UNDER	**THE COMMISSIONS OF INQUIRY ACT 1908**
IN THE MATTER OF	**ROYAL COMMISSION OF INQUIRY INTO BUILDING FAILURE CAUSED BY CANTERBURY EARTHQUAKES** **KOMIHANA A TE KARAUNA HEI TIROTIRO I NGA WHARE I HORO I NGA RUWHENUA O WAITAHA**
AND IN THE MATTER OF	**THE CTV BUILDING COLLAPSE**

STATEMENT OF EVIDENCE OF ROBERT DAVID JURY
IN RELATION TO THE CTV BUILDING

DATE OF HEARING: COMMENCING 25 JUNE 2012

STATEMENT OF EVIDENCE OF ROBERT DAVID JURY

1. My full name is Robert David Jury.

2. I am currently a Technical Director in the discipline of structural engineering in the Wellington office of Beca Carter Hollings and Ferner Ltd (Beca). I have Bachelor of Engineering (Hons) and Master of Engineering degrees conferred at the University of Canterbury in 1977 and 1978 respectively. I hold CPEng in the practise areas of structural and civil engineering. I am a Fellow of the Institution of Professional Engineers New Zealand and a Fellow of the National Society for Earthquake Engineering (NZSEE).

3. I have over thirty years' experience in the field of structural engineering consultancy, specialising in the assessment and design of structures and in particular the performance of structures in earthquakes. I have received several excellence awards for projects that I have been involved with including; the Auckland SkyTower, the Thorndon Overbridge Seismic Retrofit project and the Istanbul residential buildings seismic retrofit project. I have authored or co-authored over forty technical papers on various subjects relating to structural engineering. I was a member of the Standards' committee that developed the current New Zealand Loadings Standard (including the earthquake loading Standard), NZS1170, and also its predecessor, NZS4203. I was convenor for the NZSEE study group that prepared the Society's publication entitled "Assessment and Improvement of the Structural Performance of Buildings in Earthquakes", June 2006. I am currently a member of the Engineering Advisory Group that is providing technical advice to the Department of Building and Housing in response to the Christchurch earthquakes.

4. Attachment 'A' is a copy of my Curriculum Vitae.

EVIDENCE

5. My evidence will address, on behalf of the Expert Panel appointed by the Department of Building and Housing, the process, findings and recommendations in the Expert Panel report "Structural Performance of Christchurch CBD Buildings in the 22 February 2011 aftershock", in relation to the CTV Building, a copy of which is already publicly available and I do not attach it here.

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

The Expert Panel

7. I was a member of the Expert Panel appointed by the Department of Building and Housing. The Department initiated four technical investigations into four buildings that collapsed or suffered significant damage during the February 2011 earthquake, CTV being one of those buildings.

8. It appointed a single Expert Panel to provide guidance on methodologies being used by the technical investigations, to review and approve the technical reports and to report on their implications. I attach, marked B, the Terms of Reference for my engagement in this role.

9. The Panel produced a single report covering the four buildings. I attach, marked C, that part of the Panel's report relating to the CTV Building.

10. While there was variance among the Panel members on matters of detail (which I address below) the Panel agreed on, in relation to the CTV Building:

    a. Factors that contributed, or may have contributed, to the building's failure;

    b. Key vulnerabilities in the building design; and

    c. Potential collapse scenarios.

Panel's Oversight of Technical Report

11. The Panel met on seven occasions over the period from 30 March 2011 through to 20 October 2011 and also members of the Panel corresponded between one another on various matters relating to the investigation primarily via email over the same period. Copies of this correspondence have been provided to the Royal Commission.

12. Our role was to provide guidance and direction to the technical investigation, as set out in the terms of reference at p 12 of the Expert Panel report.

13. Presentations by the investigating consultant for CTV were given to the Expert Panel at each of these meetings. These presentations covered the progress of the investigation up to that particular point in time, particular challenges encountered, interim findings and, when they were available, preliminary conclusions reached by the investigating consultant.

14. In general Panel members relied on the factual information collected by the investigating consultant, during the course of its investigation but interpretation of this information was open to discussion. Such information included witness statements and material test results and structural analyses.

15. While there were differing views between the panel and the consultant over the use of the method of analysis (as set out below) my understanding is that in general the Panel had confidence in the balance of the investigation work undertaken by the investigating consultant.

16. Early in the investigation the Panel expressed concerns regarding the type and extent of structural analysis of the building being carried out by the investigating consultant. In particular, the concerns related to the use of elastic methods of analysis compared with methods that would allow the non-linear behaviour of materials to be modelled. As a result of these concerns, non-linear time history analyses (NLTHA) were commissioned by the investigating consultant. The Panel gave the results from the NLTHA analyses a higher weighting than those using elastic methods.

17. Some members of the Panel expressed concern that sufficient regard had not been given to other potential failure modes such as failure of the ties between the shearcore and the floor slabs and failure of the internal columns. The Panel also felt that the investigating consultant may have given undue prominence to some other potential failure modes that had been identified. These aspects received considerable attention and discussion between the Panel members and the investigating consultant who was encouraged to consider them in the investigation report.

18. It would be fair to record that there remains some differences in opinion between some Panel members and the investigating consultant. These differences are described in the Panel report (refer p34) and relate to what might be the initiating factors of the collapse, and some aspects of detail in terms of the failure modes. I have read Nigel Priestley's evidence that has been made to the Royal Commission on the CTV building and agree that, in general, it also outlines the areas where differences of opinion still exist between some members of the Panel and the investigating consultant.

19. The investigating consultant produced several drafts of the investigation report, each of which was reviewed by Panel members who provided comments back to the consultant.

20. When the investigating consultant's report was near completion the Panel report was drafted and circulated for comment to all Panel members (including the member of the investigating consultant team who was also a Panel member). Comments from all Panel members were considered and, following discussion and iteration, a final version of the Panel report was prepared that all members of the Panel were prepared to accept as reasonable.

21. 5 recommendations were made in the Panel report as a result of the CTV investigation. Specifically these relate to:

    a. The manner in which structural irregularities are dealt with in design.

    b. The adequacy of non-ductile "gravity" columns, particularly those in buildings designed prior to the introduction of NZS4203:1992.

c. The interaction between precast panels and columns, particularly when the panels are only part height.

d. The adequacy of connections between floor slabs and shearwalls, and

e. The need for improved confidence in design and construction quality.

Dated: 31 May 2012

Robert David Jury

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Rob Jury

Technical Director – Structural Engineering – Wellington Structural

Bachelor of Engineering (Hons)

Master of Engineering (Civil) University of Canterbury, New Zealand

Citizenship	New Zealand
Membership	■ Fellow, Institution of Professional Engineers (NZ) (Civil & Structural) ■ Fellow, NZ Society for Earthquake Engineering ■ CPEng ■ Member, Earthquake Engineering Research Institute (USA) ■ Member of NZ Timber Design Society ■ Chairman of the New Zealand Society of Earthquake Engineering Study Group on Earthquake Risk Buildings. ■ Member of the New Zealand Society of Earthquake Engineering Study Group reviewing the Seismic Design of Storage Tanks. ■ Committee Member of NZS/AS Joint Standard for Loadings on Buildings including NZS 1170.5 Earthquake Loadings Standard. ■ Member of Research Board providing overview to Auckland and Canterbury Research programme on seismic resistance of existing buildings.
Special competence	Design and detailing of low and high rise industrial and commercial buildings and unusual structures. Seismic design of structures including strengthening of earthquake risk buildings. Preparation of seismic design codes. Assessment of damage and probable loss due to earthquakes. Seismic risk studies. Risk Studies.
Background	1978 – present Beca Carter Hollings & Ferner, Wellington, New Zealand

Relevant experience

Beca Carter Hollings & Ferner Ltd, Wellington, 1978-Present:

1999 – present - Technical Director with Beca Carter Hollings & Ferner Ltd

1996 -2007 & 2009 - Present - Manager – Wellington Structural – Beca Carter Hollings and Ferner Ltd

Investigation of Buildings for Department of Building and Housing - 2011-2012

Responsible for preparation of investigation reports for several buildings that performed poorly during 22nd February Christchurch earthquake. Also on Expert Panel that investigated several others. Presented results to Royal Commission.

Victoria University Seismic Assessment Project 2009 - 2010

Responsible for a project to complete IEP assessments for all buildings in the University property portfolio and to prepare strengthening schemes for buildings scoring below 65%NBS.

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Whangarei Hospital Seismic Assessment Project 2009

Responsible for a project to assess hospital buildings using the IEP methodology.

Seismic Retrofit of School Buildings in Istanbul – 2008 and ongoing

Specialist seismic advisor for a project to assess and design seismic retrofit solutions for over 500 school and institutional buildings.

Review of Retrofitting Guidelines for Romania – 2007

Team Leader for a World Bank project to review guidelines on assessment and retrofit of existing buildings in Romania, prepared by a Romanian Code Committee.

Porirua City Council Consent Assessment – 2009 and ongoing

Advisor to the Porirua City Council and responsible for the Beca consent reviewers.

Wellington City Council Existing Building Assessment Project – 2007 and ongoing

Responsible for the assessment of approximately 1500 existing buildings in Wellington City using the IEP process contained with the NZSEE guidelines recommendations.

Wellington City Council Building Consent Assessment, 2004 and ongoing

Advisor to the Wellington City Council on Structural matters. Also responsible for the Beca assessment team.

Rail Bridges for ONTRACK – 2004-2007

Responsible for the preparation of designs, tender and contract documentation and site monitoring for general online replacements of rail bridges on the main trunk line.

Guidelines for the Assessment and Retrofit of Earthquake Risk Buildings - 2005

Responsible for various projects associated with this study group report for the New Zealand Society for Earthquake Engineering. Included revision of the Initial Evaluation Procedure, development of analysis procedures and preparation of worked examples using the guidelines.

Retrofit of Residential Buildings in Istanbul – 2005–2006

Structural Team Leader for a World Bank funded project to investigate the viability of seismic retrofitting for a sample group of residential buildings in Istanbul.

Plimmerton Footbridge – 2004

Responsible for the design of a 33 m span steel truss footbridge across SH1, Plimmerton.

Investigation into Standards for 500E Grade Reinforcing Steel in New Zealand – 2004

Completed an investigation into the existing Standards pertaining to 500E Grade Reinforcing Steel to determine whether these were likely to the adequate given some unexplained failures. Study carried out for NZ Building Industry Authority.

Investigation into the Seismic Performance of Hollowcore, 2003

Team Leader of an investigation into the seismic performance of hollowcore in New Zealand. Carried out for the NZ Building Industry Authority. A statistical survey of hollowcore usage in Wellington and Christchurch was completed. The information obtained was analysed to characterize the expected performance of hollowcore and to establish the risk profile in these cities following the experience from university testing.

Singapore Bridge Assessments and Retrofits – 2003-2004

Responsible for bridge assessment and retrofit programmes for Land Transport Authority, Singapore. Twenty-one bridges inspected and assessed, full retrofits prepared for seven prestressed beam and reinforced concrete bridges.

Southern Resa Underpass, Wellington Airport – 2005-2007

Responsible for the structural design of a traffic (2 lane) underpass at the southern end of the main runway.

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Seismic Assessments of Seven Bridges in Wellington & Nelson Regions, Transit NZ, 2003-2004

Team Leader and Job Director for a detailed assessment of 7 bridges located in Wellington and Nelson. Required a detailed structural assessment for seismic effects, assessment of seismic hazard and an economic analysis for each.

Seismic Hazard Analyses, 1999-2006

Responsible as verifier and Job Director for seismic hazard analyses for the following projects:

- Rodney Power Station – 2006
- Kopu Bridge, New Zealand - 2006
- Kupe Project, Taranaki, New Zealand – 2004
- Bridge Seismic Assessments – Wellington and Nelson (NZ) 2003
- North Shore Bus-way Auckland, New Zealand, Transit NZ – 2003
- Mt Maunganui, New Zealand, Mobil NZ, 2003
- Takatimu to Tasman Bridge and Approaches, New Zealand, Transit NZ – 2003
- Maui Platforms and Pohokura sites, Taranaki, Shell Todd Oil Services, 2003-2003
- Golden Bay Cement Kiln Upgrade, Whangarei, NZ, Fletcher Construction
- Newmarket Viaduct Upgrade, Auckland, NZ, Transit NZ, 2002-2003
- State Highway 2, Lower Hutt, NZ, Transit NZ, 2002-2003
- Alpurt Project, New Zealand – 2002
- Port Vila (Vanuatu), 2002 (Client: Mobil Oil Australia)
- Kings Wharf Rehabilitation, Fiji, Maritime and Ports Authority of Fiji, 2002
- University of Auckland, New Zealand – 2001
- Huntly e3p Project, New Zealand – 2001
- Queen Street Station, Auckland, New Zealand – 2001
- Marsden Point, New Zealand – 2000
- Project Waikato, New Zealand – 2000
- Bell Block Bypass, New Zealand – 2000
- Fiona Dam, 2000 (PT Inco)
- Ramu Nickel, Papua New Guinea, Highlands Pacific Australia Pty, 1999
- Lihir Gold Mine, Lihir Island, PNG, Lihir Management Company Pty Ltd, 1999

Transmission Gully Studies, 2003-2004

Earthquake engineer for team preparing an updated cost estimate including associated design concepts and risk analysis for a proposed new 27 km long four-lane motorway providing an alternative route out of Wellington.

Meridian Energy Ltd: Project Aqua, 2003-2004

Earthquake Design Specialist responsible for earthquake design philosophy and seismic design for a 60 km power scheme in the lower Waitaki Valley involving canals and six 90 mW power stations (project stopped during preliminary design phase).

Aratiatia Power Station Seismic Retrofit Peer Review – 2003-2004

Commissioned by Mighty River Power to carry out a peer review of proposed seismic upgrading works including potential for surge tank sliding and potential for damage to the gate support structures within the surge tank.

State Highway 2 Dowse to Petone Upgrade, Wellington, 2003-2010

Structural Team Leader for the structural design of 5 State Highway overbridges, interchange and retaining walls to be constructed at Petone, Lower Hutt. Includes alterations and retrofit of the Petone Pedestrian Overbridge. Value $60M.

Wellington Bulk Water Supply, 2001

Seismic engineer, reviewer and job director for a study on a pipeline segment which included both a Wellington fault crossing and the Hutt River crossing, with a characteristic movement of 3.6-5 m in a 600 year return period.

