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 JOHN BARRIE MANDER

BUDDLE FINDLAY
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 JOHN BARRIE MANDER

1. My full name is John Barrie Mander. I am a New Zealand Citizen, and currently a Permanent Resident of the United States residing in College Station, Texas. I hold the position of the Zachry Professor of Design and Construction Integration 1, within the Zachry Department of Civil Engineering at Texas A&M University.

2. 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 of Conduct'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, although in the interests of transparency, I note:

   (a) From 2008 to 2010, I funded and was the Research Supervisor of Mr Christopher Urmson as well as the Major Advisor for his Master of Science Thesis at Texas A&M University. Following the completion of his Master’s Thesis in August 2010, Mr Urmson returned to New Zealand and joined ARCL in September 2010 as a Graduate Engineer. We have remained in contact as we continue to publish papers that were derived from his Master’s research. Mr Urmson was also previously employed at ARCL as an undergraduate intern during the summer of 2005 to 2006 and upon graduating and prior to his coming to Texas A&M University, he was employed by ARCL as a Graduate Engineer.

   (b) During the summer of 2005 to 2006, my son Mr Thomas Mander was an undergraduate intern at ARCL. Upon graduating, he undertook a Master of Science degree in Civil Engineering at Texas A&M University. I was not his thesis advisor, but we did work together on a common research project and have published that work with other faculty and students. My son now resides in San Antonio, Texas. He assisted me with some aspects of analysing the behaviour of the columns in the CTV Building and in particular carried out the computer programming.

   (c) During my own PhD studies that were conducted from 1979 to 1983, I was co-supervised by Dr M.J. Nigel Priestley who is a member of the

Expert Panel. Apart from sitting on a NZSEE committee together in the early 2000s, we have not worked together since 1983.

5. I do not consider that any of these relationships impact upon my impartiality or independence in this matter.

Qualifications and experience

6. I hold a New Zealand Certificate in Engineering (1976, Christchurch Technical Institute), a Bachelor of Engineering (Hons) (1979, University of Canterbury) and Ph.D in Civil Engineering (1984, University of Canterbury).

7. I am a Fellow of the Institution of Professional Engineers of New Zealand and a member of the New Zealand Society for Earthquake Engineering.

8. From 1973 to 1987 I was employed by the New Zealand Railways (**NZR**). NZR also fully funded my education for the NZCE, BE and PhD. I was initially employed as a Draughtsman in Christchurch (1973-78), then an Assistant Engineer in Christchurch and Wanganui (1979-85), and finally as the Deputy Group Manager of the Property Business Group in Wellington (1986). In 1987 I left New Zealand to take up a position as Visiting Assistant Professor at the State University of New York at Buffalo (**SUNY**). In 1988 I was appointed to a tenure track position in the Department of Civil Engineering as an Assistant Professor, and in 1995 was promoted with tenure to Associate Professor. I returned to New Zealand in July 2000 to take up a position as Professor and Chair of Structural Engineering in the Department of Civil Engineering at the University of Canterbury. In 2007 I returned to the United States and took up my current tenured position as the inaugural holder of a newly endowed professorship in the Zachry Department of Civil Engineering at Texas A&M University.

9. I have written or co-written over 100 peer reviewed journal papers, 40 peer reviewed research reports, 14 book chapters and 140 conference papers. Amongst several diverse topics, the publications have a strong focus on structural concrete and earthquake engineering.

10. I have been the advisor or co-advisor of 95 doctoral and masters graduate students including 19 PhD students, ten of whom are currently in faculty positions at the level of Associate or full Professor in various Universities around the world.

11. The research work on the behaviour of confined concrete conducted during my PhD studies is well-known, highly cited and widely used internationally, either directly or indirectly in codes and computer programs. My early work at SUNY was part of the first generation of research investigations on the seismic behaviour of non-seismically designed concrete structures (buildings and bridges) in zones of low to moderate seismicity (such as Christchurch). I was also the developer of the theory and implementation of the fragility functions for highway bridges used in HAZUS, a US-FEMA initiative that is widely used in the United States and elsewhere, for the risk assessment of the damage to the built environment caused by natural hazards. I have conducted numerous large-scale laboratory experiments on structural components and subassemblages. In New York State, I also performed field-testing of several large bridge structures under ambient and forced vibration motion.

12. My full resume is **annexed** to this Statement.

Instructions and report

13. I have been instructed by Buddle Findlay, on behalf of ARCL, to provide independent expert evidence 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 undertake the following matters:

    (a) Review the key findings of the report of the Department of Building and Housing, as reported by Dr Hyland and Mr Smith in January 2012;

    (b) Present, analyse and discuss the results of new investigations commissioned by ARCL into ground motions, concrete tests and column tests;

    (c) Perform an analysis of the column performance; and

    (d) Present and discuss any other relevant issues, including in particular alternative collapse hypotheses.

14. I have prepared a submission which details my findings and opinions on these issues. My submission is **annexed**.

15. Also **annexed** are:

    (a) the conference paper referred to in section 2.6 of my submission (Mander, J.B., and Huang, Y., (2012) “Damage, Death and Downtime

Risk Attenuation in the 2011 Christchurch Earthquake," Proceedings of the New Zealand Society of Earthquake Engineering 2012 Conference (NZSEE 2012), Christchurch, NZ, 8-pp, Paper-016); and

(b) the presentation that accompanied the conference paper.

NTHA expert panel

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

Dated this tenth day of June 2012

JB Mander

J. B. Mander

Resume: Professor John B. Mander

Department of Civil Engineering
Texas A&M University
CE/TTI Building
3136 TAMU
College Station TX 77843

(979) 862 8078
Fax (979) 845 6554

e-mail: jmander@civil.tamu.edu

EDUCATION:

Ph.D.	1984	Civil Engineering, University of Canterbury, New Zealand Thesis: *Seismic Design of Bridge Piers* Advisors: Profs. M.J.N. Priestley and R. Park.
B.E. (Hons)	1979	Civil Engineering, University of Canterbury, New Zealand *Senior Scholar* (first class honours)
N.Z.C.E.	1976	Civil Engineering, Christchurch Polytechnic, New Zealand *Gold Medal prize*.

EMPLOYMENT HISTORY

2007-	Inaugural Zachry Professor in Design and Construction Integration I, Zachry Department of Civil Engineering, Texas A&M University
7/00-12/06	Professor and Chair of Structural Engineering, Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand
3/95-6/00	Associate Professor, Department of Civil of Structural and Environmental Engineering, State University of New York at Buffalo, NY 14260, USA.
1/89-2/95	Assistant Professor, Department of Civil Engineering, State University of New York at Buffalo.
1/88-12/88	Visiting Assistant Professor, Department of Civil Engineering, State University of New York at Buffalo.

1987	Deputy Group Manager and Strategic Planning Manager, Property Business Group, New Zealand Railways Corporation (N.Z.R.C.), Wellington, New Zealand. *Developed business plans including econometric model forecasts and divestment strategies for managing $1.2 billion of corporate real estate assets.*
1983-86	Systems Engineer, N.I.M.T. Railway Electrification Project Office, Wanganui, New Zealand.
1986-86	*Consultant to Railfreight Management. Successfully implemented single-manning of trains by using probabilistic risk analysis to investigate levels of safety associated with different size train crews.*
1984-86	*Managed computer-based critical-path construction programming of $60 million civil engineering works.*
1984-86	*Project Engineer for $5 million of retrofitting works for five tunnels for electrification, including design and construction installation of: the overhead traction system; a rock-bolt foundation support system; shotcrete strengthening of tunnel linings; and an innovative sandwich wrapped geotextile floor drainage system.*
1983-85	*Developed and implemented a risk-balanced design methodology for culvert renewals. Led design and construction team for the installation of $3 million of new culvert systems.*
1979-83	New Zealand Railways Research Fellow (PhD Candidate), University of Canterbury, New Zealand.
1979	Assistant Civil Engineer, Bridge Design Office, Chief Civil Engineers Office, New Zealand Railways, Wellington, New Zealand. *Computer-aided structural analysis for retrofitting aged iron rail bridges. Design of elastomeric bearing systems for dynamic isolation.*
1977-78	Undergraduate Student, Civil Engineering, University of Canterbury.
1973-76	Draughtsman, (Engineering Cadet for NZ Government) New Zealand Railways, Christchurch District Office. *Surveying and design of railroad layouts. Structural drafting of bridges and tunnels.*

RESEARCH CONSULTANT TO:

(1) New Zealand Railways Corporation (2005-2008)

- Senior Consultant to *Ontrack* for 5 tunnel lowering projects on the Palmerston North –Gisbourne Line
- Designed new precast concrete lining system for a deteriorated tunnel north of Dunedin.
- Asset Management Advisor for system wide drainage and culverts

(2) World Bank, [through BECA Consultants NZ] to the Romanian ministry of Construction. Advice on the development of a seismic retrofit code, including a new section on innovative protective systems. November 2006-April 2007

(3) Applied Technology Council (ATC) for the following projects:

(a) ATC 43 sub-consultant for Federal Emergency Management Agency (FEMA): *Evaluating Earthquake Damage to Wall Buildings*. 1997-8. Contributed several major sections to publications *FEMA 307* and *FEMA 308*

(b) ATC 35-1 sub-consultant for United States Geological Survey (USGS): *Ground Motion and Mapping Requirements for Earthquake Design*. Responsible as advisor on energy-based structural design. 1995-97

(c) NCHRP 12-49 Sub-consultant for development of *"Comprehensive Specification for the Seismic Design of Bridges."* This project is on behalf of AASHTO with the objective of writing the new seismic provisions for the *LRFD Bridge Design Specifications*. Responsible for the sections on concrete and timber design. 1998-2000

(4) National Institute of Building Sciences (NIBS), Washington, D.C.

- Sub-consultant to FEMA to *Develop Computational Fragility Analysis Tools for Highway Bridges*. 1998-1999

AWARDS

Fellow, Institution of Professional Engineers New Zealand

Senior Scholar, University of Canterbury, graduated with first class honours (1978).

Research Fellow at the University of Canterbury, Christchurch, N.Z., New Zealand Railways (1979-83).

PROFESSIONAL ACTIVITIES

MEMBERSHIPS

- American Society of Civil Engineers (ASCE)
  - Member of Concrete and Masonry Committee
  - Member of Wind and Seismic Effects Committee—also member of committee control group
- Institution of Professional Engineers New Zealand (Fellow IPENZ)
- American Society for Engineering Education (ASEE)
- American Concrete Institute (ACI)
- Earthquake Engineering Research Institute (EERI)
- The Masonry Society, U.S.A. (TMS)
- N.Z. Society for Earthquake Engineering (NZSEE)
- Royal Society, N.Z. (RSNZ)
- British Masonry Society (BMS)

JOURNAL EDITORSHIP

Associate Editor, *Journal of Structural Engineering*, *ASCE*:

- 1995-98 Committee on Concrete and Masonry Structures. This is ASCE's flagship journal. For the three-year term was responsible for handling the review process from receipt through publication of some 120 papers.
- 2002-2003 Seismic Effects Committee

Earthquake Engineering and Structural Dynamics: Editorial Board (2002-2010)

Electronic Journal of Structural Engineering: Editorial Board (2005-)

Advances in Civil Engineering, Associate Editor, Responsible for editor activities for those submission made in structural engineering. (2009-)

PUBLICATIONS

Summary	Total	Total at A&M
Refereed Journal papers	101	46
Refereed Research Reports	40	5
Book chapters	14	1
Peer reviewed conference papers	100	42
Other conference papers	37	

REFEREED JOURNAL PAPERS

101 Scott, R.M., Mander, J.B., and Bracci, J.M., (2011) "Compatibility strut and tie modeling: Part I—Formulation," ACI Structural Journal, *accepted, in press*

100 Scott, R.M., Mander, J.B., and Bracci, J.M., (2011) "Compatibility strut and tie modeling: Part II—Modeling," ACI Structural Journal, *accepted, in press*

99 Rodgers, G.W., Mander, J.B., Chase, J.G., (2012) "Modeling Cyclic Loading Behavior of Jointed Precast Concrete Connections Including Effects of Friction, Tendon Yielding and Dampers" Earthquake Engineering and Structural Dynamics, on-line, DOI: 10.1002/eqe.2183 http://onlinelibrary.wiley.com/doi/10.1002/eqe.2183/pdf

98 Rodgers, GW, Solberg, KM, Mander, JB, Chase, JG, Bradley, BA and Dhakal, RP (2010). "High-Force-to-Volume Seismic Dissipators Embedded in a Jointed Precast Concrete Frame," Journal of Structural Engineering, ASCE, *on-line:* http://ascelibrary.aip.org/getpdf/servlet/GetPDFServlet?filetype=pdf&id=JSENXX000001000001000242000001&idtype=cvips&prog=normal

97 Mander, J.B., Sircar, J., and Damnjanovic, I., (2012) "Direct loss model for seismically damaged structures," Earthquake Engineering and Structural Dynamics, Vol. 41, pp 571-586

96 Urmson, C.R., and Mander, J.B., (2012) "Local Buckling Analysis of Longitudinal Reinforcing Bars," ASCE Journal of Structural Engineering, 138 (1), pp 62-71.

95 Rodgers, G.W., Mander, J.B., Chase, J.G., (2011) "Semi-Explicit Rate-Dependent Modeling of Damage Avoidance Steel Connections Using HF2V Damping Devices," Earthquake Engineering and Structural Dynamics, Vol. 40, pp 977-992.

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93 Karthik, M.M., and Mander, J.B., (2011) "Stress-Block Parameters for Unconfined and Confined concrete Based on a Unified Stress-Strain Model" *ASCE Journal of Structural Engineering*, Vol. 137, No. 2, pp 270-273.

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90 Damnjanovic I., Aslan Z., and Mander, J. (2010) "Market-Implied Spread for Earthquake CAT Bonds: Financial Implications of Engineering Decisions," Risk Analysis, Vol. 30, No. 12, pp 1753-1770.

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88 Mander, T.J., Henley M.D., Scott R.M., Hite Head M., Mander J.B., and Trejo D. (2010), "Experimental Performance of Full-Depth Precast Prestressed Concrete Overhang Bridge Deck Panels," Journal of Bridge Engineering, ASCE, Vol. 15, No. 5. pp 503- 510

87 Dhakal, R.P., Mander, J.B., Xu, L., (2010) "Seismic Financial Loss Estimation of Steel Moment Frame Buildings," International Review of Civil Engineering (IRECE), Vol. 1, No. 2, May [on line].

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81 Chey, MH, Chase, JG, Mander, JB and Carr, AJ (2010). "Semi-active tuned mass damper building systems: Application," Earthquake Engineering and Structural Dynamics", Vol. 39, pp 69-89.

80 Sircar, J., Damnjanovic, I., Mander, J.B., and Aslan, Z., (2009), "Catastrophe Bonds for Transportation Assets Feasibility Analysis for Bridges," *Transportation Research Record: Journal of the Transportation Research Board, No. 2115*, pp 12–19.

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61 Bradley, B.A., Dhakal, R.P., Cubrinovski, M., Mander, J.B., and MacRae, G.A., (2007) "Improved seismic hazard model with application to probabilistic seismic demand analysis," *Earthquake Engineering and Structural Dynamics*, Vol. 36, pp 2211-2225.

60 Rodgers, GW, Chase, JG, Mander, JB, Leach, NC and Denmead, CS (2007). "Experimental Development, Tradeoff Analysis and Design Implementation of High Force-To-Volume Damping Technology," *Bulletin of the New Zealand Society of Earthquake Engineering*, Vol. 40 (2), pp 35-48

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57 Dhakal, R. P., Mander, J. B., and Mashiko, N., (2007) "Bidirectional Pseudodynamic Tests of Bridge Piers Designed to Different Standards," ASCE, J. Bridge Engrg. 12, 284

56 Rodgers G.W., Mander J.B., Chase J.G., Mulligan K.J., Deam B.L., and Carr A.J., (2007) "Re-shaping hysteretic behaviour - spectral analysis and design equations for semi-active structures," Earthquake Engineering & Structural Dynamics, Vol 36 (1) , pp 77-100

55 Chase, JG, Mulligan, KJ, Gue, A, Alnot, T, Rodgers, GW, Mander, JB, Elliott, RB, Deam, BL, Cleeve, L and Heaton, D (2006). "Re-Shaping Hysteretic Behaviour Using Semi-Active Resetable Device Dampers," Engineering Structures, Vol. 28, No. 10, pp 1418-1429

54 Dhakal, R. P., Mander, J. B., and Mashiko, N., (2006) "Identification of critical ground motions for

seismic performance assessment of structures", Earthquake Engineering and Structural Dynamics, Vol. 35, No. 8, July, pp 989-1008

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53 Dhakal, R. P., Khare R.K., and Mander, J. B., (2006) "Economic payback of improved detailing for concrete buildings with precast hollow-core floors", Bulletin of the NZ Society for Earthquake Engineering, Vol. 39, No. 2, pp 105-119.

52 Dhakal, R. P., and Mander, J. B., (2006) "Financial loss estimation methodology for natural hazards" Bulletin of the NZ Society for Earthquake Engineering, Vol. 39, No. 2, pp 91-105.

51 Xiao X., Wu H., Yaprak T.T., Martin G.R., and Mander J.B., (2006) "Experimental Studies on Seismic Behavior of Steel Pile-to-Pile-Cap Connections", ASCE Journal of Bridge Engineering, Vol 11, Issue 2, pp. 151-159

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49 Macpherson, C.J., Mander, J.B., and Bull, D.K., (2005) "Reinforced Concrete Seating Details of Hollowcore Floor Systems," Journal of the Structural Engineering Society of New Zealand, Vol. 1, pp 38-46.

48 Chase J.G., Spieth H.A., Blome C.F., and Mander J.B., (2005) "LMS-based structural health monitoring of a non-linear rocking structure" Earthquake Engineering and Structural Dynamics, Vol. 34, No 8, pp 909-930.

47 Chase J.G., Hwang K.L., Barroso L.R., and Mander J.B., (2005) "A simple LMS-based approach to the structural health monitoring benchmark problem" Earthquake Engineering and Structural Dynamics, Vol. 34, No 6, pp 575-594

46 Kim, J-H., Mander, J. B., (2005) "Theoretical shear strength of concrete columns due to transverse steel" Journal of Structural Engineering, ASCE, v 131, n 1, pp 197-199

45 Shama, A.A., and Mander J.B., (2004) "Behavior of Timber Pile-to-Cap Connections under Cyclic Lateral Loading" Journal of Structural Engineering, ASCE, Vol 130 (8), pp 1252-1262

44 Ajab J.J., Pekcan, G., and Mander, J.B., (2004) "Rocking Wall–Frame Structures with Supplemental Tendon Systems" Journal of Structural Engineering, ASCE, Vol 130 (6), pp 895-903.

43 Mander, J.B., (2004) "Beyond Ductility: The Quest Goes On", Bulletin of the New Zealand Society of Earthquake Engineering, Vol 37, No. 1, pp 35-44.

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38 Pekcan, G., Mander, J.B., and Chen, S.S. (2002), "Seismic Retrofit of Steel Deck-Truss Bridges: Experimental Investigation,” Advances in Structural Engineering, Vol. 5, No. 3, pp 173-183.

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3 Pekcan, G., Mander, J.B. and Chen, S.S., (1994) "The Seismic Response of a 1:3 Scale Model R.C. Structure with Elastomeric Spring Dampers", report to Jarret, Inc., October 19.

2 Mander J.B., Chen, S.S., Shah, K.M., and Madan, A., (1992) "Investigation of Light Pole Base Integrity" final report presented to Erie County, July 13.

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US patent:

Mander, J.B., Pekcan, G., and Chen, S.S.," Antiseismic device for buildings and works of art", US 6 256 943, July 10, 2001.

TEACHING ACTIVITIES

2007- At Texas A&M University

- CVEN 444 (501) Structural Concrete Design
- CVEN 621 Advanced Reinforced Concrete Design
- CVEN 671 Behavior and Design of Prestressed Concrete Structures

2000-2006 At University of Canterbury

- ENCI 230 – Mechanics of Materials (2003-2005) [&]
- ENCI 231 – Structural Mechanics 1 (2000-2002)
- ENCI 426 – Reinforced Concrete (2000-2002)
- ENCI 630 – Nonlinear Structural Mechanics (2002-2005) [&]
- ENCI 636 – Structural Concrete (2000-2001)

  [&] Developed these as new courses

1988-2000 At State University of New York at Buffalo

- CIE 429 - Reinforced Concrete Design (Fall '90 and Fall '99)
- CIE 467 - Special Topics in Structural Engineering (Spring '97)
- CIE 500 - Structural Design (Fall '88)
- CIE 515 - Advanced Structural Analysis (Fall '90 to '94)
- CIE 524 - Metal Structures (Fall '96)
- CIE 525 - Concrete Structures (Spring '89 to '98, Fall '99)
- CIE 527 - Design of Structural Systems (Fall '89, Spring '97, Fall'97)
- CIE 505 - Civil Engineering Seminar (Spring '91)
- CIE 521 - Plastic Analysis and Design (Spring '92-Spr '95, Spring '00)
- CIE 557/558 - Engineering Project (M.Eng., Spring '97 and Fall '98)
- EAS 451 - Engineering Computations (Spring '89 and '90)
- EAS 103 - Introduction to Engineering (Fall '92 and '93)
- EAS 140 - Engineering Solutions (Fall '95)

1997-98 Developed inaugural *Group Project* Master of Engineering *(MEng)* degree program in which 18 students successfully completed a comprehensive design of a major bridge: The *Peace Bridge* between Buffalo NY and Fort Erie Ontario. CIE 557/558 - Engineering Project (M.Eng., Spring '97 and Fall '98). Students worked in groups of six, and were responsible for two significantly different design options including all facets of geotechnical, foundation and structural design, as well as construction management, scheduling and project economics.

1985 Adjunct Lecturer, Wanganui Regional Community College, New Zealand
Advanced Mathematics 4051

1983 Teaching Assistant, University of Canterbury, New Zealand

Fortran Programming (junior course)

1981-84 National Examiner, Authority for Advanced Vocational Awards (AAVA), Wellington, New Zealand

Structures 2, fifth year (senior) NZCE course

1977-81 Adjunct Lecturer, Christchurch Polytechnic, New Zealand

Mechanics, second year NZCE course

Structures 1, fourth year NZCE course

Structures 3, sixth year post-NZCE course

SHORT COURSES

"Seismic Retrofitting of Highway Bridges", (with Prof. I.G. Buckle and G. Martin), Nashville, TN, May 28-30, 1998.

"Reinforced Concrete Design Review", for NY State Engineers, Amherst, NY (with Prof. M. Gaus), March 9-11, 1998.

"Passive Energy Dissipation for a Seismic/Wind Design and Retrofit", Irvine, CA, February 20-22, 1997, San Francisco, CA,

RESEARCH ACTIVITIES: GRADUATE STUDENT ADVISEMENT

Ph.D. Dissertations Completed: Major Advisor

1. Chang, G.A., "Seismic Energy Based Damage Analysis of Bridge Columns", Ph.D., July 1993 (presently Professor of Civil Engineering, University of Panama).

2. Mahmoodzadegan, B., "Seismic Behavior of Bridge Piers Not Specifically Designed for Earthquake Loads", Ph.D. January 1995. (presently Assistant Professor, CUNY)

3. Wendichansky, D.A., "Experimental Investigation of the Dynamic Response of Two Bridges Before and After Retrofitting with Elastomeric Bearings", April 1996 (presently Associate Professor at University of Puerto Rico).

4. Kim, J.-H., "Seismic Evaluation of Shear-Critical Reinforced Concrete Columns and Their Connections", April 1996 (presently Associate Professor, Ajou University, Su-Won, South Korea).

5. Kim, D.-K., "Experimental and Theoretical Studies on the Seismic Performance of Structural Bearing Systems", August 1996 (presently Associate Professor, Seoul National University of Technology, South Korea).

6. Cheng, C-T. "Rational Flexure-Shear Force-Deformation Modeling of Concrete Bridge Columns", January 1997 (presently Associate Professor, National Institute of Technology at Kaohsiung, Taiwan),

7. Pekcan, G., "Design of Seismic Energy Dissipation Systems for Concrete and Steel Structures", September 1998. (presently Assistant Professor, University of Nevada, Reno)

8. Dutta, A., "On Energy-Based Seismic Analysis and Design of Highway Bridges", February 1999. (presently Structural Design Engineer, Simpson Gumpertz and Heger Inc., San Francisco, CA)

9. Shama, Ayman., "On the Seismic Analysis and Design of Steel and Timber Pile-to-Cap Connections", July 2000. (presently Senior Engineer, Parson Corp., NY, NY)

Ph.D. Dissertations Completed: Co-Advisor

1. Bracci, J.M. (1992) "Experimental and Analytical Study of Seismic Damage and Retrofit of Lightly Reinforced Concrete structures in Low Seismicity Zones", Ph.D. (co-advisor with A.M. Reinhorn; presently Professor and Division Head, Zachry Department of Civil Engineering, Texas A&M.)

2. Madan, A. (1996) "Nonlinear Modeling of Masonry Walls for Planar Analysis of Building Structures" (co-advisor with A.M. Reinhorn; presently Associate Professor, IIT New Delhi, India).

Ph.D. Committees: Graduated Candidates

1. Lee, H.-H. (1990) "The Hysteretic and Dynamic Behavior of Upgraded Composite Masonry Walls under Cyclic Loadings and Strong Ground Motion" (with S.P. Prawel and G.C. Lee).

2. Kartoum, A., (1991) "Experimental and Analytical Study of a Sliding Isolation System for Bridges" (with M.C. Constantinou and A.M. Reinhorn).

3. Shi, X. (1993) "Plastic Analysis of Seismic Stress Fields" (with R. Richards and S. Ahmad).

4. Berg, E. (1993) (with J. Neal and R. Richards).

5. Lobo, R. (1994) "Inelastic Dynamic Analysis of Reinforced Concrete Structures in Three Dimensions". (with A.M. Reinhorn and M.C. Constantinou).

6. Valles, R. (1995) "Evaluation, Prevention and Mitigation of Pounding Effects in Building Structures", (with A.M. Reinhorn and M.C. Constantinou).

7. Wang, Chengbao (1995) "Advanced Development of Boundary Element Methods in Material Nonlinear Analysis" (with P.K. Banerjee and D.P. Henry).

8. Reichman, Y. (1996) "Evaluation of Bridge Structures Subjected to Severe Earthquakes".(with A.M. Reinhorn and M.C. Constantinou).

9. Guin, G. (1997) "Advanced Soil-Pile-Structure Interaction and Nonlinear Pile Behavior". (with P.K. Banerjee and D.P. Henry).

Master of Science (MS) Theses

1. Bracci, J.M. (1989) "Seismic Damage Evaluation of Reinforced Concrete Structures" (with A.M. Reinhorn).
2. Fang, L. (1990) "System Identification for Evaluating Hysteretic Response of Reinforced Concrete Structural Components Subjected to Cyclic Loads" (with A.M. Reinhorn).
3. Lao, L.F. (1990) "The Effect of Detailing on the Seismic Performance of Gravity Load Dominated Reinforced Frames" (with A.M. Reinhorn).
4. Aycardi, L.E. (1991) "The Experimental Behavior of Gravity Load Designed Reinforced Concrete Subassemblages Under Reversed Cyclic Lateral Load".
5. Panthaki, F.D. (1991) "Low Cycle Fatigue Behavior of High Strength and Ordinary Reinforcing Steels".
6. Nair, B. (1992) "An Experimental Study on the Seismic Performance of Brick-Infilled Steel Frames".
7. Kim, D.-K. (1992) "The Effects of Moment Redistribution on the Seismic Design of Structural Frames".
8. Chaudhary, M.T. (1992) "Performance of a Gravity Load Designed Reinforced Concrete Bridge Pier Model Under Reversed Cyclic Lateral Load".
9. Choudhuri, D. (1992) "The Experimental Behavior of Retrofitted Reinforced Concrete Beam-Column Joints Under Reversed Cyclic Lateral Load".
10. Shah, K. (1993) "Wind Induced Fatigue in Steel Pole Bases" (with S.S. Chen).
11. Waheed, S.M. (1993) "Experimental Behavior of a Full-Size Bridge Pier Cap-Column Subassemblage Under Reversed Cyclic Lateral Load".
12. Premus, G.J. (1993) "The Seismic Performance of 30 Year Old Steel Bridge Bearings" (with S.S. Chen).
13. Ligozio, C. (1993) "An Experimental Investigation of the Seismic Performance of a Bridge Pier Designed for Gravity Loads".
14. Kasalanati, A., "Variable Amplitude Low Cycle Fatigue Behavior of Reinforcing Steel", December 1993.
15. Pekcan, G., "Low-Cycle Fatigue Behavior of Semi-Rigid Top-and-Seat Angle Connections", February 1994.
16. Estevez, A., "Biaxial Lateral Load Resistance of a Wall Type Bridge Pier", February 1994.
17. Bhadra, S., "An Experimental Investigation of the Seismic Performance of a Retrofitted Bridge Pier Designed Only for Gravity Loads", April 1994.
18. Dutta, A., "Fatigue Failure Theories for Reinforced Concrete Bridge Columns", September 1995.
19. Jung, O., "Retrofitting of Reinforced Concrete Structures Using Wire Rope as Transverse Reinforcement", September 1995.
20. Goel, P., "Overstrength Factors for Capacity Design of Bridges," January 1996.
21. Kungunga, P.A., "Renewable Moment Resisting Beam-to-Column Connections for Precast Concrete Frame Buildings", August 1996.
22. Qi Ye, "Computational Modeling of Spring Dampers and the Seismic Retrofit of Sway Frames", January 1998.
24. Tobar Ramos, Rene E., "Digital Image-Based Measurements in Experimental Structural Mechanics", July 1998.
25. Garcia, Rudolfo, "Shaking Table Study of Rocking Column Bridge Based on Damage Avoidance Design", August 1998.
25. Kokorina, Tatiana, "Experimental Performance of Reinforced Concrete Bridge Columns with Renewable Hinges Subjected to Seismic Excitation" (August 1998).
26. Szustak, Peter W., “Development of a Soil-Structure Interaction Testing Facility and Experimental Validation of a Seismic Bearing Capacity Theory for spread Bridge Footings” (August 1998).

