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**

STATEMENT OF EVIDENCE OF BRENDON ARCHIE BRADLEY

BUDDLEFINDLAY
Barristers and Solicitors
Christchurch

Solicitor Acting: **Willie Palmer / Kelly Paterson**
Email: kelly.paterson@buddlefindlay.com
Tel 64-3-379 1747 Fax 64-3-379 5659 PO Box 322 DX WP20307 Christchurch 8140

Counsel Acting: **H B Rennie QC**
Harbour Chambers Tel 64-4-4992684 Fax 64-4-4992705 PO Box 10242 Wellington

BRIEF OF EVIDENCE OF BRENDON ARCHIE BRADLEY

1. My full name is Brendon Archie Bradley. I reside in Christchurch. I am a Lecturer at the University of Canterbury and run my own seismic engineering consultancy business.

2. In accordance with the requirements of Rule 9.43 of the High Court Rules, I confirm that I have read the Code of Conduct for expert witnesses and that my evidence complies with the Code's requirements.

3. Matters on which I express an opinion are within my field of expertise.

4. I have no interests or relationships with any parties to these proceedings.

Qualifications and experience

5. I hold a Bachelor of Engineering (Hons) (University of Canterbury, 2006) and a Doctorate of Philosophy in Civil Engineering (University of Canterbury, 2009).

6. I am a Member of the New Zealand Society for Earthquake Engineering and other relevant professional bodies.

7. Since 2010 I have lectured at the University of Canterbury, Department of Civil and Natural Resources Engineering, in courses involving structural dynamics; engineering mechanics and mathematics; geotechnical earthquake engineering; and seismic hazard and risk analysis, among others.

8. For 13 months during 2010 and 2011 I was a JSPS Postdoctoral Fellow in geotechnical earthquake engineering at Chuo University – Faculty of Science and Engineering. Between 2009 and 2010 I was employed by GNS Science as a seismic hazard modeller where I was engaged in research and consulting seismic hazard and risk analysis, tsunami reconnaissance, development of displacement-based fragility functions for earthquake and tsunami.

9. In 2010 I founded Bradley Seismic Limited, a consultancy firm through which I provide research and consultancy services in seismic hazard and risk analysis, structural and geotechnical seismic response analysis.

10. Throughout my education and professional career, I have been awarded numerous scholarships and prizes, as detailed in my resume. I have

authored or co-authored 39 published journal articles, as well as numerous conference articles, books and book chapters and other publications, all of which are detailed in my resume.

11. My expertise covers seismic hazard analysis, ground motion prediction and analysis and structure-specific seismic loss assessments.

12. My full resume is attached and marked "A".

Instructions

13. I have been instructed by Buddle Findlay, on behalf of Alan Reay Consultants Limited (**"ARCL"**), to provide independent expert advice on issues relevant to the collapse of the CTV Building on 22 February 2011 following an earthquake of magnitude 6.3. In particular, I have been asked to comment on the following issues:

    (a) A critique of the concrete modelling and statistical analysis of concrete test data presented in the CTV Building Collapse Investigation for the Department of Building and Housing prepared by Dr Clark Hyland and Ashley Smith (2012) (**"DBH report"**); and

    (b) Analysis of ground motion aspects of the Canterbury earthquakes.

14. The principal sources of information I have referred to and relied upon in preparing this evidence are referenced in my reports annexed.

Analysis of concrete test data

15. My report 'Statistical Analysis of Concrete Test Data from the Canterbury Television (CTV) Building' is annexed and marked "B".

16. As outlined in my report, as a result of the statistical analysis I have carried out, based on the 23 column test data examined in the DBH report, there is no credible evidence to suggest that the observed concrete strengths are lower than the specified concrete strengths.

17. At the time of submitting my report, I have just recently received the results of further tests commissioned by ARCL from CTL Thompson. In the time available, I have not been able to analyse these results in any detail, but I have prepared two figures that summarise the concrete strength results as compared to the Hyland results, as follows:

(a) The first is correlation between the values from Hyland and those from CTL Thompson.

(b) The second is the distributions between the two sets of results.

18. The figures are annexed and marked "C". If required, I can elaborate further on the figures and the ARCL test results at a later date.

Analyses of ground motions

19. My report 'Ground Motion Aspects of the 22 February 2011 Christchurch Earthquake Related to the Canterbury Television (CTV) Building' is annexed and marked "D".

20. My report concludes that comparisons of ground motions at the CTV site obtained during April and May 2012 from specifically deployed instrumentation, as well as comparisons with an empirically derived conditional response spectrum distribution, illustrate that the ground motion time series observed at four strong motion stations near the CTV site (i.e. CCCC, CHHC, CBGS, REHS) during the 4 September 2010 and 22 February 2011 earthquakes are appropriate for use in nonlinear seismic response history analysis of the CTV building.

21. This conclusion should inform others that are conducting nonlinear seismic response analyses.

NTHA expert panel

22. I am participating in the NTHA expert panel being facilitated by Athol Carr. I reserve the right to modify or add to my evidence following the completion of this process.

Dated this 8th day of June 2012

B A Bradley

"A"

Brendon A. Bradley

Lecturer
Department of Civil and Natural Resources Engineering
University of Canterbury
Private Bag 4800
Christchurch, New Zealand

Phone: +64 3 364 2987
brendon.bradley@canterbury.ac.nz

Education

University of Canterbury – PhD Christchurch, New Zealand
Doctor of Philosophy in Civil Engineering (Dean's List) 2007-2009
"Structure-Specific Probabilistic Seismic Risk Assessment"
University of Canterbury – BE (Hons) Christchurch, New Zealand
Bachelor of Engineering with First Class Honours in Civil Engineering 2004-2006

Professional and Academic Experience

University of Canterbury - Dept. of Civil and Natural Resources Engineering 2010-present
Christchurch, New Zealand
Lecturer 2010-present
Fixed-term lecturer 2008-2009

Lecturing structural dynamics; engineering mechanics and mathematics; geotechnical earthquake engineering; and seismic hazard and risk analysis. Research in various aspects of earthquake engineering.

Chuo University – Faculty of Science and Engineering 2010-2011
Tokyo, Japan
JSPS Postdoctoral Fellow

Geotechnical earthquake engineering, 2D and 3D seismic effective stress analysis of soil deposits with ground improvement, simplified estimation of state concept parameters, liquefaction reconnaissance of reclaimed areas of Tokyo that experienced liquefaction in the 11 March 2011 Great East Japan earthquake.

GNS Science 2009-2010
Wellington, New Zealand
Seismic hazard modeler

Research and consulting seismic hazard and risk analysis, tsunami reconnaissance, development of displacement-based fragility functions for earthquake and tsunami.

Bradley Seismic Limited **2010-present**
Christchurch, New Zealand
Founder, Director

Provides research and consultancy services in seismic hazard and risk analysis, structural and geotechnical seismic response analysis.

Research Specialization

Dr. Bradley's research and professional accomplishments have covered a wide range of fundamental and applied topics in earthquake risk mitigation and management, from seismic hazard analysis and ground motion prediction to fragility analysis of structures, as well as over-arching subjects such as performance-based frameworks, structure-specific seismic loss assessment, and their role in decision making.
His most notable research contributions include:

1. Structure-specific seismic loss assessment and methodologies for performance-based earthquake engineering. In particular, the accuracy of simplified methods against rigorous solutions; consideration of correlations between different components within a structure; methodologies for the consideration of epistemic/modelling uncertainties; various ground motion intensity measures and their impact on seismic loss assessment; and applications of seismic loss assessment for case study structures, including soil-pile-structure systems in liquefiable soils. In my opinion these contributions remain the state-of-the art in seismic loss assessment, a discipline which, preceding such contributions, was largely dominated by researchers at the Pacific Earthquake Engineering Research (PEER) Centre.
2. Empirical analysis of ground motion intensity measures, including the development of NZ-specific ground motion prediction equations, and empirical correlation models between spectral acceleration, peak ground velocity, cumulative absolute velocity, significant duration and other ground motion intensity measures.
3. Development and open-source implementation of the generalized conditional intensity measure (GCIM) approach, a probabilistic seismic hazard-consistent ground motion selection method for general seismic response problems in which aspects of ground motion more than simply response spectral amplitudes are important.
4. Advanced seismic effective stress analysis of liquefaction, soil-pile-structure interaction, and ground improvement problems, and simplified pseudo-static analyses of pile foundations in liquefying and lateral spreading soils.
5. Reconnaissance and subsequent analysis of several major earthquake and tsunami events, particularly focusing on the observed strong ground motions, liquefaction, and building and lifeline performance and fragility.

Awards

NZSEE Ivan Skinner Award, for the advancement of Earthquake Engineering Research in New Zealand	2012
Japanese Society Promotion of Science (JSPS), Postdoctoral fellow	2010-2011
NZSEE Research Prize	2008
Fulbright New Zealand Travel Award	2008
Tertiary Education Commission Top Achiever Doctoral Scholarship	2007-2009
University of Canterbury Brownlie PhD Scholarship	2007-2009
Freemasons Postgraduate Scholarship	2007
John Blackett Prize	2006
Future Building Systems Research Scholarship	2006-2007
Civil Engineering Prize for outstanding promise	2006
R W Morris Prize in Hydraulic Engineering	2006
Tonkin and Taylor Prize in Geomechanics	2006
Holmes Consulting Group Structural Engineering Scholarship	2006
CCANZ prize in Reinforced Concrete Design	2006
Lloyd L Hosking Scholarship in Civil Engineering	2006
University of Canterbury Senior Scholarship	2006
University of Canterbury Summer Scholarship	2006
Concrete Prize in Reinforced Concrete Mechanics	2005
MWH Geotechnical Engineering Prize	2005
Ian McMillan Prize	2005
Peter Bryant Memorial Prize	2004
University of Canterbury Emerging Leaders Scholarship	2004

Publications

Journal Articles (39 in total)

1. Rodgers GW, Solberg KM, Mander JB, Chase JG, Bradley BA., Dhakal RP, 2011. High-force-to-volume seismic dissipaters embedded in a jointed precast concrete frame, *Journal of Structural Engineering* 2012 Vol 138 No 3, pp. 375-386.
2. Bradley BA, Use of peak ground velocity in the selection of ground motions from active shallow crustal earthquakes. *Earthquake Spectra* 2012 Vol 28 No 1, pp. 37-54.
3. Bradley BA, Empirical correlations between cumulative absolute velocity and amplitude-based ground motion intensity measures. *Earthquake Spectra* 2012 Vol 28 No 1, pp. 17-35.
4. Bradley BA, Cubrinovski M. Near-source strong ground motions observed in the 22 February 2011 Christchurch earthquake. *Bulletin of the New Zealand Society of Earthquake Engineering, Special Issue on the 22 February 2011 Christchurch earthquake* 2011 Vol 44 No 4, pp 205-226

5. Cubrinovski M, Bradley BA, Wotherspoon L, Green R, Bray J, Wood C, Pender M, Allen J, Bradshaw A, Rix G, Taylor M, Robinson K, Henderson D, Giorgini S, Ma K, Winkley A, Zupan J. Geotechnical Aspects of the 22 February 2011 Christchurch earthquake. *Bulletin of the New Zealand Society of Earthquake Engineering, Special Issue on the 22 February 2011 Christchurch earthquake*. 2011 Vol 44 No 4, pp 181-194.

6. Stirling MW, Litchfield NJ, Gerstenberger M, Clark D, Bradley B, Beavan J, McVerry GH, Van Dissen RJ, Nicol A, Wallace L, and Buxton R. 2011. Preliminary probabilistic seismic hazard analysis of the CO2CRC Otway Project Site, Victoria, Australia. *Bulletin of the Seismological Society of America* 2011 Vol 101 No 6, pp 2726-2736.

7. Bradley BA, Empirical correlation of PGA, spectral accelerations and spectrum intensities from active shallow crustal earthquakes. *Earthquake Engineering and Structural Dynamics* 2011 Vol. 40 No. 15, pp 1707-1721.

8. Wotherspoon L, Bradshaw A, Green R, Wood C, Palermo A, Cubrinovski M, Bradley BA. Bridge Performance during the 2011 Christchurch Earthquake. *Seismological Research Letters* 2011. Vol. 82 No. 6, pp 950-964.

9. Green R, Wood C, Cox B, Cubrinovski M, Wotherspoon L, Bradley BA, Algie T, Allen J, Bradshaw A, Rix G. Use of DCP and SASW tests to evaluate liquefaction potential: Predictions vs. observations during the recent New Zealand Earthquakes. *Seismological Research Letters* 2011. Vol. 82 No. 6, pp 927-938.

10. Cubrinovski M, Bray JD, Taylor M, Giorgini S, Bradley BA, Wotherspoon L, Zupan J. Soil liquefaction effects in the central business district during the February 2011 Christchurch earthquake. *Seismological Research Letters* 2011. Vol. 82 No. 6, pp 893-904.

11. Green R, Allen J, Wotherspoon L, Cubrinovski M, Bradley BA, Bradshaw A, Cox B, Algie T. Performance of levees (stopbanks) during the 4 September 2010, Mw7.0 Darfield and 22 February 2011, Mw6.1 Christchurch, New Zealand Earthquakes. *Seismological Research Letters* 2011. Vol. 82 No. 6, pp 939-949.

12. Bradley BA, Cubrinovski M. Near-source strong ground motions observed in the 22 February 2011 Christchurch Earthquake. *Seismological Research Letters* 2011. Vol. 82 No. 6, pp 853-865.

13. Bradley BA, Correlation of significant duration with amplitude and cumulative intensity measures and its use in ground motion selection. *Journal of Earthquake Engineering 2011* Vol. 15 No. 6, pp 809-832.

14. Bradley BA, Cubrinovski M, Haskell JJM. Probabilistic pseudo-static analysis of pile foundations in liquefiable soils. *Soil Dynamics and Earthquake Engineering* 2011. Vol. 31 No. 10, pp 1414-1425.

15. Reese S, Bradley BA, Bind J, Smart G, Power W, Sturman J. Empirical building fragilities from observed damage in the 2009 South Pacific tsunami. *Earth Science Reviews* 2011. Vol. 107 No. 1-2, pp 156-173.

16. Bradley BA. Displacement spectrum intensity and its use in ground motion selection. *Soil Dynamics and Earthquake Engineering* 2011. Vol 31 No 8. pp 1182-1191.

17. Bradley BA, Design seismic demands from seismic response analyses: A probability-based approach. *Earthquake Spectra* 2011. Vol 27 No 1. pp 213-224.

18. Bradley BA. A framework for validation of seismic response analyses using seismometer array recordings. *Soil Dynamics and Earthquake Engineering* 2011 Vol 31 No. 3, pp 512-520.

19. Cubrinovski M, Green R, Allen J, Ashford S, Bowman E, Bradley BA, Cox B, Hutchinson T, Kavazanjian E, Orense R, Pender M, Wotherspoon L. Geotechnical reconnaissance of the 2010 Darfield (Canterbury) earthquake. *Bulletin of the New Zealand Society of Earthquake Engineering, Special Issue on the 4 September2010 Darfield earthquake* 2010. Vol 43 No 4. pp 243-320.

20. Bradley BA, A generalized conditional intensity measure approach and holistic ground motion selection. *Earthquake Engineering and Structural Dynamics* 2010, Vol 39 No. 12, pp 1321-1342.

21. Fry B, Bannister S, Beavan J, Bland L, Bradley BA, et al. The M_w 7.6 Dusky Sound earthquake of 2009: Preliminary report. *Bulletin of the New Zealand Society of Earthquake Engineering* 2010, Vol 43 No. 1, pp 24-40.

22. Bradley BA, Dhakal RP, Cubrinovski M, MacRae GA. Prediction of spatially distributed seismic demands in specific structures: Structural response to loss estimation. *Earthquake Engineering and Structural Dynamics*. 2010. Vol. 39 No. 6, pp 591-513.

23. Bradley BA, Dhakal RP, Cubrinovski M, MacRae GA. Prediction of spatially distributed seismic demands in specific structures: Ground motion hazard and structural response. *Earthquake Engineering and Structural Dynamics*. 2010. Vol. 39 No. 5, pp 501-520.

24. Bradley BA. Site-specific and spatially distributed prediction of acceleration spectrum intensity. *Bulletin of the Seismological Society of America* 2010. Vol. 100 No. 2, pp 792-801.

25. Bradley BA, Cubrinovski M, Dhakal RP, MacRae GA. Seismic performance and loss assessment of a bridge-foundation-soil system. *Soil Dynamics and Earthquake Engineering* 2010. Vol. 30 No. 5, pp 395-411.

26. Bradley BA. Epistemic uncertainty in component fragility functions. *Earthquake Spectra* 2010, Vol 26 No. 1, pp 41-62.

27. Bradley BA, Lee DS. Component correlations in structure-specific seismic loss estimation. *Earthquake Engineering and Structural Dynamics* 2010, Vol 39 No. 3, pp 237-258.

28. Bradley BA, Lee DS. Accuracy of approximate methods of uncertainty propagation in seismic loss estimation. *Structural Safety* 2010, Vol 32 No. 1, pp 13-24.

29. Bradley BA. Seismic hazard epistemic uncertainty in the San Francisco bay area and its role in performance-based assessment. *Earthquake Spectra* 2009, Vol 25 No. 4, pp 733-753.

30. Bradley BA, Dhakal RP, Cubrinovski M, MacRae GA, Lee DS. Seismic loss estimation for efficient decision making. *Bulletin of the New Zealand Society of Earthquake Engineering* 2009, Vol 42 No. 2, pp 96-110.

31. Bradley BA, Cubrinovski M, Dhakal RP, MacRae GA. Intensity measures for the seismic response of pile foundations. *Soil Dynamics and Earthquake Engineering* 2009, Vol. 29 No. 6, pp 1046-1058.