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Assessment and Improvement of the Structural Performance of Buildings in Earthquake, 2000-ongoing

Currently Chairman of this NZSEE Study Group to develop guidelines for the initial evaluation, detailed assessment and improvement measures for earthquake risk buildings. The guidelines document is intended to be cited in the NZBC. The work is being completed for the Department of Building and Housing.

Genesis Recreation Centre – Masterton District Council, 2000-2004

Principal Consultant for the feasibility study and design of a $7.1M project to upgrade the existing recreation centre until the design team was novated to the Design Build Contractor. Project includes refurbishment of the original stadium, upgrading and rationalisation of the original outdoor and indoor pools and construction of a new indoor aquatic centre comprising leisure pool and 25 m training pool.

Otaki River Bridge, 2000

Team Leader and Job Director responsible for a seismic assessment of this bridge carried out as part of the highway bridge seismic screening project.

Melling Railway Line Extension Bridge Study, 1999

Team Leader and job Director for an investigation of an option to provide a railway crossing of the Hutt River at Melling. Included estimation of costs.

Loadings Standard Revision (AS/NZS 1170), 1995-2004

Member of the joint Australian/New Zealand Standards lead committee considering the development of a suite of loadings standards for permanent, imposed, wind, earthquake and snow design actions.

Earthquake Loadings Standard (NZS 1170.5) 1995-2004

Member of the New Zealand Standards committee which prepared NZS 1170.5.

Whitford Brown Intersection, 2000

Deputy Team Leader for the management of this project to upgrade this intersection of SH1 near Porirua. Concept included major interchange and extension of lanes, Transit NZ.

1989 – 1999

Associate with Beca Carter Hollings & Ferner Ltd working on projects which include:

Macau Tower, 1998

Responsible as package leader for the seismic hazard assessment and structural design of the concrete shaft for this 338 m high tower currently under construction in Macau. Total expected project value US$40M.

Peer Review, Seismic Design of the Jakarta Tower, 1997

Carried out a review of the seismic design aspects of this 550 m high communication and public viewing tower proposed for Jakarta.

Seismic Hazard Analyses, 1996-1999

Responsible as verifier for seismic hazard analyses for the following projects:

- Lake Toba, Indonesia - 1996

Thorndon Overbridge Seismic Retrofit, 1996 – 1999

Engineer to the Contract and coordinator for Partnering for this project to seismically retrofit this major bridge in Wellington. Total contract value $15 million.

Lake Toba Seismic Assessment, 1996

Responsible for a seismic hazard assessment for a site near Lake Toba in northern Sumatra, Indonesia, carried out for Klohn-Crippen.

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Auckland Harbour Bridge Seismic Assessment, 1995 – ongoing

Responsible for the estimation of earthquake hazard at the site to be used in evaluation retrofit options.

Sky Tower - Auckland, 1994 – 1999

Responsible, as structural team leader, for the structural design of the 336 m high Sky Tower (communication tower), being constructed as part of the Sky City Casino project in Auckland.

Otira Viaduct Seismic Assessment, 1994

Responsible for the estimation of the earthquake hazard for the Otira viaduct site.

Thorndon Overbridge Seismic Assessment, 1993 – 1994

Responsible for the estimation of the earthquake hazard at the site to be used in evaluating retrofit options.

Nepal Building Code Project, 1992 – 1993

Responsible for the preparation of the Nepal Earthquake Code including derivation of design earthquake loads and seismic zoning for the country.

Loadings Code Revision, 1988 – 1992

Member of the Standards Association Committee which prepared the revision of the loadings standard (including earthquakes), NZS4203:1992.

North Island Control Centre, 1991

Responsible for the structural design of a control centre for Trans Power. The structure is required to be resilient to sabotage and seismic activity.

Wellington Town Hall Strengthening, 1990 – 1992

Responsible for the design of an alternative strengthening proposal for the Wellington Town Hall which was accepted as part of the successful tender.

Telephone Exchange Strengthening – 1990

Responsible for the design and construction of the seismic strengthening works of three telephone exchanges in the Wellington Region.

Included were:

- Courtenay Place Exchange ($1M)
- Lower Hutt Exchange ($0.2M)
- Naenae Exchange ($20,000)
- Miramar Exchange ($40,000)

Telecom Risk Studies, 1990

Responsible for a seismic risk study of all Telecom Telephone Exchanges in the Wellington region. Investigation required a review of building performance and how this might affect Telecom operations. An assessment of equipment performance under earthquake shaking was also undertaken.

Enclosed Switchyard Buildings, Oteranga & Fighting Bays, 1989 - 1991

Responsible for the design of these two enclosed switchyard buildings constructed as part of the DC4 Hybrid Link Project for Trans Power New Zealand Ltd.

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1987 – 1989

Specialist Engineer with Beca Carter Hollings & Ferner Ltd working on projects which include:

NZI Head Office Development - Auckland, 1988

Responsible for the design of steel work and one office tower for this three-tower $700M project. Commissioned by NZI Corporation.

Telecom Microwave Towers, 1988

Responsible for the review of a study assessing probable lifetime costs, to Telecom, resulting from structural failure of typical microwave telecommunications towers due to wind loading. This study considered likely costs associated with initial capital cost of towers and equipment, probable cost of repairs and probable loss of revenue caused by service disruption. It was shown that it is possible to determine an optimum design wind speed for a given tower so that lifetime costs can be minimised.

Hunter Building Redevelopment, 1988

Responsible for the structural design of the strengthening works for the Hunter and Robert Stout Buildings. Also responsible as principal consultant for the design and construction of Stage II of the work (Total value $15M) - Commissioned by Victoria University of Wellington.

Post-Earthquake Damage Inspection & Repair, 1987

After the 1987 Edgecumbe (NZ) earthquake, Mr Jury was part of a team who assessed damage to the Whakatane Board Mill Plant in Whakatane and directed repairs. He was a co-author of comprehensive reports recording all site damage and investigating reasons for it occurring on this site and the Tasman Pulp and Paper Mill site at Kawerau.

1982 – 1987

Senior Engineer with Beca Carter Hollings & Ferner Ltd working on projects which include:

Seismic Risk Studies

Responsibility for the preparation of reports estimating the risk and expected losses resulting from earthquakes for the following sites and structures:

- New Zealand Paper Mills - Mataura (total value at risk $135M)
- Central North Island Assets of NZFP (total value at risk $2,500M)
- Auckland Assets of NZFP (total value at risk $285M)
- Some Assets of AHI (total value at risk $320M)
- Recovery Boiler Structure - Kinleith (total value at risk $200M)
- GTG Plant, Motunui (total value at risk $1,200M)
- Lion Nathan Auckland Assets (total value at risk $495M)
- Lion Nathan Australia Assets
- Lion Nathan China Assets

Derivation of Design Response Spectra - Maui Stage II Development, 1987 – 1989

Senior Engineer responsible for the preparation of a report setting out the derivation of design spectra for the proposed second offshore platform in the Maui Gas Field (off West Coast NZ). Investigation included derivation of basement rock risk spectra, propagation of motion matching these spectra up a soil column model using LASS computer programs and derivation of design spectra. Commissioned by Shell BP and Todd Oil Services Ltd.

Earthquake Risk Assessment of the Central North Island Assets of NZ Forest Products Ltd, 1986

Senior Engineer responsible for preparation of report estimating annual and maximum credible losses for four plants in the central North Island region belonging to NZ Forest Products Ltd.

Included were; the Kinleith Industries Complex, Kinleith; Whakatane Board Mills Complex, Whakatane; Hutt Timber and Hardware Complex, Tokoroa; Taupo Totara Timber Complex, Putaruru. Required assessment of seismicity and estimation of expected damage levels by site visual inspection. (Total value at risk $2,500M). Commissioned by NZ Forest Products Ltd, Alexander Stenhouse Ltd and Sedgwicks Ltd.

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Earthquake Risk Assessment of the Auckland Assets of NZ Forest Products Ltd, 1985

Senior Engineer responsible for preparation of report estimating average annual and maximum credible losses in five plants in the Auckland region belonging to NZ Forest Products Ltd. Required assessment of seismicity and estimation of expected damage after a visual site inspection. (Total value at risk $285M). Commissioned by NZ Forest Products Ltd.

Earthquake Risk Assessment of Some Assets of Alex Harvey Industries Ltd, 1985

Senior Engineer responsible for preparation of report estimating expected losses resulting from earthquakes in AHI plants in Auckland, Hamilton, Wellington and Christchurch. Average annual losses were estimated as well as the maximum loss during the assessed maximum credible earthquake. (Total value at risk $320M). Commissioned by Alex Harvey Industries Ltd.

Mexico Earthquake Reconnaissance, 1985

Was a member of the New Zealand Reconnaissance Team which visited Mexico in the early days after the September 19 and 20 (1985) earthquakes. Reconnaissance included inspections of damaged and undamaged structures and also bridges both in Mexico City and in the Coastal Regions close to the epicentre.

Laluai Hydro Prefeasibility Study, 1985

Prepared seismic risk sections of the prefeasibility study for a hydroelectric scheme on Bougainville Island, Papua New Guinea.

Design of Rail Bridges for Ohakune, Horopito Railway Deviation, 1984 – 1985

Senior Engineer responsible for design of two rail bridges. One a 414m continuous, concrete curved bridge to replace the Hapuawhenua viaduct presented the designers with several problems related to achieving the seismic solution while still satisfying service load criteria including creep and shrinkage. The other is a 94 m long continuous concrete bridge across the Taonui Stream. The Hapuawhenua Bridge was the winner of the 1988 New Zealand Concrete Society Prestressed Concrete Award.

Seismic Upgrading of a Suspended Ceiling, Government Printing Office, Masterton, 1984

Senior Engineer responsible for the assessment of options and for the preparation of contract documents for the preferred alternative of the seismic upgrade of 10,000 m2 of suspended ceiling. (Approx value of upgrade $0.5M). Commissioned by the Ministry of Works and Development.

The Effects of Earthquakes on Tunnels, 1984

Prepared report on the effects of earthquakes on tunnels. Commissioned by Reed Stenhouse.

Earthquake Engineering for Bridges in Papua New Guinea, 1984 - 1985

Senior Engineer responsible for a complete review of the manual prepared by BCHF in 1976. Review included incorporation of the zoning scheme originally derived in 1980 for building construction and further recommendations for the design and detailing of bridges to resist seismic motions. Commissioned by the Papua New Guinea Department of Works and Supply.

Seismic Risk Analysis for Recovery Boiler Structure – Kinleith New Zealand, 1984

Senior Engineer responsible for the preparation of a report deriving suitable input motions for use in a Time History Dynamic Design Procedure of the boiler support structure. Required a calculation of the expected occurrence of given levels of structural response at the site to give a basis for scaling the strong motion records and an evaluation of site dependent characteristics. (Approximate value $100M). Commissioned by NZ Forest Products Ltd.

Design for a 21.5 Storey Building 'Wisma Sudirman MD', Jakarta Indonesia, 1983

Senior Engineer responsible for the design of the reinforced concrete shear walls and the reinforced concrete slab system to comply with the Indonesian Codes of Practice. Commissioned by Metropolitan Developments of Jakarta.

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1978-1982

Design Engineer with Beca Carter Hollings & Ferner Ltd., working on projects which included:

- Design of Reinforced Concrete Columns for the Reformer Furnace Support Structure -GTG Project Motunui, 1981 (Florida, U.S.A.)
- Carried out design and supervised detailing for the 29 m high cantilevering columns supporting the reformer furnace and roof structures. For Davy McKee Corporation USA.

Seismic Risk Study for Motunui New Zealand, 1981 - (Revised 1983)

Engineer responsible for the derivation of the seismic risk for this major petrochemical complex for converting natural gas to gasoline (approx value $1,200M). Required an assessment of the expected frequency of occurrence of ground shaking and also an assessment of the risk of liquefaction at the site caused by ground motions. Commissioned by Bechtel Pacific Corporation.

A Study of Risk from Major Natural Events - Proposed Newspring Expansion for Tasman Pulp & Paper Company Ltd, 1980

Engineer responsible for preparation of sections of the report relating to the evaluation of seismic risk at various sites within New Zealand.

Papua New Guinea Design Loadings Code, 1980

Prepared sections of this code which was commissioned by the Papua New Guinea Department of Works & Supply.

Papua New Guinea Earthquake Study - Seismic Risk Analysis, 1980

Carried out a seismic risk analysis of PNG to establish seismic zones and to evaluate seismic lateral loadings for earthquake resistant design of buildings in PNG. Commissioned by the Papua New Guinea Department of Works & Supply.

Papua New Guinea Earthquake Study - Material Code Development, 1980-1981

Prepared sections for special provisions for seismic design of materials codes for reinforced masonry, reinforced concrete and structural steel. Commissioned by the Papua New Guinea Department of Works & Supply.

Design for the Wool House Development for the New Zealand Wool Board, 1979

Engineer responsible for carrying out a computer analysis of this 14-storey shear tower structure and also responsible for the design and detailing of the braced shear towers and foundations.

Design Proof Check of the ANZ Multi-storey Development, Lambton Quay Wellington, 1978

Carried out a design proof check of this 18-storey reinforced concrete framed building. This involved checking the seismic lateral load carrying capability of the beams, columns and piles to ensure the capacity design requirements of the New Zealand Loadings code were satisfied.

Hydrological Investigation - Patea Hydro Project, 1978

Involved in a probabilistic hydrological study of the Patea River catchment to determine maximum possible flow rates in Patea River.

Canterbury University Study 1977

In partial fulfilment of the requirements for Masters Degree carried out research into the effect of earthquake motions on multi-storey reinforced concrete framed structures. In particular this involved the determination of seismic load demands on columns of reinforced concrete framed structures designed to the then proposed New Zealand building code requirements using time history techniques.

Awards

1988 New Zealand Concrete Society Prestressed Concrete Award Hapuawhenua Railway Bridge (with Hollings, Catley).

1999 ACENZ Gold Award of Excellence for the Thorndon Overbridge project.

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1997 New Zealand Concrete Society Prestressed Concrete Award Sky Tower

1997 New Zealand Concrete Society Concrete Award Sky Tower.

1998 Sesoc Award for Structural Engineering Excellence for Sky Tower.

1998 ACENZ Gold Award of Excellence for Sky Tower

1999 NZIOB James Hardie Award for Excellence in the Building Profession in the category of Innovation for the Thorndon Overbridge Seismic Retrofit Project.

