CTV BUILDING

CTV BUILDING COLLAPSE INVESTIGATION

FOR DEPARTMENT OF BUILDING AND HOUSING

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Part 1 of 3

Hyland
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GLOSSARY

Axial actions – A tension or compression action along the long axis of a structural member (e.g. a beam or column).

Axial capacity – Maximum axial compression that can be carried by a concrete column without failure. This is equal to the "squash load" when no bending moment demand occurs.

Base shear – Base shear is the lateral force due to seismic ground motion at the base of a structure. [The base shear is influenced by a number of factors including the weight of the building, the earthquake intensity, the ground conditions, and the building's structural characteristics.]

Biaxial bending – Bending of a structural member about two perpendicular axes at the same time.

Cantilever structure – A structure that is supported at one end only and that support provides full fixity.

Capacity – Overall ability of a structure or structural member to withstand the imposed demand.

Capacity design – A design process which limits actions in some structural members in order to protect others. In the capacity design of earthquake resistant structures, elements of the primary lateral load resisting system are chosen and suitably designed and detailed for energy dissipation under severe deformations. All other structural elements are then provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

Catenary – A curve formed by a chain, rope or severely damaged slab hanging freely from two points.

Centre of rigidity – If load is applied at a building's centre of rigidity, the building will not rotate or twist. Refer to Appendix E for detailed discussion for multi-storey buildings.

Compression failure – Failure of a structural member that occurs when its axial capacity in compression is exceeded.

Confined concrete – Concrete which is restrained from bursting outwards by transverse reinforcement (i.e. reinforcement at right angles to the principal reinforcement e.g. spirals around a column's longitudinal reinforcement and splices).

Critical capacity ratios – The ratio of the building's or structural member's capacity at which failure occurs to the demand placed on it,

Demand – A generic term to describe structural actions caused by gravity, wind, earthquake, and snow, etc. acting on a structure.

Damping – Damping is the process by which energy in a vibrating system is absorbed causing a decaying trend in the system's response. Damping in buildings is caused by a variety of factors including internal material energy dissipation effects, friction between components and interaction with the founding soils.

Darfield earthquake – The September Earthquake.

Dead load – The self weight of the building exclusive of any applied load.

December Aftershock – The aftershock on Boxing Day 26th December, 2010.

Deflection – Displacement measured from an at-rest or baseline starting position.

Deformation – Deformation in a structural or other member is a change in the original shape of the member. Deformation in a building occurs when it deflects or otherwise reacts to applied load.

Dependable strength – The ideal or nominal strength multiplied by the appropriate strength reduction factor.

Design capacity ratios – The ratio of dependable capacity to the design action.

Design spectra – Response spectra used for the design of buildings. The most usual measures of response are maximum displacement, velocity and acceleration relative to the natural period of vibration of the building.

Diaphragm – A structural element that transmits in-plane forces (diaphragm forces) to and between lateral force resisting elements. In buildings, floors usually act as, and are occasionally called, diaphragms.

Displacement – Displacement is the difference between the initial position of a reference point and any later position. The amount any point affected by an earthquake has moved from where it was before the earthquake.

Drag bars – Structural members that transfer lateral loads from a floor slab to the building's seismic resisting elements eg walls.

Ductile – See 'Ductility'.

Ductility – The ability of the structure or element to undergo repeated and reversing inelastic deflections while maintaining a substantial proportion of its initial load carrying capacity. The benefits of ductile design are that the building can be designed for lateral forces less than those required for elastic response. Further, the building is likely to remain standing or at least not suffer a brittle and sudden failure if it is subjected to an earthquake larger than the design earthquake.

Dynamic – Things that change with time e.g. dynamic loads.

Earthquake – A term used to describe both sudden slip on a fault and the resulting ground shaking and radiated seismic energy caused by the slip.

Earthquake-prone – The definition of an earthquake-prone building is given in section 122 of the Building Act 2004 and Section 7 of the Building regulations 2005. In summary, an earthquake-prone building is one that if assessed against current (new) buildings standards (NBS), would be assessed as not sustaining more than 33% of the minimum design actions for strength and ductility for the ultimate limit state.