32. Bradley BA, Cubrinovski M, MacRae GA, Dhakal RP. Ground motion prediction equation for SI based on spectral acceleration prediction. *Bulletin of the Seismological Society of America* 2009, Vol. 99 No. 1, pp 277-285.

33. Bradley BA, Lee DS, Broughton R, Price C. Efficient evaluation of performance-based earthquake engineering equations. *Structural Safety* 2009, Vol. 31 No. 1, pp 65-74.

34. Bradley BA, Dhakal RP. Error estimation of closed form solution for annual frequency of structural collapse. *Earthquake Engineering and Structural Dynamics* 2008, Vol. 37 No. 15, pp 1721-1737.

35. Rodgers GW, Solberg KM, Chase JG, Mander JB, Bradley BA, Dhakal RP, Li L. Performance of a damage-protected beam-column subassembly utilizing external HF2V energy dissipation devices. *Earthquake Engineering and Structural Dynamics* 2008, Vol. 37 No. 13, pp 1549-1564.

36. Solberg KS, Dhakal RP, Bradley BA, Mander JB, Li L. Seismic performance of damage-protected beam column joints. *American Concrete Institute (ACI) Structural Journal* 2008, Vol. 105 No. 2, pp 205-214.

37. Solberg KS, Mander JB, Dhakal RP, Bradley BA. Computational and rapid expected annual loss methodology for structures, *Earthquake Engineering and Structural Dynamics* 2008. Vol. 37 No. 1, pp 81-101.

38. Bradley BA, Mander JB, Dhakal RP. Experimental multi-level seismic performance assessment of 3D RC frame designed for damage avoidance, *Earthquake Engineering and Structural Dynamics* 2008. Vol. 37 No. 1, pp 1-20.

39. Bradley BA, Dhakal RP, Cubrinovski M, MacRae GA, Mander JB, Improved seismic hazard model with application to probabilistic seismic demand analysis, *Earthquake Engineering and Structural Dynamics* 2007. Vol. 36 No. 14, pp 2211-2225.

Journal Articles in Press (5 in total)

1. Bradley BA. The seismic demand hazard and importance of conditioning intensity measure. *Earthquake Engineering and Structural Dynamics* (in press).

2. Bradley BA, Stirling MW, McVerry GH, Gerstenberger M. Consideration and propagation of epistemic uncertainties in New Zealand probabilistic seismic hazard analysis. *Bulletin of the Seismological Society of America* (in press).

3. Bradley BA. Ground motions observed in the Darfield and Christchurch earthquakes and the importance of local site response effects. *New Zealand Journal of Geology and Geophysics* (in press).

4. Bradley BA. Semi-automated ground motion selection with the generalized conditional intensity measure approach. *Soil Dynamics and Earthquake Engineering* (submitted: August 2011).

5. Stirling MW, McVerry G, Gerstenberger M, Litchfield N, Van Dissen R, Berryman K, Barnes P, Wallace L, Bradley BA, Villamor P, Langridge R, Lamarche G, Nodder S, Reyners M, Rhodes D, Smith W, Nicol A, Pettinga J, Clark K, Jacobs K.. National Seismic hazard model for New Zealand: 2010 Update. *Bulletin of the Seismological Society of America* (in press).

Journal Articles Under Review (4 in total)

1. Bradley BA. A critical analysis of strong ground motions observed in the 4 September 2010 Darfield earthquake. *Soil Dynamics and Earthquake Engineering* (submitted: October 2011).
2. Bradley BA, Araki K, Ishii T, Saitoh K. Effect of ground improvement geometry on seismic response of liquefiable soils via 3-D seismic effective stress analysis. *Soil Dynamics and Earthquake Engineering* (submitted: October 2011).
3. Haskell JJM, Cubrinovski M, Bradley BA. The use of pseudo-static analysis for the design of piles in laterally spreading soils. *Soil Dynamics and Earthquake Engineering* (submitted: June 2011).
4. Bradley BA, Cubrinovski M, Simplified probabilistic state concept characterization of sandy soils and application to liquefaction resistance assessment. *Soil Dynamics and Earthquake Engineering* (submitted: February 2012).

Conference Articles (50 in total)

1. Kaklamanos J, Bradley BA, Thompson EM, Baise LG. Critical Parameters Affecting Bias and Variability in Site Response Analyses Using KiK-net Downhole Array Data. *Seismological Society of America Annual Meeting* 17-19 April, 2010. San Diego, US. (abstract).
2. Yeow T, MacRae GA, Dhakal RP, Bradley BA. Seismic sustainability assessment of structural systems: A preliminary case study. *New Zealand Society for Earthquake Engineering Conference.* 13-15 April 2012. Christchurch, New Zealand. (poster).
3. Bradley BA. A comparison of ground motions from the 2011 Mw6.3 Christchurch earthquake with those in Tokyo during the 11 March 2011 Mw9.0 Tohoku, Japan earthquake, and implications for future large events in New Zealand. *New Zealand Society for Earthquake Engineering Conference.* 13-15 April 2012. Christchurch, New Zealand. Paper 39. 8pp.
4. Bradley BA. Observed Ground Motions in the 4 September 2010 Darfield and 22 February 2011 Christchurch Earthquakes. *New Zealand Society for Earthquake Engineering Conference.* 13-15 April 2012. Christchurch, New Zealand. Paper 37. 8pp
5. Bradley BA. Consistency of Seismicity and Ground Motion Modelling with the Canterbury Earthquakes. *New Zealand Society for Earthquake Engineering Conference.* 13-15 April 2012. Christchurch, New Zealand. Paper 38. 8pp.
6. Ishikawa Y, Bradley BA. Estimated seismic performance of a standard NZS3101:2006 RC office building during the 22 February 2011 Christchurch Earthquake. *New Zealand Society for Earthquake Engineering Conference.* 13-15 April 2012. Christchurch, New Zealand. Paper 133. 8pp.
7. Robinson K, Bradley BA, Cubrinovski M. Comparison of Actual and Predicted Measurements of Liquefaction-Induced Lateral Displacements during 2010 Darfield and 2011 Christchurch Earthquakes. *New Zealand Society for Earthquake Engineering Conference.* 13-15 April 2012. Christchurch, New Zealand. Paper 54. 7pp.
8. Green RA, Cubrinovski M, Wotherspoon L, Allen J, Bradley BA, Bradshaw A, Bray J, DePascale G, Orense R, O'Rouke T, Pender M, Rix G, Wells D, Wood C, Henderson D, Hogan L, Kailey P, Robinson K, Taylor M, Winkley A. Geotechnical effects of the Mw6.1 2011

Christchurch Earthquake. *ACSE Geo-institute GeoCongress*, 25-29 March 2012, Oakland, California, US. 10pp.

9. Cubrinovski M, Henderson D, Bradley BA. Liquefaction impacts in residential areas in the 2010-2011 Christchurch earthquakes. One Year after 2011 Great East Japan Earthquake: - International Symposium on Engineering Lessons Learned from the Giant Earthquake - 3-4 March 2012. Tokyo, Japan. 14pp.

10. Ishihara K, Araki K, Bradley BA. Characteristics of liquefaction-induced damage in the 2011 Great East Japan Earthquake. *International conference on geotechnics for sustainable development - Geotec* (Keynote lecture), 6-7 October 2011, Hanoi, Vietnam. 22pp.

11. Bradley BA, Araki K, Ishii T, Saitoh K. 3-D seismic response of liquefaction-susceptible improved-soil deposits. *Disaster simulation and structural safety in the next generation* (DS11), 17-18 September 2011, Kobe, Japan. 8pp.

12. Bradley BA, Cubrinovski M. Strong ground motions observed in the 22 February 2011 Christchurch Earthquake. *2011 Southern California Earthquake Centre (SCEC) Annual Meeting.* 12-14 September 2011. Palm Springs, California, USA. (poster).

13. Bradley BA. Open-source ground motion selection using the generalized conditional intensity measure (GCIM) approach. *2011 Southern California Earthquake Centre (SCEC) Annual Meeting.* 12-14 September 2011. Palm Springs, California, USA. (poster).

14. Stirling MW, McVerry GH, Gerstenberger MC, Litchfield N, Van Dissen R, Berryman K,. Wallace L, Villamor P, Langridge R, Nicol A, Reyners M, Rhoades D, Smith W, Clark K, Barnes P, Lamarche G, Nodder S, Bradley BA, Pettinga J, Jacobs K. Precis of the New National Seismic Hazard Model for New Zealand. *9th Pacific Conference on Earthquake Engineering*, 14-16 April 2011, Auckland, New Zealand.

15. Green RA, Cubrinovski M, Ashford SA, Hutchinson T, Kavazanjian E, Bradley BA, Cox B, Orense R, Pender M, Allen J, Quigley M, Wotherspoon L, ORouke T, Algie T. Geotechnical aspects of the M_w7.1, 4 September 2010, Darfield, New Zealand, Earthquake. *Seismological Society of America Annual Meeting* 13-15 April, 2011. Memphis, US.

16. Bradley BA. Validation of seismic response analyses using seismometer array recordings. *5th International Conference on Earthquake Geotechnical Engineering* 10-13 January 2011. Santiago, Chile. 8pp.

17. Uma SR, Bradley BA. Displacement-based fragility functions for New Zealand buildings subject to ground motion hazard. *3rd Asian Conference on Earthquake Engineering* 1-3 December 2010. Bangkok, Thailand. Paper 72. 8pp.

18. Stirling MW, McVerry G, Gerstenberger M, Litchfield N, Van Dissen R, Barnes P, Berryman K, Bradley BA, et al. The national seismic hazard model for New Zealand, and the Mw7.1 4 September Darfield earthquake. *New Zealand Geoscience Society Conference* 2010 Auckland, New Zealand. 22-24th November (abstract)

19. Bradley BA. Applicability of foreign ground motion prediction equations in New Zealand. *2010 Southern California Earthquake Centre (SCEC) Annual Meeting.* 13-16 September 2010. Palm Springs, California, USA. (poster).

20. Bradley BA. OpenSHA implementation of the GCIM approach for ground motion selection. *2010 Southern California Earthquake Centre (SCEC) Annual Meeting.* 13-16 September 2010. Palm Springs, California, USA. (poster).

21. Van Dissen R, et al. Its Our Fault: Better defining earthquake risk in Wellington. *11th IAEG Congress* 5-10 September 2010. Auckland, New Zealand. 8pp.

22. Bradley BA. A generalized conditional intensity measure approach and holistic ground motion selection. *14th European Conference on Earthquake Engineering.* 30 August – 3 September 2010. Ohrid, Macedonia. 8pp.

23. Haskell JJM, Cubrinovski M, Bradley BA. Pseudo-static analysis of piles in liquefying soils: A framework for the consideration of uncertianties. *14th European Conference on Earthquake Engineering.* 30 August – 3 September 2010. Ohrid, Macedonia. 8pp.

24. Reese S, van Zijll de Jong S, Power W, Bind J, Smart G, Bradley BA, Prasetya G, Wilson K. Lessons learnt from the Samoan Tsunami 2009 – A multi-disciplinary survey. *Australasian Natural Hazards Management Conference* 11-12 August 2010, Wellington, New Zealand.

25. Power W, Wilson K, Prasetya G, Bradley BA, Wang X, Beavan J, Holden C. The 2009 South Pacific tsunami – implications for tsunami hazard in the South Pacific. *European Geosciences Union General Assembly* 2-7 May 2010, Vienna, Austria (abstract)

26. Bradley BA. A generalized conditional intensity measure approach and holistic ground motion selection. *Seismological Society of America Annual Meeting* 21-23 April, 2010. Portland, US. (poster).

27. Bradley BA. Ground motion selection for seismic response analyses. *New Zealand Society for Earthquake Engineering Conference.* 26-28 March 2010, Wellington, New Zealand (poster).

28. Cubrinovski M, Haskell JM, Bradley BA. Soil-pile interaction in liquefying soils: modelling issues. *International Workshop on Soil-Foundation-Structure-Interaction* 26-27 November 2009. Auckland, New Zealand. 8pp.

29. Bradley BA, Cubrinovski M, Dhakal RP, MacRae GA. Probabilistic seismic performance assessment of a bridge-foundation-soil system. *International Workshop on Soil-Foundation-Structure-Interaction* 26-27 November 2009. Auckland, New Zealand. 8pp.

30. Haskell JJM, Cubrinovski M, Bradley BA. Critical uncertainties in the analysis of piles in liquefying soils. 17th International Conference on Soil Mechanics and Geotechnical Engineering 2009, Alexandria, Egypt, 4pp.

31. Cubrinovski M, Haskell JJM, Bradley BA. The effect of shear strength normalization on the response of piles in laterally spreading soils. Earthquake Geotechnical Engineering Satellite Conference 17th International Conference on Soil Mechanics and Geotechnical Engineering 2009, Alexandria, Egypt, 9pp

32. Bradley BA, Cubrinovski M. Probabilistic pseudo-static analysis of pile foundations in liquefiable soils. *International Conference on Performance-Based Design in Earthquake Geotechnical Engineering – From Case History to Practice* 15-19 June 2009, Tsukuba, Japan 8pp.

33. Cubrinovski M, Bradley BA. Evaluation of seismic performance of geotechnical structures. *Performance-based design in earthquake geotechnical engineering – from case history to practice (IS-Tokyo09)* 2009. Tsukuba, Japan (Theme lecture). 17 pp.

34. Haskell JJM, Cubrinovski M, Bradley BA. Sensitivity analysis of simplifies for the design of piles in lateral spreading soils. *New Zealand Society for Earthquake Engineering Conference* 2009, Christchuch, New Zealand, 9pp.

35. Bradley BA, Cubrinovski M, Dhakal RP, MacRae GA. Probabilistic seismic performance assessment of a bridge-foundation-soil system. *New Zealand Society for Earthquake Engineering Conference* 2009, Christchuch, New Zealand, 10pp.

36. Bradley BA, Dhakal RP, Cubrinovski M, MacRae GA. Prediction of spatially distributed seismic demands in structures with application to loss estimation, *14th World Conference in Earthquake Engineering* 12-17 October 2008, Beijing, China. Paper 05-01-0176. 9 pp.

37. Cubrinovski M, Bradley BA. Assessment of seismic performance of soil-structure systems. *NZGS geotechnical symposium* 4 – 6 September 2008. Auckland, New Zealand (invited paper). 17 pp.

38. Bradley BA, Cubrinovski M, Dhakal RP. Performance-based seismic response of pile foundations, *Geotechnical Earthquake Engineering and Soil Dynamics IV* 18-22 May 2008 (GSP181), Sacramento, USA. 10 pp.

39. Bradley BA, Dhakal RP, Cubrinovski M, MacRae GA, Lee DS. Seismic loss estimation for efficient decision making. *The New Zealand Society for Earthquake Engineering Conference* 11-13 April 2008, Taupo, New Zealand. Paper no 32, 34 pp.

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1. A framework for validation of ground motion simulations emphasizing predictive power and use of seismic effective stress analyses of soil deposits *Southern California Earthquake Centre (SCEC)* Ground motion simulation validation (GMSV) *Technical Activity Group*, University of Southern California. April 2, 2012.
2. The 22 February Christchurch Earthquake: Are we properly characterizing rare/extreme events? Plenary Lecture *Southern California Earthquake Centre (SCEC) Annual Meeting*, 2011, Palm Springs, USA.
3. Seismological aspects and ground motion characteristics of the Darfield (Canterbury) Earthquake. Special Session: *5th International Conference on Earthquake Geotechnical Engineering,* 2011,Santiago, Chile.
4. The M_w7.1 Darfield Earthquake of 4th September 2010. *Chemical Grouting Company Limited,* 2010,Tokyo, Japan.
5. Preliminary findings from the M_w7.1 Darfield Earthquake of 4th September 2010. *ROSE School, EUCENTRE* 2010, Pavia, Italy.
6. A generalised conditional intensity measure approach and holistic ground motion selection. *ROSE School, EUCENTRE* 2010, Pavia, Italy.
7. Frameworks for the consideration of ground motion and seismic response uncertainties in seismic performance assessment. *International workshop on protection of built environment against earthquakes* 2010. Ljubljana, Slovenia.
8. Some preliminary findings from the 29th September Samoan Tsunami. *GNS Science Symposium* 2009 Wellington, New Zealand
9. Probabilistic seismic performance assessment of a bridge-foundation-soil system. *International Workshop on Soil-Foundation-Structure-Interaction* 2009. Auckland, New Zealand.
10. A displacement-based procedure for regional loss estimation. *GNS Science/NIWA/Geoscience Australia Symposium* 2009, Canberra, Australia.
11. Structure-specific seismic risk assessment: An example and its role in earthquake engineering. *University of Canterbury Seminar*, 2009, Christchurch, New Zealand.