1999 ACENZ Gold Award of Excellence for Thorndon Overbridge Seismic Retrofit Project

2008 ACENZ Silver Award of Excellence for Istanbul Residential Buildings Seismic Retrofit Project

Publications

Over 38 papers and presentations to Learned Societies, Technical Conferences

"Seismic Load Demands on Columns of Reinforced Concrete Multistorey Frames" Dept. of Civil Engineering, University of Canterbury, Master of Engineering Report No 78/12, 1978.

"A Numerical Method for Seismic Risk Assessment" (co-author with J P Hollings). Proceedings Institution of Professional Engineers Conference – Christchurch, February 1982.

"The Development of Seismic Zones and the Evaluation of Lateral Loadings for Earthquake Resistant Design of Buildings in Papua New Guinea. Bulletin NZNSEE, Vol. 15, No. 3, Sept. 1982. (Co-author with I A N Fraser and J P Hollings).

"A Seismic Zoning Scheme for New Zealand including Lateral Load Derivation". (Co-author with J P Hollings). Proceedings 3rd South Pacific Conference in Earthquake Engineering, Wellington, May 1983.

"A Roofing Case Study": An example of the use of life cycle costing analysis for design and maintenance decisions". Proceedings IDEA 1985 Conference, 1985, (co author with I A N Fraser).

"The September 1985 Mexico Earthquakes: Preliminary Report of the New Zealand Reconnaissance Team" Bulletin NZNSEE, Vol. 18, No. 4, DOC 1985 (co-contributor with five others).

"The Design of a Curved 414m Long Continuous Concrete Railway Bridge to Resist Earthquake Motions." (co author with J P Hollings and T J Catley) Proceedings Earthquake Engineering Symposium 1986, Sydney.

"The Design of a Recovery Boiler Support Structure to Resist Earthquakes." (co author with J.P. Hollings and R D Sharpe) Proceedings Earthquake Engineering Symposium 1986, Sydney.

"Earthquake Performance of a Large Boiler (Co-author with J P Hollings and R D Sharpe). Proceedings 8th European Conference on Earthquake Engineering, Barcelona 1986 (Presented by Professor T Paulay)

"Design of a Curved 414m long Continuous Concrete Railway Bridge to Resist Earthquake Motions" (co-author with J P Hollings and T J Catley). Proceedings Pacific Concrete Conference, Auckland, 1988. Also published. Bulletin NZNSEE, Vol 22, No.1, March 1989.

"The September 1985 Mexico Earthquakes: Final Report of the New Zealand Reconnaissance Team" bulletin NZNSEE, Vol. 21, No. 1, March 1988 (co-contributor with five others).

"The Hapuawhenua Railway Bridge" Proceedings Bridge Design and Research Seminar 1990, RRU Bulletin 84.

"Strengthening of the Wellington Town Hall" Proceedings NZNSEE Conference, 1993 also published Bulletin NZNSEE Vol 26, N° 2, June 1993.

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"NZS4203:1992 General Structural Design and Structural Loadings - An Outline Paper" (co-authored with A King) 1993 IPENZ Conference.

"Seismic Design Aspects of Sky Tower in Auckland, NZ", (co-authored with D Whittaker). Seminar on Tall Building, Singapore Institution of Engineers, 1995.

"Consideration of Earthquakes in the Design of Sky Tower", (co-authored with R D Sharpe). Proceedings NZNSEE Conference, 1996.

"Seismic Strengthening of the Odeon Theatre, Gisborne", (co-authored with S A Edmonds and J McGregor). Proceedings NZNSEE Conference, 1996.

"The Design of Sky Tower", presented at the IPENZ Conference, Dunedin, 1996.

"An Earthquake Hazard Model for the Central Southern Alps, South Island, NZ", (co-authored with A G Hull, K R Berryman). Proceedings NZNSEE Conference, 1996.

"Design of Sky Tower - New Zealand's Tallest Structure", (co-authored with D H Turkington), Proceedings NZ Concrete Conference, 1995. Also published in FIP Journal, 1995.

"The Top 100 m of Sky Tower", Proceedings of the 59th Annual Conference of the NZ Institute of Welding (Inc), 1997.

"Sky Tower Mast - Design Challenges", (co-authored with D H Turkington and H M Irvine), Proceedings ASEC Conference, 1998.

"Thorndon Overbridge Seismic Retrofit Design and Construction", (co-authored with A Powell and Z Rakovic), Proceedings ASEC Conference, 1998.

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"A"

"Hollow Core Floors – a Regulators Perspective" (co authored with four others) Proceedings, New Zealand Society for Earthquake Engineering Conference, 2007.

"Performance Assessment of Existing Buildings in New Zealand" (co-authored with 2 others) Proceedings, New Zealand Society for Earthquake Engineering Conference, 2008.

"Observations and Musings of a Practicing Earthquake Engineer – a tribute to Tom Paulay". Proceedings, New Zealand Society for Earthquake Engineering Conference, 2010.

Technical Investigation into the Performance of Buildings in the Christchurch CBD in the 22 February Christchurch Aftershock

Expert Panel
Terms of Reference

1 General

Overall Terms of Reference for the investigation are given in Attachment 1.

Investigations will look at the expected performance of the buildings, when they were built, the impact of any alterations, compliance with the code at the time, and the reasons for the collapse. The investigations will focus only on the technical findings and are not to address liability.

The Department of Building and Housing has overall responsibility for the outcome of the investigation and has appointed:

- Engineering consultants to investigate the subject buildings
- A panel of experts to assist in achieving the the overall objectives of the investigation

These Terms of Reference for Expert Panel describe the roles and responsibilities of the expert panel in the context of the overall Terms of Reference for the investigation.

2 Outline Approach and Outputs

The main outputs of the investigation will be:

- Consultant technical investigation reports on each building
- A report prepared by the Expert Panel to the Department
- A Department report to the Minister on the outcome of the investigation.

The investigating consultants will be responsible for their own work and for determining the inputs they use to reach their conclusions.

The consultant reports will be attachments to the Expert Panel Report.

The Department Report will be based on material in the consultant reports and the Expert Panel Report.

3 Roles and Responsibilities

The panel members have been chosen to provide a background of experience in the range of matters related to the planning, design, approval and construction of buildings.

In general, it is expected that, individually and collectively, panel members will help the Department to provide comprehensive, accurate and authoritative accounts of why the buildings collapsed and what the implications are for the Building Act and Code.

Particular roles and responsibilities include:

- Providing guidance and direction to the investigation

- Advising on the scope and extent of investigation necessary to achieve overall objectives
- Monitoring and reviewing the approaches, investigations, data and outputs of the engineering consultants
- Recommending to the Department any changes in the scope and nature of work necessary to address the matters for investigation fully, accurately and authoritatively.
- Reviewing and approving the engineering consultant reports on each building.
- Producing an overview report addressing the matters for investigation and indicating any issues for further consideration by the Department in their role as regulator responsible for the Building Act and Code.

4 Timeframe

The Department Report to the Minister is due by 31 July 2011. The Expert Panel Report is due by 30 June 2011. These deadlines may be revised if necessary for the investigation to achieve its objectives.

5 Conflicts of Interest

General

Panel members must declare all conflicts or potential conflicts of interest throughout the investigation. A register will be maintained which will be accessible to all members.

*Interaction with engineering consultan*ts

Panel members may provide comments to consultants in their role as panel members, but may not provide advice. Panel members are to advise other panel members of all such comments given as soon as possible.

Tonkin & Taylor may provide advice to consultants provided that Peter Millar is not personally involved.

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5.0 Canterbury Television Building

5.1 Overview

The six-level Canterbury Television (CTV) Building located at 249 Madras Street, Christchurch suffered a major structural collapse on 22 February 2011 following the Magnitude 6.3 Lyttelton aftershock. Shortly after the collapse of the building a fire broke out in the stairwell and continued for several days.

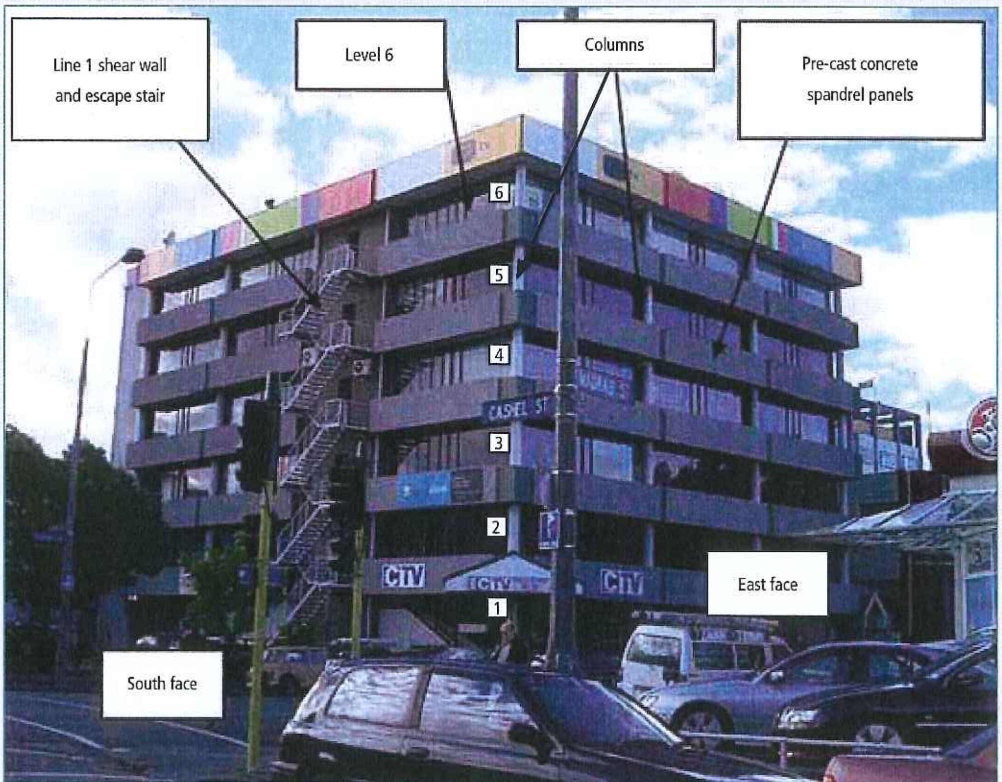

Figure 5.1: Canterbury Television Building in 2004 (Photo credits: Phillip Pearson, derivative work: Schewede66)

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.

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Factors that contributed (or may have contributed) to the failure include:

- higher than expected horizontal ground motions
- exceptionally high vertical ground motions
- lack of ductile detailing of reinforcing steel in all columns
- low concrete strengths in critical columns
- interaction of perimeter columns with the spandrel panels
- separation of floor slabs from the north core
- accentuated lateral displacements of columns due to the asymmetry of the shear wall layout
- accentuated lateral displacements between Levels 3 and 4 due to the influence of masonry walls on the west face
- the limited robustness (tying together of the building) and redundancy (alternative load paths) which meant that the collapse was rapid and extensive.

A number of key vulnerabilities were identified which affected the structural integrity and performance of the building. These included: high axial loads on columns; possible low concrete compression strength in critical columns; lack of ductile detailing and less than the minimum shear reinforcing steel requirements in columns; incomplete separation between in-fill masonry and frame members in the lower storeys on the west wall; and the critical nature of connections between the floor slabs and north structural core walls.

Examination of building remnants, eye-witness reports and various structural analyses were used to develop an understanding of likely building response. A number of possible collapse scenarios were identified. These ranged from collapse initiated by column failure on the east or south faces at mid to high level, to collapse initiated by failure of a more heavily loaded internal column at mid to low level. The basic initiator in all scenarios was the failure of one or more non-ductile columns due to the forces induced as a result of horizontal movement between one floor and the next. The amount of this movement was increased by the plan irregularity of the lateral load resisting structure. Additional inter-storey movement due to possible failure prior to column collapse of the connection between the floor slabs and the north core may have compounded the situation.

The evaluation was complicated by the likely effect of the high vertical accelerations and the existence of variable concrete strengths. It was further complicated by the possibility that the displacement capacities of columns on the east or south faces were reduced due to contact with adjacent spandrel panels. Many reasonable possibilities existed. In these circumstances it has been difficult to identify a specific collapse scenario with confidence.

The most studied collapse scenario, which was consistent with the arrangement of the collapse debris and eye-witness reports of an initial tilt of the building to the east, involved initiation by failure of a column on the mid to upper levels on the east face. Inter-storey displacements along this line were higher than most other locations and there was the prospect of premature failure due to contact with the spandrel panels. For this scenario, it was recognised that contact with the spandrel panels would have reduced the ability of the column to carry vertical loads as the building swayed. However, the displacement demands of the 22 February 2011 event were such that column failure could have occurred even if there had been no contact with the spandrels. Loss of one of these columns on the east face would have caused gravity load to shift to the adjacent interior columns. Because these columns were already carrying high vertical loads and were subjected to lateral displacements, collapse would have been likely.

The low amount of confinement steel in the columns and the relatively large proportion of cover concrete gave the columns little capacity to sustain loads and displacements once strains in the cover concrete reached their limit. As a result, collapse was sudden and progressed rapidly to other columns.

Once the interior columns began to collapse, the beams and slabs above fell down and broke away from the north core. The south wall, together with the beams and columns attached to that wall, then fell northwards onto the collapsed floors and roof.

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Other scenarios considered had different routes to the failure of a critical column, including scenarios involving diaphragm disconnection from the north core. In all cases, once the critical column failed, failure of other columns followed.

5.2 Investigation

A technical investigation into the reasons for the collapse was commissioned by the Department of Building and Housing and was undertaken by Hyland Consultants Limited and StructureSmith Limited.

The investigation consisted of:

- examination of the remnants of the collapsed building
- review of available photographs
- interviews with surviving occupants, eye-witnesses and other parties
- review of design drawings and specification for the original work and structural modifications
- structural analyses to assess the demand on and capacity of critical elements
- synthesis of information to establish the likely cause and sequence of collapse.

A separate report covering the Site Examination and Materials Testing undertaken for the investigation was prepared by Hyland Consultants Limited.[2]

5.3 Building description

The developer of the building gained a building permit from the Christchurch City Council in September 1986, and construction progressed through 1986 and 1987. The structure of the CTV Building was rectangular in plan, and was founded on pad and strip footings bearing on silt, sand and gravels. Lateral load resistance was provided by reinforced concrete walls surrounding the stairs and lifts at the north end, and by a reinforced concrete wall on the south face. Refer to Figure 5.1 and Figure 5.2. On the west face, reinforced concrete block walls were built between the columns and beams for the first three levels. Reinforced concrete spandrel panels were placed between columns at each level above ground floor on the south, east and north faces. Spandrel panels perform various functions including fire protection, sun control and architectural design.

