests derived from the axial cored 200 mm x 100 mm cylinders tested by Mr Haavik in the USA. In my first brief of evidence it was concluded from Mr Haavik's work that one could generally assume the in situ strength to be 1.5 times the specified 28 day strength; thus the expected strength is f'_{ce} = 37.5 MPa.

14. At first glance it is curious as to why the full-scale specimens infer lower concrete strengths compared to the direct compression tests on the small cylinders obtained from essentially the same concrete. This apparent difference is attributed to the effect of load-induced damage on the columns as a whole. Under sidesway, the cover concrete will inevitably experience tensile cracking over a considerable proportion of their length, specifically under the lower levels of gravity load inherent in the upper storey columns and more specifically when combined with high axial load (tensile) variations.

15. Although it is possible that the columns would crack across their entire diameter under axial tension effects particularly arising from the vertical acceleration effects, care was taken that such cracks were not present in the small cylinder tests. However, it is inevitable that such cracks would be unavoidably included in the complete column tests.

16. It should be noted that not only are the inferred compression strength values (f'_{co}) lower in the full scale column tests, but there is also a reduction in the concrete material stiffness, E_c. Young's modulus results are only 85%, 82% and 86% of the expected NZS 3101 (1982) code values. Again this is attributed to the softening effect of damage arising from the loading experienced by the columns.

17. An indication of such damage may be found in the previous work by Mander (1983). Consider for example "***COLUMN c***" which was tested under similar load conditions as for the CTV column, but with one significant difference as shown in Figure 3. ***COLUMN c*** was tested under compression-only cyclic loading. Although ***COLUMN c*** was a "well

confined" test specimen, an examination of the results is instructive in the present context.

18. From an examination of Figure 3 it is evident that the both the concrete strength and stiffness deteriorate due to the cyclic loading effects. For example, the concrete exhibits only 95% of the expected strength, and 70% of the stiffness prior to the peak strength, with the latter dropping to only 25% of the initial stiffness where the strain was cycled in the post-peak region (reversing strains greater than 0.008).

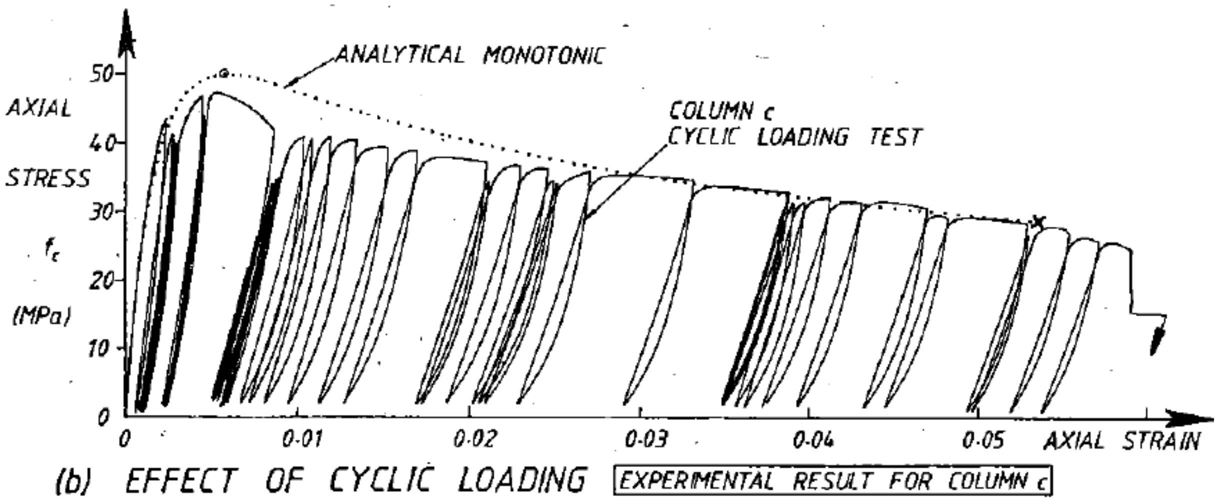

Figure 3. Results of "*COLUMN c*" tested under compression-only cyclic loading effects by Mander (1983).

Conclusion

19. Based on the foregoing analysis, the following conclusions are drawn:

(a) By solving the inverse problem, the overall post-earthquake (damaged) performance of the columns can be modeled and hence the post-damaged concrete strength inferred; damage is considered as the principal cause of the unexplained differences between the full column tests and the cylinder tests.

(b) The inferred dynamic strengths from the test results from the two "slightly damaged" columns, C5 upper column and C5 lower column, were 25 MPa and 27 MPa, respectively. These values are either at or marginally above the specified strength. If the columns were undamaged and in pristine (undamaged) condition, one would expect the results to be in the order of 31 MPa. It should be noted that this is lower than the expected (cylinder) strength of 37.5 MPa. Such a lower value is ascribed to the well-known size effect.

(c) The inferred dynamic strength of the damaged column specimen C13, was only 19 MPa, which is corroborated by the clearly visible damage.

(d) For all specimens, the stiffness is less than normally observed in such large scale tests. This is due to load-imposed damage of the earthquakes on and prior to 22 February, 2011. This demonstrably less stiffness is also the principal reason the OPUS results inferred by Schmidt Hammer test infer a lower concrete strength.

References

Karthik, M.M. and Mander J.B. (2011) "Stress-Block Parameters for Unconfined and Confined concrete Based on a Unified Stress-Strain Model" ASCE Journal of Structural Engineering, Vol. 137, No. 2, pp270-273

Mander J.B., (1983) "Seismic Design of Bridge Piers," PhD Thesis, University of Canterbury Library. http://ir.canterbury.ac.nz/handle/10092/1197

Mander, J.B., Priestley, M.J.N., and Park, R., (1988a) "Theoretical Stress-Strain Model For Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, pp 1804-1826.

Mander, J.B., Priestley, M.J.N., and Park, R. (1988b) "Observed Stress-Strain Behavior of Confined Concrete", Journal of Structural Engineering, ASCE, Vol. 114, No. 8, pp 1827-1849.

Dated this 14th day of August 2012

JB Mander.

J. B. Mander