27. Garcia, Diego Lopez, “Evaluation of Methods of Seismic Analysis for Existing Bridges” (August 1999).
28. Allicock, Dion R., “Experimental Study of Timber Piles Subjected to Reverse Cyclic Loading” (September 1999).
29. Percassi, Stephen J., Jr., “Rocking Column Structures with Supplemental Damping Devices” (February 2000).
30. Ajrab, Jack J., “Rocking Wall-Frame Structures with Supplemental Damping Devices” (May 2000).

Master of Engineering (M.Eng.) Report

1. Wojtkowski, K.M. (1990) "Experiments on Brick-Infilled Steel Frames Under Reversed Cyclic Lateral Load".
2. Chen, C.-Y. (1990) "Behavior of Confined Micro-Concrete Columns".
3. Ma, J. (1990) "Experiments on Brick Infilled Steel Frames Under Reversed Cyclic Lateral Load".
4. Carlson, Brian (1998) "A New Peace Bridge Across the Niagara River".
5. Contreras, Rosa E., "Shaking Table Experiments on Bridge Piers with Rocking Columns", July 1998.
6. Cosgrove, Arthur C. (1998) "The Design of Prestressed Concrete Arch Bridge As a Twin to the Existing Peace Bridge between Buffalo and Fort Erie".
7. Craig, Michael W. (1998) "Peace Bridge: Alternative Design Curved Cable Stay Signature Bridge".
8. Conlon, William J. (1998) "Peace Bridge Signature Bridge Alternative".
9. Heh, Wei-Ming (1998) "Peace Bridge: Construction Engineering and Management of a Proposed New Twin Segmental Prestressed Concrete Arch Bridge".
10. Holevinski, Holly A. (1998) "The Preliminary Design of an Externally Prestressed Concrete Truss Bridge for the Peace Bridge Project".
11. Keenan, Christopher T. (1998) "The Design of a Prestressed Concrete Curved Cable Stay Bridge as an Alternative to the Twinning of the Existing Peace Bridge between Buffalo and Fort Erie".
12. Kostowniak, Edward W. Jr. (1998) "Peace Bridge: Construction Engineering and Management of a Proposed New Curved Cable Stayed, Prestressed Concrete Signature Bridge".
13. Lam, Kai H. (1998) "Twin Peace Bridge".
14. Lazzaro, Donald (1998) "A New Peace Bridge Across the Niagara River".
15. Major, Timothy E. (1998) "Twin Peace Bridge Design".
16. Paksaichol, Prakarn (1998) "The Design of a Double Deck Prestressed, Precast Concrete Cable Stayed Truss Bridge Foundation as a Replacing of Existing Peace Bridge, Buffalo, USA and Fort Erie, Canada".
17. Schaller, Robert (1998) "A New Peace Bridge Across the Niagara River".
18. Schoenthal, Carl L. (1998) "The Design of a Prestressed Concrete Arch Bridge as a Twin to the Peace Bridge between Buffalo, New York and Fort Erie, Ontario".
19. Schulz, Cameron (1998) "A New Peace Bridge Across the Niagara River Twin Bridge Foundation".
20. Tschiderer, Bob (1998) "Twin Peace Bridge".
21. Huang, I-Chi (1998), “Alternative Method of Constructing Seismic Resistant Steel Frames”.

At the University of Canterbury:

PhD Degrees (Note University of Canterbury requires co-supervision)

1. Toranzo L, (2002) "The use of rocking walls in confined masonry structures"
2. Matthews, J., (2004) "Hollow-core floor slab performance following a severe earthquake"
3. Saunders, D., (2004) "Seismic performance of pre-1970's non-ductile reinforced concrete waffle slab structures constructed with plain round reinforcing steel".
4. Abdul Hamid , N (2007) "Rocking hollow core precast concrete wall buildings," Presently Senior Lecturer, Malaysia University of Technology, Mara.
5. Zaghlool B., (2007) "Behavior of three-dimensional concrete structures under concurrent orthogonal seismic excitations" Presently Senior Structural engineer, The O'Neil Group Pty Ltd, Victoria Australia.
6. Mulligan K., (2007) "Experimental and Analytical Studies of Semi-Active and Passive Structural Control of Buildings"
7. Chey Min Ho (2007) "Passive and semi-active tuned mass damper building systems" Presently Assistant Professor, Yanbian University of Science and Technology, China and Pyongyang University of Science and Technology' in North Korea
8. Rodgers, G.W. (2009) "Next Generation Structural Technologies: Implementing High Force-To-Volume Energy Absorbers," Presently, Post-doc, University of Otago Medical School, Christchurch, NZ

M E Theses

1. Holden (2001) "A comparison of the seismic performance of precast wall construction: Emulation and Hybrid approaches"
2. Lander M., (2001)
3. Martinez (2002) "Performance-based seismic design and probabilistic assessment pf reinforced concrete moment resisting frame structures"
4. Surdarno I., (2003) "Performance of thin precast concrete wall buildings under dynamic loading"
5. Davies M., (2003) "Seismic damage avoidance design of beam-column joints using unbonded post-tensioning"
6. Murahidy A., (2003) "Design, construction, dynamic testing and computer modelling of a precast prestressed reinforced concrete frame building with rocking beam-column connections and ADAS elements"
7. Arnold D., (2004) "Development and experimental testing of a seismic damage avoidance designed beam to column connection utilising draped unbonded post-tensioning"
8. Bothera J., (2004) "A shaking table investigation on the seismic resistance of a brick masonry house"
9. Liyanage L., (2004) "Biaxial lateral loading behaviour of thin concrete walls"
10. Lindsay R., (2004) "Experiments on the seismic performance of hollow-core systems in precast concrete buildings."
11. Liew H., (2004) "''Performance of hollowcore floor seating connection details"
12. Rahardjo T., (2004) "Experiments and stochastic modelling of NZ grown Pinus Radiata timber and timber piles under seismic loading"
13. Macpherson C., (2005)
14. Robertson, K.,(2006) "Probabilistic seismic design and assessment methodologies for the new generation of damage resistant structures"
15. Li L., (2006) "Further experiments on the seismic performance of structural concrete beam-column joints designed in accordance with the principles of damage avoidance"
16. Wang C., (2006) "Experimental investigation on behaviour of steel fibre reinforced concrete (SFRC)."
17. Mashiko, N. (2007) "Comparative performance of ductile and damage protected bridge piers subjected to bi-directional earthquake attack."

18. Solberg, K., (2007) "Experimental and financial investigations into the further development of Damage Avoidance Design."

At Texas A&M University

MS Theses, Graduated

1. Sircar, J., (2008) "Loss modeling for pricing catastrophic bonds," student funded
2. Henley, M., (2009) "Shear connections for the development of a full-depth precast concrete deck system," Funded by USAF
3. Reddiar, MKM (2009) "Stress-strain model of unconfined and confined concrete and stress-block parameters," student funded.
4. Brey, RW (2010) "A systematic investigation of shear connections between full-depth panels and precast prestressed bridge girders," funded from start-up
5. Scott, R.M., (2010) "Experimentally Validated Compatibility Strut and Tie Modeling of Reinforced Concrete Bridge Piers," funded on TxDOT project 5997.
6. Urmson, C.R., (2010) "Ultimate Limit State Response of Reinforced Concrete Columns for Use in Performance-Based Analysis and Design," funded from start-up.
7. Deshmukh, PB (2011) "Rapid spatial distribution seismic loss analysis for multistory buildings" Partial hourly support.
8. Ghorawat, S (2011) "Rapid loss modeling of death and downtime caused by earthquake damage to structures" Student had a TA position
9. Parkar, AS (2011) "Design and development of a continuous precast prestressed concrete bridge system for the multimodal freight shuttle project." TTI funding from Freight Shuttle project.
10. Roy, S (2011) "Design and construction integration of a continuous precast prestressed concrete bridge system." TTI funding from Freight Shuttle project
11. Yoon Mo Kim (2011) "Modal analysis of continuous structural system with tapered cantilever members." Self-supported student.

SUBMISSION TO THE ROYAL COMMISSION OF INQUIRY:

AN ALTERNATIVE COLLAPSE SCENARIO FOR THE CTV BUILDING

John B. Mander PhD, F.IPENZ
Inaugural Zachry Professor
Texas A&M University
College Station, TX 77843, USA

INTRODUCTION AND SCOPE OF THIS SUBMISSION

The first purpose of this submission is to review the key findings of the work commissioned by the Department of Building and Housing (the **DBH**) on the Canterbury Television Building (**CTV Building**) Collapse Investigation, as reported by Dr. Clark Hyland and Mr. Ashley Smith in January 2012 (**H-S Report**). This submission will show that while the H-S Report has been comprehensively executed, much of the analysis has been based on several erroneous assumptions. The claims resulting from the CTV Building Collapse Investigation are therefore faulty in their reasoning leading to incorrect conclusions.

This submission also considers the remarks made by Mr. William Holmes [BUI.MAD249.0372] who was the formally assigned external international peer reviewer of the H-S Report. Holmes is moderately critical of several technical points in the H-S Report, the foremost of which he considers the neglect of modeling the connections correctly. He goes on to point the way forward in seeking the truth to what really caused the final collapse of the CTV Building, but falls short of drawing firm conclusions.

It should be noted that during the course of the CTV Building Collapse Investigation, the Hyland-Smith team was advised by an external group appointed by the DBH. Professor Nigel Priestley was the senior engineering advisor within the external group. Because the advisory group and the Hyland-Smith team were not in agreement with the conclusions made in the H-S Report, Prof. Priestley was invited by the Royal Commission of Inquiry (the **Commission**) to make a separate submission. Like Holmes, Prof. Priestley also points out several weaknesses in the report and also points the way forward, but without coming to definitive conclusions. This submission discusses the remarks by Prof. Priestley

[WIT.PRIESTLEY.001] who criticizes some of the procedural analyses presented in the H-R Report, and hence rebuts many of the conclusions made by the Hyland-Smith team.

The second purpose of this submission is to present, analyze and discuss the results of new work done since the completion of the H-S Report. This new work includes:

- A comparative analysis of the ground motions recorded recently at the CTV Building site with the four other Geonet free-field recording stations within the vicinity of the central business district (**CBD**) of Christchurch;

- A further analysis of concrete test results on test cylinders cored from the undamaged column remnants retrieved from the CTV Building and tested by CTL Thompson Materials Engineers, Inc of Denver, CO, USA (**CTL**);

- A review of the work completed to date by Dr. Rajesh Dhakal, an Associate Professor at the University of Canterbury who was commissioned to conduct full-scale tests on large intact column remnants retrieved from the CTV Building;

- An analysis of column performance under double bending within each floor level of the CTV Building to show the sensitivity of the concrete strength and confinement effects under different levels of axial load; and

- Some general conclusions from the above points.

The third and final purpose of this submission is to provide an alternative hypothesis to the original collapse hypotheses proposed in the H-S Report. It is shown that the columns, independent of their degree of ductility capability, could have collapsed over the lower four stories from a classic type of buckling (known as Euler buckling), largely due to the overload effects arising from extremely high vertical ground motions and promoted from a deteriorated beam-column joint condition.

At the time of writing this submission, full corroboration of the alternative collapse hypothesis through advanced computational analysis is still a work-in-progress. The additional analysis, to be conducted by Compusoft Engineering Ltd (**Compusoft**), the original subcontractors for the H-S Report, through the computational non-linear time history analysis (**NTHA**) expert panel process, may provide useful insights that are expected to support or modify the alternative collapse hypothesis.

1. A CRITIQUE OF THE HYLAND-SMITH REPORT AND ITS KEY FINDINGS

The principal conclusion in the H-S Report states:

> *"The investigation has shown that the CTV Building collapsed because earthquake shaking generated forces and displacements in a critical column (or columns) sufficient to cause failure. Once one column failed, other columns rapidly became overloaded and failed."* (Executive Summary)

The above conclusion is so generic that it could apply to virtually any type of building collapse. Moreover, this conclusion is so vague it is neither helpful nor insightful. What is also not clear is what specific forces or displacements are being referred to: north-south (N-S); east-west (E-W); a torsional combination; up-down (vertical); or an unknown combination of all of these. On further reading of the H-S Report, it becomes clear that an emphasis is placed on lateral displacements as the principal trigger mechanism that initiated collapse. Initial clues to this are found in the contributing factors listed at the commencement of the Executive Summary, with a more detailed discussion in Section 8 (Collapse Scenario Evaluation) of the H-S Report. This submission responds to and critiques the supporting conclusions made in the H-S Report, addressing each of the contributing factors in turn. .

1.1 Higher than expected horizontal ground motions

The CTV Building was designed and constructed in compliance with the applicable design and building codes and the developer was granted a building permit from the Christchurch City Council. The CTV Building was also designed in accordance with the accepted industry practice of the 1980s for a structure to withstand much *smaller* elastic forces than a full "design-level" earthquake. Then when the full force of the "design-level" earthquake is applied, the structure is expected to be damaged, but without collapse. Even though damage to the structure can be tolerated, life-safety via collapse-prevention must be ensured. To illustrate the design process, and how the structure of the CTV Building measured up to the design expectation, a comparison of the as-designed seismic capacity with the two major earthquakes it was exposed to will now be given.

The CTV Building had a first mode natural period of T= 1.0 seconds, for which the loadings code NZS4203 (1984) specified a spectral acceleration coefficient of C=0.095. Implicit in this prescribed value is that structures, if appropriately detailed, should survive an earthquake some 4 times the value of C=0.095. Thus, at T = 1.0 seconds, a 5% elastic response spectral acceleration ordinate of Sa = 0.38 g is implied by the loadings code for the

CTV Building. Note g = gravitational acceleration = 9.81 m/s/s.

Recent work by Dr. Brendon Bradley (2012) has shown that for the 22 February, 2011 earthquake (**Christchurch Earthquake**), at the CTV Building site a median conditional spectral amplitude for T = 1.0 seconds is Sa [50%] = 0.75 g. However, there is considerable spread and the plus/minus one standard deviation give results of Sa [84%] = 1.0 g and Sa [16%] = 0.55 g, respectively.

> *Explanatory comment:* *Whenever an earthquake strikes, vibration waves propagate through the rocks and soils. Soils are particularly problematic because their properties vary so much, in a random type of fashion. Therefore, the manner in which the seismic waves propagate is affected by this randomness in the soil's properties—the velocity and severity of the seismic waves are altered by the soil variability. For example, at two relatively nearby sites, seismic sensors could record potentially quite different outcomes, particularly in the high frequency band. Any two earthquakes have quite different properties. In general, this randomness or variability is called "aleatory uncertainty", a type of uncertainty that can be quantified and thereby mathematically modeled in a probabilistic sense. When such known uncertainty is applied to the CTV Building site, it leads to a relatively broad band of possible outcomes. In statistical terms, this is quantified via the standard deviation. From any statistical tables this band of spread ranges from the 16th percentile (meaning 16 out of 100 similar events would have a smaller result) to the 84th percentile (or 16 out of 100 similar events would have a larger result). Or in other words, roughly two-thirds of all possible events or earthquakes like the Christchurch Earthquakes would be expected to produce vibration signatures that would fall within this plus/minus one-standard deviation range.*

Compared to code-base design motions, the CTV Building site withstood *much higher than expected horizontal ground motions.* For any structure to survive such a high level of shaking is a bonus; it was certainly not a requirement at the time the CTV Building was designed and constructed in the late 1980s. So the supporting conclusion in the H-S Report,, that "higher than expected horizontal ground motions were observed," is correct. However, the H-S Report essentially neglects the effect of earlier earthquakes on the structure of the CTV Building.

While much higher than expected ground motions were observed during the Christchurch Earthquake, focusing the discussion on this disregards the fact that the CTV Building, and indeed all structures in the Christchurch area, suffered varying degrees of damage in previous earthquakes, commencing with the magnitude 7.1 earthquake in Darfield on 4 September, 2011 (**Darfield Earthquake**). Based on the results from Bradley (2012), at the CTV Building site the Darfield Earthquake produced a median conditional spectral amplitude for T = 1.0 seconds of Sa [50%] = 0.33 g, with a plus/minus one standard deviation spread of

Sa [84%] = 0.44 g and Sa [16%] = 0.24 g, respectively. Hence it can be inferred that there is a 40% chance that the Darfield Earthquake ground motion at the CTV Building site was *larger* than the design level of ground motion. That is, the initial Darfield Earthquake alone produced essentially the same forces and acceleration that the Christchurch City Council permitted CTV Building was designed to resist. By design, significant damage would be expected from such a level of ground shaking. The fact that the CTV Building survived the design-level Darfield Earthquake, with only minor visually observed damage, is a testament to the sufficiency of the design—it met the aim and objective of the design codes.

However, as will be discussed in section 2 below, it is evident that the structure of the CTV Building must have also sustained hidden (unobserved and/or unobservable) damage. It can be argued that with the level of observed as well as hidden damage, the CTV Building should have been "Red Stickered" following the Darfield Earthquake. Inspecting engineers would have been well aware that the level of ground motions sustained was similar to the level that the design code NZS4203 of 1984 called for. This should have served as a signal that substantial inelastic response would have occurred, whether it was seen or unseen. It is therefore concerning that the inspectors did not immediately Red Sticker the CTV Building.

What is more concerning is that following the Darfield Earthquake, eyewitnesses reported on numerous occasions that the CTV Building was uncomfortably lively. Again, this should have served as a signal and as further confirmation to inspecting engineers that the CTV Building had sustained some hidden damage, and that they should take a second look to determine the source of damage. Some insights into the cause of this structural liveliness are discussed at the close of section 1.2 below.

1.2 Exceptionally high vertical ground motions

The seismic design environment of the 1980s

During the 1980s, structural engineers operated using a heuristic rule that vertical ground motions would be about two-thirds of the horizontal motions. Stiff and heavy horizontal elements such as floor slabs tend to have high vibration frequencies greater than 2 Hz, that is the vibration periods for those elements are normally T < 0.5 seconds. The 1980s design spectrum for Christchurch for short periods (that would affect higher modes, such as floors) was C = 0.125. This implies elastic spectral response accelerations of Sa [T < 0.7seconds] = 0.5 g and 0.33 g, for horizontal and vertical motions, respectively. Floor slabs tend to be excited by these vertical ground motions, and at this modest level of shaking the normal safety factors inherent in gravity load design make the floors capable of sustaining the vertical ground motion induced vibrations.

By inspecting the vertical response results in the H-S Report (Figures 51 and 56), it is evident that the CTV Building had a floor system with a vibration period of T = 0.25 seconds (4 Hz). During the Christchurch Earthquake the observed vertical spectral response accelerations results are Sa [T = 0.25 seconds] = 0.55, 0.65, 0.85 and 0.95 for the CBGS, CHHC, CCCC and REHS recording stations, respectively. Clearly, these seismic demands were *much* higher than the expected level of 0.33 g implied by design. Thus, the supporting conclusion of H-S Report that “exceptionally high vertical ground motions” helped lead to the demise of the CTV Building is correct. But again, the H-S Report essentially neglects the effect earlier earthquakes had on the structure of the CTV Building.

During the Darfield Earthquake the observed vertical spectral response accelerations results were Sa [T = 0.25 seconds] = 0.30, 0.30, 0.27 and 0.37 for the CBGS, CHHC, CCCC and REHS recording stations, respectively. These demands are about the same as the 0.33 g that would be expected for a “design-level” ground motion. Again, inspecting engineers would have been aware that this design expectation for the CTV Building either was already met or had been exceeded. Therefore, engineers should not have been surprised by reports from occupants that the CTV Building was considerably more lively after the Darfield Earthquake. *The effects of the damage as felt by the CTV Building occupants should have served as sufficient evidence that the CTV Building should have been Red Stickered.*

The Unexpected consequences of Exceptionally High Vertical Ground Motions Heading

Not referred to in the H-S Report is the fact that the CTV Building had a continuous three-span one-way slab system in the N-S direction. The relatively high vertical vibrations from earthquakes prior to the Christchurch Earthquake essentially would have “broken” the fixed-end condition to make the slabs function more like three individual simply supported units. Therefore, vibration-induced deflections would be amplified up to some 500%. Although these vibrations made the CTV Building occupants feel uncomfortable, they would not have seriously impaired the safety of the vertical load-carrying system. However, the floor-seat connection damage made the CTV Building ill-prepared to survive subsequent large earthquakes, in particular the Christchurch Earthquake, which produced force demands and accelerations that were some three times larger than those implicitly accommodated for in the code-based designs of the 1980s. During the Christchurch Earthquake, out-of-plane shaking induced damage would have impaired the ability of the floors to adequately transmit concurrent in-plane forces and may have led to a buckled folded-plate failure mode.

In light of these exceptional demands on the CTV Building, it is quite surprising that this point has not received more attention. For example, if we were to run this analysis as a graduate student project, the students would be instructed to analyze the effect of the vertical ground motions alone (without horizontal components of ground shaking) and to investigate the extent of damage caused to the structure. Naturally, all motions combined would also be analyzed as well and the results compared. Instead of the effect of horizontal actions being the primary cause of the CTV Building failure, it is contended that the *exceptionally high* vertical ground motions were a primary contributor to triggering the CTV Building's failure and subsequent collapse.

1.3 Lack of ductile detailing in critical columns

Although "confined" concrete columns have been the hallmark of building and bridge structures constructed in New Zealand since the 1970s, a liberal interpretation of the 1980s building design codes allowed for designers to choose other strategies to provide earthquake resistance. It appears that the deviance from ductile detailing in the concrete columns was contentious at the time that the CTV Building designer sought the building permit from the Christchurch City Council. This deviance from customary ductile detailing remains a contentious issue in the H-S Report. Good ductile detailing, including confinement of columns, is highly desirable in the delivery of a robust structure.

However, during the 1980s era of building construction there began a time when developers and contractors put immense pressure on structural designers to deliver buildings at low-cost, coupled with rapid construction details. The former mold was broken, moving from cast-in-place moment-frame systems that were the hallmark of a mini building boom in the 1970s, to the entirely modular precast structures of today. This modern era started around the time of the design of the CTV Building when the time-cost-of-money (interest rates ~26%) was dictating shorter project delivery times. The CTV Building was in fact quite revolutionary at that time, as the details of the design are clearly contractor-friendly. It appears to be for these reasons that the structural designer evidently sought a simpler form of construction that avoided the use of copious quantities of transverse reinforcing steel to provide a ductility capability.

Transverse reinforcement in columns provides three primary functions:

(i) It confines and strengthens the core concrete so that when the cover concrete in the end regions spalls off at high bending strains, the strength is restored due to the substitution of the strong confined core. Confined concrete also permits very large strains to exist (from sway effects) while still providing a substantial amount of stress (resistance).

(ii) It provides additional shear resistance that is not possible once the concrete cracks. Because the concrete is highly strained (and highly cracked) in the end regions of the columns, the shear strength is sustained through tightly wound spirals or closely spaced hoops.

(iii) Under high bending rotations, once the cover spalls off, the longitudinal reinforcing steel is prone to buckle. Closely spaced transverse steel inhibits this buckling and allows the reinforcing bars to maintain their high compression strains and loads.

Although it is true that the columns were not provided with substantial transverse reinforcement, this was neither a problem nor a cause of failure within the CTV Building. If it were a cause of the collapse, then there would be substantial forensic evidence indicating that most columns had significant lengths of cover concrete spalled off, substantial buckling of longitudinal reinforcing bars due to the high axial loads, and diagonal shear cracks. There is little evidence of such damage to the columns of the CTV Building. There is only some minor evidence that short circumferential rings of concrete were missing, but it is contended that this was mostly an *outcome* of the CTV Building collapse—not the cause.

There is an analogous problem to low transverse reinforcing steel in the columns: no transverse reinforcing steel in the beam-column joint regions. Transverse reinforcing steel in the beam-column joint regions in the form of horizontally oriented closely spaced spirals or hoops is normally provided to help resist high joint shear forces. Such horizontal joint steel is also called "confining steel" in the United States, whereas in New Zealand it is merely called joint-shear reinforcement. Such reinforcement is not intended for reasons of ductility per se, rather it provides the two functions listed in paragraphs (ii) and (iii) above.

The shear force demands in beam-column joints are several times greater than in the surrounding columns and beams. For robust performance, it can be shown from the fundamentals of mechanics that there should be roughly the same area of transverse reinforcing steel in the beam-column joint as there is in the top and bottom beams combined. Such highly reinforced joints are very hard to construct, and contractors certainly prefer not to place steel in the beam-column joints as it is a very slow, awkward and therefore costly within the construction process.

The fact that a beam-column joint has no transverse steel does not mean that it does not have a shear transfer mechanism. Instead, concrete arch action occurs and a diagonal concrete-strut provides the shear transfer mechanism. The concrete strut also serves as part of the axial (gravity) load path. However, the joint concrete's ability to transmit this heightened level of load due to higher stress intensities is impaired under significant transverse tension strains (ε_1), cyclic loading effects, or both. Earthquakes are of course highly cyclic in nature and the alternating loads eventually wear the concrete's resistance down to a point where it becomes rubble.

It is contended that the lack of joint-shear reinforcement is one of the principal contributing factors to the CTV Building's collapse. In fact, prior to the Christchurch Earthquake, substantial fatigue-like damage would have already existed in the joints in the CTV Building. Damage to the concrete beam-column joints resulting from the Darfield Earthquake also provides additional evidence of the lively nature of the CTV Building structure.

The simple beam-column cruciform subassemblage, as depicted in Figure B.5 of the Compusoft Report, is not complete. When properly completed, it tells the story of very high joint shear intensity, where the beams are stronger than the columns and the columns stronger than the joints. For the CTV Building, because the joints were the weak link in the chain, they were part of a primary trigger mechanism of the collapse mode.

Finally, on this point of ductility, it can be shown that if the NZS3101 code-prescribed amount of transverse reinforcement was provided in the columns, this would not necessarily have prevented the collapse of the structure of the CTV Building via the columns. This is because the 400 mm diameter columns were small with a relatively high degree of cover concrete, sized for a Darfield Earthquake type of event—a test which the CTV Building demonstrably passed. Had 500 mm diameter columns been used, along with closely spaced spiral (confinement) reinforcing in the columns, and even more closely spaced spiral in the beam-column joints, then the CTV Building would have still been damaged in the Christchurch Earthquake, but the vertical load path would have been maintained thereby giving the CTV Building a greater chance of surviving a collapse. But in the 1980s at the time of design, such columns and joints would have been considered an expensive and unnecessary luxury that would minimize the developer's profit margin.

1.4 Low concrete strength in the critical columns

When an engineered structure is being built, the contractors order materials based on a specified strength. Once purchased, there is a design-based implication that there should be a 95% chance that the observed strength of any materials sampled either meets or exceeds the specified strength. Thus the as-built strength is generally somewhat greater than the specified strength. The in-situ strength of concrete is formally defined as the average crushing stress (in MPa units) of three standard 100 mm diameter by 200 mm long test cylinders when tested at 28 days after pouring the concrete. To achieve a concrete strength to ensure a 95% exceedance probability that the provided in-situ strength will be greater than the specified strength, the ready-mix concrete batching plant uses a target mean strength that is greater than the specified strength by some 25%. Once placed, the in-situ concrete further hardens over time, such that after a few years the strength is typically 20% greater than the 28 day test results. Therefore, when assessing the strength of an as-built structure in the absence of any material test data, it is customary professional practice to use a probable strength of 1.5 times the specified strength. However, in order to conduct a full forensic analysis of any collapsed structure, it is wise to obtain and test samples of the materials used in the structure to ascertain the actual as-built strength.