Other Technical Presentations (15 in total)

1. Bradley BA. Using seismometer array recordings to validate seismic response analyses. *5th International Conference on Earthquake Geotechnical Engineering* 10-13 January 2011. Santiago, Chile. 8pp.
2. Bradley BA. A generalized conditional intensity measure approach and holistic ground motion selection. *14th European Conference on Earthquake Engineering.* 30 August – 3 September 2010. Ochrid, Macedonia. 8pp.
3. Probabilistic seismic performance assessment of a bridge-foundation-soil system. *International Workshop on Soil-Foundation-Structure-Interaction* 2009. Auckland, New Zealand.
4. Probabilistic Pseudo-static Analysis of Pile Foundations in Liquefiable Soils. *International Conference on Performance-Based Design in Earthquake Geotechnical Engineering – From Case History to Practice* 2009, Tsukuba, Japan.
5. Probabilistic seismic performance assessment of a bridge-foundation-soil system. *New Zealand Society of Earthquake Engineering Conference* 2009, Christchuch, New Zealand.
6. Prediction of Spatially Distributed Seismic Demands in Structures with Application to Loss Estimation, *14th World Conference in Earthquake Engineering* 2008, Beijing, China.
7. Performance-based Seismic Response of Pile Foundations, *Geotechnical Earthquake Engineering and Soil Dynamics IV* 2008, Sacramento, USA.
8. Seismic loss estimation for efficient decision making. *The New Zealand Society of Earthquake Engineering Conference*, Taupo, New Zealand.
9. Quasi-static Testing of a damage-protected Beam-column subassembly with Internal Lead Damping Devices, *8th Pacific Conference on Earthquake Engineering* 2007, Singapore.
10. Probable Loss Model and Spatial Distribution of Damage for Probabilistic Financial Risk Assessment of Structures, *10th International Conference on Applications of Probability and Statistics in Civil Engineering* 2007, Tokyo, Japan.
11. Improved Seismic Hazard Model for use in Performance Based Engineering. *The New Zealand Society of Earthquake Engineering Conference* 2007, Palmerston North, New Zealand.
12. Multi-level Seismic Performance Assessment of a Damage-protected Beam-column Joint with Internal Lead-dampers, *The New Zealand Society of Earthquake Engineering Conference* 2007, Palmerston North, New Zealand.
13. Performance of Damage Avoidance Beam-Column Joint Subassembly Subjected to Bi-directional Earthquake Excitation, *19th Australasian Conference on the Mechanics of Structures and Materials* 2006, Christchurch, New Zealand.
14. Dependency of current Incremental Dynamic Analysis to source mechanisms of selected records, *19th Australasian Conference on the Mechanics of Structures and Materials* 2006, Christchurch, New Zealand.
15. Modeling and Analysis of Multi-Storey Buildings Designed to Principles of Ductility and Damage Avoidance, *Tenth East Asia-Pacific Conference on Structural Engineering and Construction* 2006, Bangkok, Thailand.

Professional Affiliations and Activities

Society Memberships

New Zealand Society of Earthquake Engineering,	*2007-present*
New Zealand Geotechnical Society	*2008-present*
International Society of Soil Mechanics and Geotechnical Engineering	*2008-present*
Seismological Society of America	*2008-present*
Earthquake Engineering Research Institute	*2011-present*

Peer Review

Reviewer for the following international journals

- Bulletin of the New Zealand Society of Earthquake Engineering
- Bulletin of the Seismological Society of America
- Earthquake and Structures
- Earthquake Engineering and Structural Dynamics
- Earthquake Spectra
- Engineering Geology
- Journal of Earthquake Engineering
- Journal of Structural Engineering – ASCE
- New Zealand Journal of Geology and Geophysics
- Natural Hazards
- Soil Dynamics and Earthquake Engineering
- Structural Safety

Reviewer for the following funding agencies

- New Zealand Earthquake Commission (2010,2011)

Conference scientific committees, working groups and advisory panels

- Ground motion simulation validation (GMSV) *Southern California Earthquake Centre Technical Activity Group*, University of Southern California. April 2, 2012.
- International conference *Earthquake Design and Analysis*, Coimbatore, India. December 1-3, 2011.
- TC203: Earthquake Geotechnical Engineering, *working group on site characterization, site effects and design ground motion*. 2011-2015.
- Invited participant, *FRST Natural Hazards Research Platform Immediate Outcomes workshop*. 2010. Wellington, New Zealand.
- Invited participant, *International Workshop on Soil-Foundation-Structure-Interaction* 2009. Auckland, New Zealand.
- Invited member. *Seismic instrumentation advisory panel*, 2009. Wellington, New Zealand.

Natural Hazard reconnaissance

- 2011 Great East Japan Earthquake

- 2011 Christchurch Earthquake
- 2010 Darfield, Canterbury Earthquake
- 2009 Samoan Tsunami

Sponsored Research Projects

Total research funding of $686,000 as PI since 2010.

1. Ministry of Science and Innovation, Natural Hazards Research Platform (NHRP) 2012-2015 (PI - $500,000) "Quantification and Numerical Modelling of Source, Path, and Site-Specific Effects in the Canterbury Earthquakes"
2. New Zealand Earthquake Commission (EQC) 2011-2012 (PI - $35,000) "Strong ground motion analysis of the Canterbury earthquakes in the near source region"
3. New Zealand Earthquake Commission (EQC) 2011-2012 (PI - $42,000) "Seismic site response analysis of soil sites during the Christchurch earthquakes"
4. New Zealand Earthquake Commission (EQC) 2011-2012 (Co-PI - $42,200) "Geotechnical characterization of Christchurch's strong motion stations"
5. Southern California Earthquake Centre (SCEC) 2012 (PI - $3,000 USD) "A framework for validation of ground motion simulations emphasizing predictive power and use of seismic effective stress analyses of soil deposits".
6. Southern California Earthquake Centre (SCEC) 2010-2011 (PI - $6,000 USD) "Implementation of a generalized conditional intensity measure (GCIM) approach in OpenSHA".
7. Earthquake Commission (EQC) Biennial Grants Programme 2010-2011 (PI - $40,000) "Consideration and propagation of epistemic uncertainties in probabilistic seismic hazard analysis".
8. University of Canterbury Earthquake Recovery Doctoral Scholarship 2012-2014 (PI - $60,000) "Seismic response analysis of liquefiable soils in Christchurch and mitigation using ground improvement methodologies"

"B"

Statistical Analysis of Concrete Test Data from the Canterbury Television (CTV) Building

Technical Report Prepared for the Canterbury Earthquakes Royal Commission

Brendon A. Bradley[1,2]

[1]Director, Bradley Seismic Limited

[2]Lecturer, Department of Civil and Natural Resources Engineering, University of Canterbury

15 May 2012

Christchurch, New Zealand

Disclaimer:

This report has been prepared for the Canterbury Earthquakes Royal Commission to provide information on the statistical significance of post-earthquake column test data obtained from Canterbury Television (CTV) building. The authors accept no liability for the use of the material contained in this document by third parties.

Table of Contents

Executive Summary

This report provides a statistical examination of the observed concrete column test data obtained from the Canterbury Television (CTV) building in relation to the expected results implied from the building's design specifications.

A rigorous statistical analysis of the concrete column test data obtained from the CTV building is necessary as a result of: (i) the paucity of tests performed; (ii) the majority of the test results being based on inferred column compressive strengths from Schmidthammer testing - a correlation which shows large uncertainty; and (iii) the unknown locations of the extracted columns in the building, and the fact that the specified concrete column strengths vary from 35 to 25 MPa over the height of the structure. In order to account for these uncertainties, with particularly small sample sizes, the Monte Carlo simulation method is adopted.

The statistical analyses performed make the assumption that test results are representative of the characteristics of the concrete columns prior to the 22 February 2011 earthquake (with the exception of column C18 as discussed). Such an assumption is made because of the difficulty in quantifying the effect of collapse, fire, demolition and retrieval. The caveats in the results which result from this assumption are elaborated upon in detail when interpreting the analysis results.

The statistical analysis illustrates that the distribution of observed concrete strengths and expected concrete strengths (from design specifications) cannot be rejected as being statistically different. This result is obtained even without the consideration that the test specimens may have been comprimised as a result of damage from collapse, fire, demolition, and removal from the site. This result therefore conflicts with the opinions expressed in Hyland (2012), but is consistent with the opinions expressed, concrete materials perspective, by Mackechnie (2012) and CCANZ (2012).

1. Introduction

This report provides a statistical examination of the observed concrete column test data obtained from the Canterbury Television (CTV) building and their relation to the expected results implied from the building's design specifications.

A rigorous statistical analysis of the concrete column test data obtained from the CTV building is necessary as a result of: (i) the paucity of tests performed; (ii) the majority of the test results being based on inferred column compressive strengths from Schmidthammer testing - a correlation which shows large uncertainty; and (iii) the unknown locations of the extracted columns in the building, and the fact that the specified concrete column strengths vary from 35 to 25 MPa over the height of the structure. In order to account for these uncertainties, with particularly small sample sizes, the Monte Carlo simulation method is adopted.

Section 2 examines the observed concrete column test results obtained, as presented in Hyland (2012), and the distribution of concrete strengths from these observations, accounting in particular for the uncertainty in the correlation between concrete core compressive strength and Schmidthammer test results. Section 3 examines the distribution of concrete strengths that would be expected to be observed in tests from specimens taken from the CTV building with strengths as per the design specification, accounting for, in particular, the uncertain location of the columns from which the majority of specimens were obtained. Section 4 compares the distribution of observed concrete strengths with the distribution of expected concrete strengths, and the statistical significance of any differences. Section 5 summarizes the results of the analyses and their implications.

2. Distribution of observed concrete strengths

A total of 24 concrete column elements from the CTV building were completed (Hyland 2012). Note that this report examines only the concrete column elements and not the additional 5 tests performed by Hyland (2012) on slabs, wall, core, and beam elements (since the sample size of 1 for each of these other element types does not allow a rigorous assessment of their characteristics).

Hyland (2012) acknowledged that portions of the columns that were tested were done so on "members that had suffered distress during the collapse". Mackechnie (2012) examined photographic evidence of tested elements and devised a subjective rating system for cracking risk based on visual assessment. As a result of this rating system, test specimen "C18" in Hyland (2012) and Mackechnie (2012) was considered as significantly compromised and removed from the analysis performed in this report. Details of the remaining 23 test specimens whose data is used in this report are given in Table 1.

Table 1: Summary of concrete element test properties

Name of element	Location in structure	Compressive Strength (MPa)	Average Schmidt hammer strength	Median predicted strength from hammer (MPa)
tC1	Level 6	27.0	42.1	(29.8)[1]
tC2	Level 1	-	42	31.3
tC3	Unknown	-	42	29.0
tC4	Unknown	46.6	47.9	(44.3)[1]
tC5	Level 6	-	39.5	26.2
tC6	Level 6	-	36	20.8
tC7	Unknown	-	42	29.5
tC8	Level 5-6	-	34	18.5
tC9	Level 6	-	38	23.2
tC10	Level 1-2	-	33	18.4
tC11	Unknown	-	34	19.4
tC12	Unknown	26.7	44.1	(33.2)[1]
tC13	Unknown	-	37	22.4
tR1	Unknown	-	38	23.4
tR2	Level 6	-	41	28.1
tR3	Unknown	20.3	35.4	(20.5)[1]
tR4	Level 5-6	-	46	37.0
tR5	Level 5-6	-	44	32.4
tR6	Unknown	25.5	36.8	(22.2)[1]
tR7	Unknown	40.9	46.4	(37.7)[1]
tR8	Unknown	-	41	28.1
tR9	Unknown	-	38	23.4
tR10	Unknown	-	46.5	38.7

[1]The predicted values via hammer testing were not used since direct results from core testing were available.

2.1. Correlation between Schmidt hammer and core compressive strength

As can be seen in Table 1, only 6 of the 23 test results are based directly on concrete core strengths, while the remaining results require the use of a correlation between core strength and Schmidt hammer strengths. Figure 1 illustrates the correlation that was obtained between the directly obtained core strength and the Schmidt hammer strength. It can be seen that there is a large variability between these two strength metrics. Such variability is not unexpected, for example ASTM C805 (American Society for Testing and Materials 2008) notes that even when calibrated hammer strengths may range by $\pm 50\%$.

The best-fit line to the data in Figure 1, as also noted in Hyland (2012), can be evaluated as:

$$CS = 2.9164 \exp(0.0552 * SHS) \tag{1}$$

where CS is the core strength, and SHS is the Schmidt hammer test strength. As noted in the caption of Figure 1, the fact that CS and SHS are related in this fashion implies that for a given observed SHS value the distribution of CS is lognormal. As part of the least-squares regression to determine Equation (1) is can be found that the standard deviation of this lognormal distribution is $\sigma_{lnCS} = 0.1577$, which represents approximately a 16% coefficient of variation.

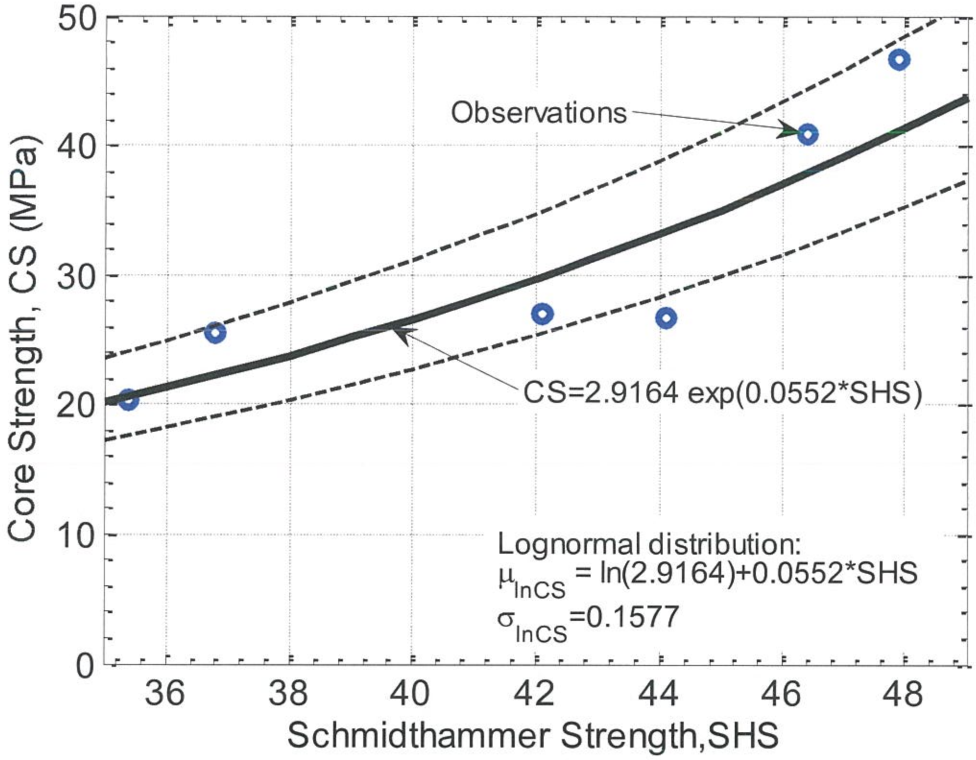

Figure 1: Correlation between Schmidthammer strength and core strength for six tests from the CTV building columns.

2.2. Distribution of observed concrete strengths accounting for correlation uncertainty

Using Equation (1), the median prediction for the core strength for the 17 concrete column test specimens without direct measurements can be computed as shown in Table 1. These 17 values, combined with the 6 values for which core strengths have been obtained directly can be considered as 23 data points representative of the observed strength of the concrete columns in the CTV building (with the caveat that they are not compromised as a result of collapse, fire, demolition, and retrieval). However, as noted in section 2.1, there is significant uncertainty in the correlation between the Schmidt hammer and core strengths which should be explicitly accounted for. For this purpose, Monte Carlo simulation is utilized to obtain random core strengths for each of the 17 test specimens with only Schmidt hammer values. The 6 core strengths that were computed directly were used as is. All values generated were multiplied by a factor of 1.08 to account for the effect of the testing orientation being transverse to the casting orientation (GBCS 1987, Hyland 2012). The effect of age is noted in the following section on expected concrete strengths from design specifications, and was not used to modify the observed concrete strength results.

Figure 2 illustrates the (empirical) cumulative distribution function of the 23 observed concrete column test results. The empirical cumulative distribution function gives the proportion of the empirical data which is less than or equal to a particular value. Thus, a value of 0.6 for a concrete strength of 30MPa implies that 60% of the observed data have value less than 30MPa. The solid black line represents the empirical distribution neglecting the uncertainty in the Schmidt hammer-core strength correlation, while the dashed red line considers this uncertainty (via Monte Carlo simulation). It can be seen that while the two results are not equal they are also not notably different, indicating that the implications to follow are not principally the result of the uncertainty in this correlation.

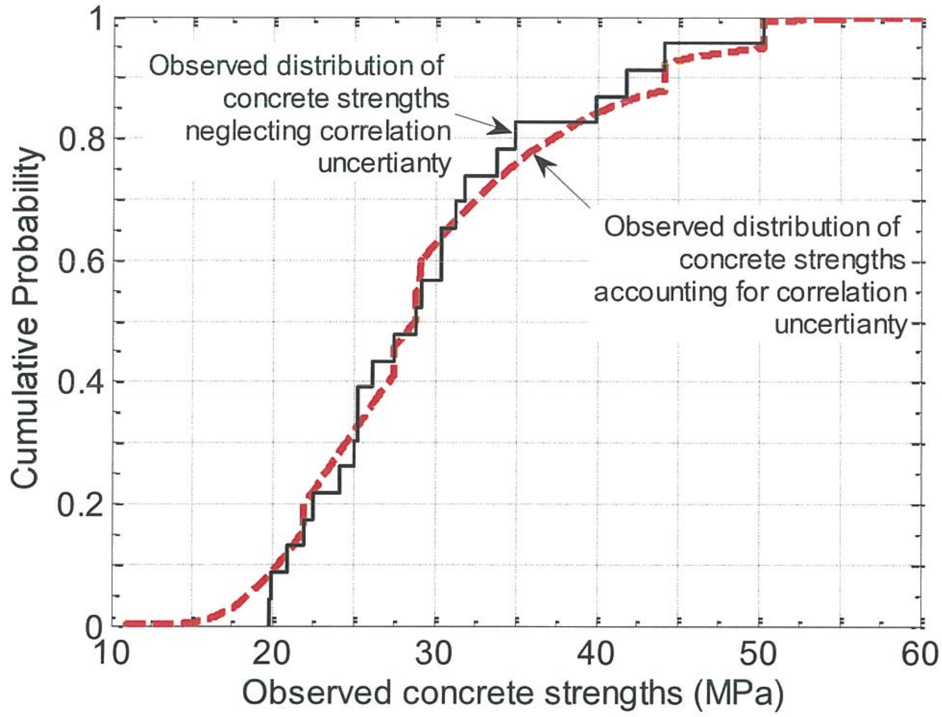

Figure 2: Cumulative distribution of the observed concrete strengths from 23 test specimens and accounting for uncertainty in the Schmidt hammer-core strength correlation.