The reinforced concrete floors were cast in-situ on permanent metal forms. The slabs were supported by reinforced concrete beams around the perimeter and internally, running in the east-west direction. The beams were, in turn, supported principally by circular reinforced concrete columns.

The building was designed with ductile reinforced concrete shear walls and with a lightweight roof supported on steel framing above Level 6. The walls of the north core and south wall were designed to provide all the lateral stability needed for earthquake actions. As such, they were required to be stiff and ductile. The columns (and the frames formed by columns and beams) were designed to carry gravity loads only on the basis that the lateral displacements of these gravity elements would then be restricted by the stiff wall elements. Provided the walls were designed to keep displacements within prescribed limits, the beams and columns were not required to be detailed to behave in a ductile manner.

The CTV Building was originally designed as an office building but changed use over time to include an education facility and radio and television studios for Canterbury Television.

Note that the six floor levels are numbered with the ground floor being Level 1 for the CTV Building. Refer to Figure 5.1.

Grid line locations are defined in Figure 5.2.

2 Report to the Department of Building and Housing on CTV Building Site Examination and Materials Tests, Hyland Fatigue and Earthquake Engineering (January 2012).

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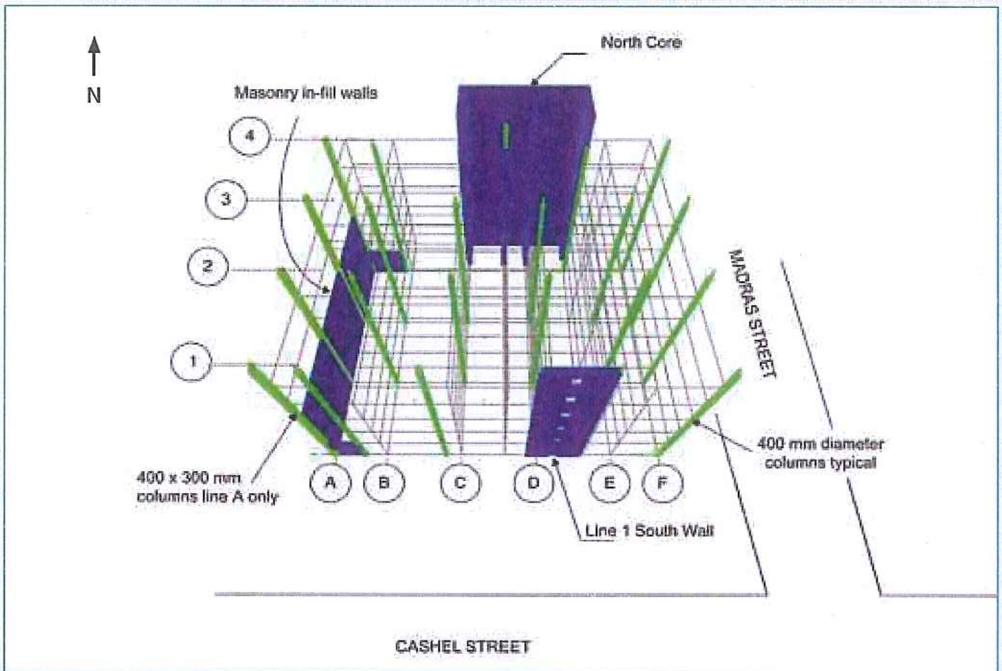

Figure 5.2: Building orientation and grid line references

5.4 Structural modifications

Following an independent consulting engineer's pre-purchase review in January 1990, drag bars were designed by the design engineer in October 1991 and subsequently installed at Levels 4, 5 and 6 to improve the connections between the floor slabs and the walls of the north core (refer to Figure 5.7). These connections were vital to the integrity of the building since the walls provide lateral stability and strength to the building.

Other structural modifications to the building included the formation of a stair opening in the Level 2 floor next to the south wall. Coring of the floors for pipes was found to have occurred at the locations where the slab pulled away from the lift core during the collapse. However, neither the stair opening nor the coring of floors appears to have been a significant factor in the collapse on 22 February 2011.

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5.5 Earthquake and other effects prior to 22 February 2011

4 September 2010

Damage to the CTV Building structure was observed and reported after the 4 September 2010 earthquake, as follows:

- Minor cracking to the south wall and adjacent floors.
- Minor structural damage including fine shear cracks in the north walls.
- Fine cracking of several perimeter columns in the upper floors.
- Several cracked or broken windows
- Floor to ceiling cracks at the junction of the lift doors wall and return walls on Level 6.

This reported damage appeared to be relatively minor and was not indicative of a building under immediate distress or having a significantly impaired resistance to earthquake shaking.

Demolition of neighbouring building

The building next door to the CTV Building began to be demolished almost immediately after the 4 September 2010 earthquake and demolition continued until a week before the 22 February 2011 aftershock. The demolition work caused noticeable vibrations and shuddering in the CTV Building which was a significant concern to the tenants. The view of the investigation team, based on a general description of the demolition operation and photos of the demolition process, was that the demolition would have been unlikely to have caused significant structural damage to the CTV Building.

26 December 2010

Eye-witnesses advised of no significant structural damage but some non-structural damage after the 26 December 2010 aftershock. There were no available reports on the condition of the building after this event, but photographs of this damage indicate that it was minor.

5.6 Collapse on 22 February 2011

The 22 February 2011 aftershock caused the sudden and almost total collapse of the CTV Building. Shortly after the collapse of the building a fire broke out in the stairwell and continued for several days.

It is evident that the building collapsed straight down almost within its own footprint and that the south wall (with stairs attached) fell on top of the floor slabs. The north core remained standing after the collapse.

Eye-witnesses spoken to as part of the investigation saw the building sway and twist violently. One, with a view of the south and east faces, described the whole exterior exploding and seeing the cladding failing and falling, and columns breaking. The upper levels of the building were seen to tilt slightly to the east and then come down as a unit on the floors below. The building appeared to collapse in on itself and this was confirmed by the final position of the collapsed slabs and the fact that external south face framing collapsed on top of the floor slabs.

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5.7 Eye-witness accounts

Interviews were conducted with 16 eye-witnesses to the CTV Building collapse in order to identify consistent qualitative observations about the collapse. Four of the eye-witnesses interviewed were inside the CTV Building at the time of collapse and 12 were in the street or in other buildings next door with a clear line of sight to portions of the CTV Building as it collapsed. These insights provided clues to what actually happened to the structure of the building in the collapse event.

Although eye-witnesses interviewed in the investigation gave varying responses on the speed of collapse of the CTV Building, the majority felt it went down in a matter of seconds. Eye-witnesses gave a range of responses on the speed of collapse, including responses such as "it crumbled in seconds", there was "only five seconds warning from the time the earthquake hit", and it came "down in 30 seconds or quicker". Where timing was mentioned, eye-witness responses referred to seconds rather than minutes for the collapse to occur.

5.8 Examination of collapsed building

Inspections and photographs

The examination of the collapsed building involved physical examination of the Madras Street site including the north core, and examinations of the columns that had been extracted from the building and taken to a secure area at the Burwood Eco Landfill. Photographs of the collapse taken by the public prior to debris being removed, and by rescue agencies and the media during the removal of debris, were used to help ascertain the likely collapse sequence and behaviour of the CTV Building.

A review of photographs taken by rescue agencies as debris was removed provided valuable information on the sequence of the collapse.

Site examination and materials testing

Following the completion of rescue and recovery efforts, the Madras Street site was examined and material samples collected and tested. Columns at the Burwood Eco Landfill were also extracted and tested. Care was taken to select samples that were not affected by the post-earthquake fire and which were away from clearly damaged areas.

Materials testing was conducted on reinforcing steel, wall concrete, slab concrete and beam concrete to assess compliance with standards of the day. The main findings from this testing included the following:

- All reinforcing steel appeared to conform to the standards of the day.
- Concrete strengths in concrete from south wall and north core wall samples were found to be greater than specified.
- Tests on 26 column samples (21% of all CTV Building columns) indicated that, at the time of testing, the column remnants from Levels 1 to 6 had a mean concrete strength of 29.6 MPa, with measurements ranging from 17.3 MPa to 50.3 MPa.

The position in relation to the column samples is summarised in Figure 5.3. The black line indicates the inferred distribution of concrete strengths from the tests. The other three distributions are the expected strength distributions at 28 days from pouring of concrete based on the specified concrete strengths (after 28 days) which were 35 MPa for Level 1 columns, 30 MPa for Level 2, and 25 MPa for Levels 3 to 6. Even though it is not known which of the measurements applies to which expected strength distribution, it can be seen that a higher than expected proportion of the results is below the specified level in all cases.

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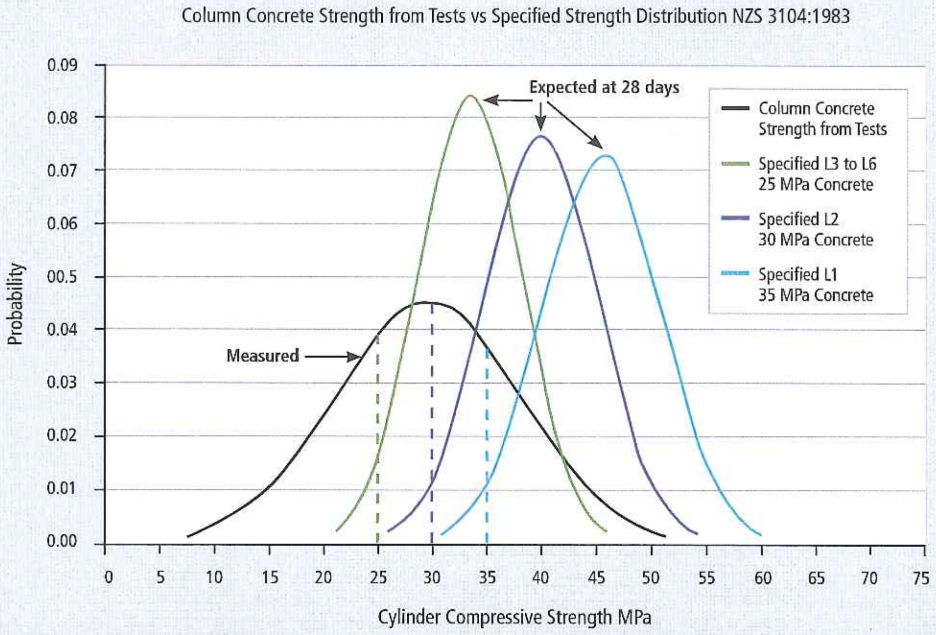

Figure 5.3: Column concrete test strengths

While it is recognised that the tests were conducted on members that had been involved in the collapse, the results indicate that column concrete strengths were significantly less than the expected strength considering the specified strengths, the conservative approach to achieving specified strengths, and the expected strength gain with age.

5.9 Collapse evaluation

Approach and limitations

The aim of the evaluation was to identify, if possible, the most likely collapse scenario. The results of the structural analyses undertaken were considered in conjunction with information available from eye-witness accounts, photographs, physical examinations and selective sampling and testing of remnants.

The analyses were needed to develop an understanding of the likely response of the building to earthquake ground motions and the demands this response placed on key structural components. It was recognised that any analyses for the 22 February 2011 event must be interpreted in the light of the observed condition of the CTV Building after the earthquake on 4 September 2010 and the 26 December 2010 aftershock, and the possibility that these and other events could have affected the structural performance of the building.

Elastic response spectrum analyses (ERSA) were undertaken similar to those required by the design standards of the time (NZS 4203:1984 and NZS 3101:1982) and also using levels of response corresponding to the ground motion records. These analyses provided insights into the design intentions and the likely response of the building in the 4 September 2010, 26 December 2010 and 22 February 2011 events.

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5.0 CANTERBURY TELEVISION BUILDING

Non-linear time history analyses (NTHA) were undertaken using actual records of the 4 September 2010 earthquake and the 22 February 2011 aftershock from other nearby sites. The response of the CTV Building to these ground motions and the structural effects on critical elements, particularly the columns and floor diaphragm connections, was assessed.

The approach taken was to:

- carry out a number of structural analyses of the whole building to estimate the demands (loads and displacements) placed on the building by the earthquakes
- evaluate the capacities (ability to resist loads and displacements) of critical components such as columns
- compare the demands with the capacities to identify the structural components most likely to be critical
- identify likely collapse scenarios taking account of other information available.

Structural analyses and evaluation included:

- elastic response spectrum analyses (ERSA) of the whole building
- non-linear static pushover analyses of the whole building
- non-linear time history analyses (NTHA) of the whole building
- elastic and inelastic analyses of the easternmost frame (Line F).

The demands from these analyses were compared with the estimated capacities of critical elements to assess possible collapse scenarios and to reconcile the results of the analyses with the as-reported condition of the building on 4 September 2010.

Overall, the approach for the analyses was to:

- use established techniques to estimate structural properties and building responses
- use material properties which were in the middle of the range measured
- examine the effects of using ground motions (or response spectra records derived from them) from several recording stations
- apply these ground motions or response spectra without modifying their nature or scale
- consider the variability and uncertainties involved in each case when interpreting results of the analyses or comparisons of estimated demand with estimated capacity.

The characteristics of the building and the information from inspections and testing required consideration of a number of possible influences on either the response of the building or the capacities of elements, or both. Principal amongst these were the following:

- The masonry wall elements in the western wall (Line A) up to Level 4 may have stiffened the frames.
- The concrete strength in a critical element could vary significantly from the average values assumed for analysis.
- The spandrel panels on the south and east faces of the building may have interacted with the adjacent columns.
- The floor slabs may have separated from the north core.

On top of this, consideration needed to be given to the variability and uncertainties inherent in structural analysis procedures. In this case, particular consideration was given to the following:

- The possibility that the ground motions or elastic response spectra used in the analyses may have differed significantly in nature and scale from those actually experienced by the building.
- The stiffness, strength and non-linear characteristics of structural elements assumed for analysis may have differed from actual values. This possibility can result in differences from reality in the estimated displacements of the structure and/or the loads generated within it.
- Estimating the effects on the structure of the very significant vertical ground accelerations was subject to considerable uncertainty.