When analyzing the concrete strength in the critical columns of the CTV Building, Opus International Consultants Limited (**Opus**) (the subcontractors) carried out the formal testing of the concrete using several core samples extracted from the CTV Building. These small diameter test cores were obtained by drilling into the sides of the CTV Building columns. To supplement only a few destructive crush-test results on these smaller than standard cylinder specimens, an alternative rapid non-destructive test method was also applied—the Schmidt-Hammer test. Once calibrated and collated, the results were averaged for all concrete and presented graphically in the form of a normal distribution, as shown in Figure 5 of the H-S Report.

Examination of the plotted test results imply that the as-built concrete only possessed a characteristic strength (that means a 95% exceedance probability) of 15 MPa. Clearly this is an unrealistic result. It is evident from a cursory inspection of the rubble at the Burwood dump site, that the quality of the concrete is mixed; much of it is damaged partly from the collapse, and also in part by the fire. However, a substantial amount of the concrete appears to be undamaged. A visual inspection reveals the quality to be quite sound and not likely to be as weak as 15 MPa.

Dr. James MacKechnie was engaged by the Commission to review the concrete tests and the associated conclusions in the H-S Report. His review [BUI.MAD249.0362.1]) casts serious doubt on the process and procedures.

Dr. Brendon Bradley was engaged by Buddle Findlay on behalf of ARCL to conduct further analyses on the concrete test results presented in the H-S Report. His rigorous and formal probabilistic analysis shows that there is _no statistical significance_ in the claim that the columns had lower concrete strength than specified.

CTL in the United States tested eight large-diameter core samples extracted from the central region of column remnants from the CTV Building collapse. All test results showed the concrete to be above the specified strength, with an average value of f'_c = 40 MPa. Even without further analysis, it is immediately apparent that the CTL results are more reasonably in keeping with what one would expect to observe with concrete aged some 25 years.

In concert with the physical strength tests conducted by CTL, Mr. Douglas J. Haavik was engaged to conduct a detailed materials study on the quality of the concrete in the columns of the CTV Building. Haavik concludes:

> "…*there is no reason to believe that there was a systematic reduction in concrete strength supplied to the project and that any such strength reduction is likely attributable only to gross error for a specific load of concrete which itself is extremely unlikely."*

In spite of the so-called "low concrete strength in the critical columns," Compusoft wisely chose to ignore this advice from Opus, and used the "specified strength + 2.5 MPa." However, in light of the more recent evidence from the CTL labs, even this assumption was on the low side of the probable strength.

Further analysis is presented below in the Section 2 of this submission on the concrete tests conducted by the CTL labs. This analysis demonstrates that had the customary practice been used of having a *probable strength of 1.5 times the specified strength,* then more realistic results would have been obtained in the non-linear time history analyses (**NLTHA**) work conducted by Compusoft. In summary, the claim in the H-S Report that the concrete had low concrete strength in the critical columns is erroneous.

1.5 Interaction of perimeter columns with the spandrel panels

Historically there have been many instances of collapsed structures where the so-called "soft-story" effect has been caused by the presence of "short" shear-critical columns. Often the short column effect is due to the presence of up to half-story high infilled frames. The presence of the spandrel panels in the CTV Building alludes to this class of failure, with the resulting collapse mechanism development also shown in Figures 17 and 18 of the H-S Report.

While the interaction of the perimeter columns with the spandrel panels in the CTV Building may have been a contributing factor in the final demise of the structure, this was neither the trigger nor the cause of the collapse. The exterior frames where the spandrels were present were more lightly loaded than their interior cousins. This lighter value of axial load reduces the *P-delta* instability effect. It will be shown later that the more heavily loaded interior columns were more critical.

When future analyses on the CTV Building are conducted, it is essential that the spandrel effects are modeled directly with the use of gapping elements that mimic the opening and closure of the clearance gap between the columns and spandrel panels. Unless this feature is modeled accordingly, it is not possible to know whether the interaction of the exterior columns on Line F in particular (the basis for the main collapse hypothesis proposed in the H-S Report) was instrumental in initiating the structural collapse of the CTV Building.

1.6 Separation of the floor slabs from the North Core

It is agreed that the separation of the floor slabs from the North Core is problematic. This separation permitted differential displacements to occur between floors. In the alternative collapse hypothesis developed in Section 3 of this submission, it will be shown that in many respects, this detail in the CTV Building would perhaps be a necessary feature of several different failure modes, including that proposed in the H-S report.

Notwithstanding this detailing feature due to the absent drag-bars, the history of the CTV Building design should be recalled. The original Christchurch City Council permit for the CTV Building construction did not require drag-bars as part of the design. Just a few years after the construction was complete, the CTV Building was up for sale and as part of the sale process, Holmes Consulting Engineers pointed out that drag-bars should be installed on all floors.

However, on the design review by ACRL, based on the required resistance for the design-level earthquake, ARCL recommended that the drag-bars need not be placed on all floors. On the one hand ARCL have been criticized for not providing sufficient redundancy in their detailing. But on the other hand, ARCL can feel vindicated because the structure survived the design-level Darfield Earthquake *without collapse*, which was a main aim of the design.

1.7 Accentuated lateral displacements of columns due to the asymmetry of the shear wall layout

Similar to the issue in section 1.5 above, while the accentuated lateral displacement of the CTV Building columns may have contributed to the collapse of the CTV Building, this factor has been overstated. (This overstatement has also been noted by Prof. Priestley.)

Although pushover analyses have been conducted by Compusoft on the CTV Building, the results were not put to good use to give further insights into the performance of the CTV Building. The NLTHA also conducted by Compusoft was in 3D. Consequently all torsional and eccentricity features are automatically captured in that more rigorous advanced method of analysis.

One simple method to check whether the lateral-torsional coupling effects are significant is to apply the results of the pushover analyses. In theory, it is possible to take the pushover curves of a structure and normalize them, then superimpose the normalized pushover curves on the Acceleration-Displacement Response Spectrum (**ADRS**). By performing this analysis, a single-degree-of-freedom (**SDOF**) simplification can be made of the structure and then used along with an appropriate damping factor for modeling overall system hysteretic behavior, in order to infer the maximum earthquake response of the structure.

When performing this simplified “capacity-spectrum” (SDOF-ADRS) method of analysis on the CTV Building, the results agree remarkably well with the NLTHA; they do not indicate that the translational displacements are significantly amplified by lateral torsional (eccentricity) effects.

1.8 Accentuated lateral displacements due to the influence of masonry walls on the west face

The west wall of the CTV Building was damaged due to two factors. First, the Darfield Earthquake left the west part of the CTV Building badly cracked. This damage and the additional aftershock damage, along with more damage that was most likely sustained as a consequence of the demolition of the adjoining building, would have left the integrity of the west wall of the CTV Building impaired. Eyewitnesses even reported seeing daylight through portions of the west wall **[WIT.HARRIS.001]**. Consequently, subsequent to the Darfield Earthquake the west wall and frame was essentially unfit for the purpose of providing a substantial degree of seismic resistance. Evidently, the deteriorated condition of the west wall was so serious that it was necessary to implement repairs which were still being undertaken at the time of the Christchurch Earthquake.

Second, the lack of integrity of the west wall may have contributed in promoting the unseating of the E-W beams along column lines 2 and 3 of the CTV Building. It is hypothesized that the pull-out of those beam connections into the west wall region led to the unseating of the beams. As a consequence, the gravity load, normally carried down the west wall frame, would need to be resisted elsewhere. This load would be mostly transferred to the neighboring interior columns along line B of the CTV Building. The additional axial load demand, along with the *exceptionally high* vertical ground motions would then cause a "P-delta" type of instability of the columns under sidesway. This may well have been the principal cause of damage of the CTV Building in the Christchurch Earthquake. This collapse concept is analyzed further in Section 3 of this submission.

1.9 Limited robustness and redundancy

The CTV Building did have a limited degree of robustness and redundancy, but this conclusion is based on different reasons than those set out in the H-S Report. The CTV Building was a one-way slab system, which is relatively unusual in the modern era of buildings. In two-way slab systems, the floor deflections are significantly smaller as the slabs are stiffer. But the integral (more robust and redundant) two-way slab-on-beam floor systems are also more expensive and slower to build. Cast-in-place concrete is normally used for the two-way systems—a slower, more labor-intensive construction method. In contrast, the CTV Building was in a different class of structural systems that consisted of mostly one-way frame and floor systems. The modern building era has adopted a variety of precast modular building systems.

As explained in section 1.1 above, the CTV Building survived its "design-event": the Darfield Earthquake. But for the structure of the CTV Building to survive a substantially larger earthquake, such as the Christchurch Earthquake, more robustness was necessary. One key item missing in the CTV Building was a series of N-S support beams between the columns. Such supports beams, although not a requirement of the design and building codes of the day, would have improved the diaphragm transfer mechanism and inhibited the possibility of the out-of-place buckling of the slabs along the E-W yield lines. This is discussed further in Section 3 of this submission.

2. Supplementary Investigation Work Conducted on the CTV Building Collapse

2.1 Ground motions

Surrounding the Christchurch CBD are four ground motion recording stations as part of the Geonet monitoring platform. These stations are:

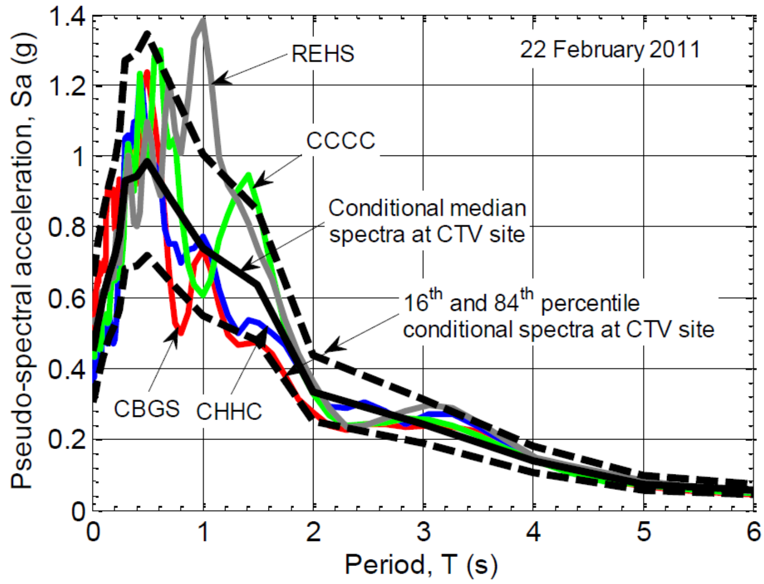
- CCCC at the Christchurch Catholic Cathedral College
- CBGS at the Botanical Gardens
- CHHC at Christchurch Hospital
- REHS at Resthaven Home

In the computational modeling using NLTHA, Compusoft used only the first three of this suite of stations; the REHS station located on Bealey Ave was not used, ostensibly because of soft soil deposits (possibly peat) near the surface. However, much of the CBD has pockets of such soils, and it seems premature to discount the REHS station for this reason alone.

It remains unknown as to what the ground motion was exactly like at the CTV Building site on 22 February, 2011 when the Christchurch Earthquake struck. The best practice is to analyze the CTV Building structure at the other recording stations, as if the CTV Building were located at those sites, then infer from the results any trends and likely outcomes that may have occurred at the CTV Building site on the corner of Madras and Cashel Streets. Naturally, the more results one can produce, the more robust the inferred outcome; greater confidence can then be placed in the resulting conclusions. It was therefore inappropriate to remove the REHS recording station *a priori;* the REHS station should remain as part of the four-station suite of earthquakes until such time that sufficient evidence is compiled to remove it.

Dr. Brendon Bradley was commissioned by Buddle Findlay to conduct an investigation on the statistical significance of either keeping or removing the REHS station from the suite of ground motions. In his report, Bradley presents a thorough analysis of the four-station suite of ground motions. As part of his analysis, Bradley employed empirical ground motion equations showing a range of results that may be expected at the CTV Building site. Bradley's empirically predicted range was then compared with the actual results from the four Geonet recording stations. With some minor exceptions, the range of results (within the probabilistically defined range for the CTV Building site) is well captured by all four ground motion stations, as shown in Figure 2.1 for the Christchurch Earthquake and the Darfield Earthquake.

(a)



(b)

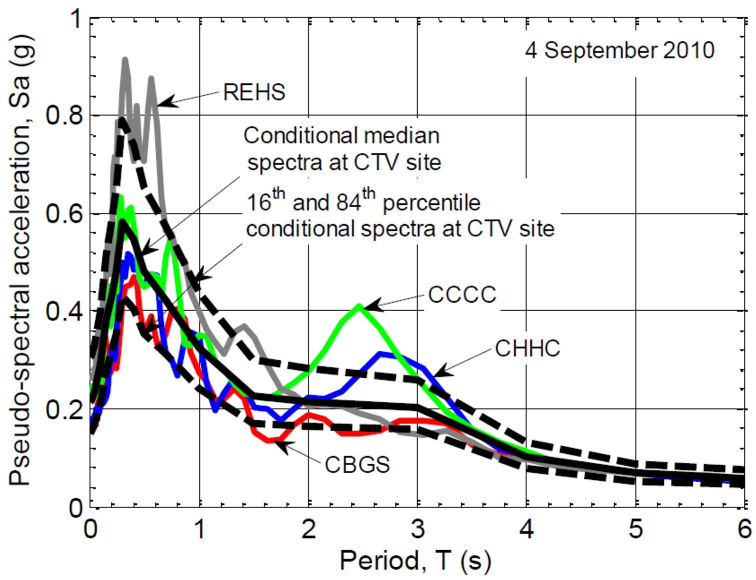


Figure 2.1. The range of seismic demands expected at the CTV Building site showing the 16th, 50th and 84th percent fractals for: (a) the Christchurch Earthquake; and (b) the Darfield Earthquake.

Although the CTV Building had a natural period of vibration in the order of T = 1.0 seconds, due to inelastic response the Compusoft results show that the CTV Building vibrated at about 1.5 seconds and 2.0 seconds for the E-W and N-S directions, respectively. Over this particular range, the four ground motions fall mostly within the 16 to 84^{th} percent fractals of probable behavior at the CTV Building site for both the Christchurch Earthquake and the Darfield Earthquake. Based on this analysis, there appears little reason to remove any one ground motion station from the suite.

Finally, Bradley went on to conclude that “the ground motions observed at the Christchurch

Police Station, Westpac building, and Pages Road Pumping Station (PRPC) are not considered appropriate for application at the CTV Building site."

Notwithstanding the above, ARCL installed a strong motion CUSP accelerograph that is compatible with the Geonet network at the CTV Building site. Several substantial ground motions of earthquakes up to magnitude 5.2 have been recorded.

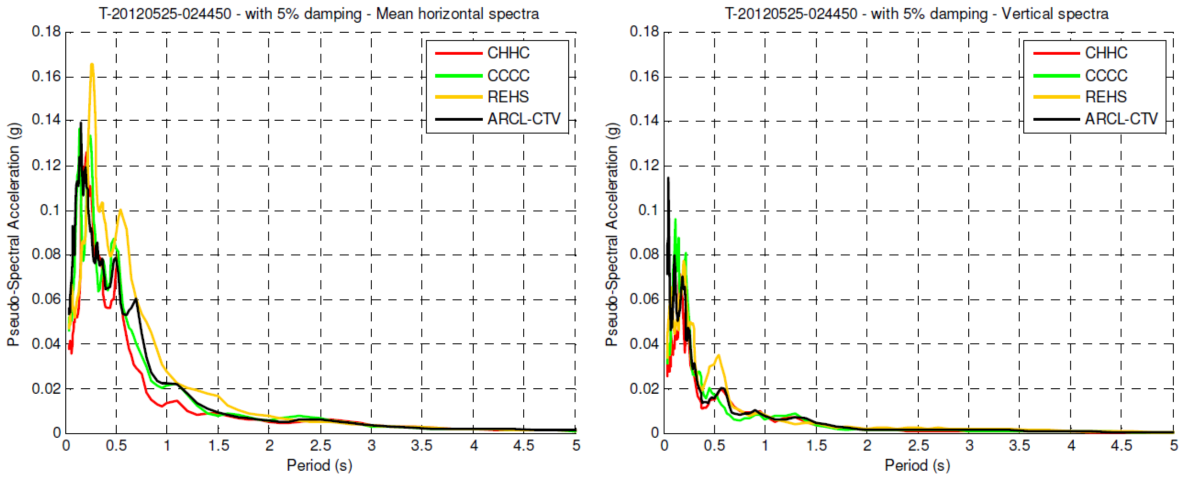
From these records the ground motions were analyzed by Dr. Geoffrey Rodgers. His work was commissioned by the author and Buddle Findlay on behalf of ARCL. Dr. Rodgers analyzed the elastic response spectra for the earthquakes with magnitudes greater than 4.5. In Figure 2.2, the results of the largest three earthquakes are compared with their companion Geonet records observed at the four CBD recording stations. It should be noted that for two of the three events presented there was one Geonet sensor inoperative—these results were not selectively removed.

As shown in Figure 2.2, the CTV Building motion generally lies in between the CCCC and REHS results. Perhaps on reflection, this similarity should not be surprising, as the CTV Building site lies midway between the CCCC and the REHS stations, forming almost a straight line-of-sight back to the epicentral region.

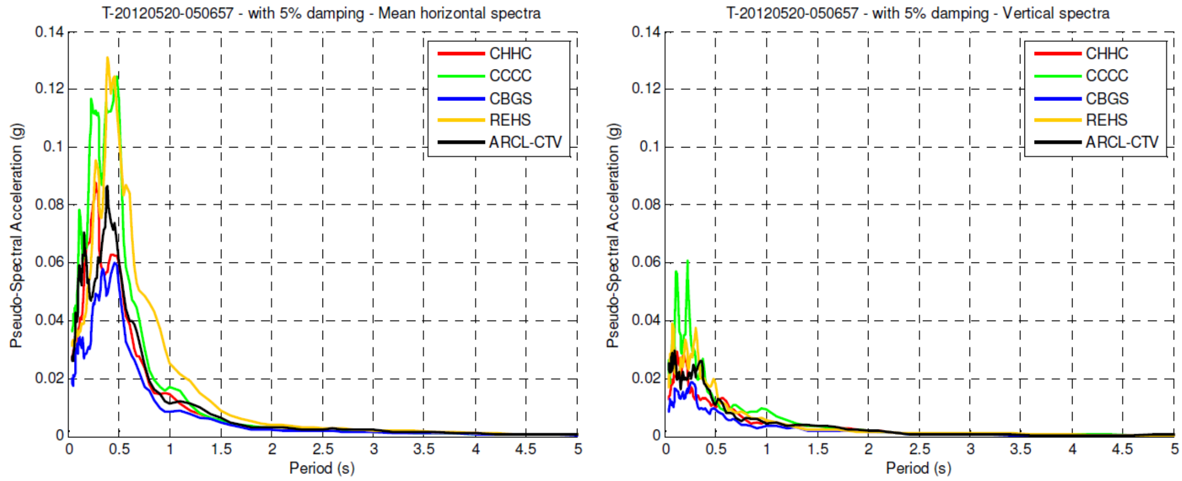
From the results in Figure 2.2, the following two key conclusions can be drawn:

(i) The three records that were used by Compusoft to analyze the likely response of the CTV Building should continue to be used. Further, the REHS station ground motion that was not used by Compusoft displays similar responses to the other three recording station (CHHC, CBGS and CCCC) ground motions that were used. This confirms Bradley's assertion that there was no valid reason to exclude the REHS station from the Christchurch CBD station suite; the REHS site should be included in the four-record suite for any future analyses.

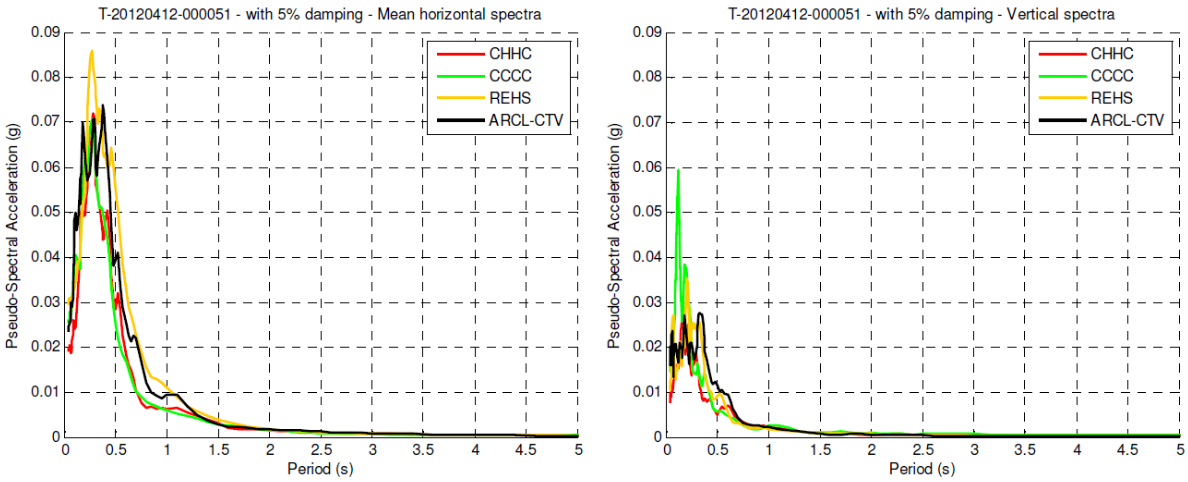
(ii) The CTV Building records are all evidently bounded by one or more of the other earthquakes in the four-station suite of CBD ground motions. This independently validates Bradley's class of conditional seismic demand envelop that also encompasses the CTV Building site.



(a) M5.2 26 May 2012



(b) M4.8, 21 May 2012



(c) M4.6 13 April 2012

Figure 2.2. Acceleration response spectra comparison at the CTV Building site with the other Geonet recording stations within the Christchurch CBD (when actively recording).

2.2 Concrete Testing

As discussed in section 1.4 of this submission, CTL tested eight 145 mm diameter core specimens, where the coring was performed parallel with the longitudinal axis of the column segments retrieved from the CTV Building after its collapse. The cores were sent to an independent concrete testing laboratory in the United States for compression strength testing by CTL, and the testing was conducted in accordance with international best practice.

The 145 mm specimens were further cored down to 99 mm to obtain standard test proportions. In ascending order, the results of the eight compression tests were as follows (in MPa units):

28.4	30.3	32.5	35.8	39.1	48.2	70.1	75.1

The above results were normalized and then plotted in the form of a cumulative distribution as shown in Figure 2.3. The process of the normalization is explained in the following.

First, it should be noted that the columns of the CTV Building had higher strength concrete in the lower stories. Specifically, levels 1 and 2 specified 35 MPa, level 3 specified 30 MPa concrete, while all other concrete in the CTV Building was specified to be 25 MPa, including the column concrete at levels 4, 5 and 6. It was not known where in the CTV Building most of the reclaimed columns were originally located.

It is reasonable and logical to assign the highest specified concrete strength (35 MPa) to the two highest test results, the intermediate strength (30 MPa) to the third ranked test result, and the most common concrete in the columns (25 MPa) to the five remaining test results. The results were then normalized using:

$$f'_c / f'_{c\ specified}$$

A cumulative distribution of these normalized results is plotted (with the staircase lines) in Figure 3. A two-parameter lognormal distribution has been fitted to these results using:

- median of $f'_c = 1.5 f'_{c\ specified}$
- lognormal standard deviation of $\beta = 0.23$.

The lognormal standard deviation is approximately equal to the Coefficient of Variation (**COV**) of a normal distribution. Good agreement between the observed test results (the staircase line, shown in red) and the empirically fitted distribution (via a standard equation that gives the smooth curve, shown in blue) is evident.

Comment: The staircase-type line (in red) exists because there are only eight samples. If there were many more test observations, and those results conformed to the above calculated modeling parameters, there would be many small steps in the staircase and the red line would tend toward the smooth mathematically modeled blue curve.

The general result that $f'_c = 1.5\, f'_{c\ specified}$ should be no surprise for two reasons. First, a well-known and common recommendation for evaluating the strength of existing aged concrete is to take the assumed strength at 50% above the specified strength. Second, the dispersion of the results (β = 0.23) is quite similar to that for concrete, where the COV can vary from 0.15 to 0.25.

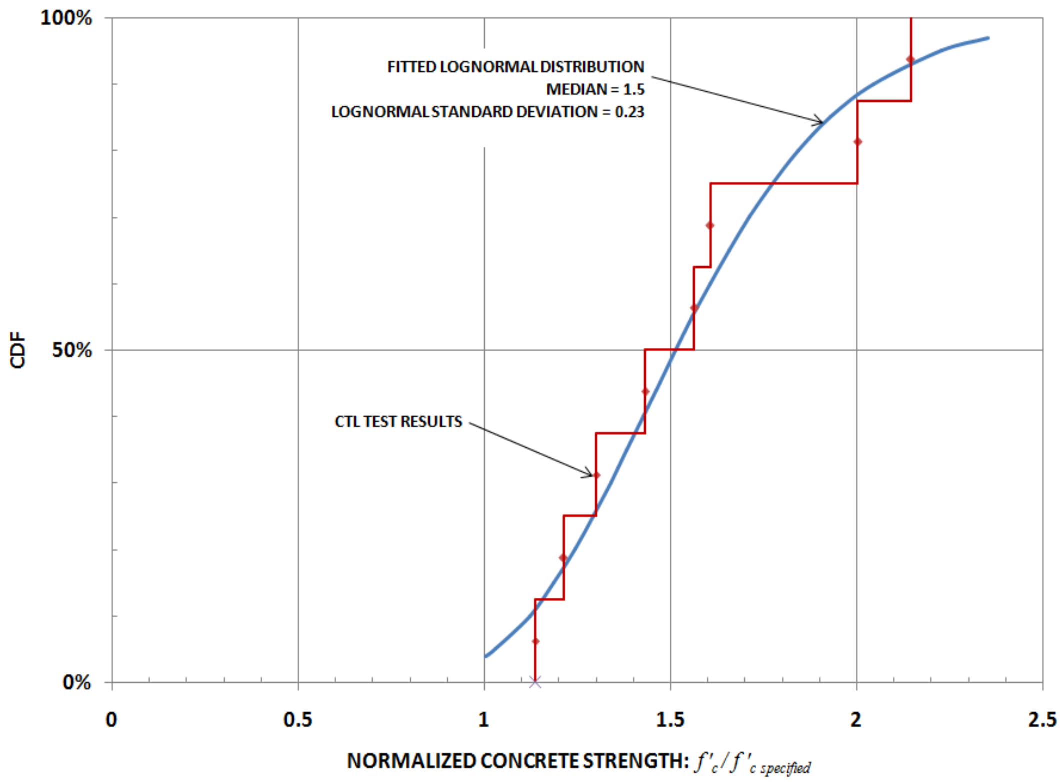


Figure 2.3. Cumulative distribution plot of the normalized concrete strength from the CTL test results. The eight test cylinder samples were cored from the longitudinal axis of the CTV Building columns.

Based on the evidence of the CTL physical test results, the comprehensive forensic tests of the concrete material and the above analysis, the following may be concluded for any future analyses, including NLTHA:

the in-situ strength for the CTV Building should be assumed to be:

$$f'_c = 1.5\ f'_{c\ specified}$$

When conducting an advanced analysis such as NLTHA, it is always prudent to perform a few "swing analyses" to examine the sensitivity of the overall outcomes to values adopted for certain key parameters. In the case of the CTV Building, the concrete strength is a very important parameter, largely because the columns are compression-critical. It is for this reason that the lower values previously used by Compusoft should be retained to model the extreme possibility of weaker concrete. The Compusoft analyses used concrete strengths amplified some 10% above the specified strength. With respect to the median concrete strength observed in the CTL tests, the Compusoft assumed concrete strengths fall approximately on the 10th percentile of the distribution (see the blue curve in Figure 3).

While it is reasonable to conduct a few analyses at the 10th percentile in the distribution of strength, it is considered to be entirely <u>*unreasonable*</u> to base all NLTHA and thereby the general conclusions on such a low level of material strength.

2.3 Additional Concrete Testing on CTV Building Columns

An inspection of the Burwood dump site revealed that there were several columns remaining from the CTV Building that were in relatively good condition. The columns were evidently from the 6th floor level, and thus would have had a specified strength of 25 MPa (see the discussion in section 2.2 above). As part of a more comprehensive forensic analysis on the CTV Building collapse it was considered essential that these columns be tested in a full-scale condition. Three specimens were retrieved by ARCL, and taken to the University of Canterbury structural laboratory for testing under concentric axial compression at seismic strain rates in the 10MN Dartec universal testing machine.

One purpose of this part of the investigation was to compare the results obtained from the CTV Building columns with similar well-known test results on unconfined and confined concrete columns in the 1980s. The earlier work has been reported in Mander (1983) and Mander, Priestley and Park (1988a,b). In those early University of Canterbury tests, Christchurch-sourced ready-mixed concrete and steel reinforcing materials were used, similar to the materials that were later used in the construction of the CTV Building. In the comparative test evaluation, the aim was to investigate whether any unusual surprises in performance existed—especially when tested under dynamic loading rates.