3. Distribution of expected concrete strengths from design specifications

Concrete strengths for columns in the CTV building varied from 35MPa for the ground floor, 30MPa for the second floor, and 25MPa for the remaining floors (3^{rd}-6^{th}). Based on these specified concrete strengths the distribution of actual concrete strength is given by the target mean and standard deviation values in the first row of Table 2.

Based on the target mean values identified by NZS3104:1983, Hyland (2012) suggested that an increase of 25% should be added to account for aging effects, based on observational data for bridges in California (second row in Table 2). However, both Mackechnie (2012) and CCANZ (2012) note that the use of such a factor for aging effects for slender elements, which do not contain Supplementary Cementitious Materials, and are sheltered from the atmosphere is inappropriate. Furthermore, Mackechnie (2012) and CCANZ (2012) both note that as the concrete used in the construction of the CTV building was from a "Special Grade plant" which allows for the use of a target strength 3 MPa lower, as a result of a higher level of quality control. It is these target mean values suggested by CCANZ (2012), in the third row of Table 2, which are utilized in the results to follow.

Table 2: Specified and target concrete strengths as implied by NZS3104:1983

<table>
<tr><th></th><th>Specified Strength (28 days)</th><th>25 (MPa)</th><th>30 (MPa)</th><th>35 (MPa)</th></tr>
<tr><td rowspan="3">Target Mean</td><td>NZS3104:1983</td><td>33.5</td><td>40.0</td><td>45.5</td></tr>
<tr><td>NZS3104:1983 increased by 25% for aging (as applied by Hyland (2012))</td><td>41.9</td><td>50.0</td><td>56.9</td></tr>
<tr><td>NZS3104:1983 reduced by 3MPa for Special grade TMS (as noted by Mackechnie (2012) and CCANZ (2012))</td><td>30.5</td><td>36.5</td><td>42.5</td></tr>
<tr><td colspan="2">Target Standard deviation</td><td>4.70</td><td>5.20</td><td>5.46</td></tr>
</table>

The test specimen data presented in Table 1 is also notable in that only 10 of the 23 specimens come from known floor locations within the CTV building. This is particularly important in light of the fact that the specified strengths of the concrete columns vary from 35 MPa on the first floor, 30MPa on the second, and 25MPa on the 3rd to 6th floors. As a result, it is necessary to account for the uncertainty in the location of 'unknown' elements when determining the distribution of concrete strengths which would be expected based on the design specifications in Table 2.

Similar to the computation of the observed distribution of strength data in the previous section, to compute the distribution of concrete strengths expected to be observed based on the specified concrete strengths, Monte Carlo simulation was employed. Where the location of the test specimen was known a random value of the expected concrete strength was simulated based on the target mean and standard deviation in Table 2 (e.g. a random strength values expected for specimen tC1, located on the 6th floor was obtained from a normal distribution with mean 30.5MPa and standard deviation 4.70MPa). Where the location of a particular test specimen was unknown it was considered as equally likely to have come from any of the 6 floors in the CTV building. As such, Monte Carlo simulation was used to first simulate a random floor corresponding to the specimen. Based on this random floor, a random core strength was simulated from the properties in Table 2 (as in the case where the location of the specimen is specifically known).

Figure 3 illustrates the distribution of concrete strengths expected, for the 23 column test specimens, if the strengths conform to those specified as per NZS3104:1983 (i.e. Table 2). Also shown in Figure 3 is the distribution of concrete strength for specified strengths of 25, 30, and 35 MPa (i.e. simply a normal distribution based on the target mean and standard deviation values given in Table 2). Firstly, it can be seen that the expected distribution of the 23 test specimens has a larger variability than each of the three specified strength distributions, as a result of the fact that the test specimens correspond to more than one strength specification (because they are based on columns which occur at multiple, although often unknown, floors in the structure). Secondly, it can be seen that the overall distribution is most closely aligned with the 25MPa distribution as a result of the fact that many of the test specimens come from unknown locations in the structure, and there is a 4/6 chance that such column specimens are from the 3rd – 6th floors (which have specified strengths of 25MPa).

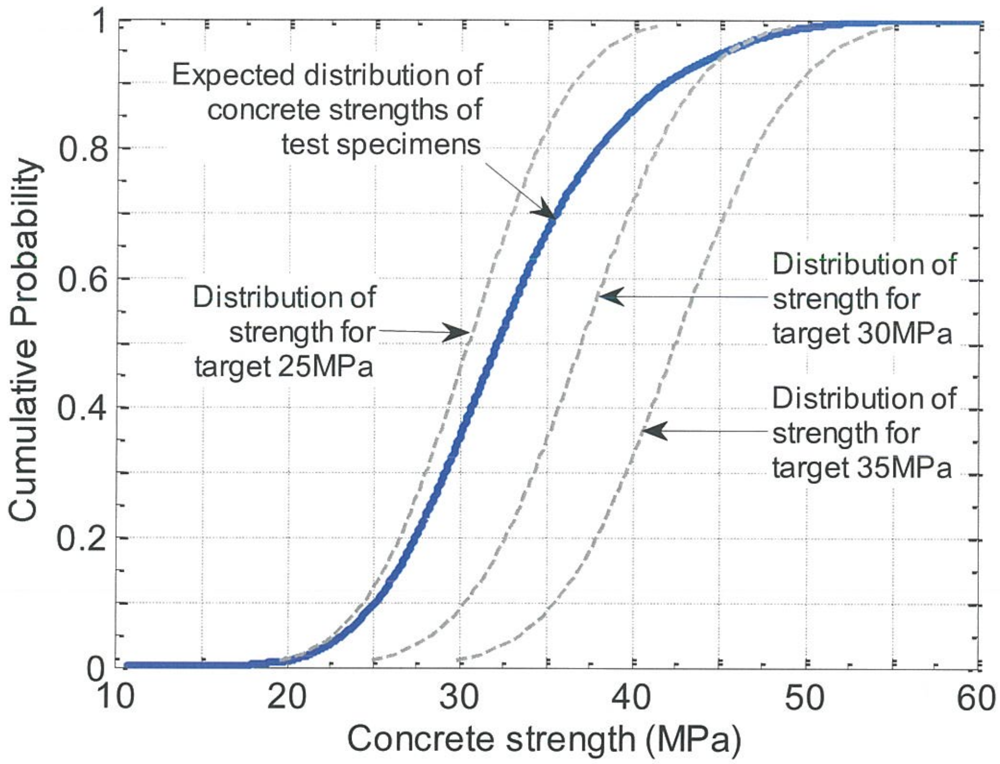

Figure 3: Distribution of concrete column strengths for the 23 column tests expected if the strengths conform to the specified strengths in Table 2. This distribution accounts for the uncertain location within the CTV building of the majority of the test specimens.

4. Comparison of observed and predicted strength distributions

Figure 2 provides the distribution of observed concrete strengths from the 23 column test specimens in Table 1, including the effect of uncertainty in the correlation between Schmidt hammer and core strength. Figure 3 provides the distribution of concrete strengths that would be expected to be observed from the 23 column tests, accounting for the unknown locations of the specimens in the CTV building.

Figure 4 illustrates a comparison between the observed distribution of concrete strengths (i.e. from Figure 2) and the expected distribution of concrete strengths based on those specified in Table 2 (i.e. from Figure 3). From inspection alone it can be seen that the two results are not dissimilar. From these two distributions it can be quantitatively stated that there is a probability of 0.65 (i.e. 65%) that any given observed strength will be less than a random value of the expected distribution of concrete strengths based on design specifications. While this value may first be perceived to be large given that it is greater than 50%, it actually implies that there is a 35% probability that the observed concrete strength is greater than the design specification. In statistical hypothesis testing it is conventional that the level of significance would have to be greater than 90% (so that there was less than a 10% probability of being incorrect) to confidently distinguish differences these two distributions. Thus, if the 'null hypothesis' was that there is no difference between the observed concrete strengths and those specified via design, then based on Figure 4 this hypothesis could not be rejected at a significance level of 90%. The fact that the probability of 65% is significantly less that this critical 90% value shows that, based on the 23 column test data examined, there is no credible evidence to suggest that the observed concrete strengths are lower than the specified concrete strengths.

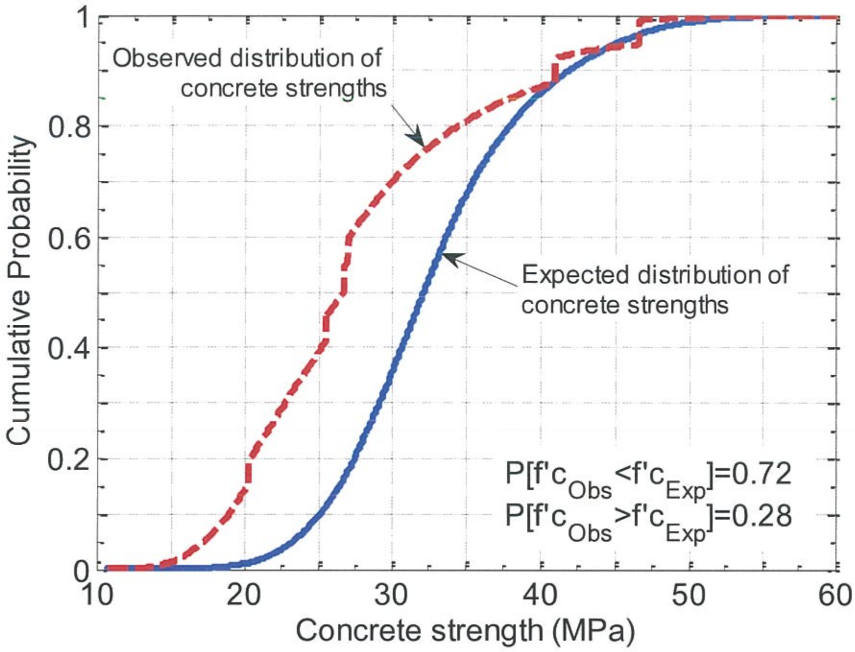

Figure 4: Comparison of the distribution of observed concrete strengths and expected concrete strengths based on design specifications. The probability that a single test observation is greater than a random value of the specified strength is 0.65 (i.e. 65%). This means that there is a 35% probability that the observed strength is actually greater than the specified strength.

5. Implications

Hyland (2012) expressed the opinion that the observed concrete strengths from the 23 test specimens were likely inconsistent with those specified in design. The following caveats in the analyses of Hyland (2012) are worthy of note:

(i) The uncertainty in the correlation between Schmidt hammer and core strengths was not considered

(ii) The target mean strength was considered to have been increased by 25% based on the consideration of such a factor for bridges in California. However, for slender elements, which do not contain Supplementary Cementitious Materials, and are sheltered from the atmosphere, such a factor is considered inappropriate (Mackechnie (2012) and CCANZ (2012)). Furthermore, the target mean strength used by Hyland (2012) was not reduced by 3 MPa to account for the fact that Special Grade plants have a greater quality control.

(iii) The uncertainty in the location of the test specimens from within the structure was not considered which is important given that the specified design strength varies from 35 MPa on the first floor, 30 MPa on the second floor, and 25 MPa on the 3^{rd} -6^{th} floors.

In this report, the above factors have been explicitly considered. The results obtained suggest that there is only a 65% probability that the observed concrete strengths are less than the specified concrete strengths (implying a 35% probability that the observed concrete strengths are actually greater than the specified concrete strengths). Following conventional statistical inference, the hypothesis that the observed and specified concrete strengths are equal cannot be rejected at the 90% significance level.

As part of this report, a sensitivity study was conducted based on several qualitative assumptions which informed the analysis. If the 8% increase in observed concrete strengths due to testing in the transverse direction was ignored then the 65% probability previously noted becomes a 72% probability, still well below 90% and therefore not affecting any of the aforementioned statements. It should be noted also that it is generally acknowledged that concrete core testing yields strengths with are 10-20% lower than cylinder tests (upon which specified strengths are based) (Mackechnie 2012), hence this neglected effect more than offsets the additional 8% increase if transverse loading effects were neglected, and also any actual increase in strength that has not been considered.

Finally, it is to be explicitly restated that the analyses considered in this report have not considered any deterioration in the test specimens resulting from specimen cracking due collapse, demolition and retrieval, nor from exposure to prolonged high temperatures from fire. These effects, however significant, will tend to give further confidence in the statement that the observed and design specified concrete strengths are not statistically different.

6. Conclusions

On the basis of the statistical analyses performed as well as the caveats noted, it can be stated that, based on the 23 column test data examined, there is no credible evidence to suggest that the observed concrete strengths are lower than the specified concrete strengths.

7. Recommendation

On the basis of the results presented, it is recommended that the design specified values of concrete strength be used in any response history analysis of the CTV building for forensic purposes. As part of any such analysis, the sensitivity of the results to key input parameters should be considered. Therefore, it is recommended that the concrete strengths used in the analysis be considered at their mean, 16^{th} and 84^{th} percentile values (16^{th} and 84^{th} percentiles corresponding to ± 1 standard deviation). Table 3 provides specific values for each of the three different strength specifications in the CTV building.

Table 3: Concrete strengths suggested for use in sensitivity studies when performing response history analysis of the CTV building

Sensitivity Case	Specified Strength (28 days)		
	25 (MPa)	30 (MPa)	35 (MPa)
Mean	30.5	36.5	42.5
16th percentile[1]	25.8	31.3	37.0
84th percentile[1]	35.2	41.7	48.0

[1] 16th and 84th percentiles correspond to the mean ±1 standard deviation.

8. References

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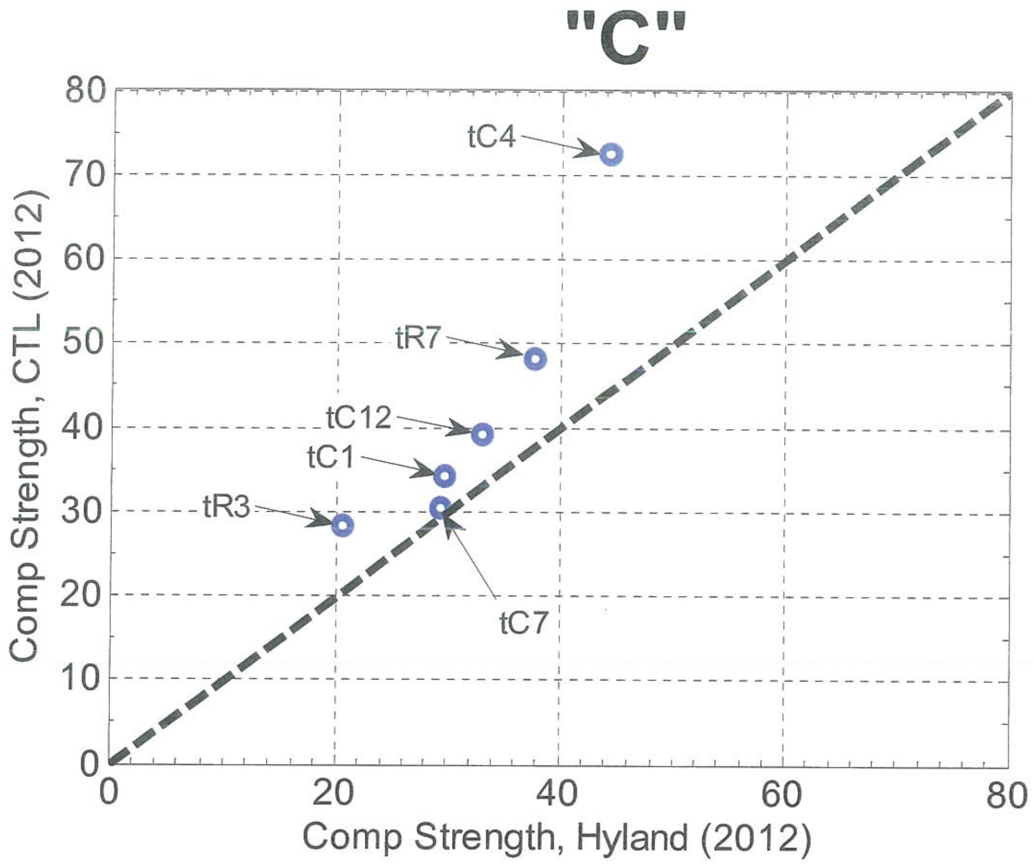
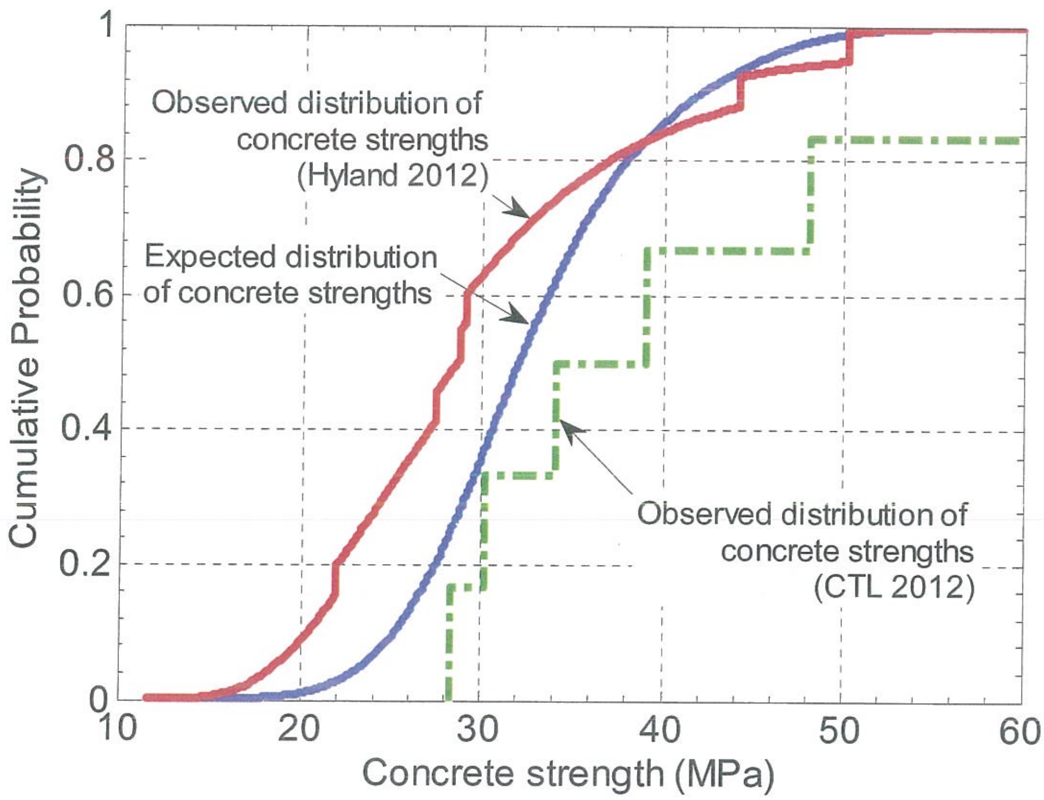

"C"
tC4
tR7
tC12
tC1
tR3
tC7
Comp Strength, CTL (2012)
Comp Strength, Hyland (2012)
0
10
20
30
40
50
60
70
80


Observed distribution of concrete strengths (Hyland 2012)
Expected distribution of concrete strengths
Observed distribution of concrete strengths (CTL 2012)
Cumulative Probability
Concrete strength (MPa)
0
0.2
0.4
0.6
0.8
1
10
20
30
40
50
60

"D"

Ground Motion Aspects of the 22 February 2011 Christchurch Earthquake Related to the Canterbury Television (CTV) Building

Technical Report Prepared for the Canterbury Earthquakes Royal Commission

Brendon A. Bradley[1,2]

[1]Director, Bradley Seismic Limited

[2]Lecturer, Department of Civil and Natural Resources Engineering, University of Canterbury

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Christchurch, New Zealand

Disclaimer:

This report has been prepared for the Canterbury Earthquakes Royal Commission to provide information on ground motion aspects related to the Canterbury Television (CTV) building. The authors accept no liability for the use of the material contained in this document by third parties.