Earthquake risk buildings – A building is assessed as an earthquake risk building if when assessed against the minimum requirements in current buildings standards, it sustains between 34% and 66% of the minimum design actions for strength and ductility for the ultimate limit state.

Eccentricity – A measure of the distance from the point of load application to the centre of rigidity. The greater the eccentricity, the greater the rotation.

Ecu = 0.004 – The condition where strain at the extreme compression fibre has reached 0.004.

Elastic – Structural behaviour, sometimes referred to as linear elastic, where an element or part springs back to its initial position when load is removed (no energy is absorbed in the process).

ERSA – Elastic Response Spectra Analysis. An acceleration response spectra is applied to a building to investigate the distribution of resulting actions on the basis of its elastic response.

February Aftershock – The aftershock on 22 February, 2011 in which the CTV Building collapsed. Sometimes it is referred to as the Lyttelton earthquake or aftershock.

First Yield or **Yield Moment** – First tension yield in outermost bar.

Fixity – Measure of the amount of rotation in a structural member at the support point. A cantilever which by definition has full fixity has no rotation at the face of its support. A pin (or roller or hinged) support provides no fixity and allows the structural member to rotate freely at the face of the support under applied load.

Flexure – Bending induced action.

Flexural cracking – Cracking as a result of flexure.

Flexible soils – Soils which deflect significantly under load.

Floor diaphragms – Broad horizontal structural floor members (e.g. concrete slabs) that carry horizontal load to the building's seismic resisting elements (e.g. frame or shear wall).

Geotechnical – Referring to the use of scientific methods and engineering principles, to acquire, interpret, and apply knowledge of earth materials for solving engineering problems.

Hinge zone – That portion of a structural member which undergoes inelastic deformations.

Horizontal shear – Shear in a horizontal direction.

Ideal strength – The ideal or nominal strength of a member.

Inelastic – The member or element goes beyond its elastic limit (it does not return to initial position and energy is absorbed).

In-plane – Along the face of, or parallel to, the structural member under consideration.

In-situ concrete – Concrete poured on site.

Inter-storey drift – Horizontal displacement of a floor relative to the floor immediately below.

Kilopascals (kPa) – Measurement of pressure being equal to one thousand Pascals. A Pascal being the pressure resulting from the force of one Newton applied over an area of one square metre.

Lap zone – Zone where reinforcement is overlapped so as to maintain its structural continuity.

Lateral displacement – Movement in a sideways or horizontal direction.

Liquefaction – Loss of resistance to shear stress of a water-saturated, silty-sandy soil as a consequence of earth shaking, to the extent that the ground behaves as a liquid rather than a solid.

Linear (refer to Elastic)

Linear static analysis – Another term for 'equivalent static analysis'.

Live load – The applied load or weight borne by a structure.

Lyttelton earthquake or aftershock – the February Aftershock.

Masonry infill wall – Infill panel between structural members made of masonry construction.

Modal analysis – Analysis of the building that considers and combines the various modes of vibration to determine the building's total response. (See ERSA).

Moment demands – The flexural demands on a structural member.

Moment frame – A structural frame which resists applied vertical and lateral loads, primarily in bending or flexure.

Moment-resisting – Able to resist the moment demands placed on it.

Nominal strength– Theoretical strength of a member section calculated using the section dimensions as detailed and the minimum specified reinforcement strengths and compressive strength of concrete or those inferred from testing. The maximum strain at the extreme compression fibre at the development of nominal strength is assumed equal to 0.003.

Non-ductile – Prone to sudden or brittle failure.

Non-linear – Describes behaviour beyond linear (or elastic).

NPA – Non- linear Pushover Analysis. Loading is progressively applied to investigate the development of failure conditions in a structure.

NTHA – Non-linear Time History Analysis technique to analyse the response of a building to a specific earthquake ground motion record.