A second purpose of the full-scale testing was to investigate any size effect that may have been present. The so-called "size-effect" in concrete structures is based on the fact that when the size is increased by a factor of 4 (from the 100 mm diameter test cylinders, to the 400 mm diameter prototype column), the failure stresses are not the same. Empirical evidence shows that the larger scale leads to a smaller failure stress; in simple terms this reduction can be thought of as being akin to the weakest-link-in-the-chain theory. In the case of the University of Canterbury tests performed in the 1980s, the size effect was found to be a 15% reduction in capacity.

A third purpose of this testing was to examine the performance of concrete column elements that exhibited a poor post-collapse condition. The third specimen, yet to be tested, visually appears to be in poor condition; the concrete may have been damaged, either from the collapse or the fire. A photograph of the three specimens prior to testing is presented in Figure 4. The damaged central column shown in the photograph will be used to develop the third test specimen. It is expected that the performance may be inferior, but as to what degree it is not clear. The results of the third specimen test may give some insight into the low concrete strengths inferred as a result of the Opus testing.

Figure 2.4. The three column portions retrieved from the CTV Building used in the full-scale testing conducted at the University of Canterbury

Dr. Rajesh Dhakal was commissioned to conduct the testing in the Dartec universal testing machine at the structures laboratory at the University of Canterbury. At the time of writing this submission, the results of the final specimen test along with further analysis are not yet complete. Provisional results from the first two tests reported to date show the following:

- The concrete strength is above the specified value of f'_c = 25 MPa.
- There is a size-effect present. This may be in the order of $f'_{co} = 0.85 f'_c$, where:
    - f'_{co} = the in-situ strength of the full scale structural concrete; and
    - f'_c = the standard 100 mm x 200 mm test cylinder strength for the concrete taken from the same pour.

Finally, once the testing and analysis is complete, more definitive recommendations can be made on the precise concrete strengths to be used in any future NLTHA on the CTV Building.

2.4 Column Performance Analysis

Figure 5 presents the CTV Building under the sidesway motion effects of an earthquake. For the E-W components of ground motion, the CTV Building really exists in two parts: (i) the frame; and (ii) the shear wall systems which consist of the South shear-wall and the North Core. To view more simply how this dual system functions, Figure 5(a) shows an elevation of a typical interior frame connected through rigid links to the wall system. The links provide the in-plane connection that represents the floor diaphragm. The aim of the present analysis is to examine how half-high column components would function under the same type of lateral displacements, as shown in Figure 5(b).

Earthquake structural engineers commonly refer to the interstory displacements as the “drift” or more strictly the total story drift-angle $\theta_t = \Delta / H_s$, where:

- Δ = the lateral displacement of one floor with respect to the floor below; and
- H_s = the story height or the distance from a given floor to the floor level below.

As shown in Figure 2.5, during an earthquake, the drift on any one story in the frame (θ_t) is imposed by the displacement compatibility with the North Core (shear wall) and South shear wall of the CTV Building—both the frames and the walls are constrained to have the same drift angles.

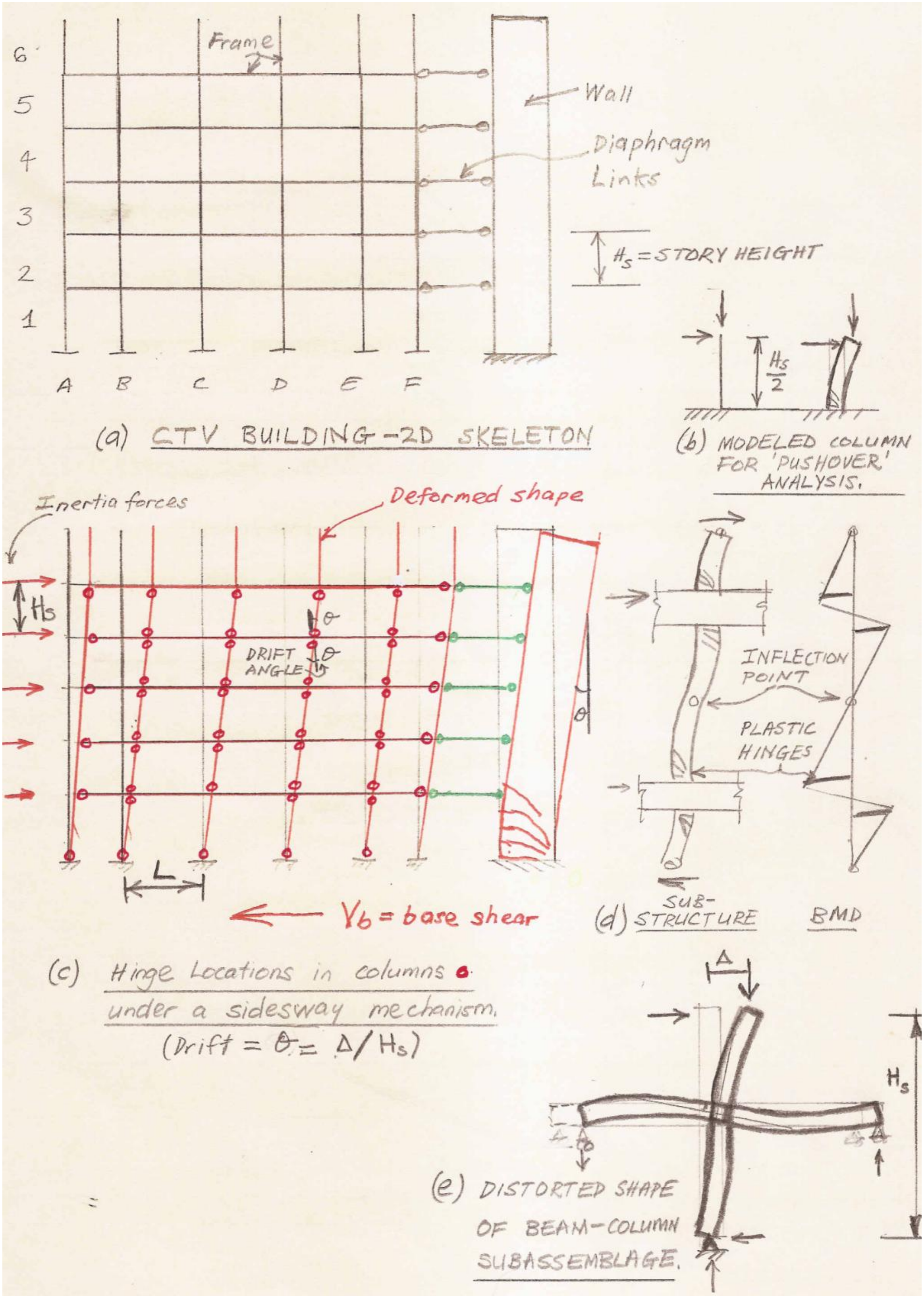


Figure 2.5. Initial seismic behavior of the CTV Building in a "sidesway mode"

The NLTHA results presented in the Compusoft report (refer to levels 2 to 5 in Figure G.6) show that the walls swayed laterally essentially at a constant *drift* angle with respect to the ground. It is for this reason that the performance of the critical components can be determined by seeking the critical columns in the structure through modeling the components as in Figure 5(b)—each column is constrained to have the same displacement field, only the axial load will change as the story changes.

The total drift angle (θ_t) on the system is composed of three components as follows:

$$\theta_t = \theta_c + \theta_b + \gamma_j$$

in which

θ_c = the rotational angle (drift) contribution of the columns;

θ_b = the rotational angle contribution of the beams; and

γ_j = the distortion angle of the beam-column joint region.

The interstory drift for each story level is essentially constant, thereby putting the columns into double *bending* with inflection points at mid-story height. Thus over the height of the walls, the subassemblage representations shown in Figure 2.5(b) and also Figure 2.5(d) and 2.5(e) are a reasonable approximation of general seismic performance.

For a subassemblage over the lower four stories of the CTV Building, an analysis shows that for the interior connections, the *columns* are weaker than the *beams*. Therefore, “pushover” analyses on a half-high column component (Figure 2.5(b)) have been conducted for the interior columns as single column components. The results are presented in Figure 2.6 and Figure 2.7.

Figure 2.6 presents results for the case where only normal gravity load exists on the columns; that is, there is no concurrent amplification from vertical earthquake motion effects. An analysis was conducted for each story using the two concrete strengths. In the left-hand column, results are presented assuming the specified concrete strength + 2.5 MPa. Recall that this is also the strength Compusoft assigned in their analyses. According to the more recent test results described in Section 1 above, the Compusoft assumed concrete strengths would fall on approximately the 10th percentile of the probable range, so there is a 90% chance that the concrete is stronger than Compusoft assumed.

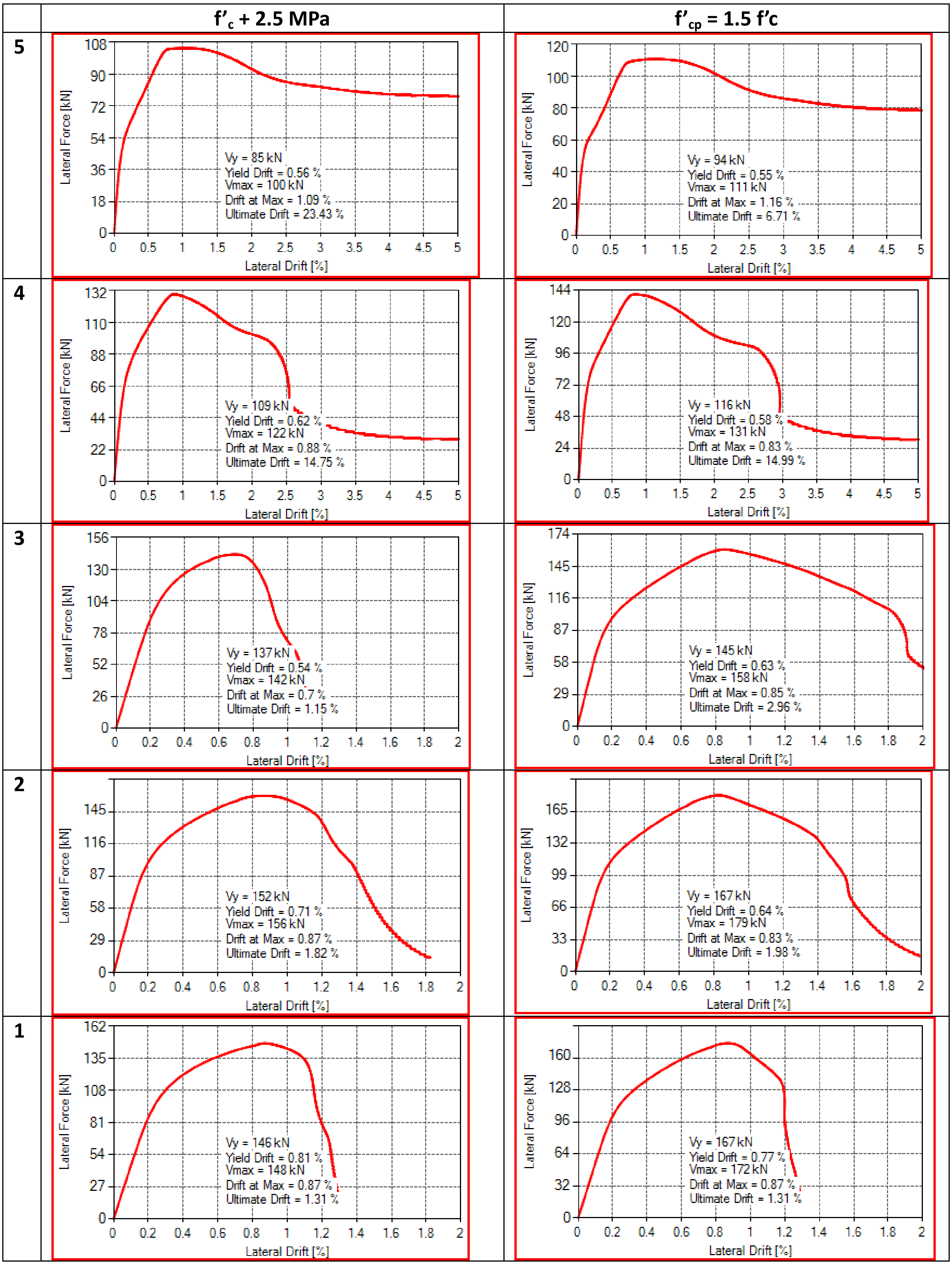


Figure 2.6. CTV Building column performance for <u>normal gravity axial load effects</u>.
(The analysis assumes double curvature implying a <u>mid-story inflection point</u> leading to the most adverse column shear force demand)

The Compusoft analysis case is therefore considered to be representative of a lower-bound strength condition. The right column presents the results for the median value concrete strength based on the CTL test results (50th percentile = 1.5 f'_c of the specified strength), and is considered to be more representative of the probable in-situ condition of the concrete.

From a general inspection of the results in Figure 2.6, it is evident that both the lateral load resistance (which is the same as the shear force in the column for the particular story) and the deformability of the frame (the drift capacity) improve as the concrete strength increases. An accurate definition of "drift capacity" is difficult to determine, but is generally considered to be when the structure has lost some ability to carry substantial lateral load. In lieu of a more precise definition, "drift capacity" is often taken as the drift when the post-peak lateral load (column shear) falls to 80% of the peak value.

Figure 2.6 also shows that in the lower stories the deformability of the structure becomes more restricted as the axial load increases with respect to the concrete strength. More specifically, the third floor appears the most critical in terms of strength and drift capacity. This will be examined below for the more conservative of the two cases (f'_c + 2.5 MPa).

Figure 2.7 presents results in a similar fashion to those shown in Figure 2.6, but with one key difference. The axial loads used in each analysis were taken as the *maximum* axial loads registered for the CCCC station ground motion NLTHA results for the Christchurch Earthquake, as depicted in Figures 52 to 55 of the Compusoft Report. During the Christchurch Earthquake, there were large amplifications of vertical (axial) load due to the *extremely high vertical ground accelerations*.

Because the frequency of the vertical components is some 5 times greater than the horizontal response (3 Hz for the floors-columns system vs. 0.67 Hz for the E-W frames), it is inevitable that the two displacement and force maxima will coincide momentarily, producing extremely high loading and stress demands on the materials. If the materials are overloaded, this means at least some partial damage (breakage) of the weakest of those vulnerable components.

When compared to Figure 2.6, the results in Figure 2.7 do indicate that the structure of the CTV Building was vulnerable to the vertical motions as a consequence of the extremely high dynamic axial load effects. It should be noted that this aspect of vertical horizontal load coupling was not correctly modeled in the Compusoft analyses. In its modeling, Compusoft reported the level of axial load and moment, but did not adjust the moment-axial load failure surface accordingly. It is for this reason that Figure 15 of the H-S Report displays inadmissible results. Numerous data points are plotted *outside* the failure surface—such performance is (theoretically) physically not possible. When the load path reaches the "failure surface" (think of this as a fence) something has to "fail", either:

(i) the steel yields; this happens for low levels of axial load in the upper stories under normal gravity load (less than 1100 kN in Figure 15 of the H-S Report); or

(ii) the concrete crushes; this occurs in the lower stories under normal gravity load, and also the mid-height stories under the high level of axial load caused by vertical acceleration effects (as for axial loads more than 1700 kN in Figure 15 of the H-S Report).

While the outcome in paragraph (i) is desirable, the outcome in paragraph (ii) in contrast may be catastrophic. This is especially true if the column is in an unconfined condition, as was the case for the CTV Building. Had the column axial load-moment interaction been modeled correctly, then many (yield or failure) points would be plotted on the outer curves, not beyond them. What this means is that the post-peak performance of the CTV Building, and the consequent redistribution of forces, has not been tracked correctly.

Therefore, the clearly demonstrated modeling inaccuracies of the failure criteria puts a large cloud of doubt over most of the results in the H-S Report, and in particular the interpretation of the results. This is particularly true for the lower two stories of the CTV Building, where the failure trigger may have initiated. Take for example the column at level 2, by considering the left-hand column graphs in both Figures 2.6 and 2.7. In both cases, the effect of the higher axial load was to reduce both the resistance force (roughly a 10% reduction in strength, and embrittle the column. Embrittlement here means that the column has less ductility or deformability capability, specifically a 50% reduction in drift capacity.

If there was a sway failure, such as that modeled in Figures 2.6 and 2.7, the structure of the CTV Building would have attained only modest drift and then collapsed; indeed a collapse following the Darfield Earthquake even would have been conceivable. Moreover, there would be supporting forensic evidence of observable damage amongst the rubble. One would expect to observe many columns, at least all of the interior columns of one story along lines 2 and 3, to fail in this way. The damage would propagate out from the joints and cover much of the length of the columns—yet this was not the failure model for the CTV Building collapse.

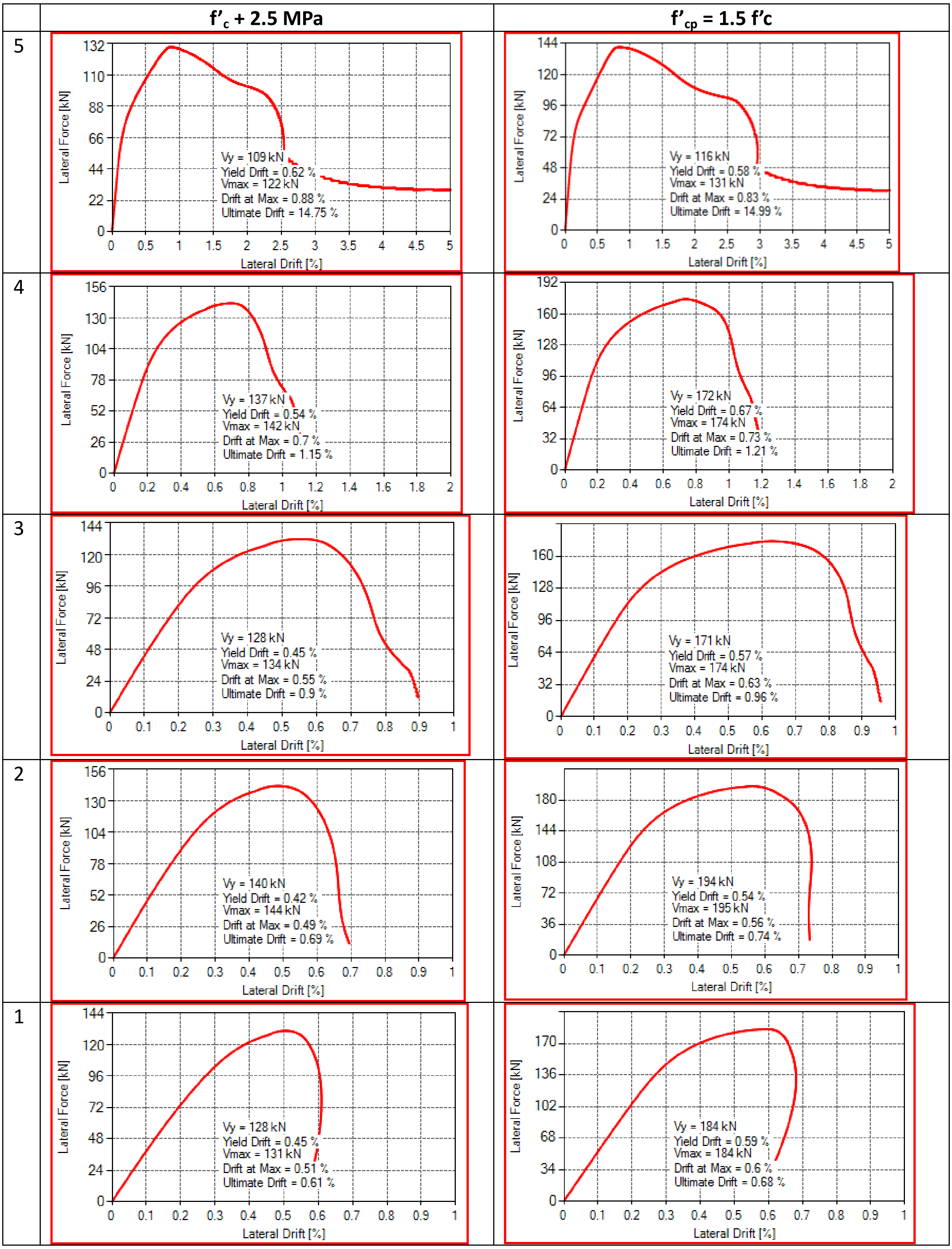


Figure 2.7. CTV Building column performance assuming <u>gravity axial plus seismic axial load effects</u>.

So what was the actual cause or trigger for the failure and eventual collapse of the CTV Building? Some clues, but not the complete answer, are given in the section 2.5 below on beam-column joints, and a full solution is postulated in Section 3 of this submission.

2.5 The Problem with the Beam-Column Joints

In the failure of structures, there exists a strength hierarchy, where the failure generally originates within the weakest link of the chain of resistances. In the context of a normal seismic design of frame structures, the strength hierarchy (from weakest to strongest) is normally:

(i) Beam bending (flexure). Beams are chosen to be the weakest link in a chain of resistance because in a building there are many plastic hinge regions at the ends of beams that serve as the hysteretic energy dissipation system under large sway reversals.

(ii) Column bending (also called column flexure). Columns are generally designed to be stronger than beams, and deliberate strength enhancement is typically 100% or more.

(iii) Joint shear. Joints are protected from failure by the presence of tightly wound spirals or closely spaced hoops.

(iv) Foundation capacity. The substructure is normally designed to be stronger than the superstructure, as damage is difficult to observe and/or repair when below ground.

In the case of the CTV Building, under an E-W sidesway analysis for the type of substructures presented in Figure 2.5 above, the strength hierarchy, (from weakest to strongest) is:

(i) Joint shear

(ii) Column flexure

(iii) Beam flexure

(iv) Wall Capacity

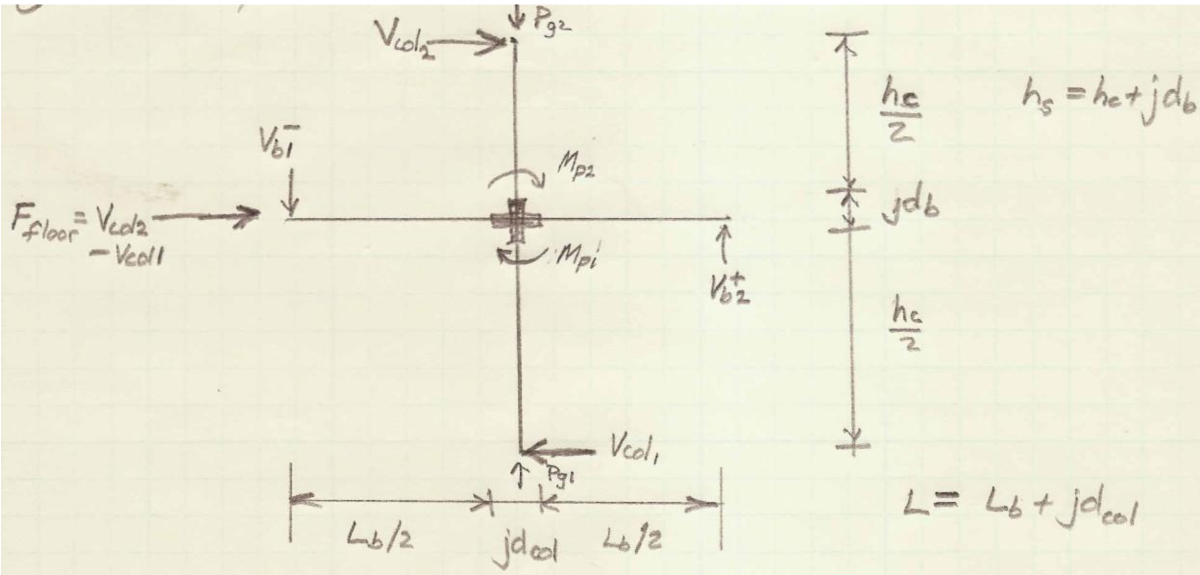
There are several reasons the beam-column joints in the CTV Building were the weakest, and thus most vulnerable, elements. First, the joints have a small cross-sectional area (note that the joint shear strength is proportional to the concrete area). Second, there was no transverse spiral reinforcement within the joints surrounding the longitudinal column bars; if present and closely spaced, such spirals can add substantially to the joint strength. And third, the shear force demands are significantly higher in the joint regions compared with the adjoining beams and columns.

The problematic high joint shear stress intensity is illustrated via the analysis presented in Figure 2.8. First, a subassemblage is extracted, as shown previously shown in Figure 2.5(e). The force actions at the inflection points (which are at the end of the members shown) are shown in Figure 2.8(a). Next, if the beams are removed, their effect must be replaced by applying equivalent beam-end forces arising from the loads carried by the reinforcing bars going into and out from the joint region, as depicted by Figure 2.8(b). The left-hand side of Figure 2.8(b) shows the bending moment diagram (**BMD**) for the column, including the effects through the joint region. By differentiating the column BMD over its member length, the shear force diagram (**SFD**) is derived as shown on the right hand side. V_{jh} is the (horizontal) shear force intensity through the joint; calculations show that this will be some 5.3 times greater than the column shear force, (V_{col}) for the CTV Building.

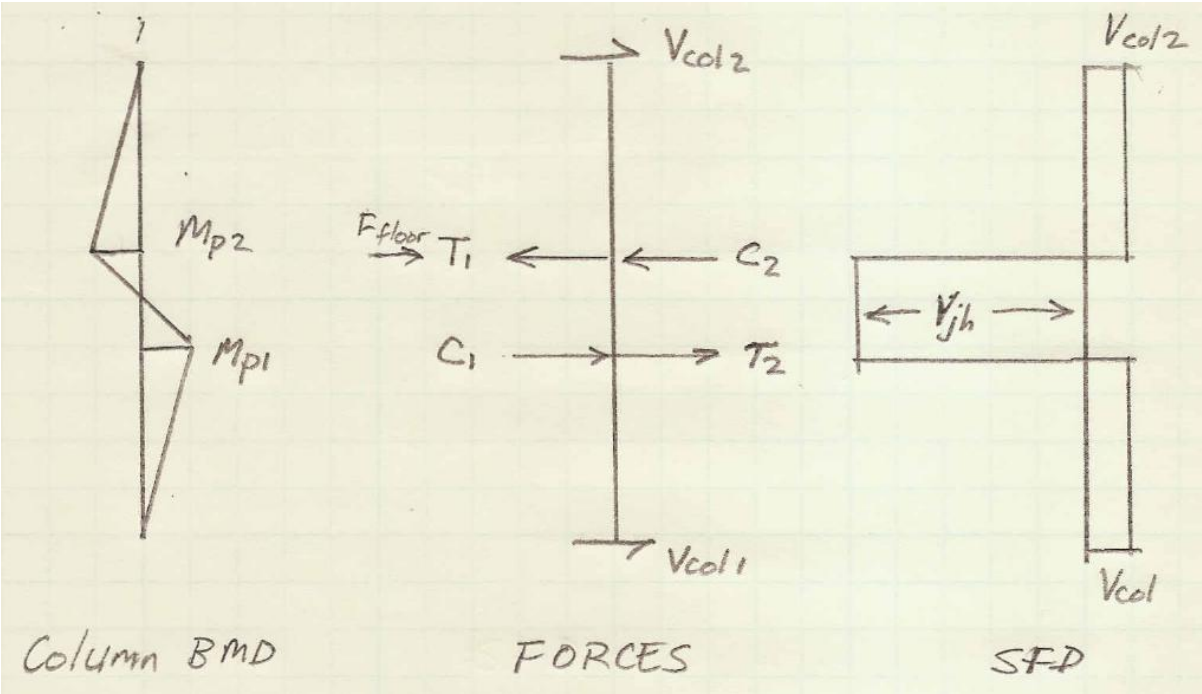
As the joints do not possess transverse reinforcement, the joint shear resistance is provided by a corner-to-corner arch (or strut) within the joint, as shown in the drawing of the beam-column joint in Figure 2.9. The magnitude of this joint-strut force cannot be resisted without the concrete within the joint becoming overstressed. Furthermore, reinforcing bars and concrete on the (opposite) off-diagonal of the joint are in tension, and this tensile action causes a progressive weakening of the compression resistance of the concrete within the strut.

On the first pulse of the Christchurch Earthquake, if the inertia forces pushed the CTV Building from left to right (as shown in Figure 2.9), the forces in the joint *may* have been resisted without too much damage to the concrete. But if the axial load in the columns is high, as it was in the lower stories of the CTV Building, at least some damage will be done. It is this damage, promoted by the tensions in the off-diagonal that leads to progressive “softening” or weakening of the concrete on subsequent cycles.

Calculations have been performed that show that the overall joint forces will restrict the potential input forces from the columns to about 70% of the potential maximum of that shown in Figure 2.7. Therefore the columns, apart from during the initial cycle, will remain mostly undamaged. Yet the condition of the beam-column joints will continue to deteriorate as the cycling progresses.



(a) A typical interior beam column joint subassemblage showing the seismic loading actions under the frame, sidesway from left to right.