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In summary, the analyses were necessarily made with particular values, techniques and assumptions, but the above limitations were considered when interpreting the output. It should be evident that determination of a precise sequence of events leading to the collapse is not possible. Nevertheless, every effort was made to narrow down the many options and point towards what must be considered a reasonable explanation even though other possibilities cannot be discounted.

Overall, the output of the NTHA analyses was not inconsistent with the reported condition of the building after 4 September 2010. The limited available evidence of the building condition after 4 September 2010 leaves room for a range of interpretations of the likely maximum displacements in the 4 September 2010 event. However, the conclusions drawn from the analyses are not particularly sensitive to the level of demand assumed by the NTHA, with indications that collapse could have occurred at lower levels of demand.

Comparisons of demand and capacity of structural elements have been made with general acknowledgement of the possibility that the actual building response may have differed from that calculated in any analysis.

The Panel supports the general conclusions as to the reasons for the collapse of the CTV Building. However, because of the range of factors noted above which are subject to variability and uncertainty, there was considerable debate between Panel members and the consultants on the relative weight that should be given to each of those factors. Although in agreement on the key outcomes, some Panel members and the consultants are not of one mind in relation to some of the detail presented in the consultants' report, particularly some detailed technical issues relating to the ERSA and NTHA analyses, the identification of critical columns, the extent of influence of the spandrel panels, and the timing of any separation that may have occurred between the floor slabs and the north core.

Soils and foundations

Surveys of the site after the collapse indicated that there had been no significant vertical or horizontal movement of the foundations. There was no evidence of liquefaction within the site. Soil and foundation elements were modelled in the structural analyses based on specialist geotechnical advice.

Ground shaking records for analyses

For the non-linear time history analyses, seismic ground motions at the CTV site were deduced from four strong-motion recordings surrounding the CBD, as follows:

- Botanical Gardens – CBGS
- Cathedral College – CCCC
- Christchurch Hospital – CHHC
- Rest Home Colombo Street – REHS.

The NTHA analyses were carried out using records from the CBGS, CCCC and CHHC sites so as to provide some indication of the effects of variability in ground shaking. While the REHS record showed significantly higher amplification than the others, both with respect to Peak Ground Accelerations (PGA) and spectral accelerations (building response), the soil profile was markedly different from that at the CTV site. The sites of the other three stations (CBGS, CCCC, CHHC) were considered to have generally similar soil profiles to the CTV site, consisting of variable silts, silty sands and gravels overlying dense sands. Geotechnical specialists recommended that the REHS record be disregarded and that the CTV site response be taken as similar to the average of the other three stations.

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For the elastic response spectrum analyses, spectra were developed for the September, December and February events using the closest sites possible at the time with compatible geotechnical conditions. These included the Westpac building and the Police Station, CHHC and CCCC. The average of the resultant response at each period of vibration recorded from the various instruments was used to develop an averaged maximum response spectra for analysis.

Critical vulnerabilities

Examination of the CTV Building design drawings indicated a number of vulnerable features or characteristics that could have played a part in the collapse. These vulnerabilities, which are outlined below, were the focus of attention during the investigation.

Columns

Details of a typical 400mm diameter column are shown in Figure 5.4. Vulnerabilities identified in relation to column structural performance were:

- non-ductile reinforcement details in the columns
- less than required minimum spiral reinforcing for shear strength
- relatively large proportion of cover concrete in the columns
- possibility of significantly lower than specified concrete strength in critical columns
- lack of ductile detailing in beam-column connections.

The lack of ductility in the columns made them particularly vulnerable and they were the prime focus of the analyses. The ability of a column to sustain earthquake-induced lateral displacements depends on its stiffness, strength and ductility. Established methods were used to estimate the capacity of critical columns to sustain the predicted displacements without collapse.

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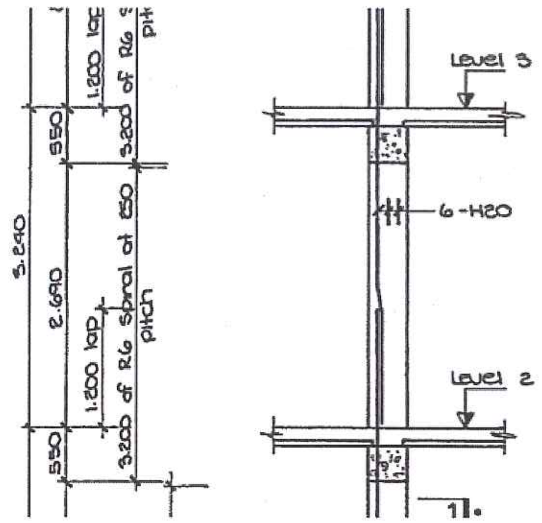
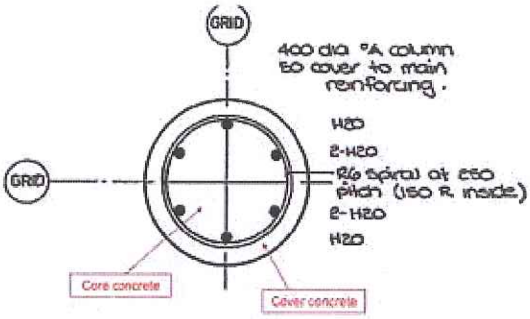
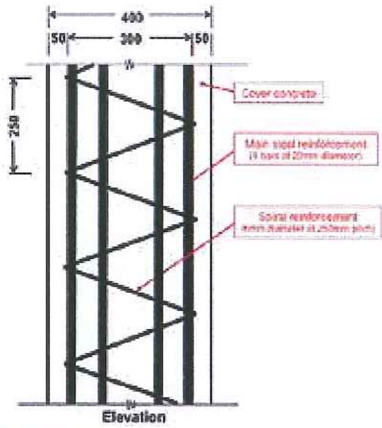


Typical column reinforcing
(from Design Engineers drawings)



Cross-section



Elevation

Figure 5.4: Typical 400mm diameter column

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Spandrel panels

A plan and a cross-section of the typical column and spandrel panel arrangement are shown in Figure 5.5.

The pre-cast reinforced concrete spandrel panels were fixed to the floor slab and were placed between columns. The gap between the ends of adjacent spandrels was specified to be 420mm giving a nominal 10mm gap either side between the spandrel and the column. It is possible that these gaps varied from the nominal 10mm and it is estimated they may have ranged between 0 and 16mm. It is not known what the sizes of the gaps actually were, but analyses showed a significant reduction in column drift capacity for the case where no gap was achieved. Forensic evidence indicated that interaction may have occurred between some columns and adjacent spandrel panels in the 22 February 2011 event. There were also indications of cracking reported in some of the upper level columns after the September earthquake that may have indicated some interaction with the spandrel panels.

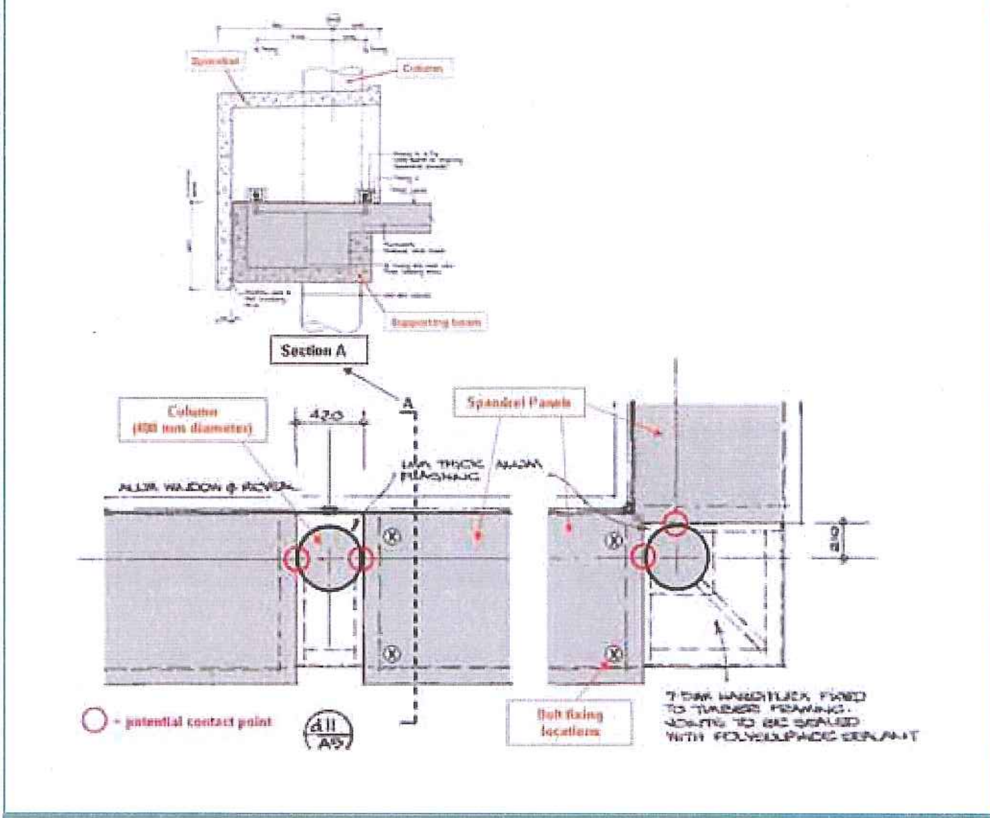

Figure 5.5: Spandrel panel detail

Irregularities/lack of symmetry

Potential vulnerabilities identified were:

- lack of symmetry in plan of the concrete shear walls (north core and south wall)
- vertical and plan irregularity due to lack of separation between the frame and masonry infill walls on the west face.

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It was considered that the lack of symmetry in plan could cause displacements on the south and east faces to increase as the building rotated in plan. Figure 5.6 illustrates the results of one examination of this effect. The centre of mass indicates where the lateral forces would act. The centre of rigidity indicates where lateral forces, at Level 4, would be resisted. The horizontal distance between these points is a measure of the tendency of the building to twist when subject to horizontal ground motions.

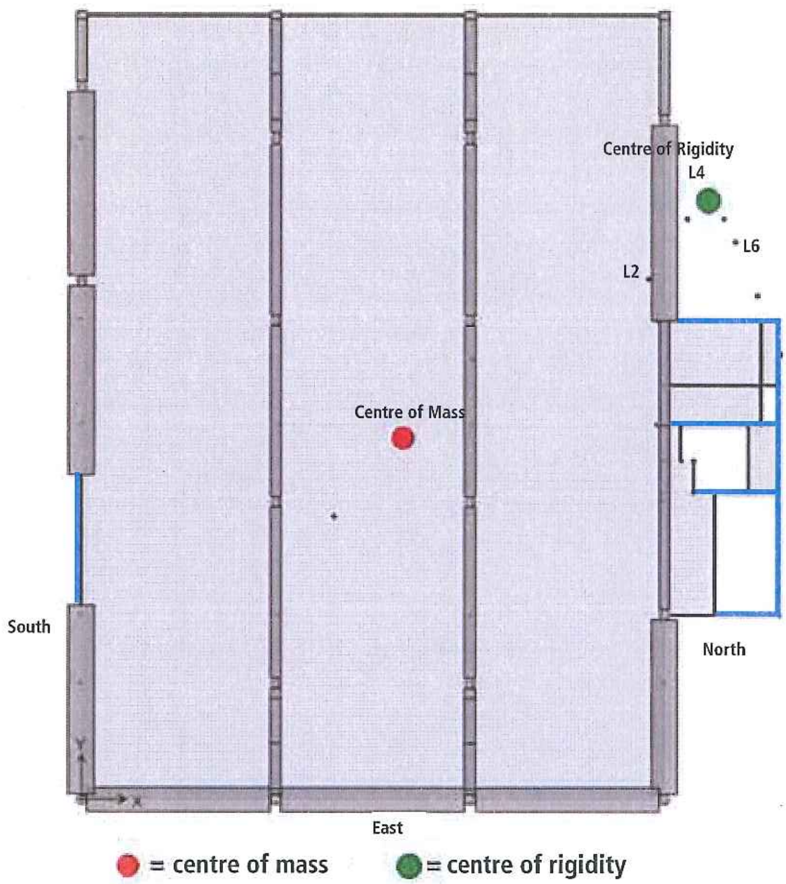

Figure 5.6: Plan irregularity

Diaphragm connection

Figure 5.7 shows plans of the area where a typical floor slab (shaded grey) meets the stabilising walls of the north core (shaded blue). The large lateral forces from the floor slab must be transferred to the walls at the (limited) places where slab and wall elements meet and through the drag bars (shaded red) which were added at Levels 4, 5 and 6 in October 1991. These connections were seen as vulnerable and there was a possibility that the diaphragm (slab) would separate from the walls, resulting in increased lateral displacements and higher demands on critical columns.