OIE – Owner's Inspecting Engineer. The consulting engineering firm that was engaged by the Building owner to inspect the CTV Building after the September Earthquake and prepare an Earthquake Damage Report.

Out-of-plane – At right angles to the face of, or perpendicular to, the structural member under consideration.

P-delta effects – Destabilizing effects due to (significant) horizontal displacement of the centre of gravity of a structure (e.g. from an earthquake). When a structure is displaced, P-delta effects reduce the resistance of the structure to further displacement in the same direction. P-delta effects are important considerations in ductile (flexible) structures.

Planar – In the plane of, or parallel to, the structural member.

Pounding – Effect of two objects (buildings) impacting against or striking each other.

Pre-cast concrete – Concrete poured at a location remote from the building site and later transported to and placed on the site.

Response spectra – Response spectra are derived from the response of single degree of freedom oscillators subjected to an earthquake ground excitation. The peak accelerations (or velocity or displacements) are plotted against the period of vibration.

Retrofitting – Reinforcement or strengthening of existing structures to become more resistant and resilient to earthquakes.

Return period – The average time in years between earthquakes of a given magnitude on a fault or in a locality. The magnitude of the earthquake and the associated actions are assumed to increase with the return period.

Section capacities – The ideal strength in bending, shear or axial load that a cross-section in a structural member (e.g. beam or column) can withstand without failure.

Seismic frame – A frame, comprising columns and beams, that contributes to the building's lateral resistance enabling it to withstand earthquake actions.

Seismic gap – A separation between buildings or building elements which allows them to move during earthquakes.

Seismic response spectra – See Response spectra.

Seismicity – Refers to the geographic and historical distribution of earthquakes and their effects.

September Earthquake – Refers to the 4 September, 2010 Darfield earthquake.

Shear – A force applied at right angles to the vertical axis of a building or longitudinal axis of a structural member.

Shear wall – A wall that contributes to the building's lateral resistance enabling it to withstand earthquake actions.

Spalling – The loss of cover concrete, being the concrete between the external face of a structural member (e.g. beam or column) and the main reinforcing steel.

Spandrel Panels – Panels on the external face of the building. Spandrel Panels normally extend from ceiling level on one floor to window sill height on the floor above. Spandrel Panels are often used to provide fire separation between floors but can also have a structural function or comprise part of the building's cladding.

Tensile – Relates to tension in a structural member.

Tensile failure – Failure of a structural member as a result of tension.

Torsion – Twisting of a structural member or building as occurs when loads are applied other than through the member or building's centre of rigidity.

Vertical acceleration – Earthquake acceleration measured in the vertical direction.

Wall fins – Structural members at right angles to a wall to provide lateral stability.

Yielding – Inelastic deformation of material

Yield Moment or **First Yield** – First tension yield in outermost bar or 0.002 compression strain in concrete.

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EXECUTIVE SUMMARY

OVERVIEW

The six-level Canterbury Television building (CTV) located at 249 Madras Street, Christchurch suffered a major structural collapse on 22 February 2011 (Figure 1), following the Magnitude 6.3 Lyttelton Aftershock ("the February Aftershock"). Shortly after the collapse of the building a fire broke out in the stairwell and continued for several days.

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.

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 due to the influence of masonry walls on the west face.
- 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 and 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 February Aftershock 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.

Figure 1 - The CTV Building seen from the corner of Cashel and Madras Streets immediately after the collapse, and prior to debris being shifted and removed. The escape stair on the collapsed South Wall can seen laying on top of the rubble. Fractured Line F columns and precast concrete Spandrel Panels have fallen onto cars parked in Madras Street. A portion of floor slab from in front of the lift doors at Level 5 hangs precariously from the North Core in the distance (MSN).

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.

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.

INVESTIGATION

The technical investigation into the reasons for the collapse of the CTV Building was commissioned by the Department of Building and Housing and was undertaken by Hyland Consultants Limited ("HCL") and StructureSmith Limited ("SSL").

This report has been prepared under the direction of a Panel appointed by DBH for investigations into the collapse of the CTV Building during the February Aftershock.