Table of Contents

Executive Summary

This report provides an examination of ground motion aspects related to the Canterbury Television (CTV) Building. Specifically, information on observed ground motions in the vicinity of the CTV site is examined, and based on the characteristics of the locations at which these ground motions were observed, their appropriateness for representing the unknown ground motions at the CTV site during the 4 September 2010 and 22 February 2011 earthquakes is presented.

Comparisons of ground motions at the CTV site obtained during April and May 2012 from specifically deployed instrumentation, as well as comparisons with the empirically-derived conditional response spectrum distribution, illustrate that the ground motion time series obtained at four strong motion stations near the CTV site (i.e. CCCC, CHHC, CBGS, REHS) during the 4 September 2010 and 22 February 2011 earthquakes are appropriate for use in nonlinear seismic response history analysis of the CTV building in lieu of the unknown ground motion which actually occurred. The ground motions observed at the Christchurch Police Station, Westpac building, and Pages Road Pumping Station (PRPC) are not considered appropriate.

It is recommended that the ground motion time series obtained from a given location (i.e. CCCC, CHHC, CBGS, REHS) in both the 4 September 2010 and 22 February 2011 earthquakes be utilized in the same nonlinear seismic response analysis scenario. Hence, with four strong motion stations this will result in a total of four different input ground motion combinations to be considered.

Because of the intensity of ground motion shaking in all three orthogonal directions, all three components of ground motion should be considered simultaneously in nonlinear seismic response history analyses. Furthermore, in order to adequately account for such effects, the constitutive models for critical elements should explicitly consider the influence of combined actions (that is, bi-axial moment, bi-axial shear, and axial load).

1. Introduction

This report provides an examination of ground motion aspects related to the Canterbury Television (CTV) building during the 22 February 2011 Christchurch earthquake. Only those aspects related to the CTV building are examined, while general ground motion aspects in this event can be found in Bradley (2012b) and references therein.

Earthquake-induced ground motions are generally referred to as being affected by source-, path- and site-effects. That is, the ground motion at a specific location is affected by the characteristics of the earthquake source (i.e. the time evolution of slip on the causative fault); the path of the emitted seismic waves through the earth's crust (including the reflection and refraction of waves, and their superposition), and the influence of (nonlinear) seismic response of surficial soils. At a qualitative level, these effects on surface ground motions are well understood, however an accurate quantification of these effects on a case-specific basis remains elusive.

Instrumental records of observed strong motion are the most precise means to estimate surface ground motions, and coupled with nonlinear response history analyses, their impact on engineered structures. This report provides information on observed ground motions in the vicinity of the CTV site, and based on the characteristics of the locations at which these ground motions were observed, their appropriateness for representing the unknown ground motions at the CTV site during the 4 September 2010 and 22 February 2011 earthquakes.

Section 2 outlines the strong motion instruments in the vicinity of the CTV site and the time series of acceleration that were recorded at these locations in the 4 September 2010 and 22 February 2011 earthquakes. Section 3 briefly discusses the local geotechnical conditions inferred at the CTV site. Section 4 examines ground motions observed directly at the CTV site from recently deployed instrumentation relative to the ground motions at strong motion stations in the vicinity. Section 5 examines the empirically-derived conditional distribution of response spectra and hence the representativeness of the observed ground motions at nearby sites for inferring the unknown ground motion at the CTV site resulting from the 4 September 2010 and 22 February 2011 earthquakes. Sections 6, 7 and 8 summarise the analyses conducted and the representativeness of the observed ground motions for the CTV site.

2. Strong ground motion instrument recordings in proximity to the CTV site

2.1. Strong motion station locations

While no instrumental records of the ground motion at the CTV site were obtained from the 4 September 2010 Darfield and 22 February 2011 Christchurch earthquakes, numerous ground motions were observed in the general vicinity of the site. The four closest strong motion stations which are part of the permanent GeoNet instrumentation network are shown in Figure 1. The approximate distances of these stations to the CTV site are 720m (CCCC), 1300m (REHS), 1250m (CHHC), and 1850m (CBGS).

Figure 1: Location of the four nearest strong motion stations to the CTV site that are part of the permanent GeoNet instrumentation program. CCCC = Christchurch Cathedral College; REHS =Christchurch Resthaven; CHHC = Christchurch Hospital; and CBGS = Christchurch Botanical Gardens.

2.2. Spatial variability of observed ground motions

While the four aforementioned GeoNet stations, and the CTV site, appear to be in close proximity to one another, strong ground motions, particularly at high frequencies, can vary significantly over such distances. Figure 2 illustrates the spatial distribution of the acceleration time series of ground motion observed from instrument recordings throughout Christchurch resulting from the 22 February 2011 Christchurch earthquake. In all cases it can be seen that the nature of the ground motion is complex and that even a visual inspection of the acceleration time series illustrates how significantly the amplitude, frequency content, and duration of the ground motion varies over short spatial distances.

Figure 2: Observed acceleration time series at various locations in the Christchurch region from the 22 February earthquake: (a) fault-normal horizontal; (b) fault-parallel horizontal; and (c) vertical components (the inferred causative fault plane is shown as a rectangle) (after Bradley and Cubrinovski (2011a)).

2.3. Response spectra observed from the 22 February 2011 Christchurch earthquake

Linear elastic response spectra provide a simple interpretation of the severity of a ground motion on engineered structures. Figure 3 illustrates the pseudo-acceleration response spectra obtained from the ground motions at the four "CBD" strong motion stations as compared to the 'elastic' response spectrum (using S=5 for reinforced concrete) prescribed by the standard to which the CTV building was designed against seismic hazards (NZS 4203:1984). It is clear that the ground motion at all of the four "CBD" strong motion stations was well in excess of the 'elastic' design spectrum. What is also important, for the purposes of this report, is the variability in the response spectra of the ground motions observed at these four locations. It can be seen that the variability is a function of the vibration period considered, with very little difference in the response spectra for $T > 2s$, but differences in the order of a factor of 2 for $0.5 < T < 1.5s$. This large variability is significant, given that the period of the CTV structure is estimated to be on the order of $T = 1s$ and given that the exact ground motion at the CTV site is unknown. The vibration period dependence of this variability in ground motions is physically understood, and occurs because of the fact that long period ground motion has significantly longer wavelengths than short period ground motion. Short period ground motion, which have very short wavelengths (on the order of 20m for a wave with vibration period of 0.1s travelling through soft soil with $V_s = 200$m/s), is significantly modified (reflected, refracted, amplified) by the local geotechnical characteristics of the site. Furthermore, because of the short wavelengths, the incident high-frequency ground motion below each site will also be different (i.e. even before the ground motion is modified via propagation through the local geotechnical conditions at each site). In contrast, long period ground motion is generally not significantly affected by near surface geotechnical characteristics, and also the longer wavelengths mean the incident ground motion will be similar below each site.

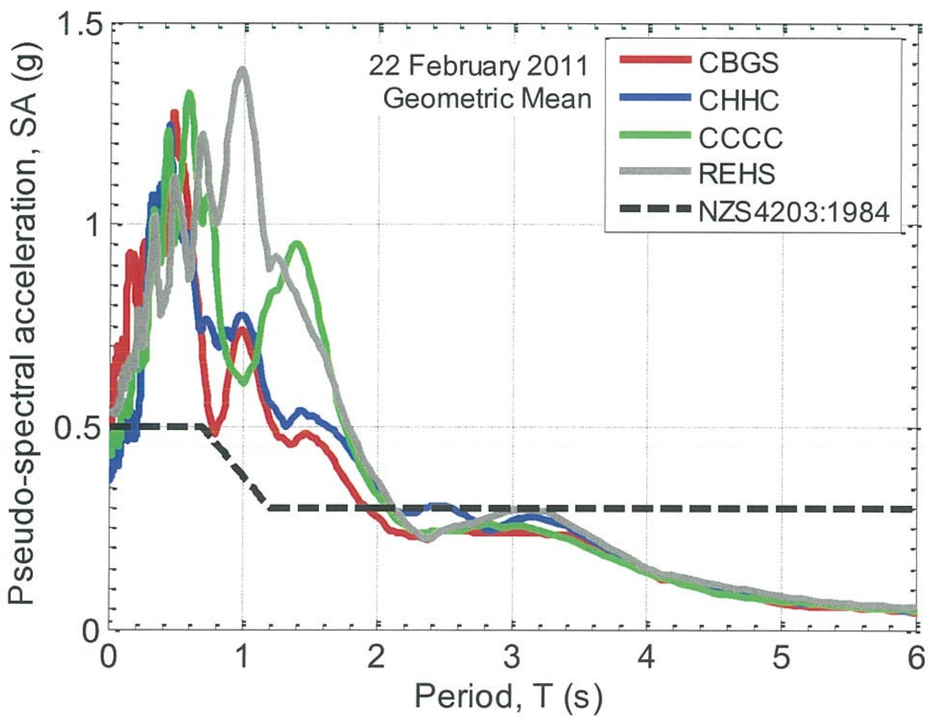

Figure 3: Pseudo-acceleration response spectra of ground motions observed during the 22 February 2011 Christchurch earthquake. The response spectra are for the geometric mean horizontal component (see Bradley (2012b) for discussion of alternative representations).

Figure 3 presented the elastic response spectra in terms of the pseudo acceleration response. The pseudo-acceleration is that conventionally used in force-based seismic design, since when multiplied by the seismic mass it provides the force required to be resisted. Often the peak displacement is also of interest. Because the peak displacement and pseudo-acceleration are directly related it is possible to plot them on the same figure, in what is known as an "acceleration displacement response spectrum" (ADRS) as given in Figure 4 for the 22 February 2011 Christchurch earthquake. In an ADRS plot, lines of constant vibration period are radial lines from the origin, as shown for vibration periods of $T = 0.5, 1.0, 1.5, 2.0$, and 3.0s. As such plots can be used with a nonlinear 'pushover' curve of the structure to approximately assess seismic performance, ADRS plots are provided in Appendix A for damping ratios of 5, 10, 15, and 20%.

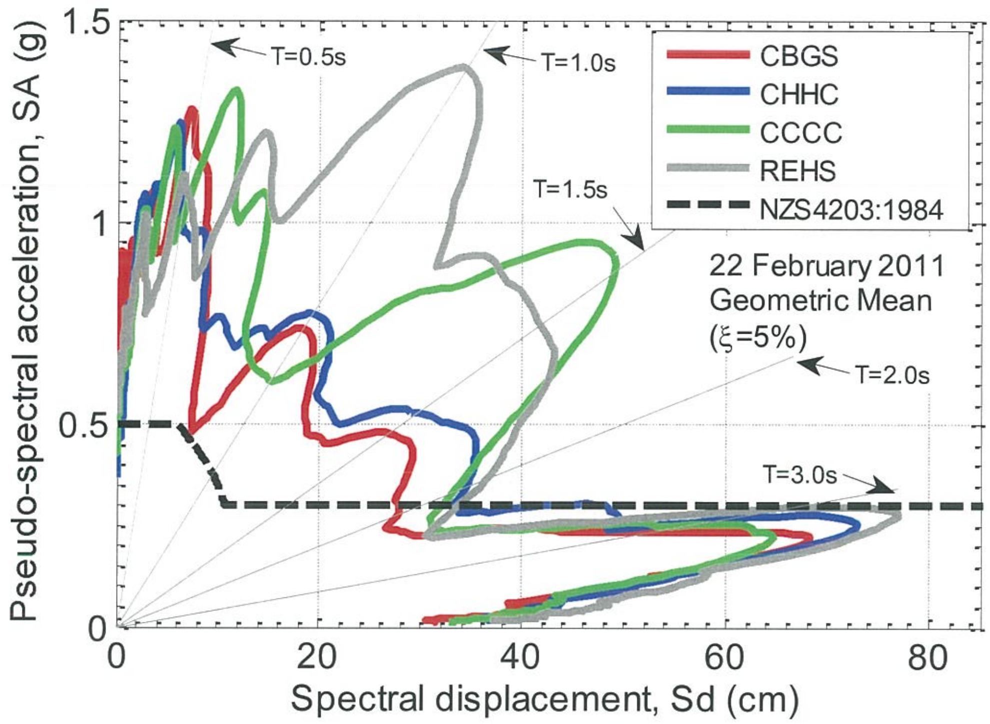

Figure 4: Acceleration displacement response spectra ($\xi = 5\%$) of ground motions observed during the 22 February 2011 Christchurch earthquake.

2.4. Tri-directional ground motion aspects of the 22 February 2011 Christchurch earthquake

As noted in Figure 3 and Figure 4, the geometric mean horizontal ground motion recorded at strong motion stations in the vicinity of the CTV site was notably above the ground motion intensity for which the structure was designed. In addition, the vertical ground motions in the vicinity of the CTV site were also significant (i.e. Figure 2c). Figure 5 illustrates the response spectra of vertical ground motions at the aforementioned four stations in proximity to the CTV site. It can be seen that the peak vertical ground motion acceleration was typically in excess of 0.5g, with spectral accelerations at short vibration periods significantly exceeding 1.0g.

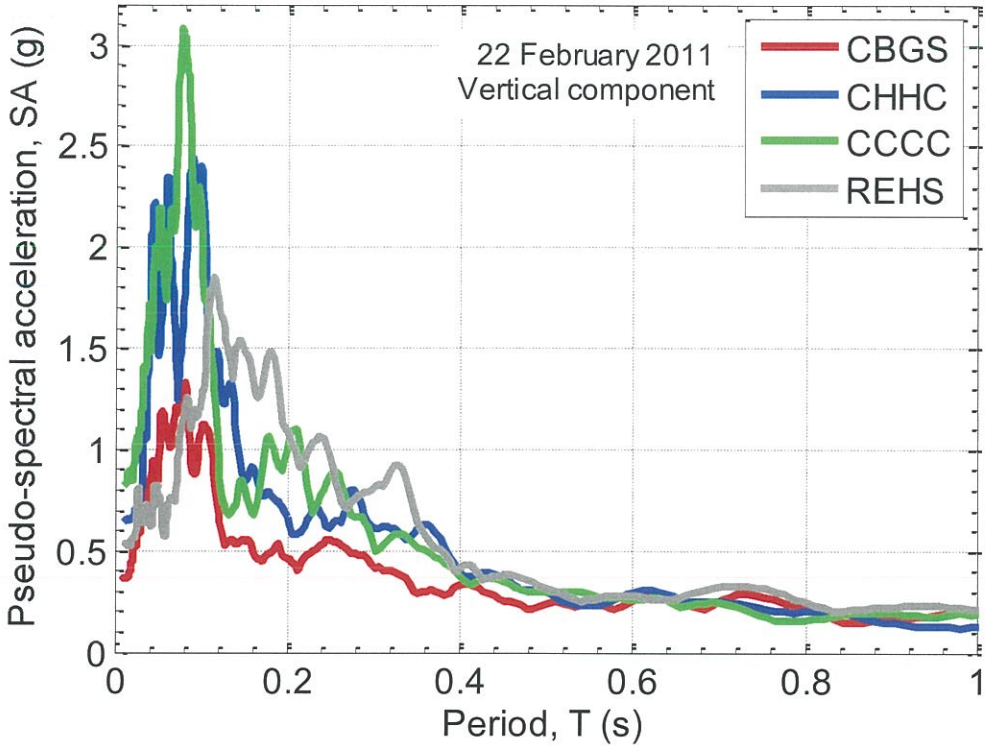

Figure 5: Response spectra of vertical ground motions observed during the 22 February 2011 Christchurch earthquake.