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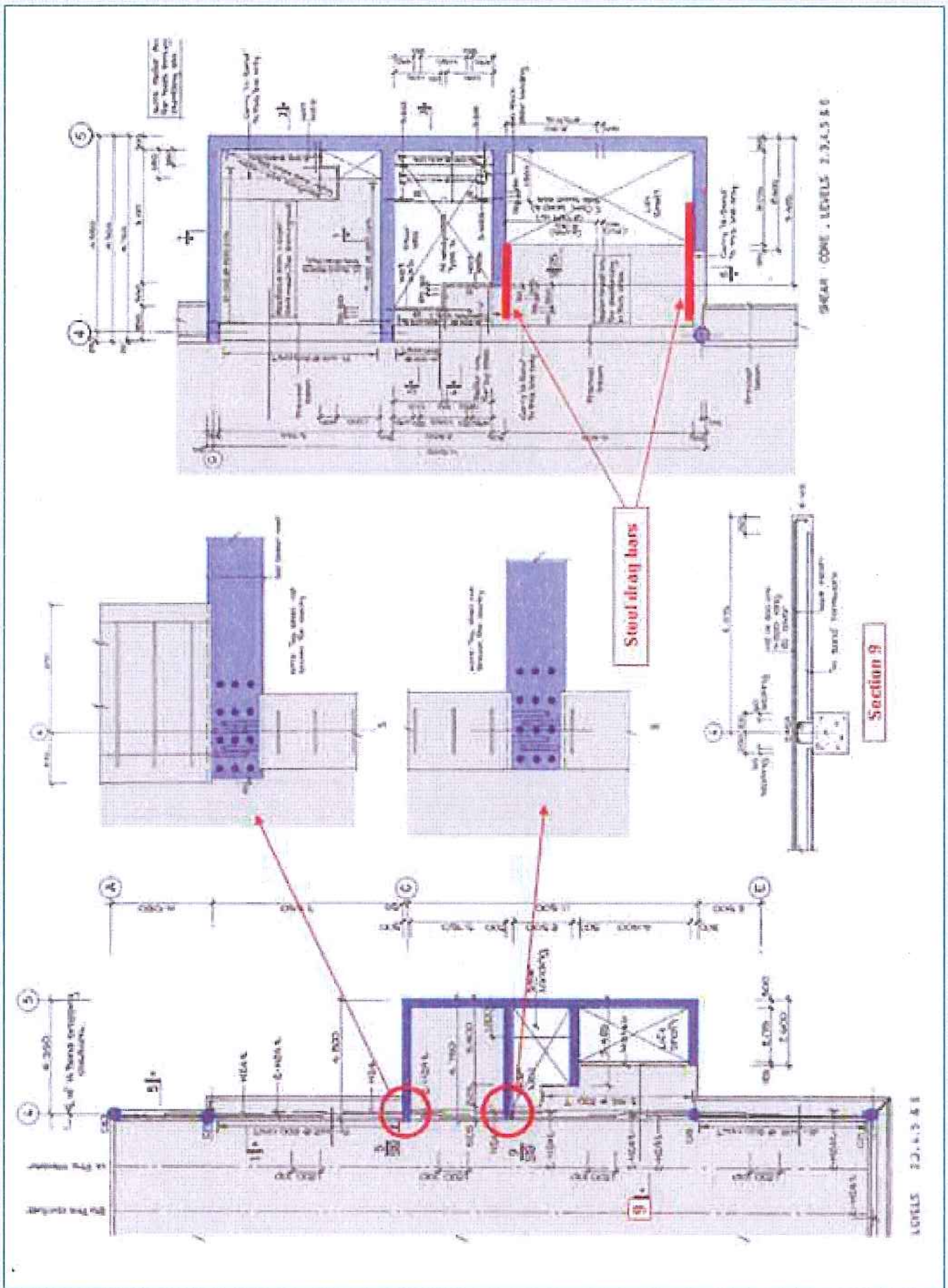

Figure 5.7: Diaphragm connections at north core

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Collapse initiators examined

Four potential collapse initiation scenarios were identified for evaluation:

1. Column failure on Line F or Line 1. This involved collapse initiation as a result of column failure on one of these lines, probably in a mid to upper level, with or without the influence of spandrel interaction. A Line F initiation was noted as being consistent with the arrangement of the collapse debris and eye-witness reports of an initial tilt to the east.
2. Column failure on Line 2 or Line 3. Collapse in this case would be initiated by failure of a column at mid to low level, under the combined effects of axial load (gravity and vertical earthquake) and inter-storey displacement. Low concrete strength could have made this scenario more likely.
3. Column failure due to diaphragm (slab) disconnection from the north core at Level 2 or Level 3. In this scenario, the diaphragm separated from the north core causing a significant increase in the inter-storey displacements in the floors above and below. The nature of the separation and resulting movement of the slab would have an influence on which of these highly loaded columns was the most critical. It was noted that no drag bars were installed at these levels.
4. Column failure due to diaphragm (slab) disconnection from the north core at Levels 4, 5 or 6. This scenario has similar characteristics to scenario 3 but involves failure of drag bars and adjacent slab connections to the north core. A compounding factor in this scenario is the effect of uplift of the slab/wall connection due to northwards displacement of the north core.

The effects of diaphragm (slab) disconnection were not modelled but disconnection at any level would lead to increased lateral displacements.

Figure 5.8 outlines the key considerations involved in evaluating these scenarios.

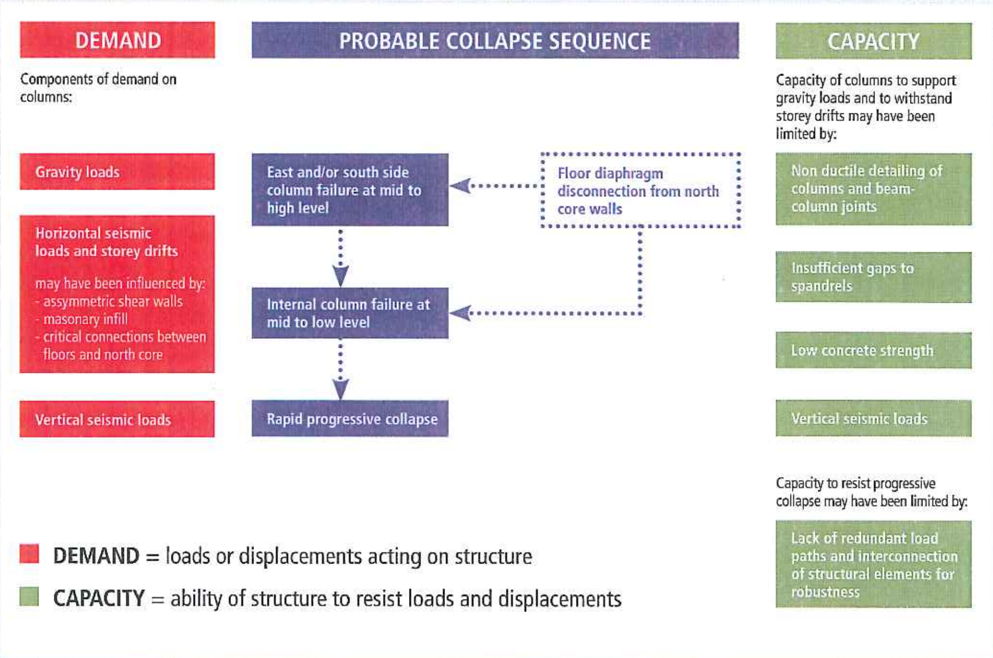

Figure 5.8: Collapse sequence evaluation

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Critical column identification

Analyses showed that drift (ie lateral displacements) demands were generally greater at the upper levels of the structure than at lower levels. For drifts in the north-south direction, the Line F (east side) columns were more vulnerable than columns on other lines because they formed a moment frame with the stiff façade beams and they may also have interacted with the spandrel panels. Drift demands in the east-west direction were greater towards the southern side of the building, being more distant from the stiff and strong north core walls. Line 1 (south side) columns also formed a moment frame with the stiff façade beams, and would have been subject to high drift demands in the east-west direction. However, the columns on Line 1 were protected to some extent by the south wall and so were considered to be less vulnerable than the columns on Line F.

The columns on Line 2 were seen as potentially vulnerable. While the lateral displacements (drifts) may have been less than on Line 1, these internal columns supported additional gravity load (with floor slabs all around). They also may have been more vulnerable to vertical acceleration effects due to the higher axial loads carried. Thus it was recognised that the reduced drift demand could have been matched or exceeded by a reduction in capacity to sustain the drifts imposed.

Taking the above factors into account, critical columns were identified on Lines F and 2 by examining the ratio of drift demand to column capacity at various levels. This process resulted in the identification of two "indicator" columns – one at Level 3 at grid position F2 and one at Level 3 at grid position D2. These particular columns were chosen because, based on maximum drifts from the NTHA, and assuming average concrete strengths, the ratio of lateral displacement demand to column capacity would be greatest in these columns.

In making these comparisons, it was recognised that the existence of low concrete strength, vertical acceleration effects, diaphragm separation and/or a different level of interaction with a spandrel panel could mean that a column in another location could have initiated failure.

5.10 Key data and results

Elastic response spectra

Figure 5.9 shows the basic "response spectra" used in the elastic response spectrum analyses. In this graph, the vertical axis represents the expected response of a building to the ground shaking. The horizontal scale shows the natural period of vibration of a building (low buildings generally having low periods and high buildings having high periods). The natural vibration period of the CTV Building was around 1.0 second.

The graphs give an indication of the relative intensities of ground shaking and expected building response on 4 September 2010, 26 December 2010 and 22 February 2011 (solid lines) and the response spectra used for design in 1986 when the CTV Building was designed (dotted lines). The upper dotted line represents full "ultimate" demand level which may be compared with the solid lines derived for the earthquake events.

Although direct comparison of such spectra can be misleading, it can be seen that at a period of 1.0 second, the acceleration shown for the 22 February 2011 event significantly exceeds the full 1984 value required for the design of elastically responding structures, while the acceleration shown for the 4 September 2010 event is around 65 percent of the full 1984 value. The CTV Building had been designed for ductile response using forces derived from the lowest design spectra shown in Figure 5.9.

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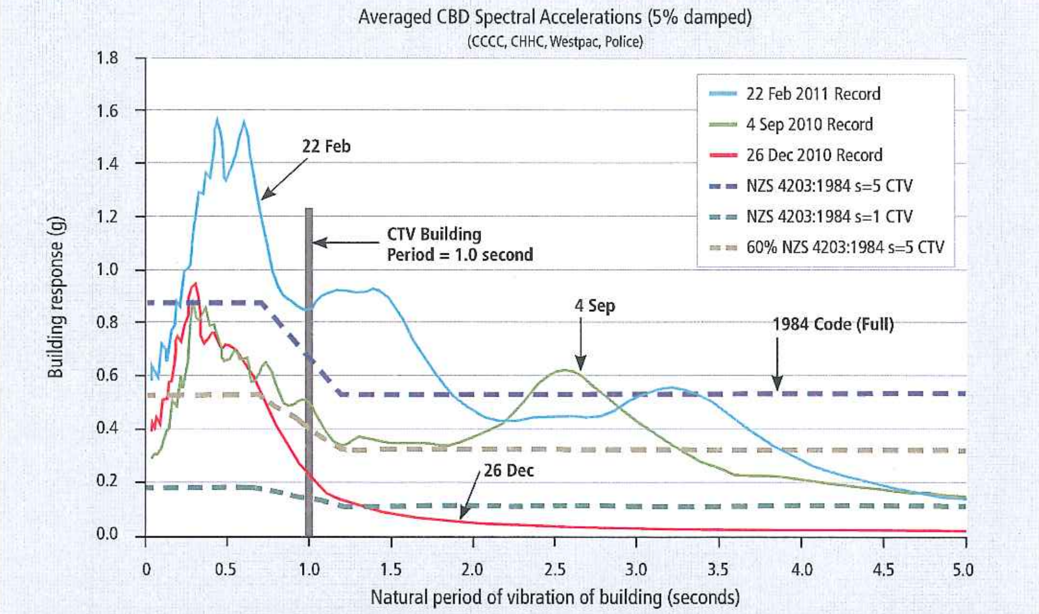

Figure 5.9: Response spectra used in CTV elastic response spectra analyses

Demand versus capacity

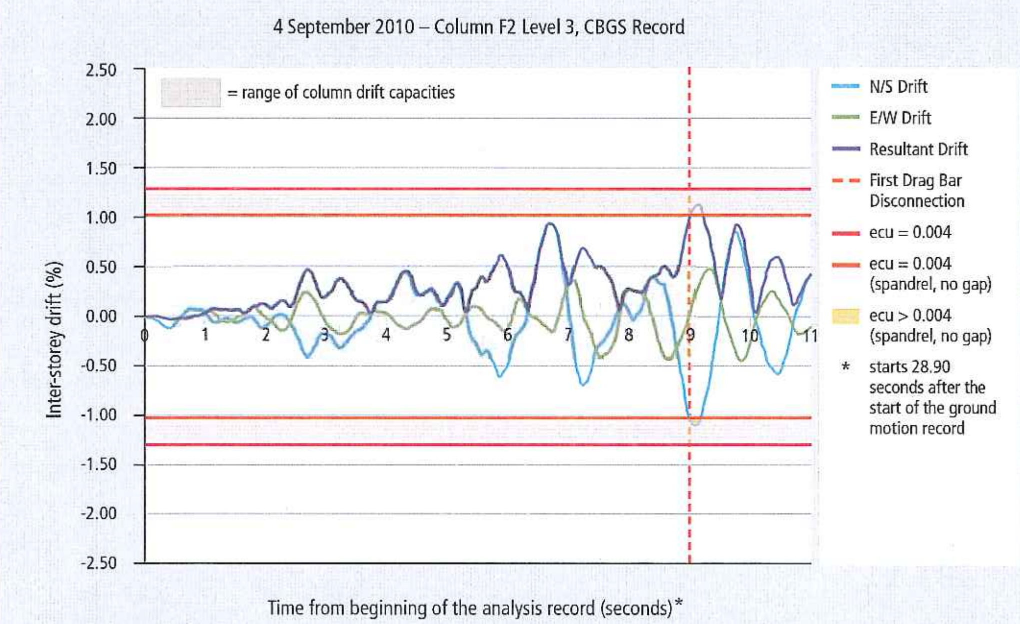

Figure 5.10: Comparison of drift demand and capacity – column F2 Level 3

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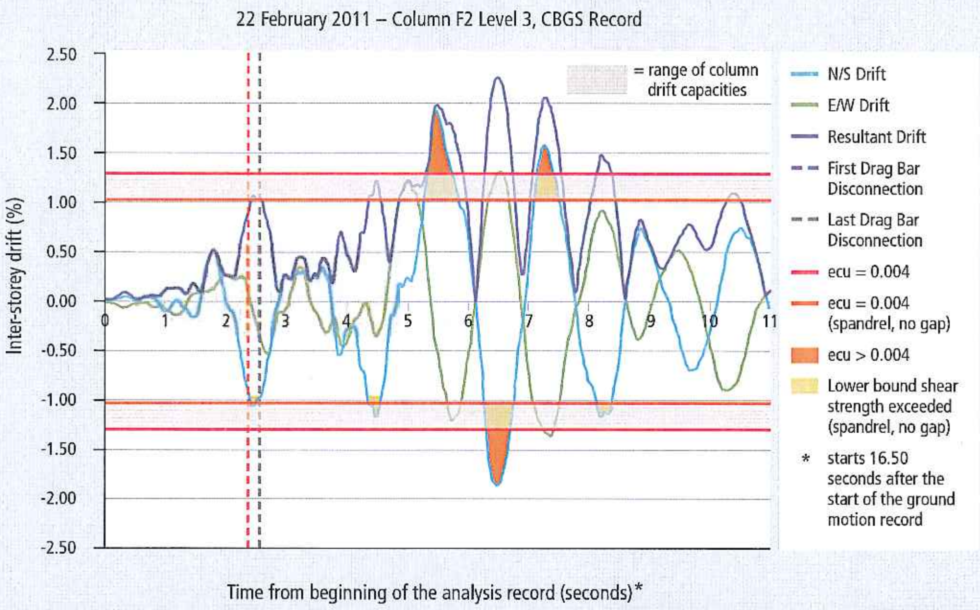