The investigation consisted of:

- Examination of the remnants of the collapsed CTV Building.
- Review of available photographs.
- Interviews with surviving occupants, eye-witnesses and other parties.
- Review of design drawings ("the 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 sequence of and reasons for the collapse.

A separate report covering the Site Examination and Materials Testing undertaken for the investigation ("the Site Examination and Materials Testing") was prepared by Hyland Consultants Limited (Hyland 2012).

The investigation commenced in the second week of April, 2011.

DBH made a public call for surviving occupants, and eyewitnesses of the collapse and those involved with the building over the years to come forward with information to assist the investigation. Over 25 people were formally interviewed for the purposes of this report, and many more contributed photos and sent in emails with information that has greatly helped the investigation.

Photos of the collapse debris taken by the Police, Fire Service and the public, immediately after the collapse and during its removal over the following days, and discussions with those involved in that process helped to give a better understanding of the condition of the structure and how it collapsed.

The Council building file was made available including the Drawings of the structure for which the building permit had been issued.

The consulting engineering firm that had prepared the structural engineering design of the CTV Building ("the Design Engineer"), made available design calculations, and the structural specification for the building. Sketches and calculations were also provided by the Design Engineer for the steel angle Drag Bars ("the Drag Bars") that were designed in 1991 and then installed to connect the floor slabs at Levels 4 to 6 to the lift shaft walls in the North Core.

The soils investigation report prepared for the Design Engineer at the time of the design was reviewed by geotechnical engineers Tonkin and Taylor Limited as part of the investigation.

BUILDING DESCRIPTION

The developer of the building gained a building permit from the Christchurch City Council ("the 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 the North Core and South Wall. The North Core consisted of reinforced concrete walls surrounding the stairs and lifts at the north end ("the North Core"). The South Wall was a reinforced concrete wall on the south face adjacent to Cashel Street ("the South Wall"). Those are shown in Figure 2 and Figure 3. On the west face reinforced concrete masonry 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 ("the Spandrel Panels"). Spandrel Panels performed various functions including fire protection, sun control and architectural design.

The composite concrete on profiled metal deck floor slabs were cast in-situ. 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 strength and stability 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, medical clinic and radio and television studios for Canterbury Television.

The change of use to education facility at Level 3 in 2001 was noted in correspondence on the Council file. In relation to the other education tenancy that occupied Level 4, no building consent application relating to that change of use was found. The occupancy of these two floors by "schools" could have triggered change of use provisions in the Building Act however, no record of any structural engineering reports or changes to the building as a result of these occupancies was found.

Grid line locations are defined in Figure 3. Note that floor levels are numbered with the ground floor being Level 1 for the CTV Building.