(b) *The column extracted from the subassemblage in (a) above. Note: the beams have been removed, but the incoming and outgoing forces provided by the beam reinforcement are shown instead.*

Figure 2.8. An interior beam-column joint subassemblage

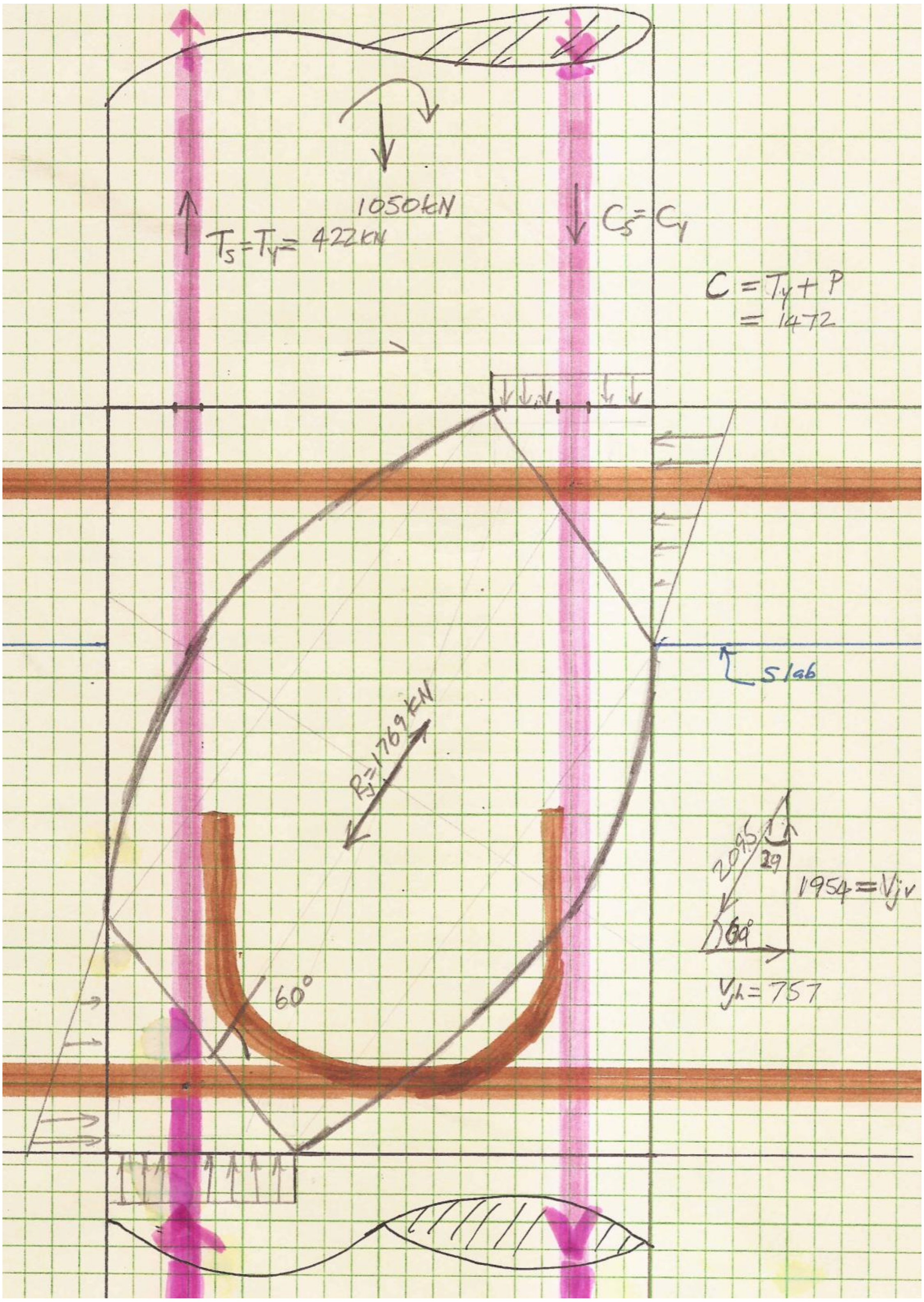


Figure 2.9. The beam column joint region

Shown are the vertical column steel (pink), the beam bars (brown) and the diagonal joint-strut. The concrete strut resists the column axial load plus the bending actions from the columns and beams.

The weaker joints in the CTV Building were a mixed blessing. The weaker joints actually will have acted like a fuse and therefore protected the columns from any further damage. However, over time the concrete will have worn down to the point where it could no longer sustain the axial load passing from the story above through the joint to the story below.

In spite of the deterioration in the beam-column joint zones, if the structure remains well tied together by the floor diaphragm, and also tied back to the shear wall system, the columns remain "trapped" and unable to fail due to a sidesway action. Of course the joints must continue to be capable of transmitting the vertical load. Providing the axial load path can be maintained and the joint concrete does not crush excessively, the joints continue function as a fuse. This initial phase of the partial failure, where the joint system acts as a fuse, is shown in Figure 2.10.

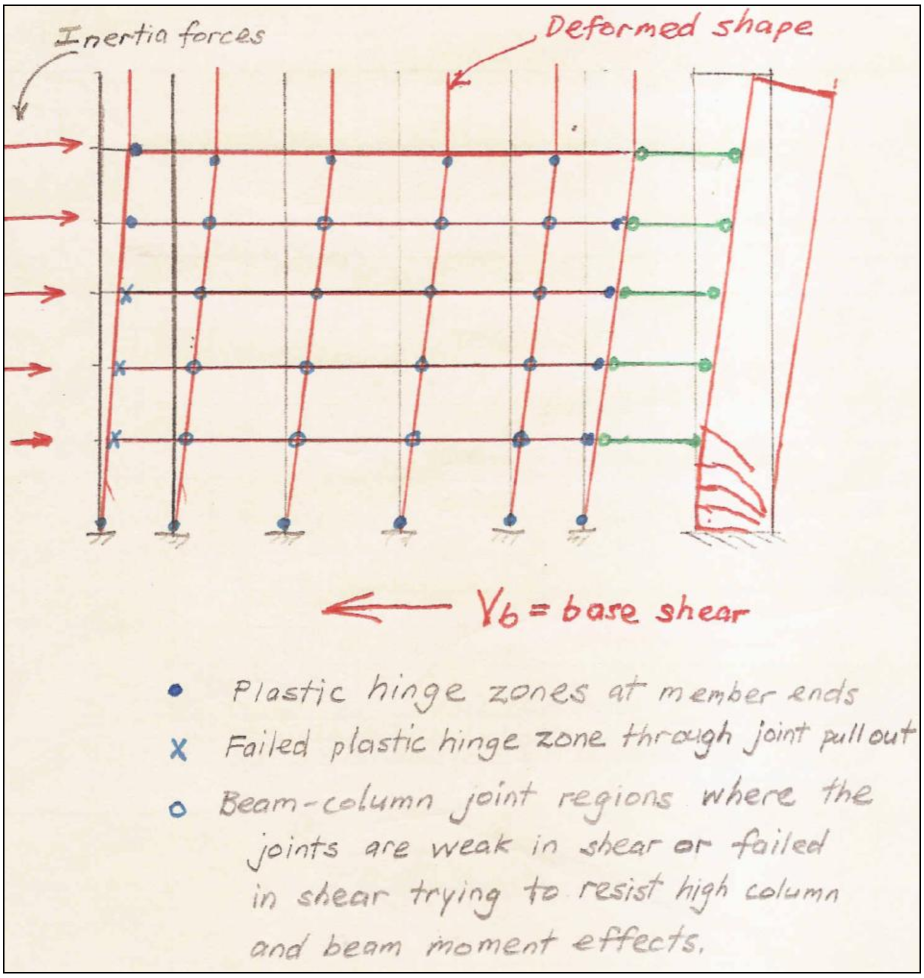


Figure 2.10. The modified sidesway mechanism arising from the presence of "weak" beam-column joint regions.

(Note the joints act like "fuses" and protect the columns from further deterioration.)

Based on an examination of all the beam-column joints in Figure 2.10, it may be noted that the *exterior* beam-column joints may have "failed". Failure of these connections is considered to be one of the primary triggers that "releases" the neighboring columns, giving them room to move laterally (sideways) at one floor level with respect to the floors above and below. It is hypothesized that this is the "trigger mechanism" that eventually led to the collapse of the CTV Building. But it should be noted that for such a failure to occur after the "trigger mechanism" has released the beam, no further external loads need be applied, instead the gravity load alone is sufficient to collapse the structure.

2.6 Expected Seismic Performance of an Exemplar Structure in the Christchurch Earthquake.

One might wonder how other buildings built in accordance with contemporary codes of practice perform in the Christchurch Earthquake. This topic has been investigated and recently reported at the 2012 New Zealand Society of Earthquake Engineering (Mander and Huang, 2012).

For many years senior undergraduate civil engineering students at the University of Canterbury have been taught the principles of design of multi-story reinforced concrete buildings, with a particular emphasis on seismic loading effects and the detailing of the reinforcement for ductility. The exemplar structure used as part of the educational process is the so-called "Redbook" building. This is a 10 story precast concrete structure, it could perhaps be considered a modern rendition of the CTV Building.

A comprehensive computational analysis was undertaken for 20 different strong earthquake ground motions whereby "incremental dynamic analyses" (**IDA**) were performed at increasing levels of seismic intensity until the structure "collapsed." The results were then characterized in a probabilistic sense so that the median response and the dispersion of the outcomes identified in a risk-based format similar to that described in Mander et al (2012). The computational analysis results of the general ability of the exemplar "Redbook" structure to strong earthquakes were then compared to the seismic demands imposed to similar structures in the Christchurch region. The outcomes were characterized in terms of a damage ratio with respect to the distance to the epicenter of the Christchurch Earthquake, that is the cost of repairs or replacement to that of a similar structure constructed under stable economic conditions prior to the earthquake.

The analysis was also expanded to investigate the ramifications of the likelihood of fatalities arising from a collapse and the expected downtime due to the earthquake-induced damage.

Additionally, several swing analyses were conducted to examine the sensitivity of the structural strength and reinforcing details on the general seismic performance. A summary of the certain key findings from the investigation are described below, full details may be found in Mander and Huang (2012).

Figure 2.11 presents a so-called Damage Attenuation relationship for the "Redbook" class of building to the Christchurch earthquake. The results from the advanced computational simulations are presented in a probabilistic fashion, so that an idea of the spread of potential outcomes can be viewed. It should be noted that one cannot be emphatic about a certain outcome as the results contain the uncertainties in the structural response, the uncertainties in the distribution of ground motions due to soil variability and the difference of the as-measured earthquake signatures at different sites based on actual Geonet data from the Christchurch earthquake. Also, the volatility in the cost of contracting and reconstruction after an earthquake is accounted for in the modeling. Therefore, there is considerable variation in the outcomes.

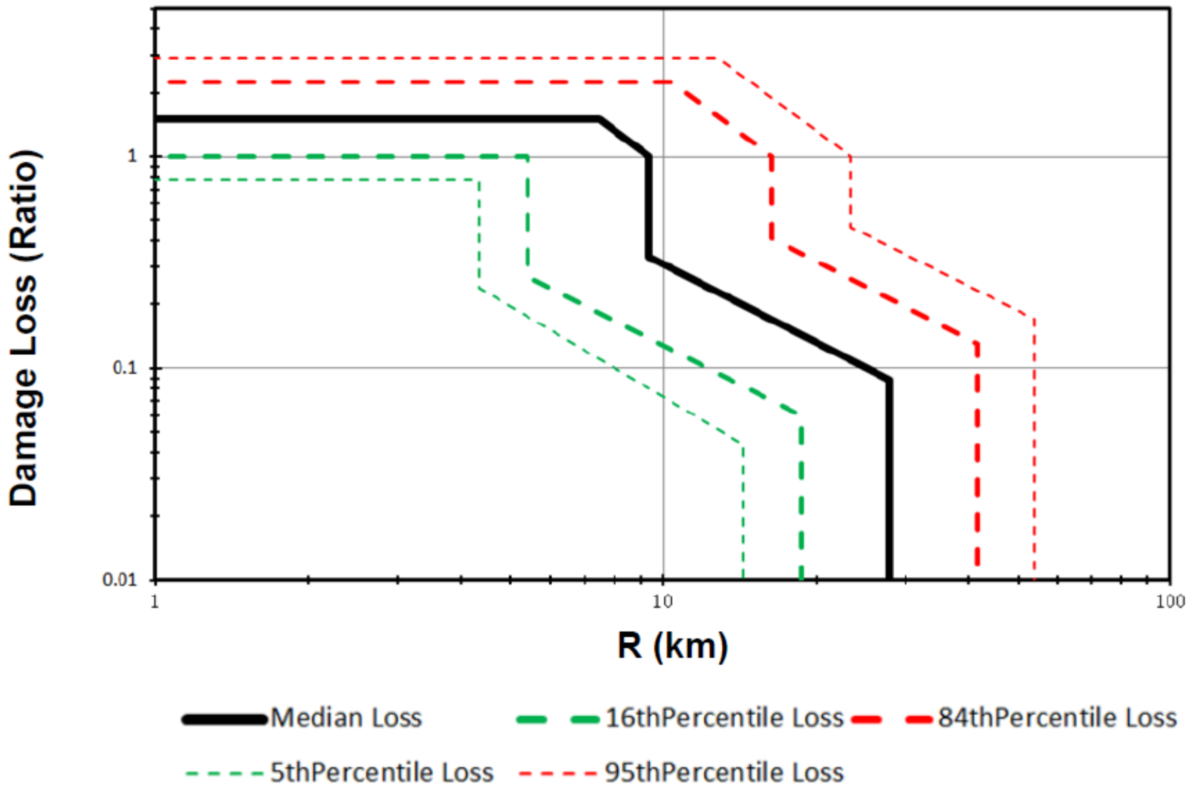


Figure 2.11. Damage loss attenuation of the Redbook Building for the Christchurch Earthquake. *(Note, the loss ratio is defined as the repair to replacement cost with respect to the construction cost prior the earthquake when stable economic conditions existed. R is the distance from the epicenter of the earthquake to the location of the building under consideration.)*

The extent of the CBD ranges from some 5 to 9 km from the epicenter. From an examination of Figure 2.11, it is evident that most structures (at least some 70%), particularly those closer than 10 km to the epicenter would not (theoretically) survive, and would require demolition and reconstruction. In fact there is already sufficient anecdotal evidence to support this analytical result. Thus in spite of modern buildings being constructed to “textbook” standards, they could not have been expected *a priori* to survive the Christchurch Earthquake.

Another question arising from this work is could one expect to see fatalities as a consequence of the damage arising from the ground shaking? Analysis results show (Mander and Huang, 2012), that deaths are not likely providing the structure conforms to the present day code-based design. However, loss to life and limb cannot be ruled out, and the modeling results show there is at about a 10% chance that if a structure collapsed occupants could be killed.

3. An Alternative Gravity-Dominated Collapse Scenario

3.1 Overview of alternative hypotheses

During the Canterbury earthquake sequence commencing with the Darfield Earthquake and leading up to the Christchurch Earthquake, substantial damage had already been inflicted upon structures throughout Canterbury. The CTV Building, in particular, had already met or exceeded the seismic design limits of it structural system. In the design of the CTV Building, the design engineer chose to transmit all seismic inertia forces accumulated by the mass distributed throughout the structure, back through the floor diaphragms to the shear walls, and then in turn to the foundations. The remainder of the structure was detailed principally for gravity loads, and a check was made that the principal gravity load bearing components (the columns) were not put under excessive sidesway displacements for the design seismic loads.

It is evident however that the first earthquake in the sequence, the Darfield Earthquake, exerted inertial loads that either met or exceeded the design expectation. The Darfield Earthquake, similar to the Christchurch Earthquake, also had very high vertical motion acceleration components. Historically by design, vertical accelerations have been expected to be about two-thirds of the horizontal acceleration components. This was roughly the case for the Darfield Earthquake, but only over a relatively narrow frequency band. For high vibration frequencies greater than about 3 Hz (period < 0.33 sec), the vertical acceleration components were exceptionally high (Sa [T < 0.3s] > 0.35 g)—considerably more than the normally expected two-thirds of the horizontal components.

These exceptionally high vertical accelerations tend to vibrate the vertical load bearing elements, such as columns and floor slabs. While the exceptionally high vertical motions were not the sole cause of failure, they certainly added considerably to the resulting damage. It is for this reason that people did not want to work in the CTV Building—they were uncomfortable with their work environment. The slabs in particular were evidently not behaving as they should have, by design. And it is for this reason that the CTV Building should have been Red-Stickered. The liveliness of the CTV Building was the primary evidence that the structure had damaged connections and that the CTV Building was ill-prepared to survive further shaking, in particular an earthquake that was greater than another design-level event.

As eyewitnesses from both inside and outside the building reported, when the Christchurch Earthquake struck, initially the CTV Building swayed violently in all directions. After several seconds of this violent shaking it seemed as though the structure had come to a rest, then collapsed (for example, **[WIT.WILLIAMS.0001.3, WIT.CAMMOCK.0001.5, WIT.LEE0001.4]**). Although the H-S Report is rather vague in its conclusions, it does elude to a collapse of the CTV Building initiated primarily from sidesway motions. The supporting Compusoft analysis was strictly unable to arrive at any other result, because the dynamic hysteretic moment-axial load interaction effects were not properly modeled in the Compusoft analysis. For example, the computational model simulations were unable to capture the possibility of a classic Euler buckling-type of columns failure due to column compression overload induced by the "exceptionally high" vertical vibrations. Furthermore, the connections between structural elements were modeled as rigid blocks. This is a customary approximation made in design-based simulations, but for a forensic analysis when demonstrable damage of the beam-column joints was clearly discernable, the assumed simplification was not sufficient.

In the remainder of this Section 3, alternative collapse hypotheses will be presented. Where appropriate, the hypotheses draw from the reported data in the H-S Report along with eyewitness evidence, to arrive at different conclusions. The analysis does not rely on the faulty assumptions inherent in the original H-S Report. The specific erroneous assumptions were that the concrete (as-built) was substandard and that the beam-columns joint zones were rigid. In contrast, the alternative collapse hypotheses use rational mechanics, supported by eyewitness statements, to deduce a type of behavior that conceivably occurred which led to the collapse of the CTV Building.

It should be noted that this collapse mode is not a radically new idea; Mr. Holmes points this out in his peer review of the H-S Report (BUI.MAD249.0372.9). Holmes also rightly points out the deficiency in the modeling of the joint strength and the dependence on sidesway as an explanation of the failure mode. He then goes on to propose that a collapse mode over more than one story was necessary for a collapse trigger mechanism to form. Holmes stops somewhat short of completing the solution, but it is considered that he was certainly heading in the correct direction.

Early in the Christchurch Earthquake, there was a substantial velocity pulse in the NE-SW direction. The velocity pulse was about 0.7 m/s. Due to its diagonal orientation with respect to the N-S facing building, this pulse would excite the structure of the CTV Building in both the E-W and N-S directions. The collapse mechanisms are the considered by decomposing the overall ground motion effects into each of the two orthogonal directions: E-W and N-S.

An alternative collapse hypothesis is first examined by considering the motions in the E-W direction, from which it is shown that previous damage along the West wall, as well as inadequate lock-in details of the E-W beams into their seats, led to the unseating of those beams along line A. This eventually led to a subsequent overload of the neighboring columns. Those neighboring columns would have been overloaded in axial compression, especially when considering concurrent vertical vibrations arising from the *exceptionally high* vertical accelerations.

The second part of the collapse hypothesis considers the motions in the N-S direction. The N-S direction has gathered much discussion by others, which all refers back to the perceived inadequacy of the drag-bars. The lack of, or failed drag bars would be affected by a northward pulse, where the inertial forces are directed south causing the floor diaphragm to pull away from the North core. It will be hypothesized however, that the opposite action is also likely—that under northward inertial forces the floors may "crumple up" (in technical terms, buckle downwards). This leaves sufficient movement room so floor slabs from one floor to the next can move such that the columns to take up a buckled shape over four-floor levels; when coupled with excessive vertical overload, buckling of the columns ensues, along with a global collapse mechanism.

There is a corollary of the abovementioned northward motion induced collapse. A similar collapse mechanism occurs due to southward movement of the floors. Because of the absence of drag bars in the lower stories, the floors are somewhat free to move away from the North core permitting a buckled shape to form.

It should be noted that in both cases the formation of the collapse mechanism is in three parts. First there must be an action that leads to a trigger, this leads to incipient failure or the first part of the failure, and finally there must be a statically admissible mechanism that can form that lead to the collapse mechanism proper.

3.2 Collapse Mechanism in under East-West Shaking

The Trigger

Figure 3.1 presents the sequence of events that led to the trigger action. According to the Compusoft results, the effects of the large velocity pulse as recorded at the CCCC station would be felt from about 4.5 to 6 seconds. Although the veracity of the Compusoft results are questionable for various reasons already stated, it serves as an interim indicator of the displacement demands experienced (refer to Compusoft, figure G.5). Here, interstory drifts of about 3% are indicated for all floors, or in other words a differential movement from one floor to the next (either above or below) of some 100 mm.

Sequence

Stage 1

The building sways to the west with a large velocity pulse. The E-W beams on column lines 2 and 3 at the West wall are required to form large negative moments that cause the joint core concrete and the beam-soffit cover concrete to crush.

Stage 2

During the next half-pulse the building lurches eastward. The beam along line 2 and 3 pull away from the west wall and their line A column seats to form the alternating positive moment. The crushed cover concrete from the previous reversal spalls off and the beam slumps down a little, with a partial or full loss of seating. Due to the loss of seating at the support line A there is a transfer of the previous gravity load from the tributary area of the beam onto the neighboring columns on line B. This action an axial force increase of up to 40% the columns along line B

Stage 3

As the building attempts to return to an upright condition by moving west, the unseated beams are inhibited from fully returning due to the presence of the west wall.

Sketches

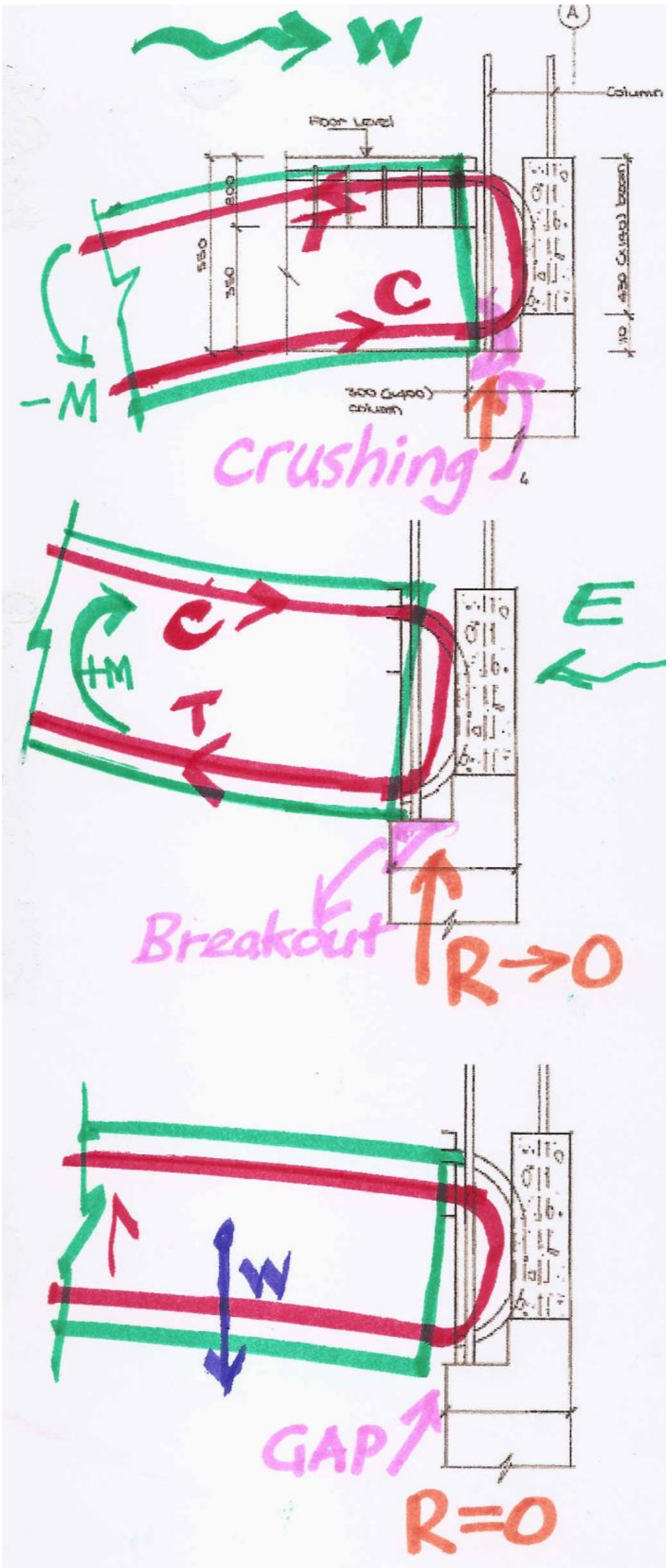


Figure 3.1. Beam-to-column connection failure sequence at the west wall; a trigger for the East-West Collapse Failure Mode

Stage 4

Permanent differential deformations remain, that inhibit the columns along line be from remaining straight. This sets the columns up for a classic Euler buckling type failure, especially under further axial load derived from vertical accelerations and their consequent vibrations

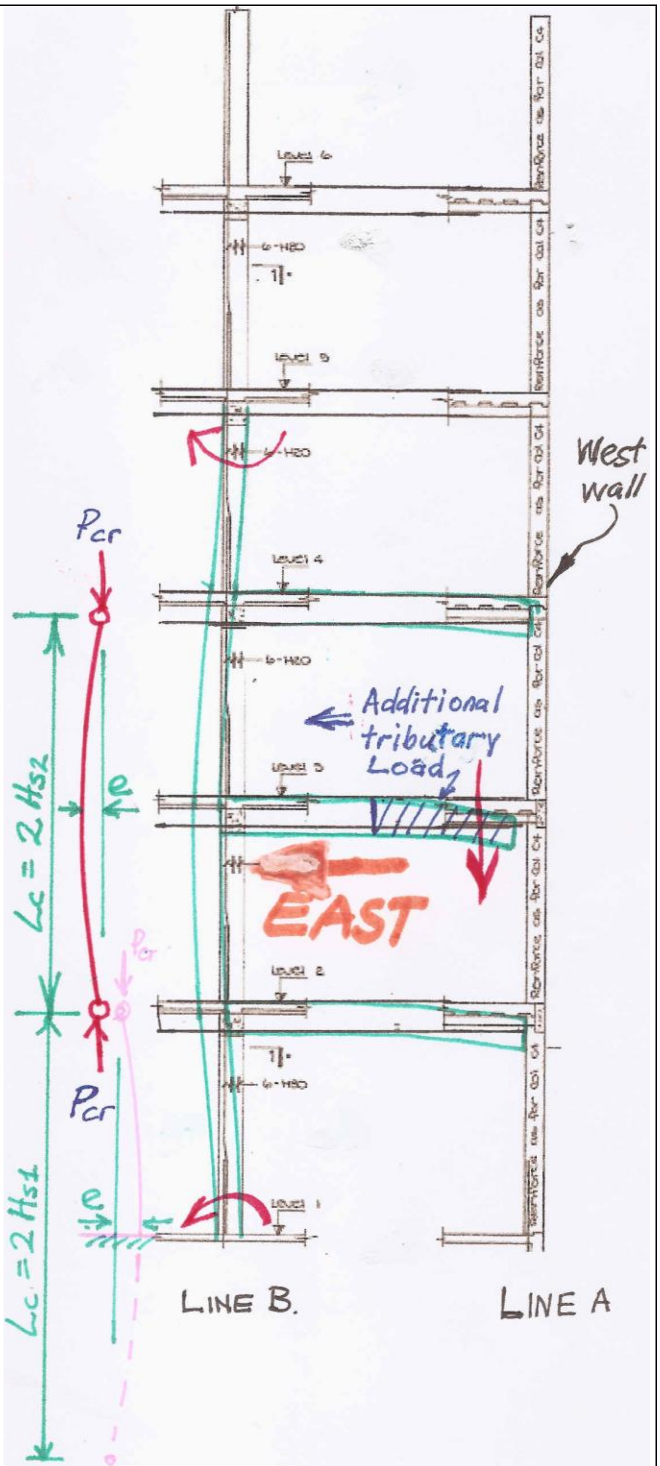


Figure 3.2. Four-story double bending buckling failure starting on column Line B Leading to the East-West Collapse Failure Mode

The effects of such movement lead to the initiating trigger action are shown in Stage 1 of Figure 3.1 where the E-W beams move eastward causing a large negative moment (tension in the top steel, compression at the beam soffit) to form. The concrete at the beam soffit would be expected to crush, as well as the weaker concrete into the joint. On motion reversal toward the east, any crushed/spalled concrete is expected to breakaway as shown in Stage 2 of Figure 3.1. The reaction on the soffit in turn vanishes.