Figure 5.11: Comparison of drift demand and capacity – column F2 Level 3

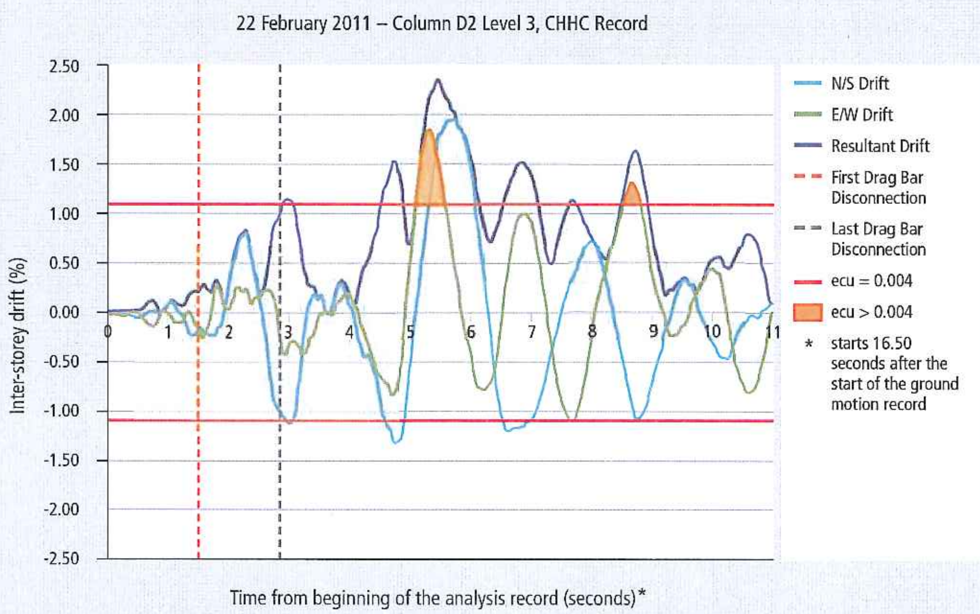

Figure 5.12: Comparison of drift demand and capacity – column D2 Level 3

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Figure 5.10 and Figure 5.11 show output from the non-linear time history analyses for column F2. Figure 5.12 shows output from the analyses for column D2. The vertical axis shows the amount of inter-storey displacement (drift) at this column location. The horizontal axis is the time from start of shaking (as input into the analysis). The wavy lines plot the drift level over time and are based on application of the full ground shaking record in the analyses. This drift is a key measure of demand on the column. The light blue line shows the north-south drift which is critical for the grid F columns, taking into account the stiff façade beams and the potential interaction with spandrel panels. The dark blue line indicates the resultant drift of the north-south and east-west drifts.

Note that the time shown on the horizontal scale in Figure 5.10, Figure 5.11 and Figure 5.12 is the time from the start of the analysis. For the 4 September 2010 case, analysis started at 28.90 seconds from the start of the ground motion record. For the 22 February 2011 cases, analyses started at 16.50 seconds. Thus the maximum response shown on Figure 5.11 at around 5.5 seconds into the analysis corresponds to 22 seconds from start of the ground motion record.

The horizontal lines represent the estimated capacity of this column to sustain the drift without failing according to various criteria (assuming average concrete strength and without vertical earthquake effects). The band between the horizontal lines in Figures 5.10 and 5.11 reflects the difference between "no interaction with the spandrels" (higher value) and "full interaction with the spandrels". The areas where the drift has exceeded the estimated capacity are shown shaded dark orange. The band showing the range of capacities would be wider if allowance was made for the effect of variable concrete strength and vertical earthquake forces in the column.

Estimates were made of the influence of axial load and concrete strength on the drift capacities of columns in different locations. Three key capacity points were identified for each case: the displacement to cause initial yield in the reinforcing steel, initiation of concrete crushing, and the displacement to cause the ultimate strain in the concrete (at which failure was taken to occur).

An important feature of this analysis was that for heavily loaded columns, the displacement to cause yielding of the main column bars was close to the displacement to cause failure. This is significant because it indicates that significant displacements, such as occurred on 4 September 2010, could be sustained with little evidence of distress, yet collapse could occur due to a relatively small additional displacement.

The key points to note are that, for the 4 September 2010 event, the maximum displacement demands are about half those calculated for the 22 February 2011 event. Although there are two places where the 4 September 2010 displacements are shaded, only one of these is for the north-south drift. There are no cases where they exceed the maximum assessed capacity. On the other hand, the 22 February 2011 demands have many "excursions" shown shaded and three that exceed the maximum value by a noticeable margin.

Similar plots were made for column D2 at Level 3, shown in Figure 5.12, with similar conclusions being reached regarding the likely performance of this column in the 22 February 2011 event.

Such comparisons provide valuable insights into the relativity of demand and capacity, but must be interpreted with care.

These comparisons give some indication of the challenges of determining which column or mechanism initiated failure. However, the plots indicate clearly that there is a strong likelihood that the demands of the 22 February 2011 event were enough to cause column failure, whereas the demands of 4 September 2010 were not.

Although the vertical accelerations at the site could have been high during the 22 February 2011 event, the analyses completed indicated column failure was possible without the additional effects from vertical accelerations.

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Displacements for column D2 on Level 1 (ground floor) (for the full record) were well below the assessed capacity of this column for 4 September 2010 and only marginally exceeded the capacity for the 22 February 2011 analysis. This is a broad indication that this column is less likely to have been the initiator of the collapse. However, this possibility cannot be ruled out because this column may have had lower than average concrete strength and/or suffered more from the effects of the considerable vertical forces generated in the 22 February 2011 event.

The vertical accelerations measured in the 22 February 2011 aftershock were exceptionally high and may have contributed significantly to vertical forces and columns and walls. The extent of this contribution is generally difficult to quantify, but analyses of the CTV Building indicated that vertical accelerations could have doubled the vertical forces in some critical, heavily loaded columns and this may have reduced the capacity of those columns to sustain lateral displacements by up to 40%, depending on concrete strength. If concrete strength in those critical columns had been less that the values that the analyses were based on, the displacement capacity would have been further reduced.

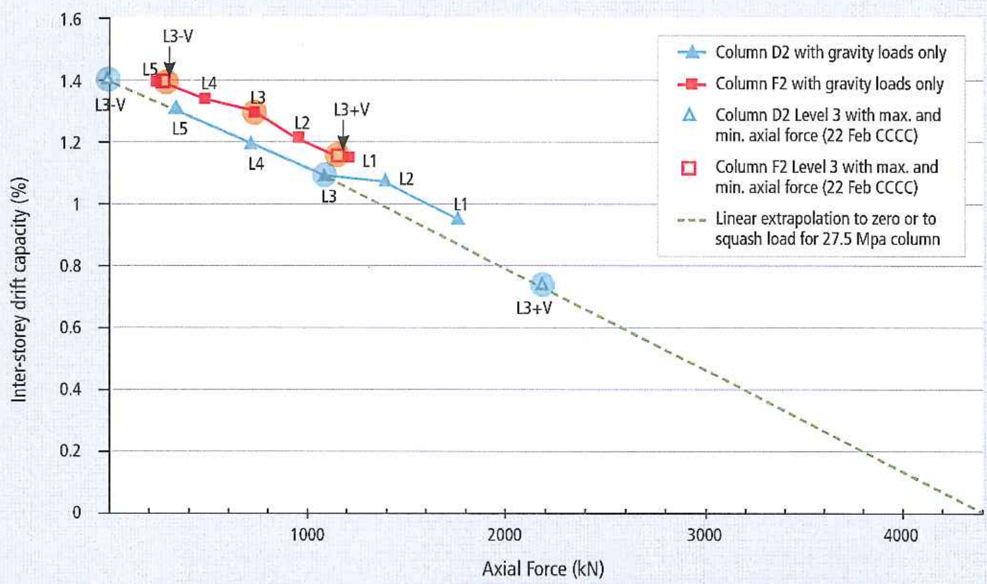

Figure 5.13: Effect of axial force on drift capacities at failures

The highlighted markers on each plot in Figure 5.13 show the situation for two columns at Level 3. The middle highlighted marker shows the axial force and drift capacity without earthquake effects. The highlighted markers to the right and left show the effects of vertical earthquakes, reducing (left mark), or increasing (right mark) the column axial force (compression). It can be seen that in these examples, the effect on drift capacity is significant, ranging up to more than 30%.

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Drift demand capacity comparison

Tables 5.1a and 5.1b show a comparison of calculated drift demands for the CBGS record and capacities for two indicator columns, column F2 at Levels 3 to 4 and column D2 at Levels 3 to 4.

A. Column on grid F2 at Level 3		
Demand or Capacity	**Event/Condition**	**North-South Column drifts (% of floor height)**
		Full Record
Demand	22 February 2011 (NTHA – CBGS)	1.9
	26 December 2010 (estimate)	0.5
	4 September 2010 (NTHA – CBGS)	1.0
	1986 Non-ductile detailing	0.6
	1986 Ultimate	1.1
	2010 Ultimate	2.3
Capacity	Failure (No spandrel effect)	1.2 - 1.3 (range)
	Failure (Full spandrel effect)	0.9 - 1.0 (range)

Table 5.1a: Indicative drift demand and capacity values

B. Column on grid D2 at Level 3		
Demand or Capacity	**Event/Condition**	**East-West Column drifts (% of floor height)**
		Full Record
Demand	22 February 2011 (NTHA – CHHC)	1.9
	26 December 2010 (estimate)	0.4
	4 September 2010 (estimate)	No analysis
	1986 Non-ductile detailing	0.5
	1986 Ultimate	1.0
	2010 Ultimate	1.8
Capacity	Failure (No spandrel effect)	1.1 - 1.2 (range)
	Failure (Full spandrel effect)	No spandrel

Table 5.1b: Indicative drift demand and capacity values

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Each table shows the maximum drift demand for 4 September 2010, 26 December 2010 and 22 February 2011 for the full record. For the 22 February 2011 event, the range shown represents the maximum drifts found for three separate analyses using records from the CCCC, CHHC and CBGS stations. Also shown are two 1986 standard design limits for the CTV Building:

- The "1986 Non-ductile detailing" figure is the drift demand computed in accordance with 1986 standards to determine the need or otherwise for ductile detailing of the columns. Non-ductile detailing would be allowed provided that the actions induced in the column at this point did not exceed a specified strength limit.
- The "1986 Ultimate" drift is the maximum expected drift demand calculated for the CTV Building indicator columns by the ERSA using the elastic design spectra and standard methods applicable in 1986.

The "2010 Ultimate" drift is also shown to indicate the level of drift demand current design requirements would place on the CTV Building indicator columns. As such it is a measure of the difference between 1986 design requirements and those of current standards – which require all columns, irrespective of drift, to be detailed for at least nominal ductility.

The "Failure" values in the Capacity part of the tables are the estimated drifts at which failure of the column was calculated to occur using average measured properties and without vertical earthquake effects.

5.11 Possible collapse scenario

Collapse was almost certainly initiated by failure of a circular column when the lateral displacement of the building was more than the column could sustain. Several possible scenarios leading to column failure were identified. Variability and uncertainty in physical properties and the analysis processes do not allow a particular scenario to be determined with confidence. However, the results of the analyses, taken together with the examination of the building remnants, eye-witness accounts and inspection of photos taken after the collapse, point to scenario 1, involving initiation of failure on Line F, as being a strong possibility.

An interpretation of this scenario is that collapse was initiated by the failure of one or more columns on the east face of the building. These columns experienced high drift demands and may have made contact with the pre-cast concrete spandrel panels placed between them, reducing their ability to cope with building displacement. Loss of these columns immediately put large additional gravity loads on the adjacent interior columns which were highly loaded at the lower levels.

The progression of collapse through the building would have been rapid. The columns were relatively small in cross-section and had a low amount of confinement steel. Even if the columns had been more closely confined, loss of cover concrete would have resulted in a substantial increase in compressive stress and extreme demands on the remaining confined section. The columns thus had little capacity to sustain load and absorb greater than anticipated displacement of the building.

Once the interior columns began to collapse, the beams and slabs above fell down and broke away from the north core, and the south wall and the beams and columns attached to it then fell northwards onto the collapsed floors and roof.

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Figure 5.14 shows the situation for this scenario with no spandrel interaction (A) and with spandrel interaction (B and C). Figure 5.16 illustrates the case of failure of ground floor columns on Line D for this scenario and the subsequent collapse of the floor slabs and frames for this inferred collapse sequence. Figure 5.15 shows the case along Line 2 of the scenario involving initiation on Line F.

Concrete strengths that were lower than the average used in the analyses would have reduced the load capacities of critical columns. Vertical accelerations from the ground motions may have added to the demands on columns and reduced their capacities to tolerate lateral displacement. The lack of symmetry of the lateral load-resisting elements is likely to have placed further demands on the critical columns by causing the building to twist and displacements to be larger than expected. Failure of diaphragm connections between floors and the north core walls, if it occurred prior to collapse initiating elsewhere, may have resulted in additional displacement demands on the critical columns.