While ground motions are certainly not coherent in three-directions simultaneously, the concurrent occurrence of intense ground motion in multiple directions can place significantly greater seismic demands on structures than consideration of each ground motion component independently. Figure 6 illustrates a trajectory of ground motion velocity in the horizontal plane (i.e. that which would be observed from a 'bird's-eye' view of the ground). The velocity time series has been considered, as it is known to be well correlated with ground motion intensity at moderate vibration periods (Bradley 2012a), which approximately encompasses the inferred fundamental periods of the CTV building in each plan direction. It can be seen that the peak ground velocity of nearly 60cm/s occurs almost simultaneously in both the NS and EW directions. Of more importance to a torsionally-sensitive structure, such as the CTV building, is the large, clover-shaped, velocity peaks in the second and third quadrants of the velocity trajectory. Similar to Figure 6, Figure 7 illustrates the trajectory of velocity in a plane which would be seen from an observer facing the north-south direction (i.e. as if one was standing on Cashel Street looking north to the CTV building). It can be seen that at the time of the aforementioned peak acceleration in the horizontal plane, there is also vertical ground motion velocity on the order of 15cm/s.

In order to proceed from the qualitative discussion of the importance of tri-directional ground motion on seismic response discussed in the last paragraph, nonlinear seismic response history analysis is needed (hence why only examples of tri-directional ground motion were illustrated). This is because ultimately the importance of tri-directional ground motion dependents on the concurrent occurrence of large responses of the structures elements under bi-axial moment, bi-axial shear, and axial loading. In addition to considering three directions of ground motion in nonlinear seismic response history analyses, it is obviously necessary to have element constitutive models which can account for the combined effect of bi-axial moment, shear and axial load. This is not a trivial task, given the state of knowledge of these combined actions coupled with the availability of models in commercial software. Needless to say, if adopted constitutive models for critical elements (i.e. beam-column joints and columns) do not have the capability to consider combined actions, then the significance of tri-directional ground motion will be under-estimated.

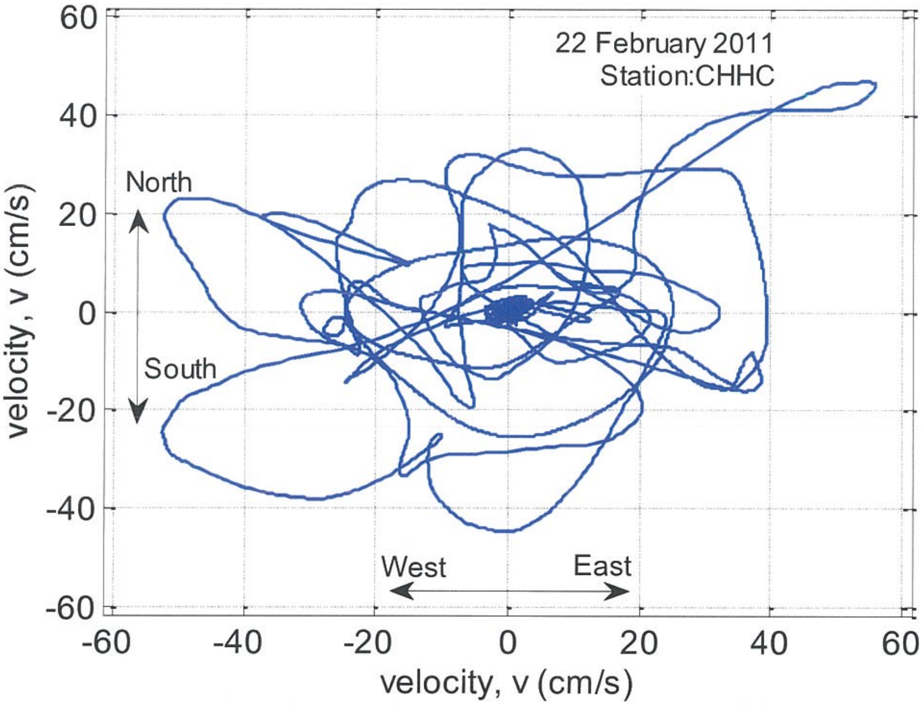

Figure 6: Trajectory of velocity in the horizontal plane at the CCCC station during the 22 February 2011 earthquake.

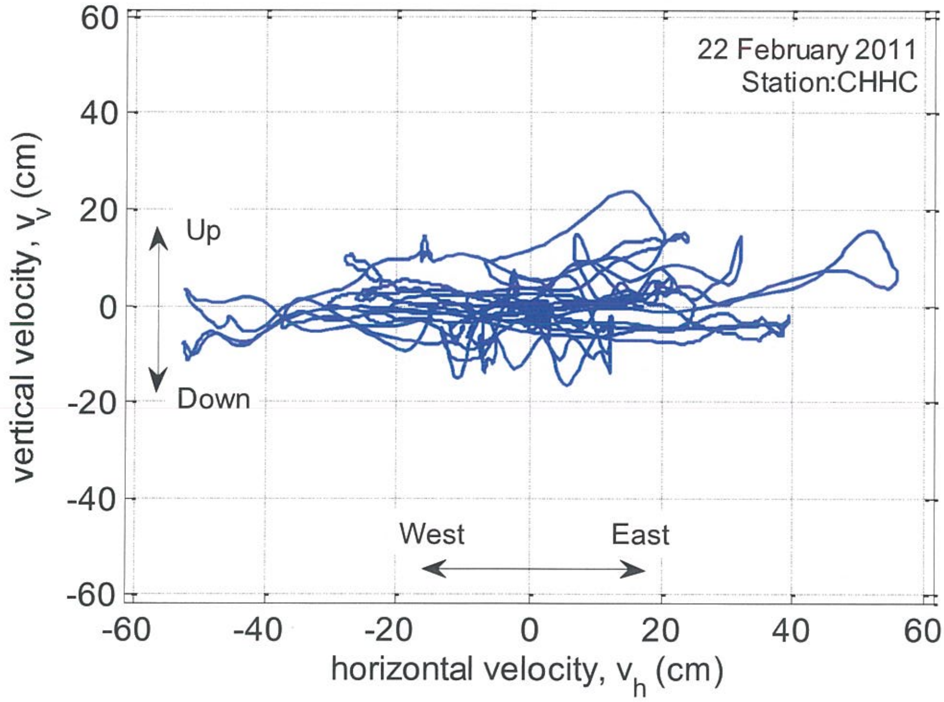

Figure 7: Trajectory of velocity in the plane comprizing the vertical and East-West directions at the CCCC station during the 22 February 2011 earthquake.

2.5. Response spectra observed from the 4 September 2010 Darfield earthquake

Since cumulative effects from ground shaking prior to the 22 February 2011 Christchurch may be significant in the collapse of the CTV building, the pseudo-acceleration response spectra and ADRS are shown for the 4 September 2011 earthquake in Figure 8 and Figure 9, respectively. It can be seen that the ground motion intensity is approximately equal to the 'elastic' design response spectrum near the inferred fundamental period of the CTV structure ($T \sim 1s$), and hence that given the structure was designed to be ductile, that some level of nonlinear response would be expected from the ground motion experienced from the 4 September 2010 earthquake.

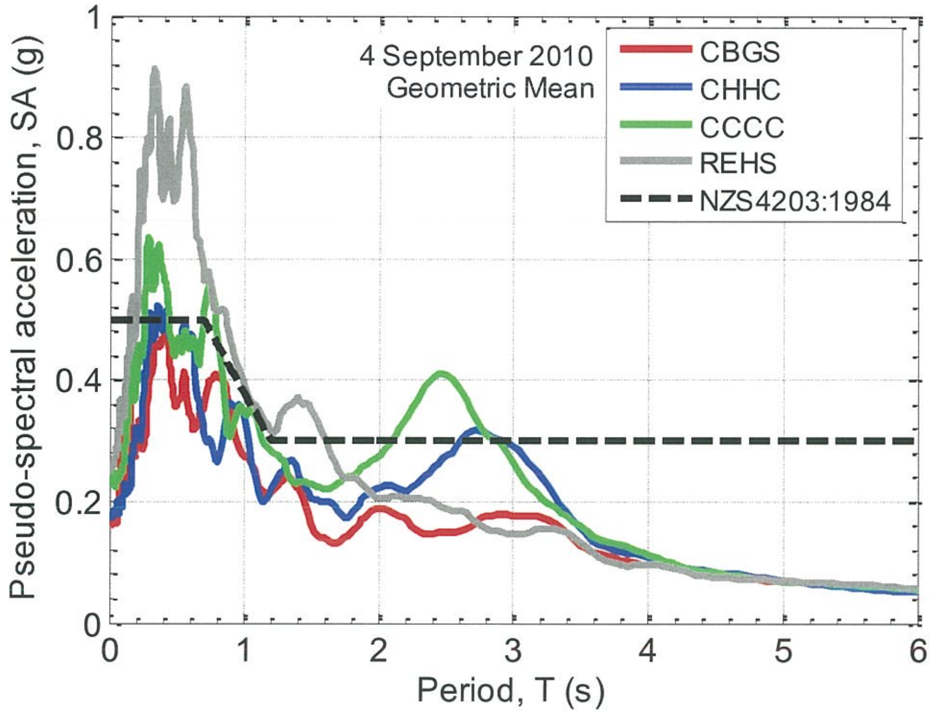

Figure 8: Pseudo-acceleration response spectra of ground motions observed during the 4 September 2010 Darfield earthquake.

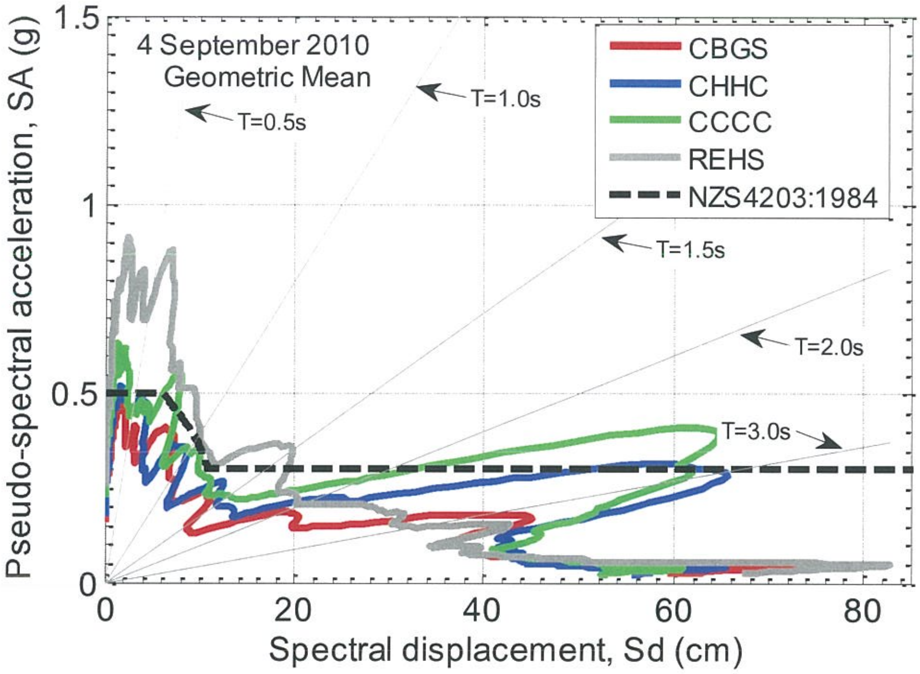

Figure 9: Acceleration displacement response spectra of ground motions observed during the 4 September 2010 Darfield earthquake.

2.6. Additional strong motion stations in the vicinity of the CTV site

In addition to the aforementioned four strong motion stations (i.e. CCCC, REHS, CHHC, CBGS), temporary strong motion instruments have also been installed in the Christchurch Police Station and Westpac Building (Sinclair 2012, Webb 2012). These two additional strong motion instruments are located in the basements of multi-storey office buildings. As a result, the ground motions recorded in these buildings are influenced by the kinematic interaction of the multi-storey structure with the ground (so-called "soil-structure-interaction"), and therefore do not represent "free-field" ground motions (which the CCCC, REHS, CHHC, and CBGS can be assumed to represent). The Christchurch Police Station and Westpac Building instrument recordings are therefore considered inappropriate for inferring the nature of the ground motion shaking at the CTV site, as will be briefly illustrated subsequently.

Sinclair (2012) also commented on the applicability of the strong motion records observed from the Pages Road Pumping Station (PRPC). While Sinclair (2012) notes that the PRPC instrument is located at a similar focal distance from the causative fault of the 22 February 2011 earthquake, the azimuth direction is significantly different, implying that seismic waves travel a significantly different path from the source to the site. Because of this fact, combined with the significantly different local geotechnical conditions at the PRPC site (Sinclair 2012), the ground motion at PRPC during the 22 February 2011 earthquake was notably different than the aforementioned four instruments in the Christchurch Central Business District (CBD) (Bradley and Cubrinovski 2011a, Bradley and Cubrinovski 2011b). As a result, the recorded PRPC ground motion during the 22 February 2011 earthquake is, in the author's opinion, not representative of that which would have occurred at the CTV site.

3. Local geotechnical characteristics at the CTV site

The CTV site has had relatively little site investigation to estimate its geotechnical and geophysical properties. A summary of the geotechnical investigation; comprising 8 hand-augered boreholes (3-4m depth), three machine-augered boreholes, and two cable-tool boreholes, performed when the building was initially designed; is given by Sinclair (2012). It is to be noted that such geotechnical investigations illustrated a large variability of ground conditions across the site, and while the interpretation by the design geotechnical engineer was 'reasonable', alternative interpretations are possible (Sinclair 2012).

Geophysical information on the CTV site can also be inferred based on a Multi-channel Analysis of Surface Waves (MASW) profile which was performed along Madras Street beside the CTV site. The interpreted results of this site are shown in Figure 10. It is important to note the 'lenses' of high velocity material at approximately 9m depth at the north end of the footprint of the CTV site. This high velocity material may be inferred as gravels, as suggested by Sinclair (2012), however it should be noted that this is inconsistent with the interpretation of the site conditions based on the 1986 hand-auguered boreholes, which suggested that the North-east corner of the site was particularly soft (i.e. Figure 1 of Sinclair 2012). From the MASW results, the average shear wave velocity of the top 30m of the site can be estimated as V_S =330m/s, which corresponds to site class D according to the current seismic loadings standard, NZS1170.5 (2004). It should be noted that the methodology used to develop Figure 10 (e.g. not 'stacking' the recordings, very close spacing of receivers relative to sources, and a large number of assumed layers leading to lack of constraint in the Vs inversion) means that the above value of 330m/s is indicative only and, in the authors opinion, should not be used in a quantitative sense. In particular, the lack of constraint in the inversion may mean that imaged high velocity 'lenses' at approximately 9m depth may not exist physically.

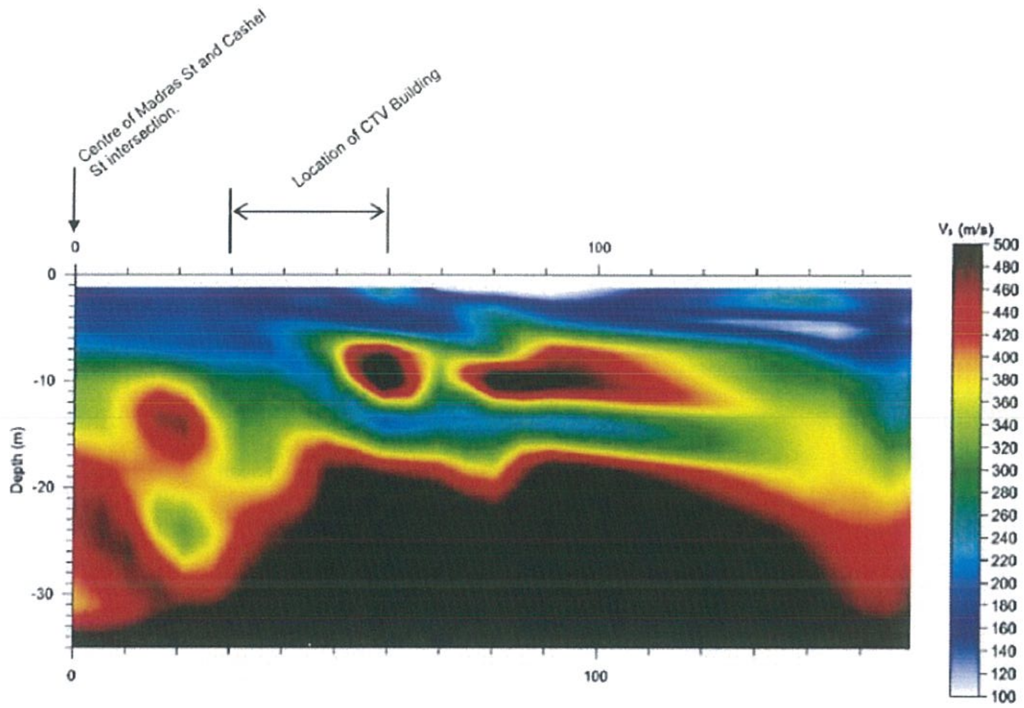

Figure 10: Inferred shear wave velocity of local soils along Madras Street in the vicinity of the CTV site (after Sinclair (2012)).

4. Ground motions observed from instrumentation at the CTV site

A strong ground motion instrument was deployed at the CTV site in March 2012. While no ground motions were recorded at the CTV site from the 22 February 2011 Christchurch earthquake, ground motions observed at the site since March 2012 can be compared with those concurrently observed at the nearby GeoNet strong motion stations, in order to investigate any peculiarities in the ground motions at the CTV site.