In Stage 3 of the sequence, the reaction is instantly transmitted to the neighboring column on line B. It is estimated the when including vertical motion effects, there is a 400 kN increase in the axial load on the second storey level of columns. This effect leads to the formation of the incipient collapse mechanism as shown in Figure 3.2.

The Incipient Failure

Figure 3.2 presents the formation of the incipient failure mechanism. For this to occur there needs to be a relatively small “perturbation” or inherent fault. In this case a small differential displacement over the floor height suffices. The mechanism consists of a column under double bending over four-floors. This concurs with eyewitness reports (for example, **[WIT.SPENCER.0001.3, WIT.FORTUNE.0001.5, WIT.GUTTERIDGE.0001.3]**).

It should be noted that for this mechanism to form, the demands on the beam column joints are relatively modest. For example, there is no moment within the joints at level 2 and 4, while level 3 has high moment through the joint and essentially no moment. Calculations show that incipient collapse would occur once the differential story movement of 37 and 42 mm (equivalent to a single bending drift in this case 1.15% and 1.3%) for the cases of columns with concrete strengths of f’c+2.5MPa and 1.5f’c, respectively.

The Collapse Mechanism

If the collapse is initiated at the west wall, then it follows there is an eastward failure from moving from column lines B to F. This is explains why more debris fell near the Madras Street corner of the building. The collapse mechanism is presented in Figure 3.3. It should be noted that the mechanism once fully formed, will push the walls out first at level 3 at the east end along column line F, then secondly at level 2 the lower column will blow out due to the now very large lateral displacements in the columns. This is consistent with eyewitness observations.

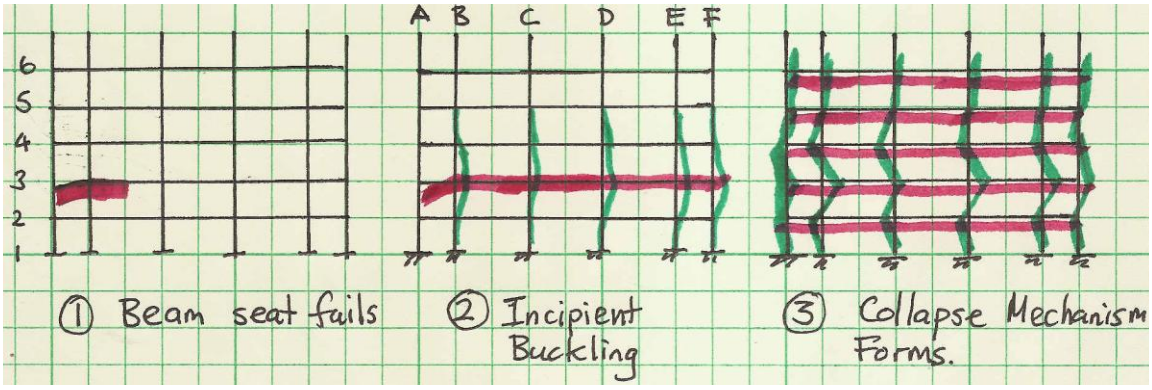


Figure 3.3. Formation of the E-W Collapse Mechanism

3.3 Collapse Mechanism under North-South Shaking.

3.3.1 Northward Mechanism

The Trigger

Figure 3.4 presents the sequence of events that led to the trigger action. From Figure G.3 of the Compusoft results, the effects of the large velocity pulse as recorded at the CCCC station would be felt from about 4.5 to 6 seconds where interstory drifts in the range of 2.3 to 2.5% are reported. Coupled with this are substantial vertical vibrations in the slab arising from vertical ground motion. Given the pre-existing damage that was evidently observed by eyewitnesses due to the liveliness of the CTV Building, it is possible that much of the metal tray-deck had de-bonded, with the floor slab going into catenary action. The vertical vibrations in the Christchurch earthquake would have caused further damage. And along with inertia forces in the northerly direction the combined effect led to the downward shape buckle (or folded plate), as depicted by the red curves in Figure 3.4.

The Incipient Failure

The folded plate action would provide sufficient movement for the columns at levels 2, 3 and 4 to also translate northward, permitting a double bending buckled shape to be set up over the lower four stories. This is shown by the green curve in Figure 3.4. The calculations are similar to those in the E-W mechanism described above, except the extra beam weight is not added. Calculations show that incipient collapse would occur once the differential story movement of about 1.2% interstory drift occurs.

<u>The Northward Collapse Mechanism and it Southward Corollary</u>

If the collapse mechanism is initiated, it would be most likely along column lines 3 at possibly rows C or D. Once these columns collapse downward they release load which in turn must be carried by their neighbors. Consequently, the surrounding columns are also over loaded, bringing the entire structure down. It should be emphasized that the main reason this mechanism can occur is because the building possessed only one-way slabs that were beamless in the N-S direction. The diaphragm stiffness was consequently low, thus the slabs had a high propensity to out-of-plane buckling due to in-plane seismic loads. Again, this downward out-of-plane slab-buckling was exacerbated by the *exceptionally high vertical accelerations.*

3.3.2 Southward Mechanism

<u>The Trigger;</u>

There is a corollary to the above described northward motion-induced buckled plate/column collapse mechanism. Suppose a large pulse acts in the northerly direction, inertia forces act southward and the floor slabs are dragged away from the wall. Irrespective of the merits of whether the drag bars had sufficient capacity to restrain these inertia forces, the fact remains that there were no drag bars in the lower stories. Such lack of restraint permits the lower level floors to move relatively freely southward, especially at the eastern side of the building where there was a frame, but no wall (as on the west side) that would otherwise provide some additional restraint.

<u>The Incipient Failure</u>

As the columns on lines 2 and 3 are free to move, they will form a buckled shape, as shown in the green line in the lower diagram of Figure 3.4.

<u>The Southward Collapse Mechanism</u>

The structural columns were the most heavily loaded along column lines 2 and 3. Once one or more of these columns become overloaded and tend to collapse downward, the loads they previously carried needed to be transmitted to neighboring columns, which in turn become overloaded. Once several columns are overloaded, a general buckling of all columns along a line develops, bringing the entire structure down. The relative lateral movement, initiated by the pullout of the wall anchorage led to a general buckling mode of failure, this would exacerbated by the very high horizontal accelerations.

SEQUENCE | **SKETCHES**

Step 1

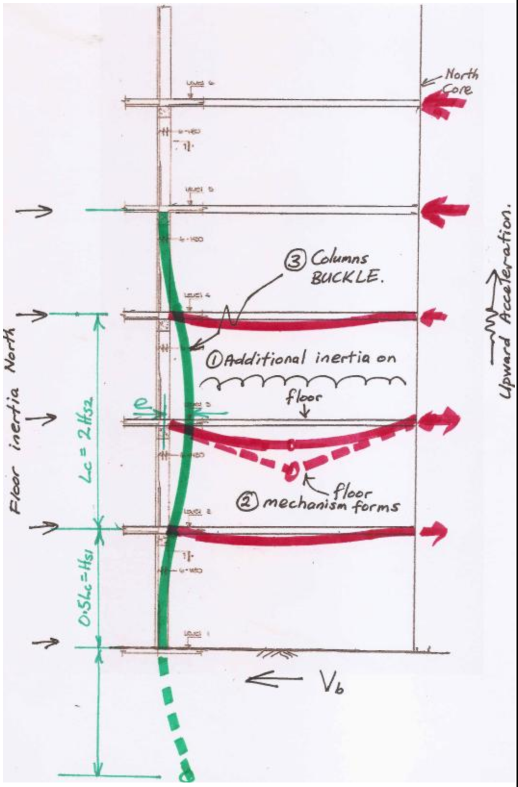
Due to the lack of beams in the N-S direction and very high vertical motions, the in-plane stiffness is low. The slab buckles downward due to a combination of upward vertical acceleration, and N-S sideway of the frame.

Step 2

Because the slab buckles, and the columns lack lateral support in the N-S direction, a 4-storey, double bending column buckling mechanism forms.

Step 3

Column buckles and collapses.



Northward Collapse Mechanism

Step 1

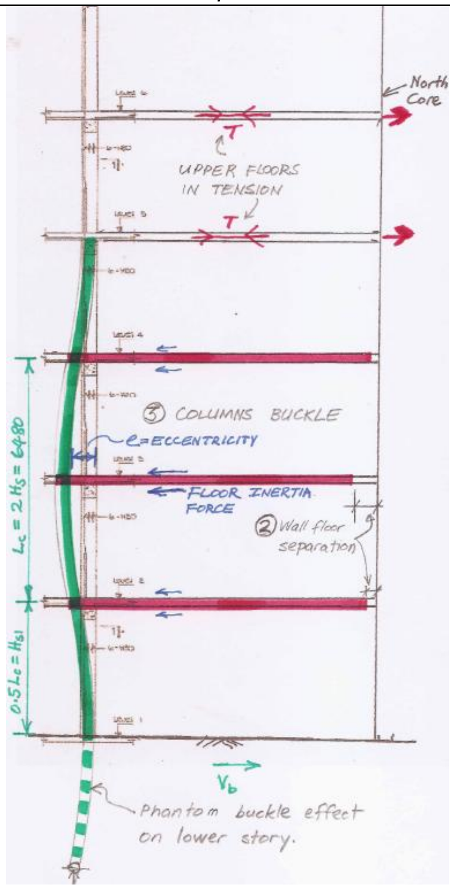
Due to the absence of drag bars in the lower stories, there is a large strain demand placed on the slab steel connecting with the North core. After one or two cycles the bars fracture due to low cycle fatigue.

Step 2

The columns on lines 2 and 3 lack lateral restraint from moving independently southward, therefore they move away from the north core. A double-bending column buckling mechanism forms in the lower four stories.

Step 3

Several columns buckle and the structure collapses downward.



Southward Collapse Mechanism

Figure 3.4. North-South Collapse Failure Modes

3.3.3 Summary remarks on failure modes

There have been three general failure modes postulated above.

- A four-story double bending buckling failure starting on column Line B leading to the E-W collapse failure mode
- A northerly motion induced collapse failure mode
- A southerly motion induced collapse failure mode

What is common amongst all three failure modes is they require the same class of buckled columns over the lower four stories. In fact it is conceivable that a combination of these modes could coexist under a torsional (twisting) action of the structure. The failure modes that led to the general collapse of the structure are consistent with eyewitness statements. Because it was the lower four stories that collapsed, the only people who survived the collapse were those in the upper two stories on levels 5 or 6 of the CTV Building.

4. CONCLUSIONS

Based on the points raised and the analysis presented in this submission, the following conclusions are drawn:

4.1 The CTV Building was designed and constructed in an innovative fashion. This structure was one of the first in a new generation of multistory buildings in the 1980s that used precast components. Instead of using a ductile moment frame as had been the custom for cast-in-place structures of the day, the CTV Building was designed with a "strong" wall system coupled with an "elastic" frame of columns and beams to support a proprietary type of floor system composed of a lightly reinforced slab cast on galvanized steel metal-rib decking. The Building was designed to the NZS 4203 Loadings Code, and a deflection check was made to ensure the displacements under the code-specified seismic loading were not excessive and that the columns remained within the elastic range.

4.2 When the Darfield Earthquake struck, it imposed ground accelerations that were essentially similar to the design limits for which the structure of the CTV Building had been designed. As a consequence, the structure was damaged; such damage would be expected, by design. The structure did not collapse, and met its design objective of ensuring life-safety.

4.3 In light of the possibility of a large aftershock, and given the fact the engineers knew many structures around Christchurch had either met or exceeded their design expectations, they strictly should have been immediately Red Stickered by fiat; a site inspection was not even necessary to make this decision. Following this period, such buildings should have been both inspected and analyzed for collapse potential in subsequent earthquakes. If necessary, gravity critical structures (such as the CTV Building) should have been shored up to ensure collapse prevention while valuables could have been retrieved and repair/retrofits implemented.

4.4 The CTV Building was inspected after the Darfield Earthquake and damage noted and the building deemed safe to reoccupy. However, the owners/engineers evidently did not pay heed to the many reports from the CTV Building occupants that the building felt uncomfortably lively. Further questions should have been raised regarding the soundness of the structure by the owners and thoroughly investigated by the assigned inspecting engineers.

4.5 The CTV Building tragically collapsed in the Christchurch Earthquake with a significant loss of life. An investigation into the collapse by the DBH led to the H-S Report. This report has been discussed and critiqued in this submission. There are several assumptions and various aspects of the H-S Report that bring into question the veracity of the claims and conclusions. In fact the peer reviewer Holmes, as well as the DBH expert advisor Priestley, are not in agreement with key aspects of the report. It is for this reason further work is essential.

4.6 One of the key areas leading to faulty conclusions in the H-S Report concerns the concrete strength. Testing and analysis commissioned by ACRL, and undertaken by independent experts, demonstrated that the concrete was not deficient as claimed in the H-S Report. In fact the concrete strength is likely to be in the range of 1.5 times the specified design strength.

4.7 Another key area of deficiency in the analysis is the correct modeling of the columns, coupled with the degrading strength of the beam-column joints. Axial load-moment interaction was not correctly considered in the NLTHA. Also, the beam-column joints that had no transverse reinforcement were modeled as rigid end blocks. As such the strength deterioration that occurs when the joint core concrete cracks was not modeled.

4.8 Further nonlinear time history analysis is needed to fully understand the nature and causes of the collapse of the CTV Building. In those analyses it will be essential that all <u>four</u> Geonet motions recorded during the Christchurch Earthquake are included in order to correctly gauge the spread of results that might have conceivably happened at the CTV Building site on February 22, 2011. Moreover, it is essential that the effect of the weakened structure following the Darfield earthquake be captured. This is most easily done via an end-on-end analysis, where the damage done in the Darfield Earthquake is captured. In previous analyses detailed in the H-S Report on the work performed by Compusoft, the program was stopped at the completion of the Darfield Earthquake and then restarted as if the structure was undamaged at the commencement of the Christchurch Earthquake.

4.9 Analyses as part of this submission show that a sway failure is unlikely, and that a classic elastic Euler buckling failure over the lower four stories is possible in either the E-W or the N-S directions. Such a failure does not rely on significant, if any, post-elastic performance. The lower four stories were able to buckle due to the relative movement of the floors with respect to the shear wall system, and the relative movement necessary to achieve this need only be small, in the order of 30 mm. The collapse is primarily caused by the substantial increase in axial loads in the columns due to the exceptionally high vertical accelerations.

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Damage, Death and Downtime Risk Attenuation in the 2011 Christchurch Earthquake

J.B. Mander & Y. Huang

Zachry Department of Civil Engineering, Texas A&M University,

College Station, TX 77843 USA.

2012 NZSEE
Conference

ABSTRACT: Following a catastrophic earthquake, 3d (damage, death and downtime) rapid estimates of the extent and severity of losses are desperately needed in order to better aid in post-event response and recovery. A quantitative risk analysis approach is advanced to investigate the 3d loss types for typical New Zealand construction as exemplified by the Redbook building for the 2011 Christchurch earthquake. The results are presented in terms of attenuation curves which show that within the central business district: the expected loss ratio (related to insurance claims) is about 50% of the asset value; the expected chance that someone is killed is 5%; and the expected downtime is 1-year. However, considerable uncertainties also exist, thus one can only be 90% confident that these results will not be higher than: 100% damage, implying collapse is a distinct possibility; 10% chance of fatalities, implying there may also be some deaths and possibly significant injuries; and 2-year downtime due to reconstruction demand surge. These salutary results demonstrate that in spite of structures being well-engineered, *downtime* in particular is unacceptably large. Two methods can be used to solve this problem: to make the structure slightly stronger and more robust through damage avoidance design.

1 INTRODUCTION

Historically, engineers have principally aimed at ensuring life-safety through collapse-prevention due to earthquakes. However, over many years of research and more recently in practice, partial damage, death and downtime losses are gradually being taken into consideration as they turn out to contribute a significant portion of the overall earthquake-induced losses, especially when one considers equivalent monetary value. In this research, the general four-step quantitative risk analysis approach of Mander and Sircar (2011) is extended from the general "all-hazard" based loss model to an earthquake-specific or "scenario-based" 3d loss model.

The well-known "Redbook Building" is considered to be an exemplar of the state-of-the-practice for good structural design in New Zealand (CCANZ, 1998). This building, which is based on using reduced design strength along with ductile detailing, is adopted as the basis for conducting a 3d seismic risk analysis. It will be shown that significant damage can be expected when subjected to the Christchurch earthquakes. Some deaths may also be expected. But it is the downtime that is most worrying aspect, as this is proving to be most undesirable from a building owner's standpoint. Therefore, three alternative design scenarios are explored and discussed. The first of these is to merely make the building stronger to defray the onset of damage. The second maintains the existing strength, but employs Damage Avoidance Design (DAD) details. While both of these aforementioned strategies are shown to make some improvements in performance along with the reduction in damage, it is finally demonstrated that what is really needed are buildings that are both stronger and have better (damage avoidance) performance attributes.

2 "SCENARIO-BASED" 3d LOSS MODEL

Recently, Mander and Sircar (2011) developed a quantitative risk analysis approach for the loss estimation of structures. The approach develops an equation-based inter-relationship using a four-step procedure between (a) seismic hazard; (b) structural response; (c) damage; and (d) loss. In this research, the "all-hazard" based analysis of Mander and Sircar (2011) is extended to a "scenario-based" risk analysis method. It is well known that for any given specific earthquake there exists an "attenuation relationship" that relates the shaking intensity with respect to the distance to the earthquake's epicentre. Such an attenuation relationship can be substituted into the Mander and Sircar (2011) theory. As shown in Figure 1, the "scenario-based" four-step risk analysis consists of: (a) hazard-intensity attenuation modelling; (b) structural analysis; (c) damage analysis; and (d) loss-attenuation estimation.

Figure 1 shows that each of the four steps are essentially linear when plotted in log-log space. Therefore, a single compound equation may be written to express the interconnection between each of the four graphs.

$$\frac{L}{L_r} = \left|\frac{\theta}{\theta_r}\right|^{c} = \left|\frac{S_a}{S_{ar}}\right|^{bc} = \left|\frac{R}{R_r}\right|^{-abc} \tag{1}$$

in which r = a reference (scenario) earthquake event; L = a loss ratio; θ = structure drift, an engineering demand parameter (EDP); S_a = spectral acceleration, an intensity measure (IM); R = radial distance from the epicentre of the earthquake; and a, b, c and d are the slopes shown in the four graphs in Figure 1, these are interrelated such that $d = -abc$.

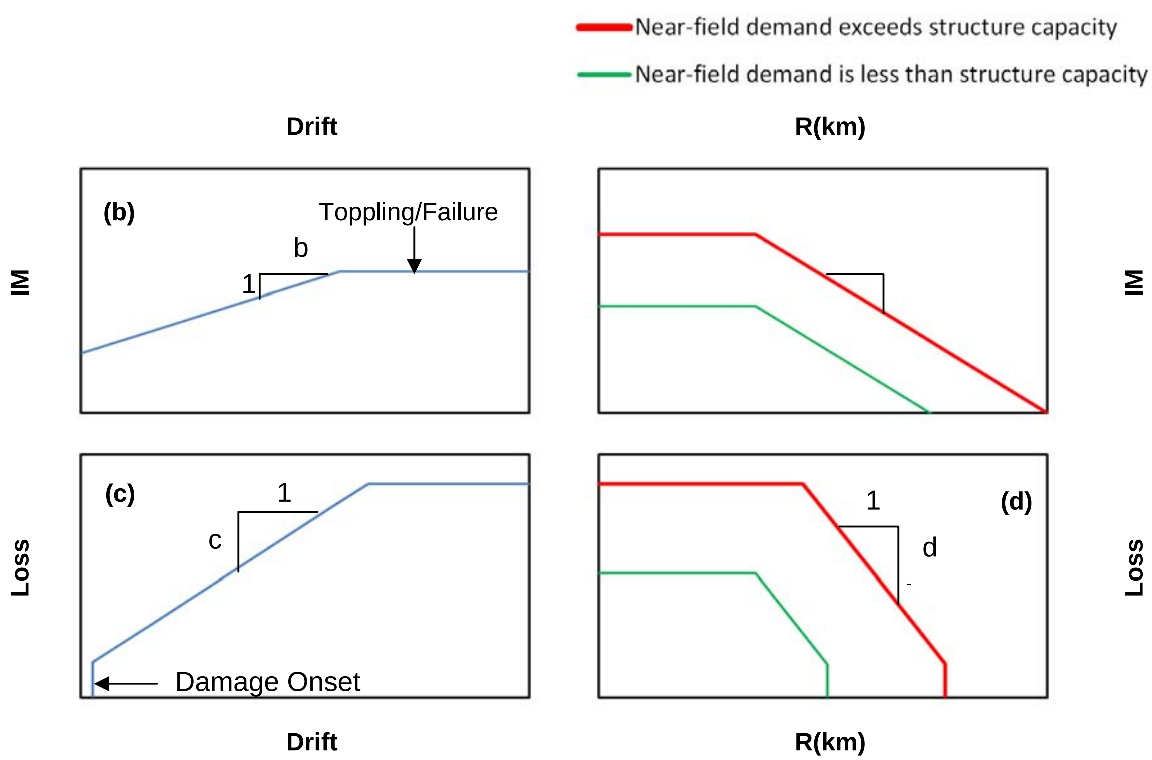

Figure 1: "Scenario-based" 3d Loss Model: (a) seismic hazard intensity-attenuation model; (b) structural analysis; (c) damage analysis; and (d) loss-attenuation estimation

In the first step, instead of associating the intensity measures with annual frequency as Mander and Sircar (2011) proposed, a simple attenuation relationship is proposed that relates the intensity measure with the radial distance from the earthquake epicentre. In the second step, the response of the structure which is exposed to different levels of ground shaking is predicted. In the third step, the structural drift is related to the damage losses. In the fourth step, damage or other losses (such as death and downtime) at a certain radial distance from the earthquake epicentre are modelled.

It should be noted that earthquakes in the near-field (approximately < 8 km) may impose seismic demands that are either greater than the capacity of the structure, or less than the structural toppling/failure capacity, as depicted in Figure 1. This input (in Figure 1a) affects the outcome, as shown in Figure 1d.

2.1 **"Scenario-based" Physical Loss Model**

The model for calculating the physical damage loss ratio is:

$$\frac{L}{L_c} = \left|\frac{\theta}{\theta_c}\right|^c ; L_{on} \le L \le L_u = 1.3 \tag{2}$$

where L_c = unit loss, which taken as the monetary value of the structure under steady-state (non-disaster) pricing, or as L_c = 1.0 for comparative (unit-price) studies; θ_c = structure drift at the onset of complete collapse; L_{on} = physical loss ratio at the onset of damage state 2; θ_{on} = structure drift at the onset of damage 2; and L_u = physical loss ratio at the complete collapse of structure, $L_u > 1$ with a suggested median value $L_u = 1.3L_c$ presumes a 30% post-event price surge.

2.2 **"Scenario-based" Death Loss Model**

Fault and Event trees are used to analyse the probability of death loss arising from building damage due to the earthquake. The value of the slope in the damage analysis was calibrated using the fault and event trees, and is taken as c = 2.6 as proposed in Ghorawat (2011). The model for calculating the probability of death loss is:

$$\frac{DL}{DL_c} = \left|\frac{\theta}{\theta_c}\right|^c ; DL_{on} \le DL \le DL_u = 0.75 \tag{3}$$

where DL_c = probability of death loss at the onset of complete damage, which is generally taken as DL_c = 0.1 (Mander and Elms 1994); DL_{on} = probability of death loss at the onset of damage; DL_u = probability of death loss of complete damage, which is normally taken as DL_u = 0.75, assuming the maximum occupancy of a building is 75%.

2.3 **"Scenario-based" Downtime Loss Model**

Guided by earlier studies (Mander and Basoz, 1999; and Ghorawat, 2011) a downtime loss model is proposed as

$$\frac{DT}{DT_c} = \left|\frac{\theta}{\theta_c}\right|^c ; DT_{on} \le DT \le DT_u = 150 \tag{4}$$

where DT_c = the downtime at the onset of complete damage; DT_{on} = the downtime at the onset of damage; and DT_u = the downtime at complete damage, which is taken as DT_u = 150 (weeks).

2.4 **Uncertainty and Randomness**

Consider the uncertainty and randomness in the seismic demand and in the establishment of the model, β is used to represent the dispersions through which the median values could be transformed to other fractiles. The dispersion of all combined uncertainty and randomness β_T is given (Kennedy et al. 1980) using the following expression:

$$\beta_T = \sqrt{\beta_D^2 + \beta_U^2 + \beta_C^2} \tag{5}$$

where β_D accounts for the variabilities in demand; β_U accounts for the uncertainty in modelling, which is taken as β_U = 0.25; and β_C accounts for the variabilities in structure capacity, which is taken herein as β_C = 0.2. Further, to account for the overall dispersion in loss with respect to the radial distance from the epicentre, it can be shown that:

$$\beta_{L|R} = \sqrt{\beta_{UL}^2 + c^2 \cdot \beta_T^2} \tag{6}$$

where β_{UL} = 0.35 is assumed to account for the uncertainty in loss (Mander and Sircar, 2011).

3 3d LOSS MODEL CONSIDERING SPATIAL DISTRIBUTION OF LOSSES

A commonly adopted conservative assumption is that the damage of a building is uniformly distributed over the entire height. Thus a "Maximum Loss Model" is defined as:

$$\frac{L_{\max}}{L_c} = \max\left(\left|\frac{\theta_i}{\theta_c}\right|^c\right) \tag{7}$$

$$\left|\frac{\theta_{\max}}{\theta_c}\right| = \left|\frac{L_{\max}}{L_c}\right|^{1/c} \tag{8}$$

where L_{max} = maximum 3d loss ratio; θ_{max} = maximum structure drift in the structure; and θ_i = structure drift of the i^{th} storey.

As buildings become taller, the conservative assumption that the maximum loss is spread equally over all storeys can lead to a substantial overestimation of total loss. This is because the most severe damage tends to be concentrated within a few floors, typically the lower storeys. Deshmukh (2011) developed a method to address this issue and proposed an "Average Loss Model". This requires the calculation and summation of the loss at each storey of the building and then averaged over the entire height of the building, as follows:

$$\frac{L_{avg}}{L_c} = \frac{\sum_{i=1}^{n} \theta_i^c}{n \cdot \theta_c^c} \tag{9}$$

$$\theta_{avg} = \left(\sum_{i=1}^{n} \frac{\theta_i^c}{n}\right)^{1/c} ; \theta_{\max} \le \theta_c \tag{10}$$

where L_{avg} = average 3d loss ratio (for physical damage loss, $L_{avg} \le$ 1.0; for death loss, $DL_{avg} \le$ 0.1; and for downtime loss, $DT_{avg} \le$ 75); n = total number of storeys of the building; and θ_{avg} = average structure drift in the structure.

In reality, neither the maximum loss model, nor the average loss model will hold universally true for all potential earthquake shaking intensities. For example, under stronger shaking if only one storey is near collapse, then insurers will condemn the entire structure in spite of most other storeys being in pristine condition. This is a case where building replacement is necessary and thus the maximum loss model is applicable.

Therefore, a proposed new loss model is developed by combining the maximum and the average loss models to give a composite conditional loss model. For physical *damage* loss, the proposed model is expressed in terms of whether the building is repaired or replaced as follows:

- The building is *repaired* when:

$$L_{eff} = L_{avg} \quad (L_{on} \le L_{\max} < 1.0) \tag{11}$$

- The building is *replaced* when:

$$L_{eff} = L_{\max} \quad (1.0 \le L_{\max} \le L_u) \tag{12}$$

where L_{eff} = the effective physical damage loss ratio for the proposed physical damage loss model.

Similarly, for the *death* loss, the proposed model is expressed as:

$$DL_{eff} = DL_{avg} \quad (DL_{on} \le DL_{\max} < 0.1) \tag{13}$$

$$DL_{eff} = DL_{\max} \quad (0.1 \le DL_{\max} \le DL_u) \tag{14}$$

where DL_{eff} = the effective probability of death loss for the proposed death loss model.

And for the *downtime* loss, the proposed model is expressed as:

$$DT_{eff} = DT_{avg} \quad (DT_{on} \le DT_{\max} < 75) \tag{15}$$

$$DT_{eff} = DT_{\max} \quad (75 \le DT_{\max} \le DT_u) \tag{16}$$

where DT_{eff} = the effective downtime loss for the proposed downtime loss model.

Other variables to represent the key coordinates in the proposed loss model are calculated by

$$R_{rr} = R_r \left| \frac{L_{rr}}{\overline{L}_r} \right|^{1/d} \tag{17}$$

$$L_{rl} = \overline{L}_r \left| \frac{R_{rr}}{R_r} \right|^{d} \tag{18}$$

where R_{rr} = radial distance from the epicentre of the earthquake corresponding to the 3d loss ratio L_{rr} (for physical damage loss, L_{rr} = 1.0; for death loss, DL_{rr} = 0.1; for downtime loss, DT_{rr} = 75), note that this distance separates repair and replacement outcomes; L_{rl} = 3d loss ratio of the average loss model corresponding to the radial distance from the epicentre of the earthquake R_{rr}.