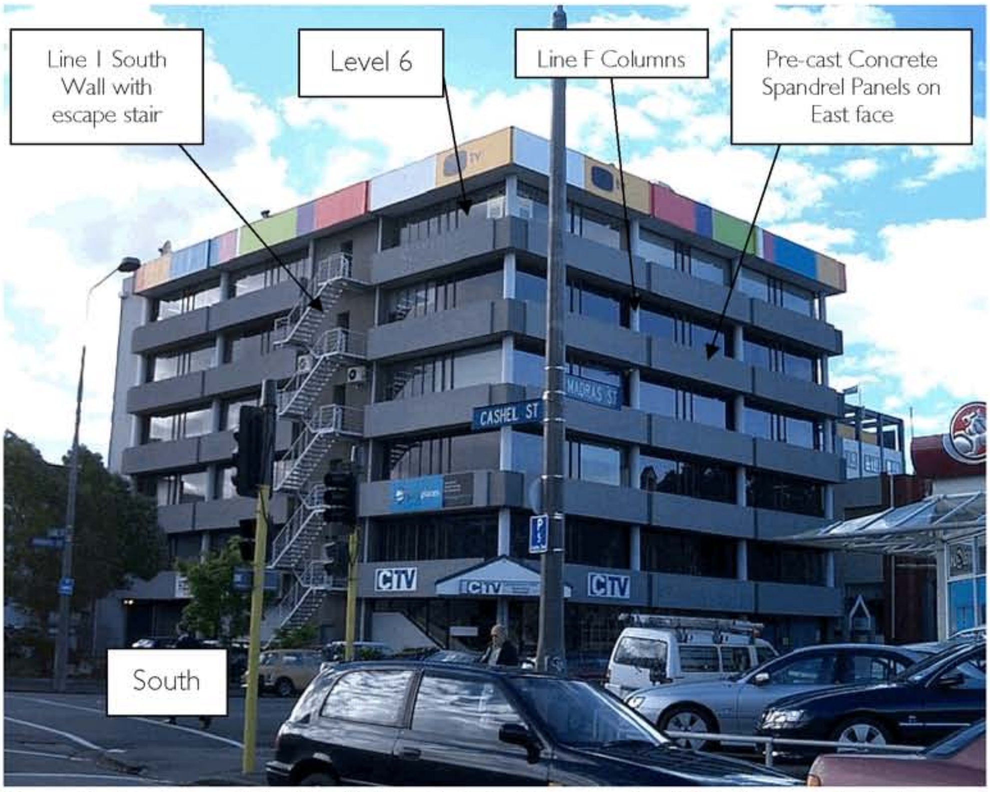

Figure 2 - Canterbury Television Building in 2004 (Photo credits: Phillip Pearson, derivative work: Schwede66) This shows some of the critical features relevant to the collapse such as the Line F columns, the pre-cast concrete Spandrel Panels, and the South Wall.

STRUCTURAL MODIFICATIONS

Following an Independent Consulting Engineer's ("the Independent Consulting Engineer") pre-purchase review in January 1990, the Design Engineer designed Drag Bars in October 1991 that were installed at Levels 4 to 6 to improve the connection between the floor slabs and the walls of the North Core. This connection was vital to the integrity of the building since the walls provided 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 locations where the slab pulled away from the North 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.

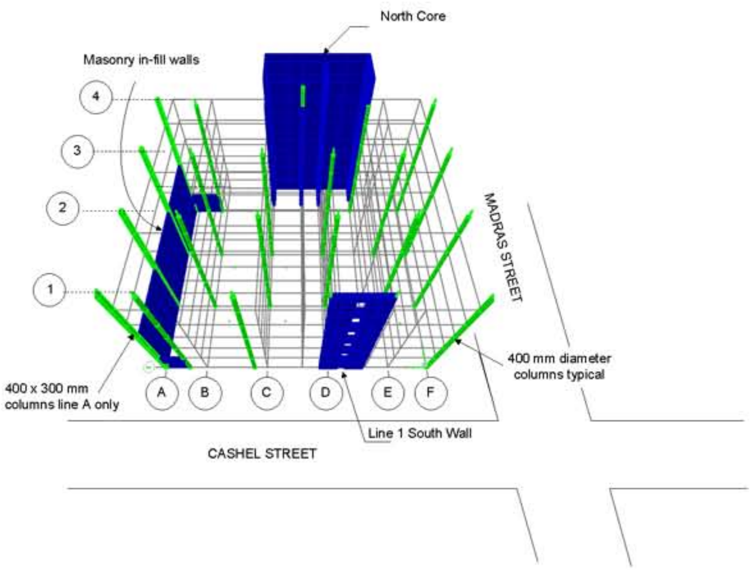

Figure 3 - Building orientation and grid lines referred to in the report. (Note that this diagram is not to scale nor is the building positioned accurately relative to the roads)

EARTHQUAKE AND OTHER EFFECTS PRIOR TO 22 FEBRUARY

4 September 2010 Earthquake

Damage to the CTV building structure was observed and recorded in an inspection report prepared by the building Owner's Inspecting Engineer ("the OIE") after the 4 September 2010 Earthquake ("the September Earthquake"). The OIE noted that they did not have a copy of the Drawings. They also may not have been aware of the Drag Bars that had been installed. The OIE findings and those of tenants interviewed are summarised as follows:

- Diagonal shear cracking and cracking of construction joints had occurred in the North Core and South Wall
- Their belief was that there had been no yielding of the reinforcement in the North Core and South Wall and that structurally their integrity was still sound
- Fine cracking had occurred in several perimeter columns in the upper floors
- No damage had occurred to the masonry infill wall on the west face (Line A)
- Several cracked or broken windows.
- Damage to non-structural partitions, and floor to ceiling cracks in the corners of the lift door wall and return walls at Level 6.