In examining the earthquake events which have occurred since March 2012, only those events which were significant were considered and are presented here. In particular, the ground motions for events with magnitude less than 4.0 were not considered, because (i) constraints on their location are weaker than larger events, and (ii) the amplitude of the corresponding ground motion will be small relative to the background noise at the site (i.e. a low signal to noise ratio).

Figure 11 provides a comparison between the response spectrum of ground motion observed at the CTV site with those of the aforementioned nearby strong motion stations in three earthquakes of magnitudes 4.62, 4.8, and 5.2, respectively, all located offshore to the east of New Brighton. Several points are worthy of note in the examination of Figure 11. Firstly, because of the large source-to-site distance of the events relative to the distance of the CTV site to the nearby strong motion instruments, then no correction to the amplitude of the response spectra was made to account for geometric spreading. Secondly, in all events it can be seen that the amplitude and 'shape' of the response spectrum at the CTV site is consistent with those at the other 4 CBD stations (i.e. CCCC, CHHC, REHS, CBGS). Comparison of Figure 11 for different events again illustrates the event-to-event variability in the response spectra, and thus this figure serves merely to illustrate that the site response at the CTV site cannot be rejected as being different than the site response at these other four stations.

Finally, in both the first two events (i.e. Figure 11a and Figure 11b) it can be seen that the ground motion recorded at the Christchurch Police Station basement is lower than those at other 'free-field' stations, emphasising its inappropriateness for representing the ground motion at the CTV site, as it is not a "free-field" site.

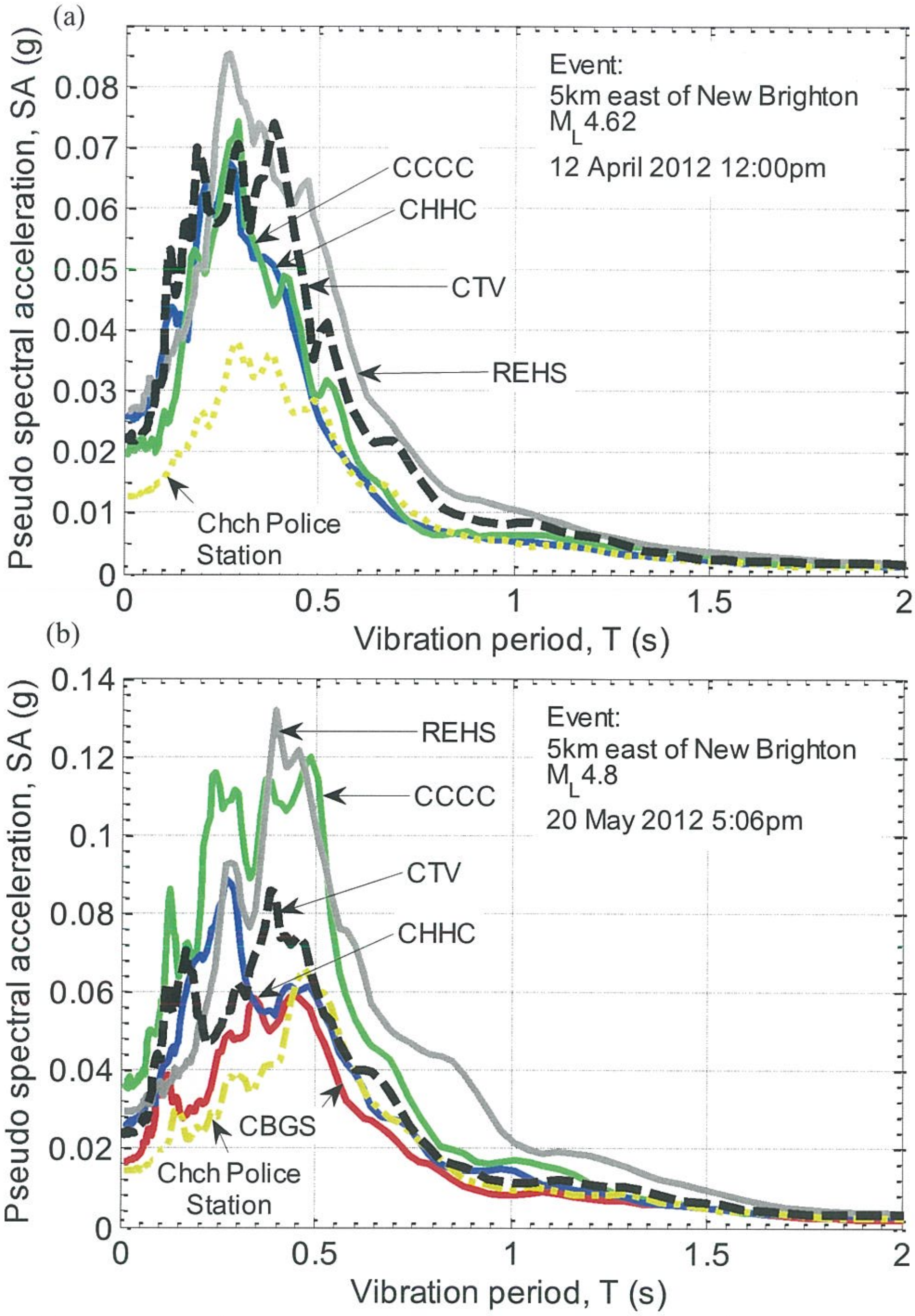

Figure 11: Comparison of pseudo-acceleration response spectra observed at the CTV site during three events since March 2012: (a) a magnitude 4.62 event east of New Brighton; (b) a magnitude 4.8 event east of New Brighton; and (c) a magnitude 5.2 event east of New Brighton.

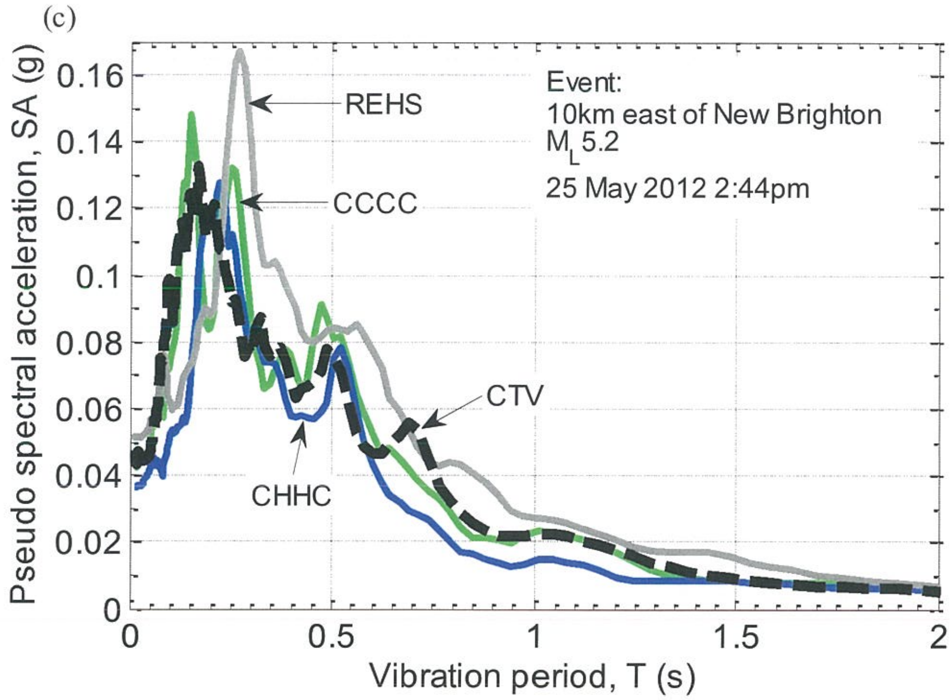

Figure 11 cont.

5. Conditional response spectrum predicted during the 22 February 2011 earthquake

While the ground motion at the CTV site was not directly recorded during the 22 February 2011 Christchurch earthquake, it is possible to make use of empirical ground motion prediction equations, and observations at nearby sites to infer the distribution of the ground motion response spectrum at the CTV site. Appendix B of this report provides the theoretical details related to the computation of the conditional response spectrum at the CTV site. In the results presented below, the NZ-specific ground motion prediction equation (GMPE) of Bradley (2010), and the spatial correlation model of Goda and Hong (2008) were adopted. The applicability of the Bradley (2010) GMPE for the Canterbury earthquakes was explicitly demonstrated in Bradley (2012b), while the author also found from preliminary analyses that the Goda and Hong (2008) relationship is applicable.

Figure 12 provides a comparison between the unconditional and conditional distributions of the ground motion response spectrum at the CTV site from the 22 February 2011 Christchurch earthquake. The unconditional response spectrum distribution is simply that which would be predicted by a ground motion model by knowing the location of the causative fault relative to the CTV site, the event magnitude, and the CTV site classification (assumed D as noted in the previous sections). In contrast, the conditional response spectrum distribution is obtained by utilizing: (i) the unconditional distribution; (ii) the observed ground motion at strong motion stations (specifically the difference between the observed and predicted response spectra at these locations); and (iii) the relative distance between the strong motion stations and the CTV site. It can be seen that the median of the conditional response spectrum is generally greater than the median of the unconditional response spectrum (the exception being for $T < 0.3s$), as a result of the fact that the ground motions in the vicinity of the CTV site (e.g. CCCC, REHS, CHHC, CBGS), on average, had ground motion amplitudes slightly larger than that predicted by the Bradley (2010) GMPE. Close comparison of the unconditional and conditional distributions also illustrates that there is a reduced standard deviation (i.e. a reduced difference between 16th, 50th (median), and 84th percentiles) in the conditional distribution because of the additional information that has been utilized from the observed ground motions. This reduction in standard deviation is greatest at long vibration periods, since as previously noted, long period seismic waves have greater wavelengths and are therefore more coherent over a spatial region.

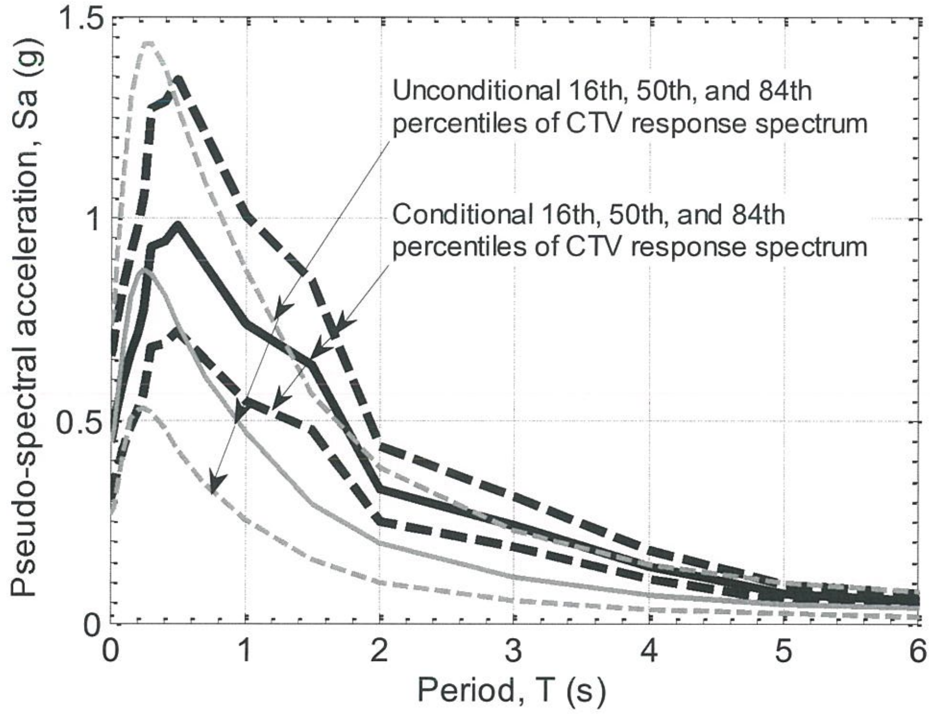

Figure 12: Comparison of unconditional and conditional response spectra predicted at the CTV site from the 22 February 2011 Christchurch earthquake.

Figure 13 provides a comparison between the conditional response spectrum distribution predicted for the CTV site using the aforementioned methodology, and the individual response spectra of the ground motions at the four stations in the vicinity of the CTV site. It can be seen, generally speaking, that these four ground motions are consistent with the conditional distribution. It is worth noting that at the inferred fundamental period of the CTV building ($T{\sim}1s$), the CBGS, CCCC and CHHC ground motions have response spectrum amplitudes which are below the median of the conditional distribution, and only the REHS ground motion is above the median.

Since cumulative effects from ground shaking prior to the 22 February 2011 Christchurch maybe significant in the collapse of the CTV building, ground motion time series from the 4 September 2010 earthquake are also needed for the purpose of nonlinear seismic response analysis. Figure 14 provides a comparison between the conditional response spectrum distribution predicted for the CTV site, and the individual response spectra of the ground motions at the four stations in the vicinity of the CTV site, all corresponding to the 4 September 2010 Darfield earthquake. It can be seen that these ground motions are consistent with the conditional response spectra, and are therefore considered appropriate for use in seismic response analysis.

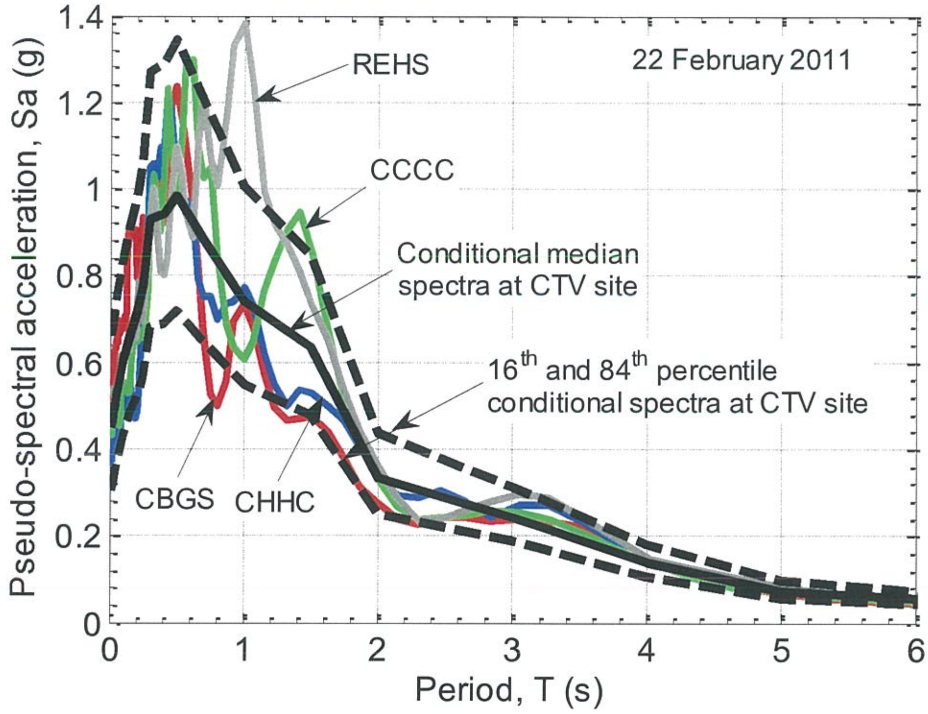

Figure 13: Comparison of the ground motions observed at the four 'CBD' strong motion stations (i.e. CCCC, CHHC, REHS, CBGS) with the conditional response spectrum distribution at the CTV site for the 22 February 2011 Christchurch earthquake.

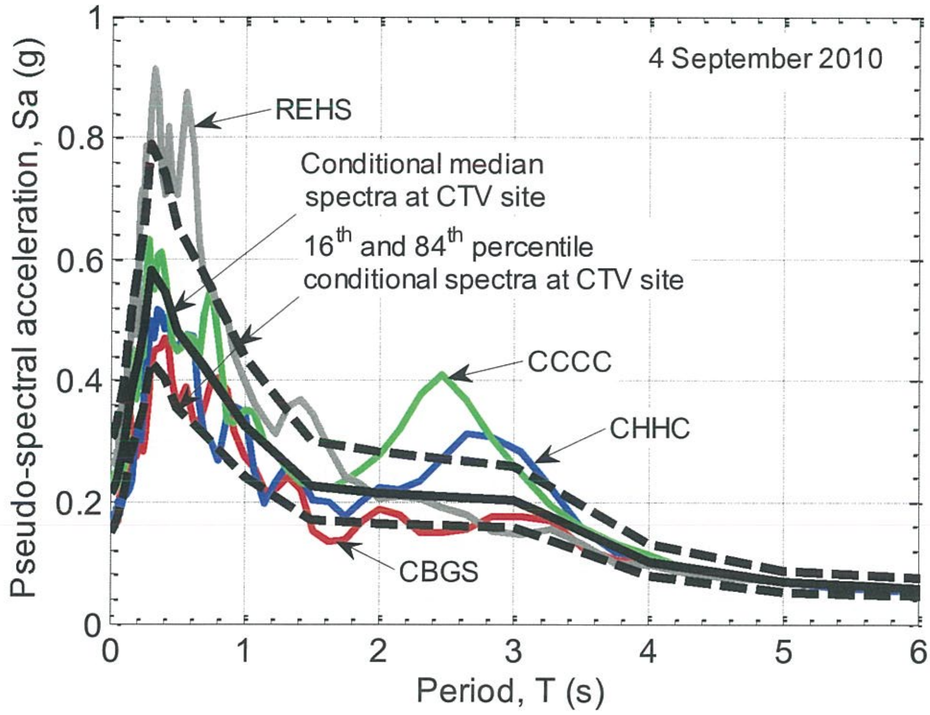

Figure 14: Comparison of the ground motions observed at the four 'CBD' strong motion stations (i.e. CCCC, CHHC, REHS, CBGS) with the conditional response spectrum distribution at the CTV site for the 4 September 2010 Darfield earthquake.