4 RESULTS AND IMPLICATIONS

From the "scenario-based" 3d loss models, the losses at a certain radial distance from the earthquake epicentre can be easily determined. The well-known 10-storey reinforced concrete "Redbook Building" (CCANZ 1998) was selected as the exemplar structure in this research. The "scenario-based" 3d loss models were implemented for the "Redbook Building" based on the 22 February 2011 Christchurch earthquake. Compared to the maximum loss model, the estimated 3d losses based on the "scenario-based" 3d loss models, considering spatial distribution of damage losses over the height of the buildings, are considerably smaller.

The 3d loss model results show that for the standard "Redbook Building", within the central business district taken as some 17 km radial distance away from the earthquake epicentre: (i) the expected physical damage loss ratio is about 50% of the asset value; (ii) the expected probability of killing someone is about 3%; and (iii) the expected downtime is 6 months. Considering the randomness and uncertainties, one can have 90% confidence that the losses will not be higher than: (i) 100% physical damage loss ratio; (ii) 5% of probability of death loss; and (iii) 1-year of downtime loss.

When considering the spatial distribution of losses, the analysis results show that for the standard "Redbook Building", at a 17 km radial distance from the earthquake epicentre: (i) the physical damage loss is about 20%; (ii) the probability of death loss is only 0.7%; and (iii) the downtime loss is about 2 months. Compared to the maximum loss model, it is observed that the estimated losses are significantly reduce when considering spatial distribution of damage over the height of the structure.

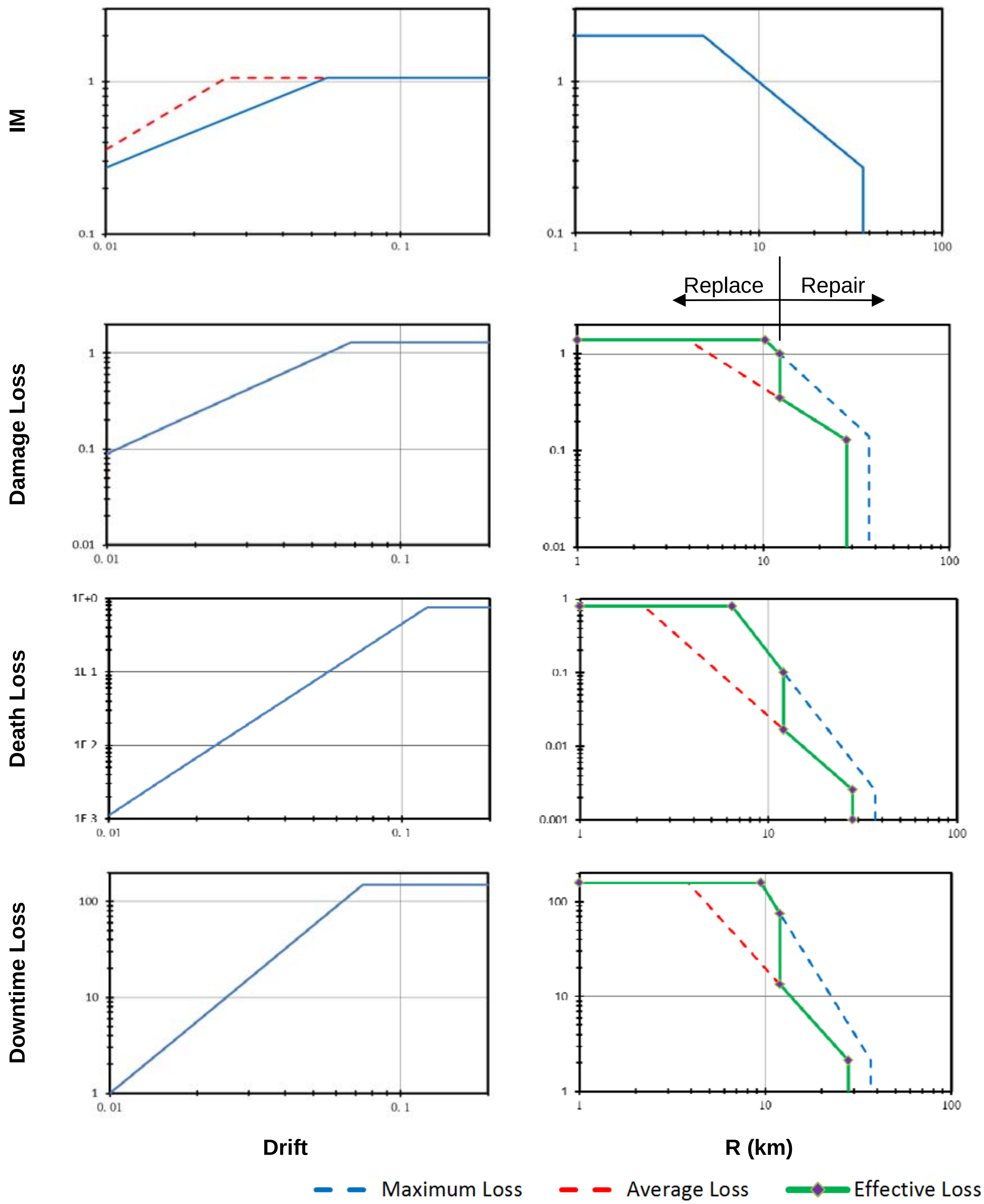

Figure 2: Proposed 3d loss model for standard "Redbook Building"

5 DISCUSSION: WHAT CAN BE DONE TO AMELIORATE LOSSES?

In monetary terms it has been shown that the *downtime* loss is the most significant loss compared to the physical *damage* loss and *death* loss (Ghorawat, 2011). It is therefore necessary to take downtime loss more seriously into consideration in the pre-event analysis and design. To investigate how one might ameliorate losses, and in particular minimize downtime by design, a sensitivity (swing) analysis has been performed. The Cases considered consist of:

(a) Making the building 30% stronger (this degree of strengthening is in keeping with the proposed increase in the seismic hazard for Christchurch);

(b) Making the building more deformable and as damage-free as practicable, without making the building stronger. Using the Damage Avoidance Design (DAD) armouring details along with re-centering attributes as proposed in various recent studies by Rodgers et al. (2008, 2012) and Solberg et al (2008).

(c) A combination of Cases (a) and (b) above.

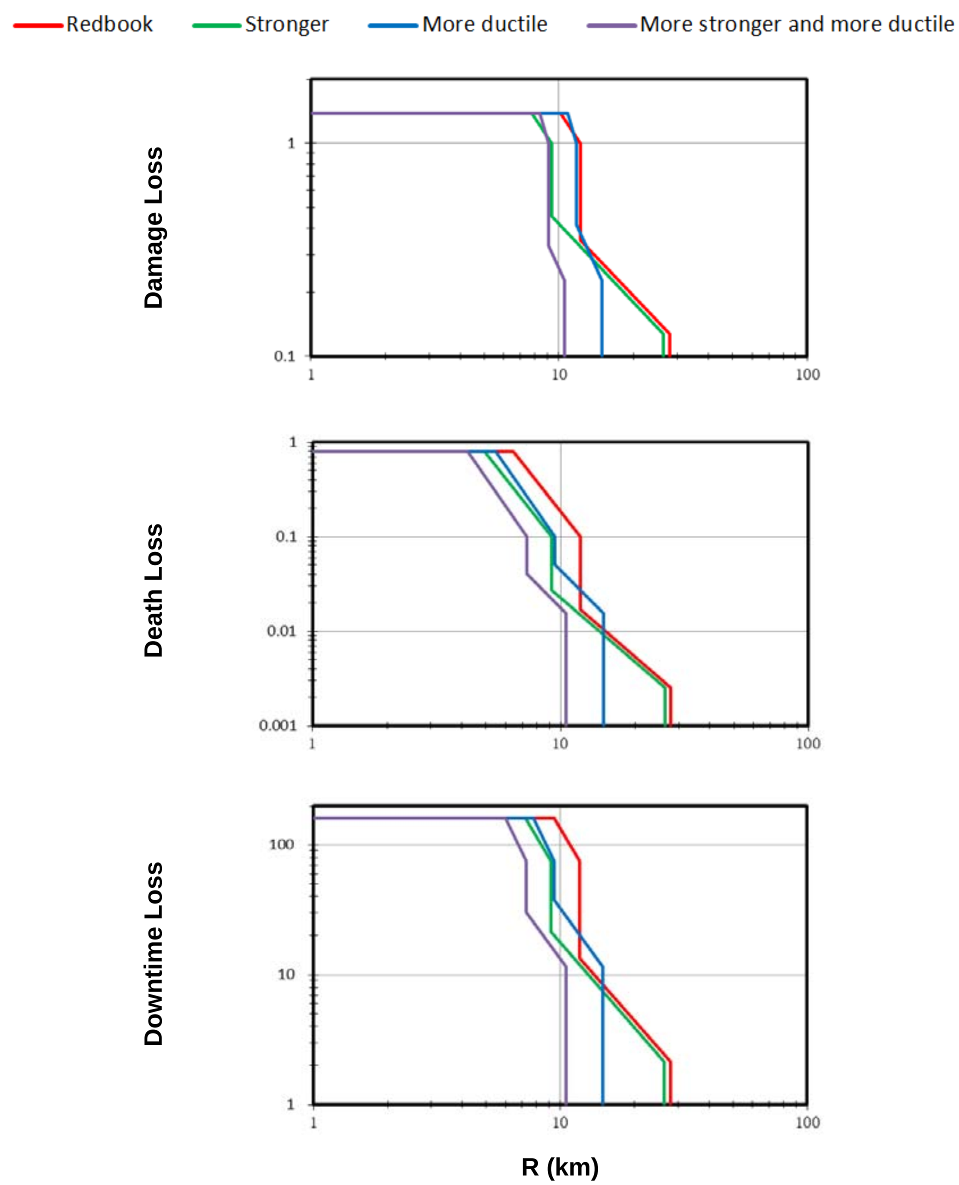

Figure 3: Proposed 3d loss model results for different buildings

Results of this swing analysis are presented in Figure 3. When compared with the "Redbook Building" as the benchmark, one can observe that for Case (a) the stronger building: (i) the physical damage loss ratio declined to about 20%; (ii) the probability of death loss reduces to 0.4%; and (iii) the downtime loss is about 1 month. The values indicate that the building with a stiffer construction can clearly decrease the damage losses, especially the downtime loss. For Case (b), the more ductile building, the 17 km radial distance from the earthquake epicentre is just at the onset of the damage loss area, which means that no damage is done to the building is possible. However, at distances closer than 17 km complete damage requiring replacement is not really improved. For Case (c), where the building is both strengthened and detailed to avoid damage, both replacement and repairs are reduced.

The swing analysis indicates that a stronger building will help inhibit collapse and thus reduce replacement losses, but repairs are still required at moderate distances from the epicentre (R > 9 km). A more deformable (DAD) construction is evidently more effective in reducing the repair losses, but under severe shaking replacements may still be necessary; there is no substitution for strength when complete toppling/collapse/replacement is concerned.

However, the more ductile building is noticeably more effective in decreasing the earthquake inflicted damage and almost eliminates the need for building repairs. Therefore, to reduce the need for both *replacements* and *repairs*, structures need to be respectively made both *stronger* and *more deformable* (with DAD details). The stronger and more ductile building can achieve the best performance and have the minimum 3d losses compared to the other three types of buildings.

For the case of modifying the “Redbook Building” by increasing the strength by 30 percent, plus altering the connections to armoured DAD details, all damage beyond 10 km from the epicentre can theoretically be eliminated. To reduce this entirely the building would need to be made stronger again.

6 CONCLUSION

By making a building stronger by design, the need for complete replacement is reduced, but damage and thus repairs along with the inevitable downtime can still be expected. Conversely, by making a building more deformable, the need for repairs will be reduced, but toppling or complete failure will only be eliminated if the building is also made stronger.

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Damage, Death and Downtime Risk Attenuation in the 2011 Christchurch Earthquake

John B. Mander

Inaugural Zachry Professor

Zachry Department of Civil Engineering

Texas A&M University

3d Seismic Losses

- <u>DAMAGE</u>

•Relates to the cost of repairs or replacement

•What are the insurance and finance needs?

- <u>DEATH</u>

•Relates to the need for <u>first responders</u>

•How many first responders and where?

- <u>DOWNTIME</u>

•Relates to the <u>recovery needs</u>

•How long will this facility be out of action?

Swing Analysis

"Redbook Building" (benchmark)

- 30% Stronger Building
- More Ductile Building
- Both Stronger and More Ductile Building

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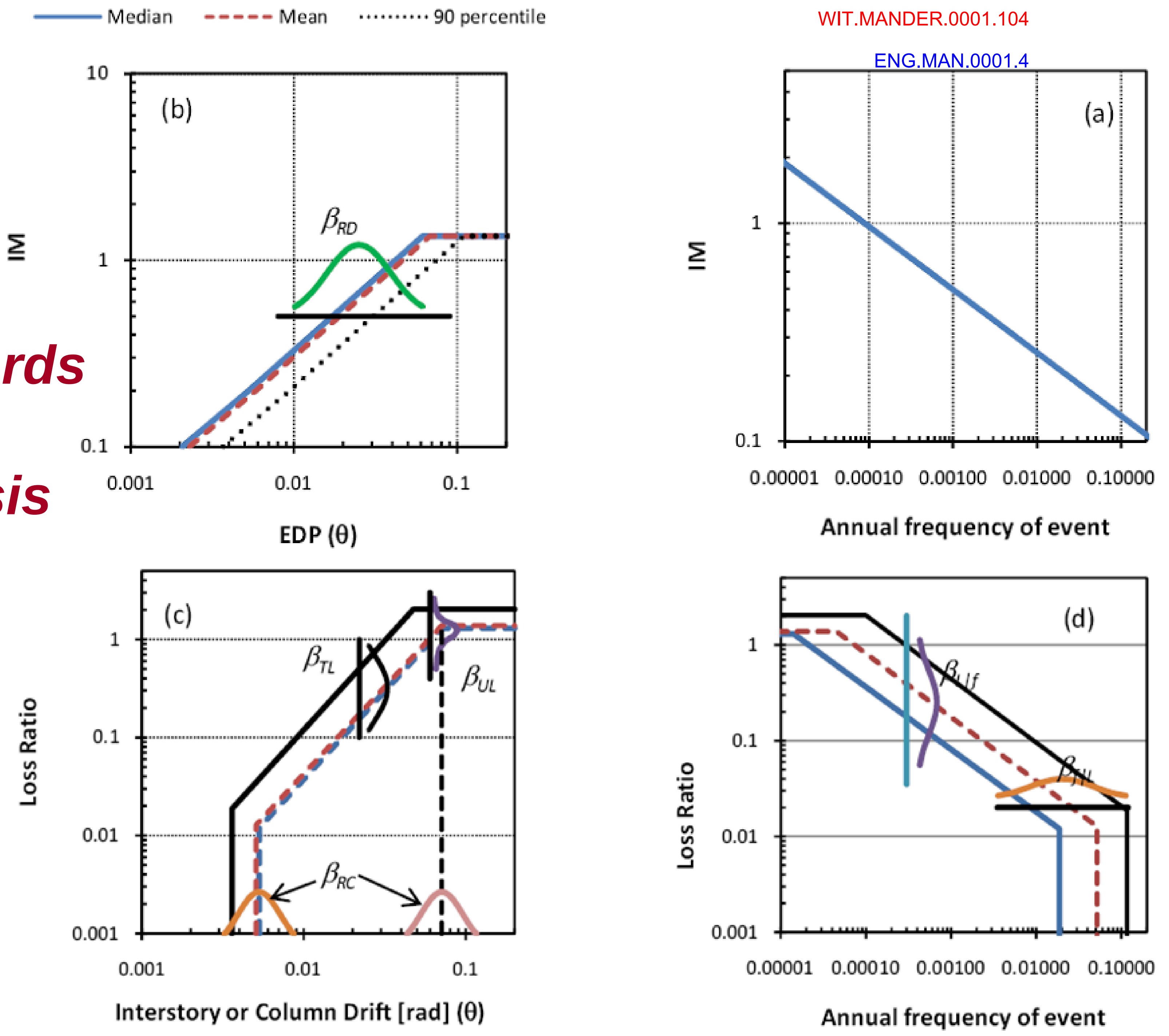
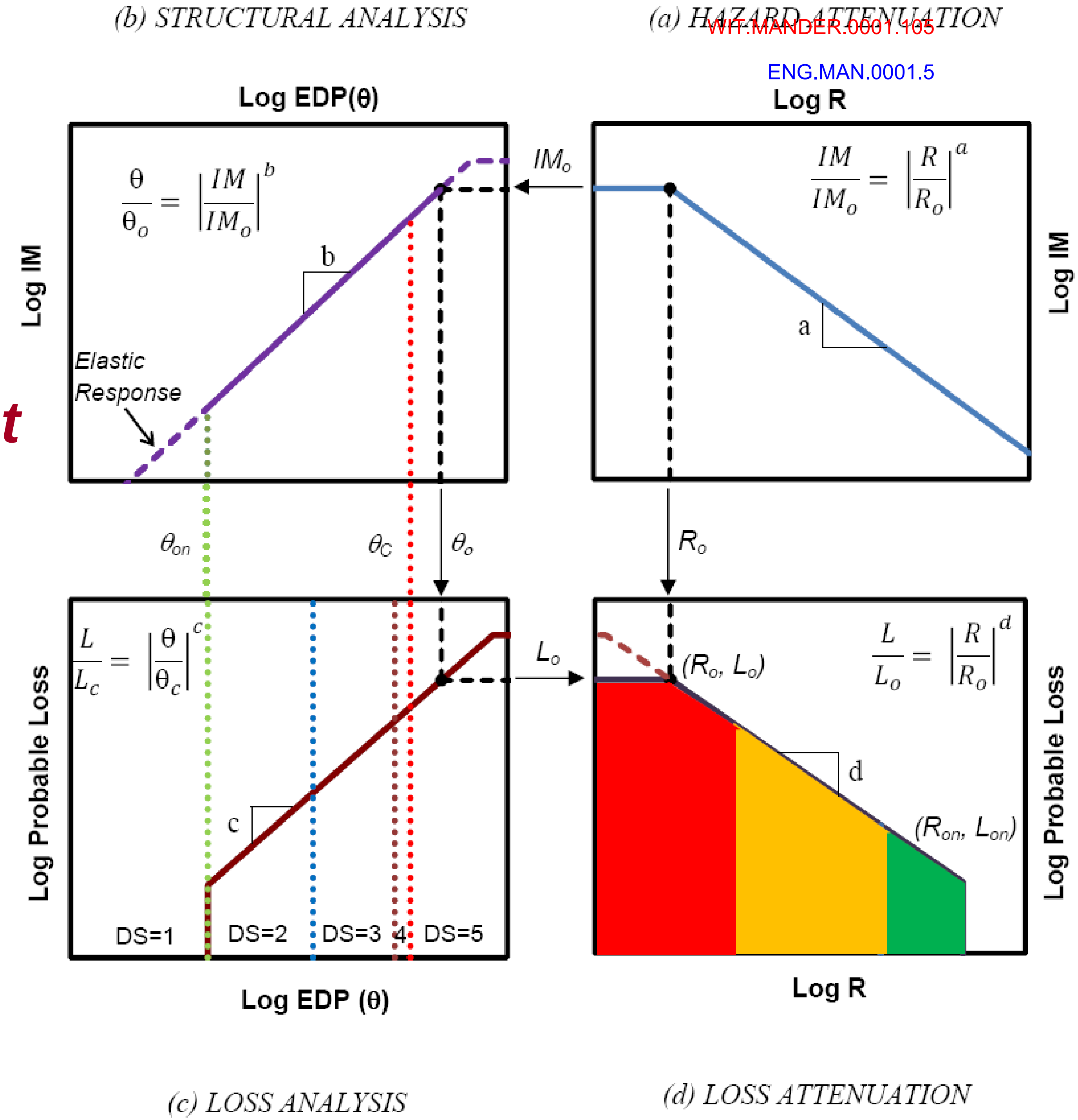
All-hazards Risk Analysis



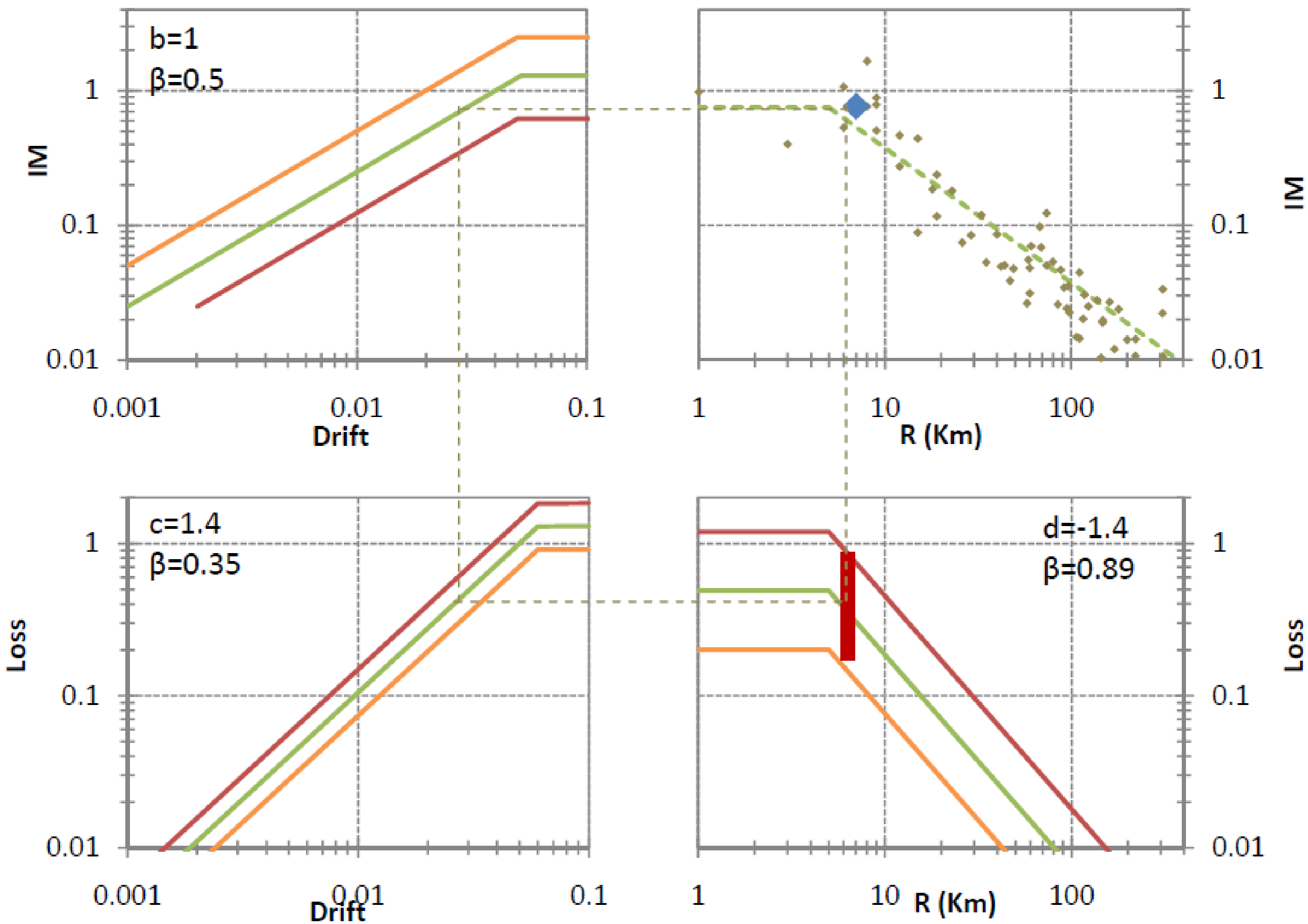
Scenario Event Risk Analysis



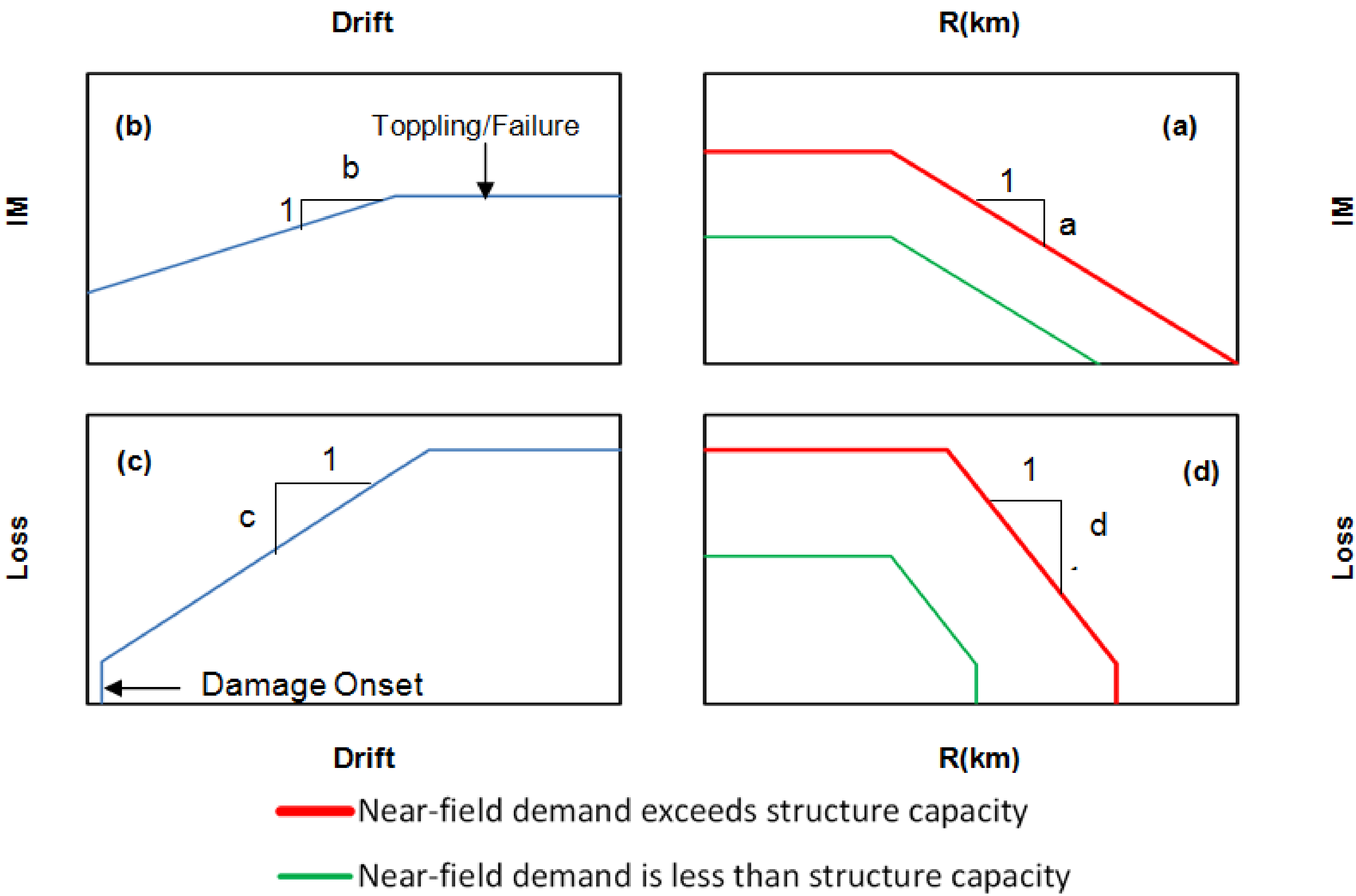
Hazard Intensity Attenuation Model

22 February 2011 Christchurch Earthquake in NZ

"Redbook Building"



“Scenario-based” 3d Loss Model



“Scenario-based” 3d Loss Model: (a) seismic hazard intensity-attenuation model (b) structural analysis (c) damage analysis (d) loss-attenuation estimation