The damage overall appeared to have been 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 September Earthquake and continued until a week before the February 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.

Workers were preparing the outer face of the masonry infill wall on the west face of the CTV Building for re-cladding at the time of the February Aftershock. One of them described the outside face of the infill masonry wall as having no gaps between the columns and the masonry, and not having been fully filled with concrete grout, particularly the top courses. On the inside face the OIE had found there to be flexible sealant between the masonry and the columns and light coming through a gap next to the column in the northwest corner at Level 2. These observations have been taken into account when modelling the effect of the masonry wall.

26 December 2010 Boxing Day Aftershock

Witnesses and tenants advised that no significant structural damage had occurred but that some non-structural damage occurred after the 26 December 2010 Boxing Day Aftershock ("the December Aftershock"). There were no available engineering reports on the condition of the building after this event, but photographs of this damage indicate that it was minor.

COLLAPSE ON 22 FEBRUARY 2011

The February 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

Figure 6 shows an aerial view of the collapse scene not long after the building collapsed. 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.

Eyewitnesses spoken to as part of the investigation saw the building sway and twist violently. Eyewitness 14, 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 onto 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.

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.

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. It also involved examination 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 4. 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 strength distribution, it can be seen that a higher than expected proportion of the results is below the specified level in all cases.

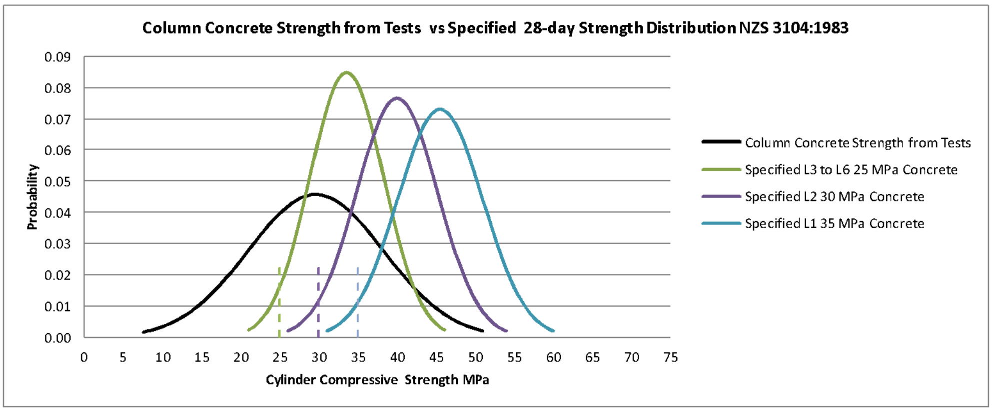

Figure 4 - Column concrete test strengths compared to specified strength distributions.