6. Representativeness of observed ground motions for nonlinear seismic response history analysis of the CTV building

The previous results have illustrated that the ground motion time series recorded at four strong motion stations in the vicinity of the CTV site (i.e. CCCC, CHHC, CBGS, REHS) can be considered applicable for use in nonlinear seismic response history analysis of the CTV building.

At present, the CCCC, CHHC, and CBGS ground motion time series have been utilized in analyses of the CTV building (Hyland and Smith 2012), however the REHS ground motion was not considered. The omission of the REHS ground motion is presumably based on the recommendation of Sinclair (2012), who noted that the REHS site contains a significant thickness of "very soft organic silt and very soft peat", consistent with other boreholes in the vicinity. Based on the limited site investigations performed in 1986 it is inferred that such soils are not present at the CTV site.

While the author does not disagree with the inferred minor difference in near surface soil conditions at the REHS strong motion station with those at the CCCC, CBGS, and CHHC sites; given the limited amount of site investigation at the CTV site; the fact that strong ground motion is affected by not only site effects, but also source and path effects, and that the remaining three ground motions (i.e. CCCC, CHHC, CBGS) have low spectral amplitudes in the vicinity of the inferred vibration period of the CTV building; it is suggested that the response of the nonlinear seismic response history analysis model to the REHS ground motion also be examined.

The ground motions at the four stations near the CTV site during the 4 September 2010 are consistent with the conditional distribution of ground motion, and therefore all should be used in nonlinear seismic response history analysis. Hyland and Smith (2012) considered only a single ground motion record from the 4 September 2010 earthquake, yet three ground motion time series from the 22 February 2011 earthquake. Given the large variability in seismic response which is known to result from ground motion variability, it is, in the author's opinion, inappropriate to use this single motion for the 4 September 2010 earthquake. It is recommended that the ground motion time series obtained from a given location (i.e. CCCC, CHHC, CBGS, REHS) in both the 4 September 2010 and 22 February 2011 earthquakes are utilized in the same nonlinear seismic response analysis scenario. Hence, with four strong motion stations this will result in a total of four different input ground motion combinations to be considered.

7. Conclusions

Comparisons of ground motions at the CTV site obtained during April and May 2012 from specifically deployed instrumentation, as well as comparisons with an empirically derived conditional response spectrum distribution, illustrate that the ground motion time series observed at four strong motion stations near the CTV site (i.e. CCCC, CHHC, CBGS, REHS) during the 4 September 2010 and 22 February 2011 earthquakes are appropriate for use in nonlinear seismic response history analysis of the CTV building, in lieu of the unknown ground motion which actually occurred. The ground motions observed at the Christchurch Police Station, Westpac building, and Pages Road Pumping Station (PRPC) are not considered appropriate.

8. Recommendation

It is recommended that the ground motion time series obtained from a given location (i.e. CCCC, CHHC, CBGS, REHS) in both the 4 September 2010 and 22 February 2011 earthquakes be utilized in the same nonlinear seismic response analysis scenario. Hence, with four strong motion stations this will result in a total of four different input ground motion combinations to be considered.

Because of the intensity of ground motion shaking in all three orthogonal directions, all three components of ground motion should be considered simultaneously in nonlinear seismic response history analyses. Furthermore, in order to adequately account for such effects, the constitutive models for critical elements should explicitly consider the influence of combined actions (that is, bi-axial moment, bi-axial shear, and axial load).

9. References

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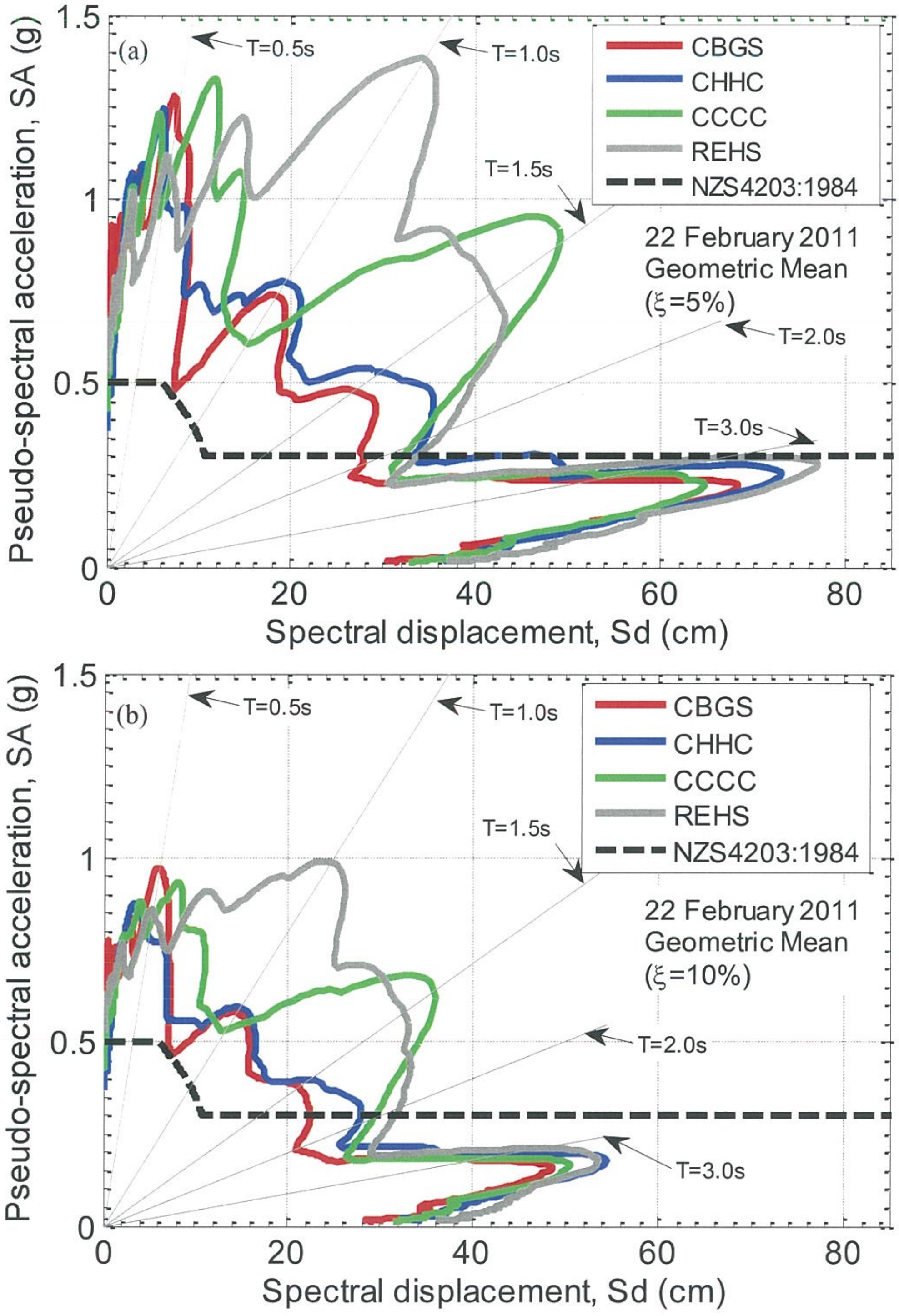
10. Appendix A: Acceleration-displacement response spectra (ADRS) for various damping ratios

This appendix contains ADRS plots of the ground motions recorded in the vicinity of the CBD site for different viscous damping ratios.



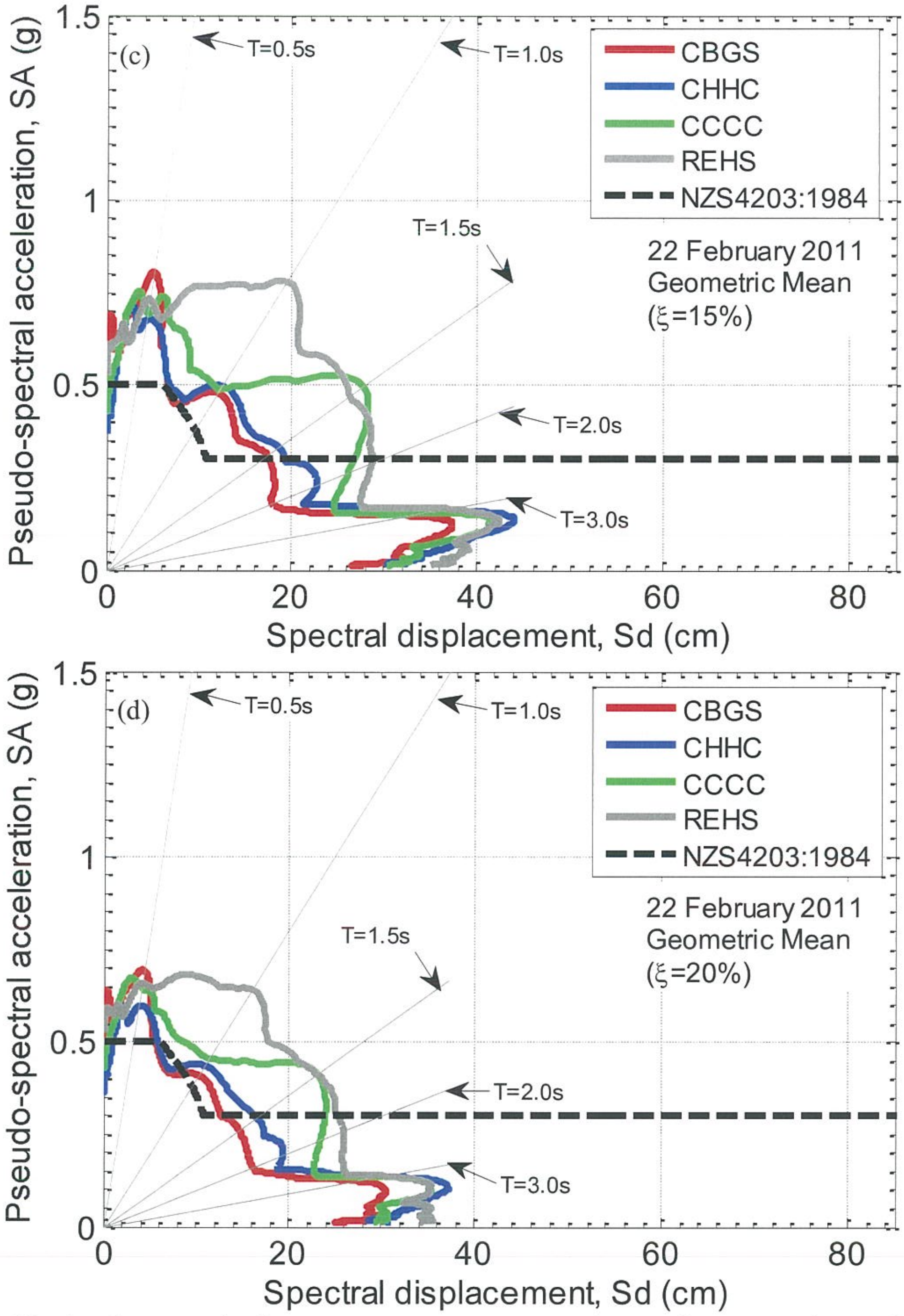


Figure 15: Acceleration displacement response spectra of ground motions observed during the 22 February 2011 Christchurch earthquake for viscous damping ratios of: (a) 5%; (b) 10%; (c) 15%; and (d) 20% of critical.

11. Appendix B: Conditional prediction of ground motion response spectra

Because of the complexity of a ground motion time series, the engineering representation of ground motion severity typically comprises one or more ground motion intensity measures. Here only the intensity measure of spectral acceleration (SA(T)) is considered, although the theory below is applicable to any other intensity measure.

The representation of SA(T) at a single location *i,* for the purposes of ground motion prediction, is generally given by:

$$lnSA(T)_i = ln\overline{SA(T)_\iota}(Site, Rup) + \eta + \epsilon_i \tag{1}$$

where $lnSA(T)_i$ is the (natural) logarithm of the observed SA(T); $ln\overline{SA(T)_\iota}(Site, Rup)$ is the median of the predicted logarithm of $SA(T)$ as given by an empirical ground motion prediction equation (GMPE), which is a function of the site and earthquake rupture considered; η is the inter-event residual; and ϵ_i is the intra-event residual. Based on equation (1), empirical ground motion prediction equations can provide the (unconditional) distribution of ground motion shaking as:

$$lnSA(T)_i \sim N(ln\overline{SA(T)_\iota}, \sigma_\eta^2 + \sigma_\epsilon^2) \tag{2}$$

where $X \sim N(\mu_X, \sigma_X^2)$ is short-hand notation for X having a normal distribution with mean μ_X and variance σ_X^2.

By definition, for a given ground motion intensity measure, (e.g. $SA(T)$) all observations from a single earthquake event have the same inter-event residual, η. In this regard, the inter-event residual represents the correlation between all observations from a single event, which may occur as a result of a unique effect occurring during the earthquake rupture, which subsequently affects the ground motion at all locations in a systematic manner. On the other hand, the intra-event residual, ϵ_i varies from site to site. In this regard the intra-event residual represents all other randomness which leads to a difference between the observed ground motion intensity, the predicted median ground motion intensity, and the systematic inter-event residual. While the intra-event residual varies from site to site, it is correlated spatially as a result of similarities of path and site effects between various locations.

Based on the aforementioned properties of η and ϵ_i, use can be made of recorded $SA(T)$ values at strong motion stations to compute a conditional distribution of $SA(T)$ at an arbitrary site of interest. The required steps are discussed below.

Firstly, an empirical ground motion prediction equation (GMPE) is used to compute the unconditional distribution of ground motion intensity at the strong motion stations where ground motions were recorded. A mixed-effects regression (Abrahamson and Youngs 1992, Pinheiro et al. 2008) can then be used to determine the inter-event residual, η, and the intra-event residuals, ϵ_i's, for each strong motion station.

Secondly, the covariance matrix of intra-event residuals is computed by accounting for the spatial correlation between all of the strong motion stations and the site of interest. The joint distribution of intra-event residuals at the site of interest and the considered strong motion stations can be represented by:

$$\begin{bmatrix} \epsilon^{site} \\ \boldsymbol{\epsilon}^{SMstation} \end{bmatrix} = N\left(\begin{bmatrix} 0 \\ \mathbf{0} \end{bmatrix}, \begin{bmatrix} \sigma^2_{\epsilon_{site}} & \boldsymbol{\Sigma}_{12} \\ \boldsymbol{\Sigma}_{21} & \boldsymbol{\Sigma}_{22} \end{bmatrix} \right) \tag{3}$$

where $\boldsymbol{X} \sim N(\boldsymbol{\mu_X}, \boldsymbol{\Sigma})$ is short-hand notation for $\boldsymbol{X}$ having a multivariate normal distribution with mean $\boldsymbol{\mu_X}$ and covariance matrix $\boldsymbol{\Sigma}$ (i.e. as before, but in vector form); and $\sigma^2_{\epsilon_{SMstation}}$ is the variance in the intra-event residual. In Equation (3) the covariance matrix has been expressed in a partitioned fashion to elucidate the subsequent computation of the conditional distribution of ϵ^{site}. The individual elements of the covariance matrix can be computed from:

$$\boldsymbol{\Sigma}(i,j) = \rho_{i,j}\sigma_{\epsilon_i}\sigma_{\epsilon_j} \tag{4}$$

where $\rho_{i,j}$ is the spatial correlation of intra-event residuals between the two locations i and j; and σ_{ϵ_i} and σ_{ϵ_j} are the standard deviations of the intra-event residual at locations i and j. Based on the joint distribution of intra-event residuals given by Equation (3) the conditional distribution of ϵ^{site} can be computed from (Johnson and Wichern 2007):

$$\begin{aligned} \left[\epsilon^{site} \middle| \boldsymbol{\epsilon}^{SMstation}\right] &= N\left(\boldsymbol{\Sigma}_{12} \cdot \boldsymbol{\Sigma}_{22}^{-1} \cdot \boldsymbol{\epsilon}^{SMstation}, \sigma^2_{\epsilon_{site}} - \boldsymbol{\Sigma}_{12} \cdot \boldsymbol{\Sigma}_{22}^{-1} \cdot \boldsymbol{\Sigma}_{21}\right) \\ &= N\left(\mu_{\epsilon^{site}|\boldsymbol{\epsilon}^{SMstation}}, \sigma^2_{\epsilon^{site}|\boldsymbol{\epsilon}^{SMstation}}\right) \end{aligned} \tag{5}$$

Thirdly, using the conditional distribution of the intra-event residual at the site of interest given by Equation (5) and substituting into Equation (2), the conditional distribution of peak ground acceleration at the site of interest, PGA_{site} can be computed from:

$$\begin{aligned} &\left[lnSA(T)_{site} | \boldsymbol{lnSA(T)_{SMstation}}\right] \\ &\qquad = N\left(\overline{lnSA(T)_{site}} + \eta + \mu_{\epsilon^{site}|\boldsymbol{\epsilon}^{SMstation}}, \sigma^2_{\epsilon^{site}|\boldsymbol{\epsilon}^{SMstation}}\right) \end{aligned} \tag{6}$$

That is, the conditional distribution of $SA(T)$ at a specific site is a lognormal random variable (i.e. the log of $SA(T)$ is a normal random variable) which is completely defined via the conditional median and conditional standard deviation.

It should be noted that in cases where the site of interest is located far from any strong motion stations the conditional distribution will be similar to the unconditional distribution, and for sites of interest located very close to a strong motion station the conditional distribution will approach the value observed at the strong motion station.