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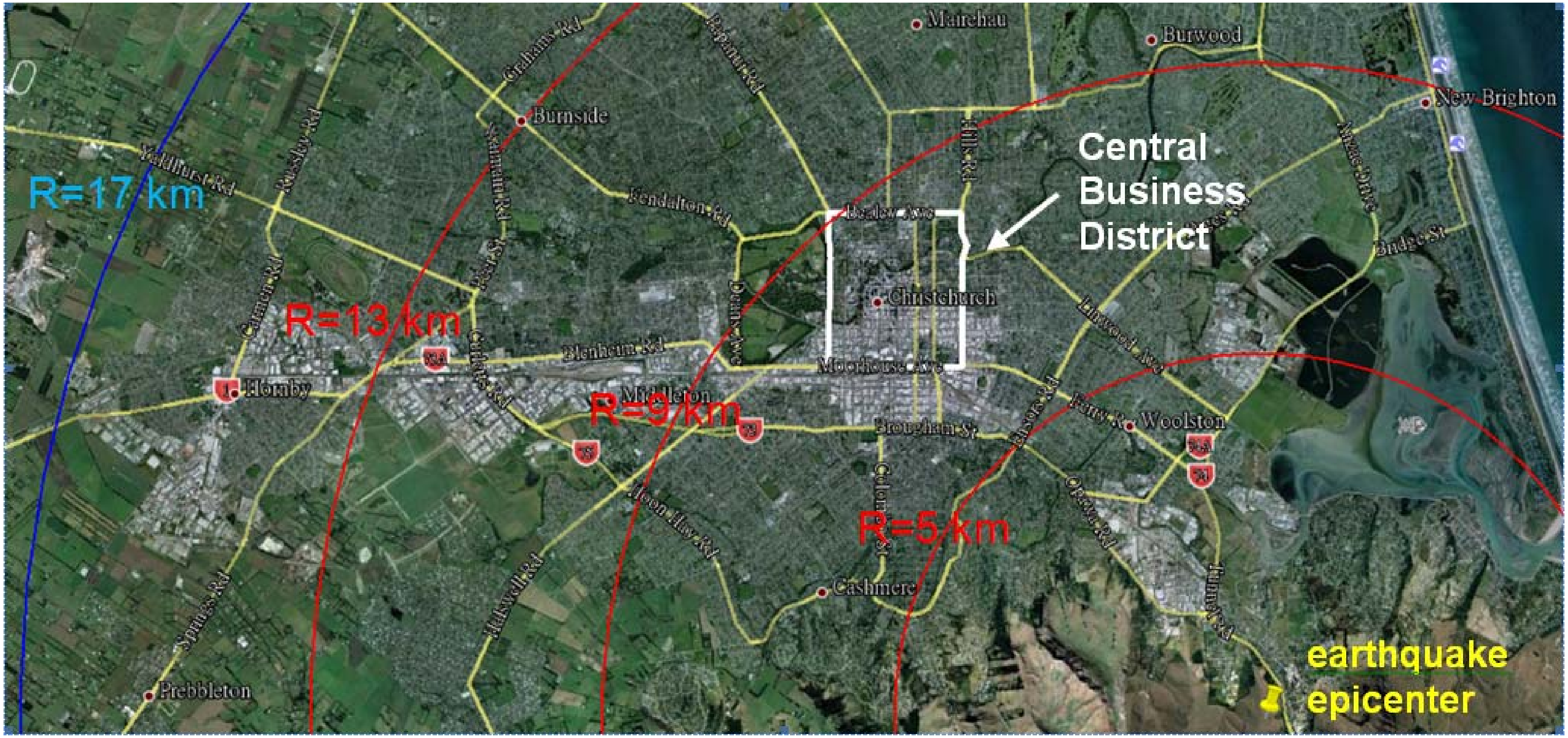
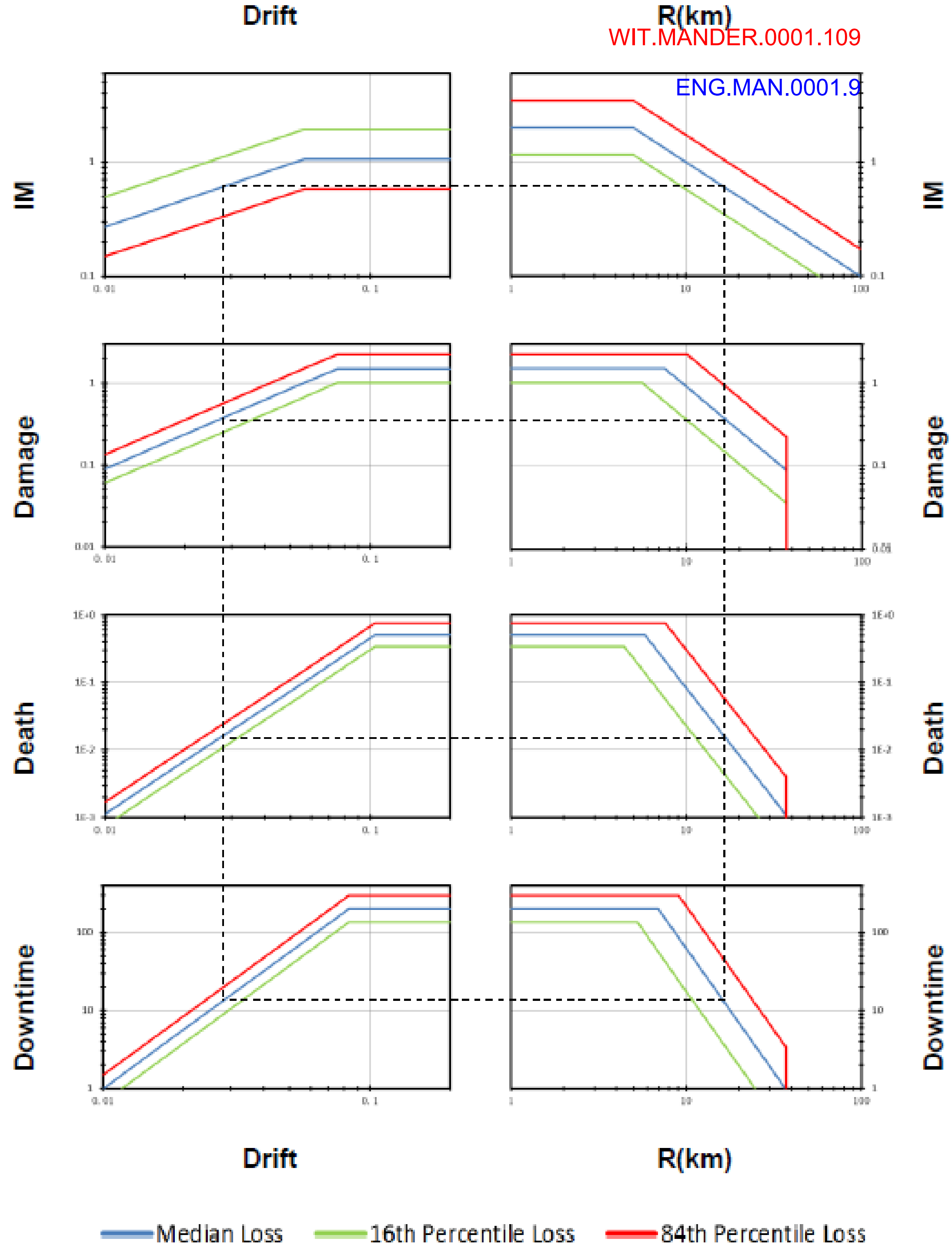
Christchurch Map (Google)



3d Losses of "Redbook Building" for 2011 Christchurch Earthquake

R=17km



Expected 3d Losses at *R = 17km* ***(R = 10km)***

Buildings \ Losses	Damage	Death	Downtime (weeks)
Redbook Building (benchmark)	**50% (154%)**	**4% (19%)**	**24 (131)**
30% Stronger Building	30% (92%)	1% (9%)	18 (66)
More Ductile Building	— (164%)	— (9%)	— (59)
Stronger and More Ductile Building	— (55%)	— (3%)	— (20)

Maximum Loss Model + **Average Loss Model**

↓ *Conditional Loss Model*

Proposed Loss Model

(Considering Spatial Distribution of Losses over the Height of Buildings)

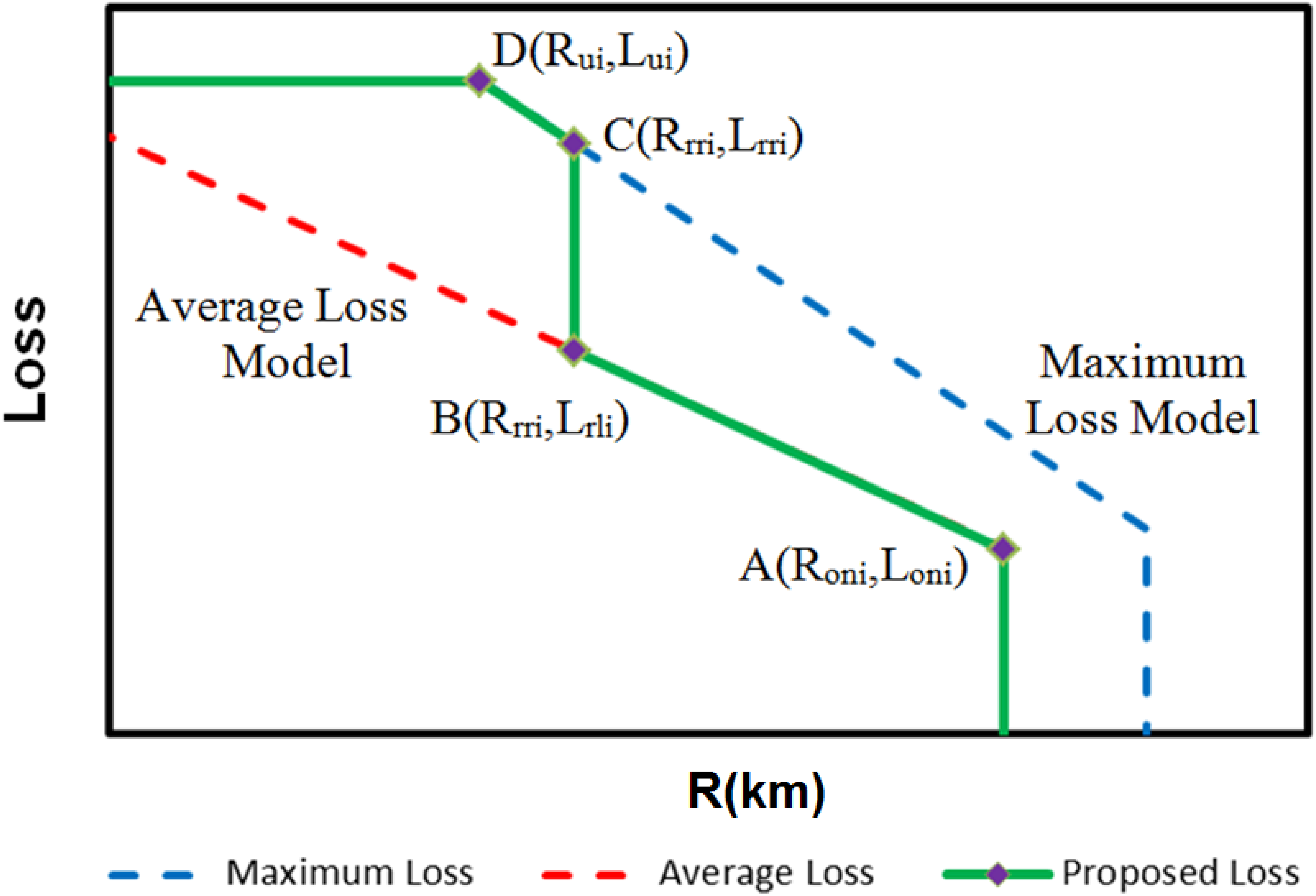
- Losses that require the building to be repaired:

$$L_{effi} = L_{avgi} \qquad (L_{oni} \le L_{\max i} < L_{rri})$$

- Losses that require the building to be replaced:

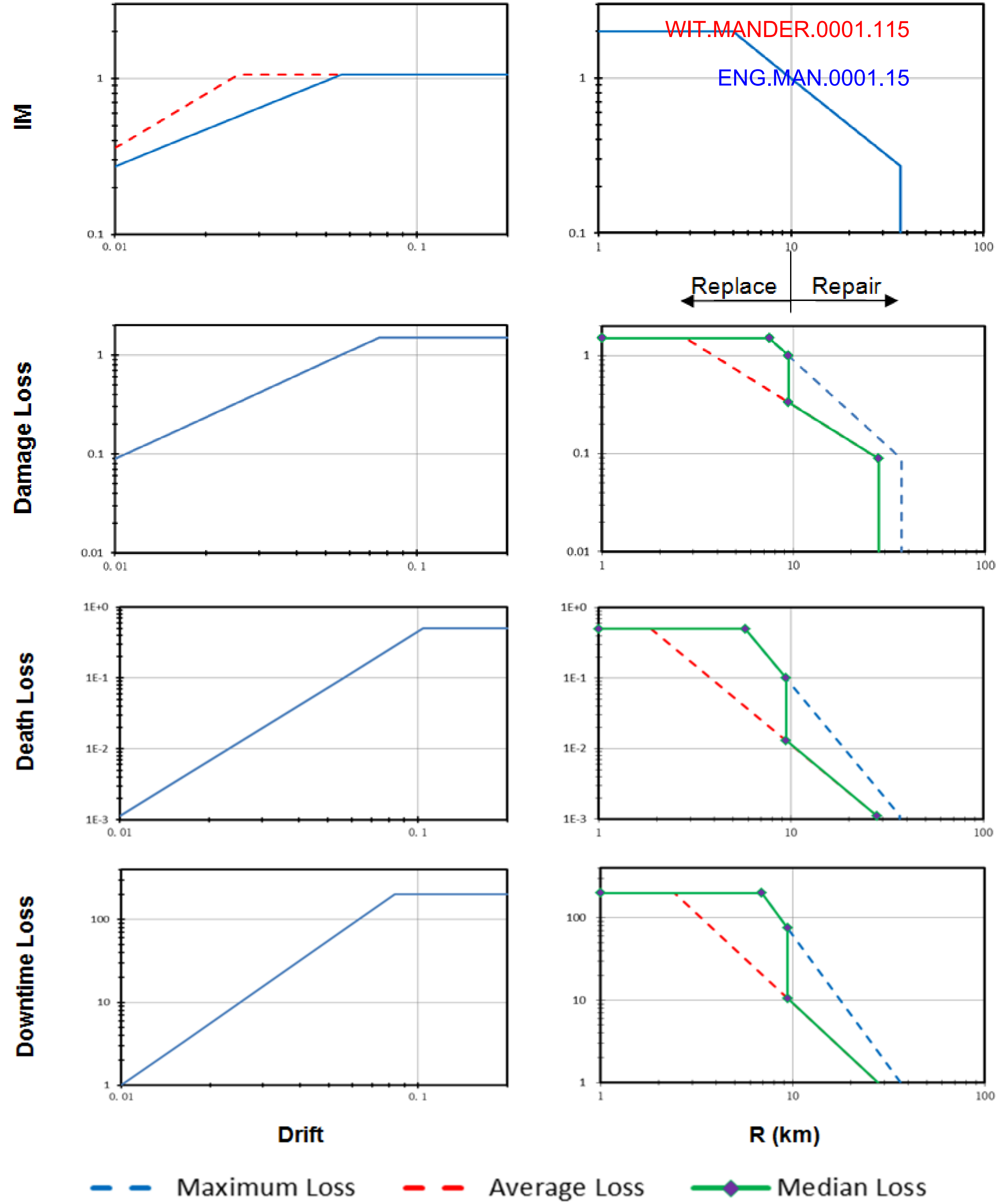
$$L_{effi} = L_{\max i} \qquad (L_{rri} \le L_{\max i} \le L_{ui})$$

	Damage $i = 1$	Death $i = 2$	Downtime $i = 3$
L_{rr}	1.0	0.1	75

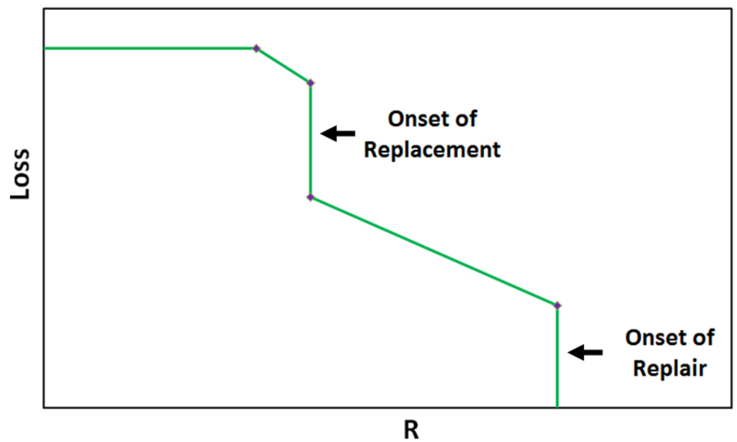

D(R_{ui},L_{ui})
C(R_{rri},L_{rri})
Average Loss Model
B(R_{rri},L_{rli})
Maximum Loss Model
A(R_{oni},L_{oni})
Loss
R(km)
Maximum Loss
Average Loss
Proposed Loss

3d Losses of "Redbook Building"

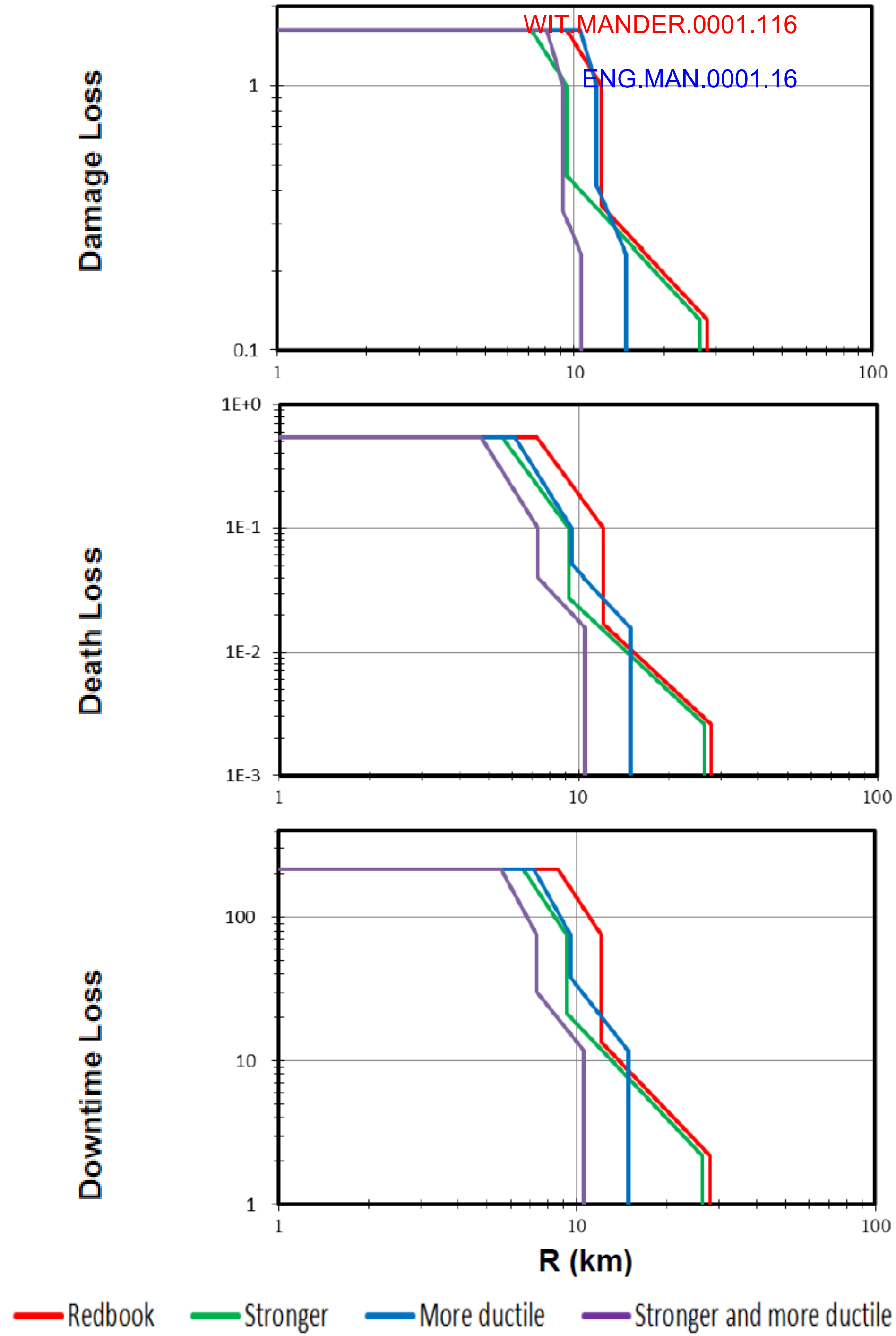
Distinguish *Repair & Replacement* Losses



Expected 3d Losses of the Swing Analysis



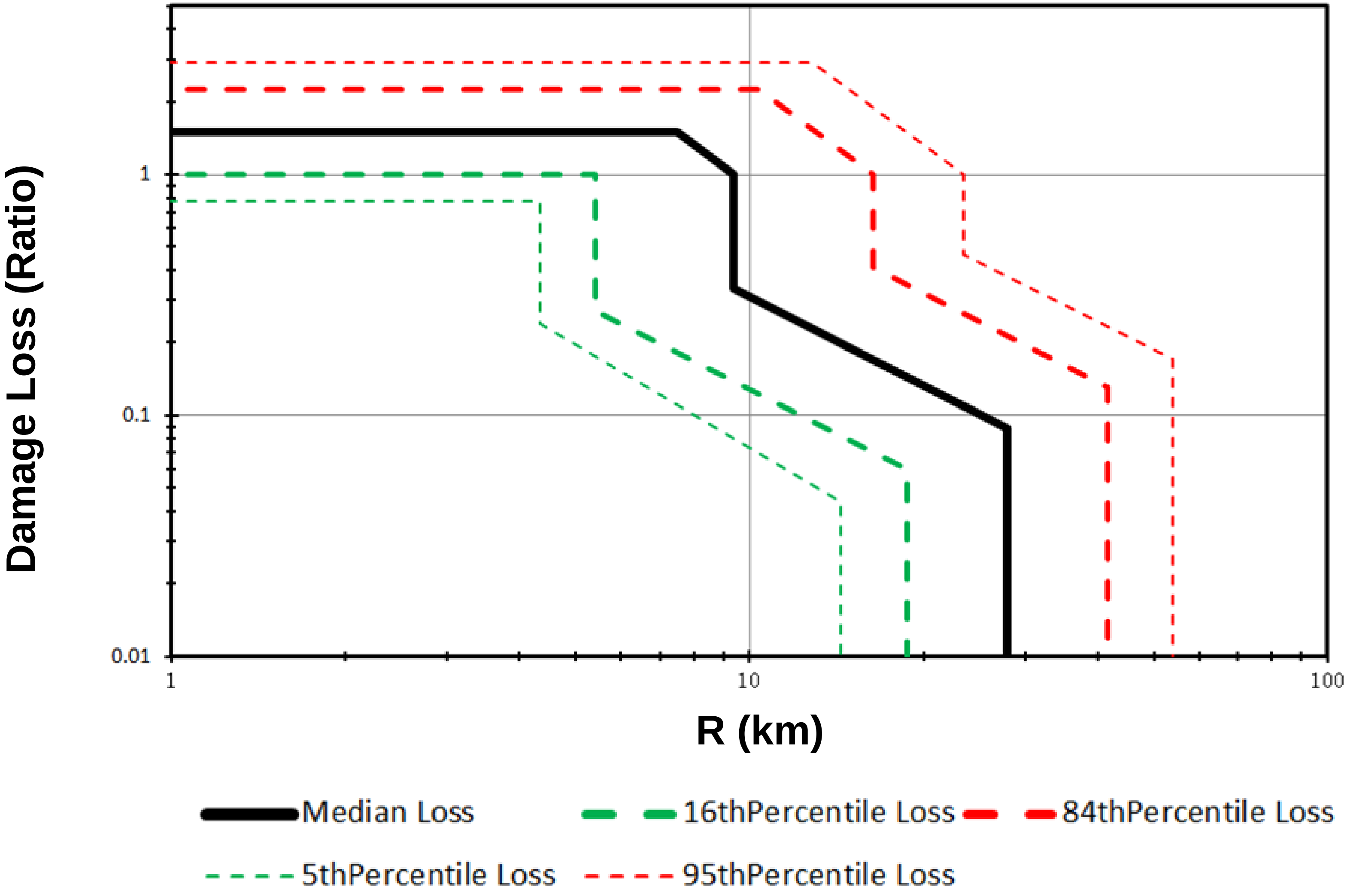
- Stronger: *Replacement Losses*
- More Ductile: *Repair Losses*



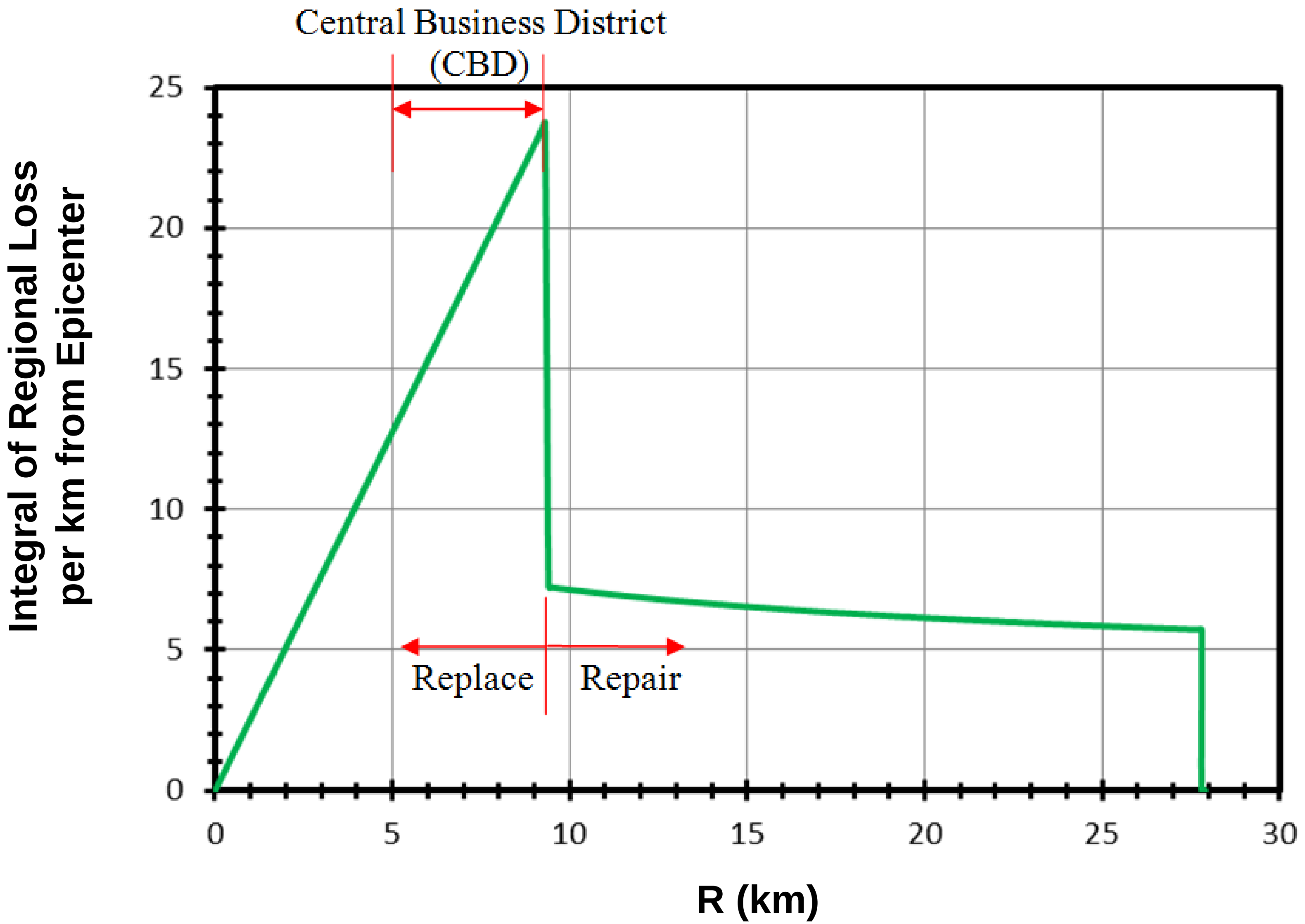
Radial Distance from the Earthquake's Epicenter at the onset of repair & onset of replacement

Losses / Buildings	Onset of Repair (km)	Onset of Replacement (km)
Redbook Building (benchmark)	***28***	***12.3***
30% Stronger Building	26	*9.5*
More Ductile Building	*15*	11.9
Stronger and More Ductile Building	*11*	*9.2*

Various Percentile Damage Loss for “Redbook Building”



Regional Loss for "Redbook Building"



Concluding Remarks

- A stronger building will help inhibit collapse and thus reduce replacement losses, but repairs are still required at moderate distances from the epicenter.
- A more deformable construction is noticeably more effective in reducing the earthquake inflicted damage and almost eliminates the need for building repairs.
- By making a building stronger and more deformable, it achieves the best performance attributes and both repair and replacement losses are evidently reduced.
- For regional loss per km from the epicenter, the building replacement loss increases uniformly with the radial distance from the earthquake epicenter; and a constant loss rate trend shows to happen for the repair cost.