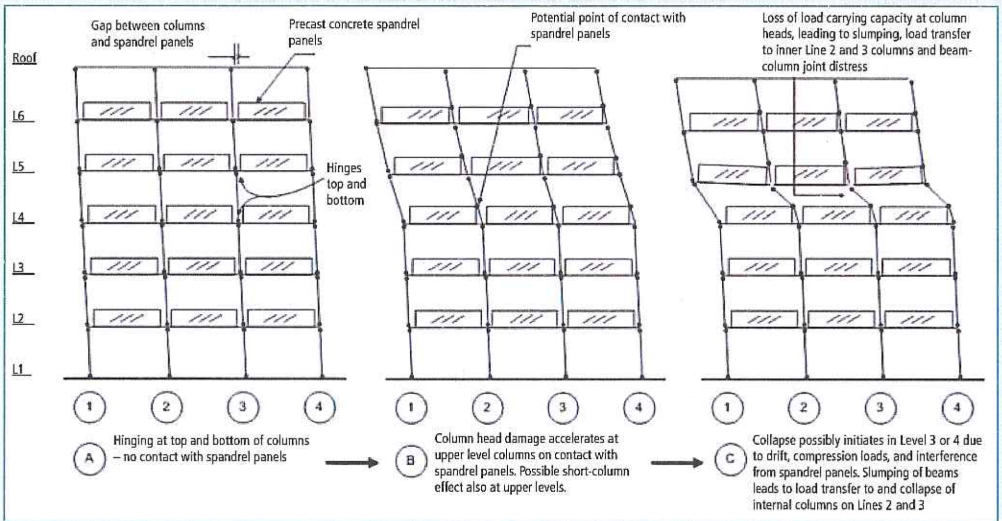

Figure 5.14: Inferred collapse initiation on Line F

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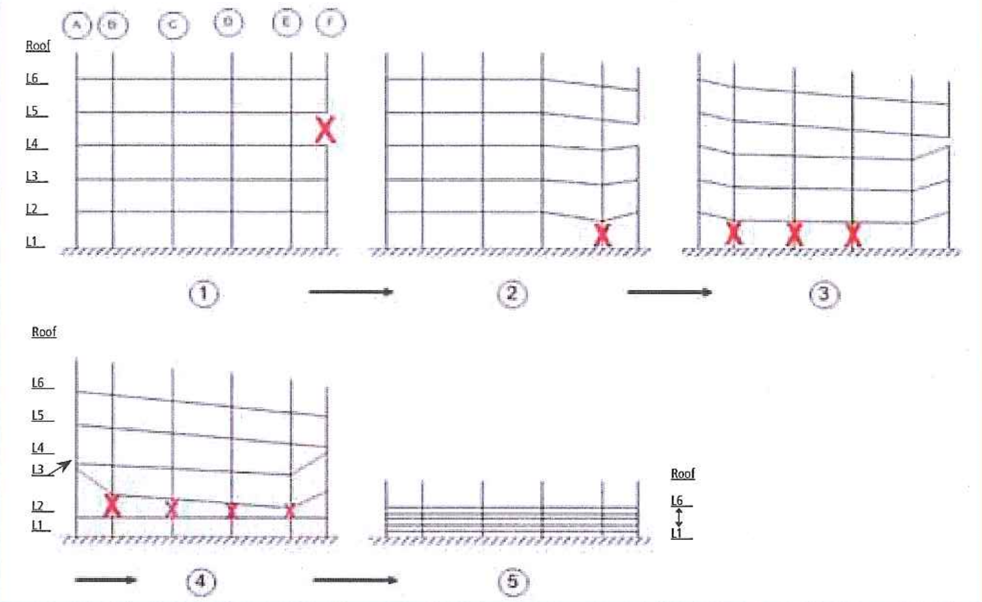

Figure 5.15: Inferred collapse sequence on Line 2

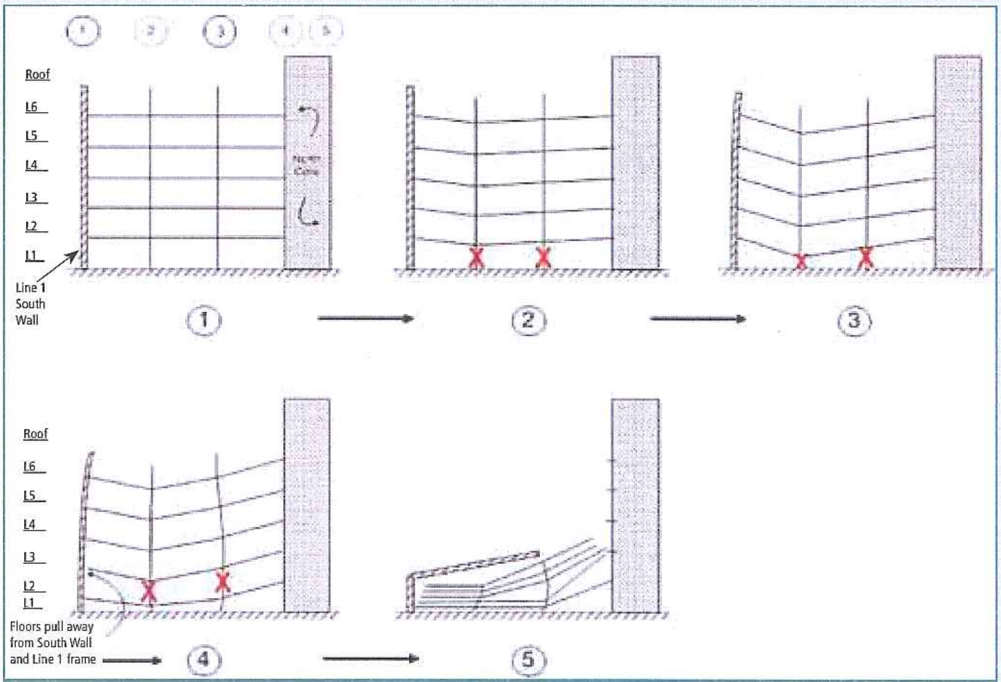

Figure 5.16: Inferred collapse sequence on Line D

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5.12 Compliance/standards issues

While it was not a primary objective, the investigation looked at how the CTV Building compared with the design and construction standards of the day. Issues assessed included those where the design, the construction, or the standards of the day as applied to the CTV Building could have been potential contributors to the collapse. These are outlined below:

Building inter-storey drift limits

When the building was designed in 1986, the building as a whole was required to have sufficient stiffness to limit the computed inter-storey displacement to below 0.83% of the inter-storey height. The CTV Building as a whole was found to have satisfied this inter-storey drift requirement of the standard.

Drift capacity of columns

The beams and columns on Lines 1, 2, 3, 4, A and F were found to have been designed as Group 2 secondary structural elements, not forming part of the primary seismic force resisting system.

The structural design standard applicable at the time of design had a general requirement that all important structural members be detailed to sustain loads at the maximum expected earthquake displacements of the building. The design standard also made a recommendation that all secondary frames be designed for ductility. The concrete design standard applicable at the time of design contained clauses that allowed "secondary" structural members to meet a less stringent ductility requirement. Under this interpretation, adequate performance of the secondary member was required to be demonstrated at 55% of the maximum expected earthquake displacements. For the CTV indicator columns the applicable displacement for this check is the "1986 Non-ductile detailing" figure in Tables 5.1a and 5.1b. The CTV columns should have been detailed for ductility in either case.

In a similar way, because they are an integral part of the columns, the beam-column joints were required to be detailed for ductility.

There needs to be a review of current requirements for ductile detailing of members, particularly those columns which are not regarded as part of the primary lateral load-resisting structure. It is important that design criteria are seen as adequate in the light of the ground shaking experienced in Christchurch and the performance of the CTV and other buildings.

Minimum column shear reinforcement

The concrete design standard applicable at the time of design had minimum requirements for shear reinforcement in columns, such as those in the CTV Building. The reinforcement in the CTV columns did not meet these requirements.

Spandrel panel separation

The spandrel panels were required to be separated from the columns to allow adequately for seismic movement and construction variations with allowable tolerances. A total gap allowing for seismic drift and construction tolerance of approximately 19mm would have been required unless specific requirements for an absolute minimum gap with tighter tolerance was specified. The drawings showed a nominal 10mm gap with no specific reference to it being a seismic separation gap.

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Plan asymmetry and vertical irregularity

The main seismic resisting elements (ie the concrete shear walls) were not located symmetrically about the centre of mass as recommended in NZS 4203:1984. The centre of stiffness of the designated primary seismic resisting elements was significantly eccentric to the centre of mass. The two main stabilising elements, the north core and the south wall, had significantly dissimilar stiffness and strength, and were outside recommended design limits for static analysis. However, there were no specific restrictions on geometric irregularity if ERSA was used. Specific warnings remained in the Loadings Standard about the ability to predict the performance of very irregular buildings with greater than moderate eccentricity such as the CTV Building.

Wall on Line A

From the design calculations it appears that the Line A masonry infill wall was intended to be separated from the structure. The appearance and performance of this wall suggests that it was not separated from the structure.

Diaphragm connection

No specific reinforcing steel was shown on the structural drawings to connect the north core lift shaft walls into the floor slabs. This omission was picked up after construction during a pre-purchase review for a potential purchaser by an independent consulting engineer and, in October 1991, resulted in the design and subsequent installation of steel angle connectors (drag bars) on Levels 4, 5 and 6.

The retrofitted drag bars were designed according to the requirements of the loadings standard of the day (NZS 4203:1984).

Although this standard had provisions for designing diaphragms and their connections, the provisions were found in the investigation to be insufficient to ensure the diaphragm connection was strong and/or ductile enough for full performance of the north core and south wall. This may be a concern for other buildings relying on floor diaphragm connections to shear walls and designed using the same standard. A review of current standards is needed.

Documentation

The gap between the spandrel panels and the columns was not identified as a minimum for seismic separation purposes.

It was noted that the top course masonry infill on Line A was shown as fully grouted which would have prevented the desired horizontal slip.

The pre-cast beams on Lines 1 and 4 between lines A and B had no starter bars shown extending into the slab on the drawings. This may have compromised the diaphragm performance in the south-west and north-west corners, and reduced robustness as the collapse developed.

Percentage New Building Standard assessment

When compared to the current standards for new buildings, the CTV Building would have achieved 40% to 55% NBS (New Building Standard). This figure applies to the pre-September 2010 condition and is based on detailed analyses of column drift demand and capacity carried out as part of this investigation. The lower figure is based on significant spandrel interaction with the columns and the higher figure on no spandrel interaction.

Geotechnical compliance

The soils investigation report prepared for the design engineer at the time of the design was reviewed by a leading geotechnical consultant as part of this investigation. The consultant considered that the geotechnical investigation carried out in 1986 was typical of the time and appropriate for the expected development.

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Construction issues

A number of areas were identified where construction issues could have introduced potential weaknesses in the building including the following:

- **Concrete strength** – Tests on 26 columns after the collapse found that the concrete in many columns was significantly weaker than expected. Cores taken from the Line 4-D/E columns were found to have traces of silt.
- **Construction joints** – In many construction joints the concrete surface was not roughened in accordance with the requirements of the concrete construction standard.
- **Bent-up bars** – Some of the beams on the north face of the building were found not to have their reinforcing steel properly connected into the west face of the north core on a number of floors.
- **Separation of elements** – Some of the reinforced masonry infill walls constructed between beams and columns appeared to have been constructed so that the intended structural separation was not fully achieved.

Construction supervision and monitoring

The investigation highlights the need for buildings to be built in accordance with the drawings and specification, and the need to have confidence that the design intent also has been interpreted correctly and followed through. Effective quality assurance measures need to be developed and implemented during construction. This includes having appropriately trained and qualified personnel undertaking the work, adequate supervision, approvals and audit by the consenting authority, and construction monitoring by the design engineer and architect.

5.13 Conclusions

The investigation found that there was no evidence to indicate that the damage to the structure observed and/or reported after the 4 September 2010 earthquake and the 26 December 2010 aftershock caused significant weakening of the structure with respect to the mode of collapse on 22 February 2011.

Although there is some scope for interpretation of the reported building condition, the estimated response of the building using the 4 September 2010 ground shaking records and the assessed effects on critical elements are not inconsistent with observations following the 4 September 2010 event. Analyses using the full 22 February 2011 aftershock ground motion records indicate displacement demands on critical elements to be in excess of their capacities even assuming no spandrel interaction and no vertical earthquake accelerations.

The following factors were identified as likely or possible contributors to the collapse of the CTV Building:

- The stronger than design-level ground shaking.
- The low displacement-drift capacity of the columns due to:
  - the low amounts of spiral reinforcing in the columns which resulted in sudden failure once concrete strain limits were reached
  - the large proportion of cover concrete, which would have substantially reduced the capacity of columns after crushing and spalling
  - significantly lower than expected concrete strength in some of the critical columns
  - the effects of vertical earthquake accelerations, probably increasing the axial load demand on the columns and reducing their capacity to sustain drift.
  - loss of diaphragm connection to the north core at Lines D and E.

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- The lack of sufficient separation between the perimeter columns and the spandrel panels which may have reduced the capacity of the columns to sustain the lateral building displacements.
- The plan irregularity of the earthquake-resisting elements which further increased the inter-storey drifts on the east and south faces.
- Increased displacement demands due to diaphragm (slab) separation from the north core.
- The plan and vertical irregularity produced by the influence of the masonry walls on the west face up to Level 4 which further amplified the torsional response and displacement demand.
- The limited robustness (tying together of the building) and redundancy (alternative load path) which meant that the collapse was rapid and extensive.

Surveys of the site after the collapse indicated that there had been no significant vertical or horizontal movement of the foundations. There was no evidence of liquefaction.

5.14 Recommendations

The performance of the CTV Building during the 22 February 2011 aftershock has highlighted the potential vulnerability in large earthquakes of the following:

- **Irregular structures**
  Geometrically irregular structures may not perform as well as structural analyses indicate. There is a need to review the way in which structural irregularities are dealt with in design standards and methods.
- **Non-ductile columns**
  Buildings designed before NZS 3101: 1995, and especially those designed prior to NZS 4203: 1992 (which increased the design drift demand), with non-ductile gravity columns may be unacceptably vulnerable. They should be checked and a retrospective retrofit programme considered.
- **Pre-cast concrete panels**
  Existing buildings with part-height pre-cast concrete panels (or similar elements) between columns may be at risk if separation gaps are not sufficient. Such buildings should be identified and remedial action taken.
- **Diaphragm connections**
  Buildings with connections between floor slabs and shear walls (diaphragm connections) designed to the provisions of Loadings Standard NZ 4203 prior to 1992 may be at risk. Further investigation into the design of connections between floor slabs and structural walls is needed.
- **Design and construction quality**
  There is a need for improved confidence in design and construction quality. Measures need to be implemented which achieve this. Design Features Reports should be introduced and made mandatory. Designers must have an appropriate level of involvement in construction monitoring. There should be a focus on concrete mix designs, in-situ concrete test strengths, construction joint preparation and seismic gap achievement.

It is recommended that the Department take action to address these concerns as a matter of priority and importance. The first four recommendations identify characteristics that, individually and collectively, could have a serious effect on the structural performance of a significant number of existing buildings. It is suggested that these issues be addressed collectively rather than individually.

The Panel recommends that the Department leads a review of the issues raised around design and construction quality. The Department should work with industry to develop and implement changes to relevant legislation, regulations, standards and practices to effect necessary improvements.