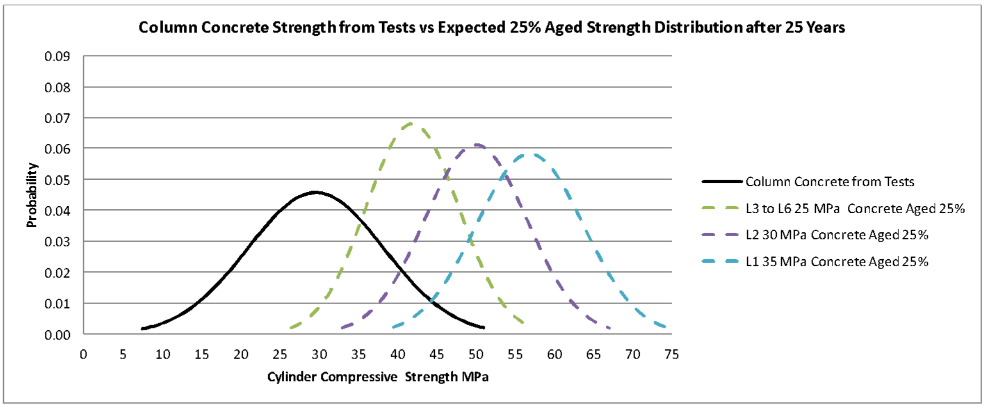

Figure 5 - Column strengths compared to expected strength distributions with 25% allowance for aging after 25 years.

While it is recognised that the tests were conducted on members that had been involved in the collapse, the results in Figure 5 indicate that column concrete strengths were significantly less than the expected strengths at the time of their testing. An expectation of 25% increase in strength during 25 years since construction is considered to be a general guide considering the specified strengths, the conservative approach to achieving specified strengths, and the expected strength gain with age.

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 eyewitness 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 February Aftershock must be interpreted in the light of the observed condition of the CTV Building after the September Earthquake and the December Aftershock and, and the possibility that these and other events could have affected the structural performance of the building.

Figure 6 - The CTV Building collapse shortly after machinery began to remove debris. (NZ Herald)

Elastic response spectrum analyses ("ERSA") were undertaken similar to those required by the design standards of the time (NZS 4203:1984 and NZS3101:1982) and also using levels of response corresponding to the actual ground motion records. These analyses provided insights into the design intentions and the likely response of the building in the September Earthquake, the December Aftershock and the February Aftershock.

Non-linear time-history analyses ("NTHA") were undertaken using actual records of the September Earthquake and the February Aftershock to assess the response of the building to the likely ground motions and the structural effects on critical elements, particularly the columns and floor diaphragm connections.

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 the following:

- Elastic response spectrum analyses of the whole building.
- Non-linear time history analyses of the whole building.
- Non-linear static pushover analysis of the whole building ("NPA").
- Equivalent inelastic analyses of the frames on Line F and Line 2.

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 after the September Earthquake.

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 members, or both. Principal amongst these were:

- 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 from the average values assumed for analysis.
- The Spandrel Panels on the south and east face 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 possibility that the response of the computer models to the ground motions or response spectra records used may differ significantly in nature and scale from the actual response of the CTV Building.
- The stiffness, strength and non-linear characteristics of structural members assumed for analysis may differ from actual values. This 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 is subject to considerable uncertainty.

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.

Generally, the output of the NTHA analyses was regarded as not inconsistent with the reported condition of the building after the September Earthquake. However it could be that a lower level of response may have occurred than was indicated by the analyses using the full records. The limited available evidence of the building condition after the September Earthquake leaves room for a range of interpretations of the likely maximum displacements in the September Earthquake. However, the authors agreed that the conclusions drawn from the analyses were 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.

Due to the range of factors noted above, which are subject to variability and uncertainty, there remain some issues where interpretations by the authors varied. These have been noted in the report and relate to; interpretation of some collapse evidence and photos; reconciliation of some damage reported compared with that implied by NTHA and ERSA; limitations of the applicability of ERSA in the assessment of earthquake demands on structures; and interpretation of the requirements for design of secondary structural elements. While interpretations varied it was recognised that the resolution of these was not critical to the findings of the investigation.

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 (NTHA), 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 North (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 from the other three stations.

For the elastic response spectrum analyses (ERSA), spectra were developed for the September Earthquake and the December and February Aftershocks using the closest sites possible at the time with compatible geotechnical conditions. These included the Westpac building (503A) and the Police Station (501A), 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 resultant 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 7. 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 have been the focus of the analyses. The ability of a column to sustain inter-storey drift depends on its stiffness, strength and ductility. Established methods were used to estimate the capacity of critical columns to sustain the drift without collapse.

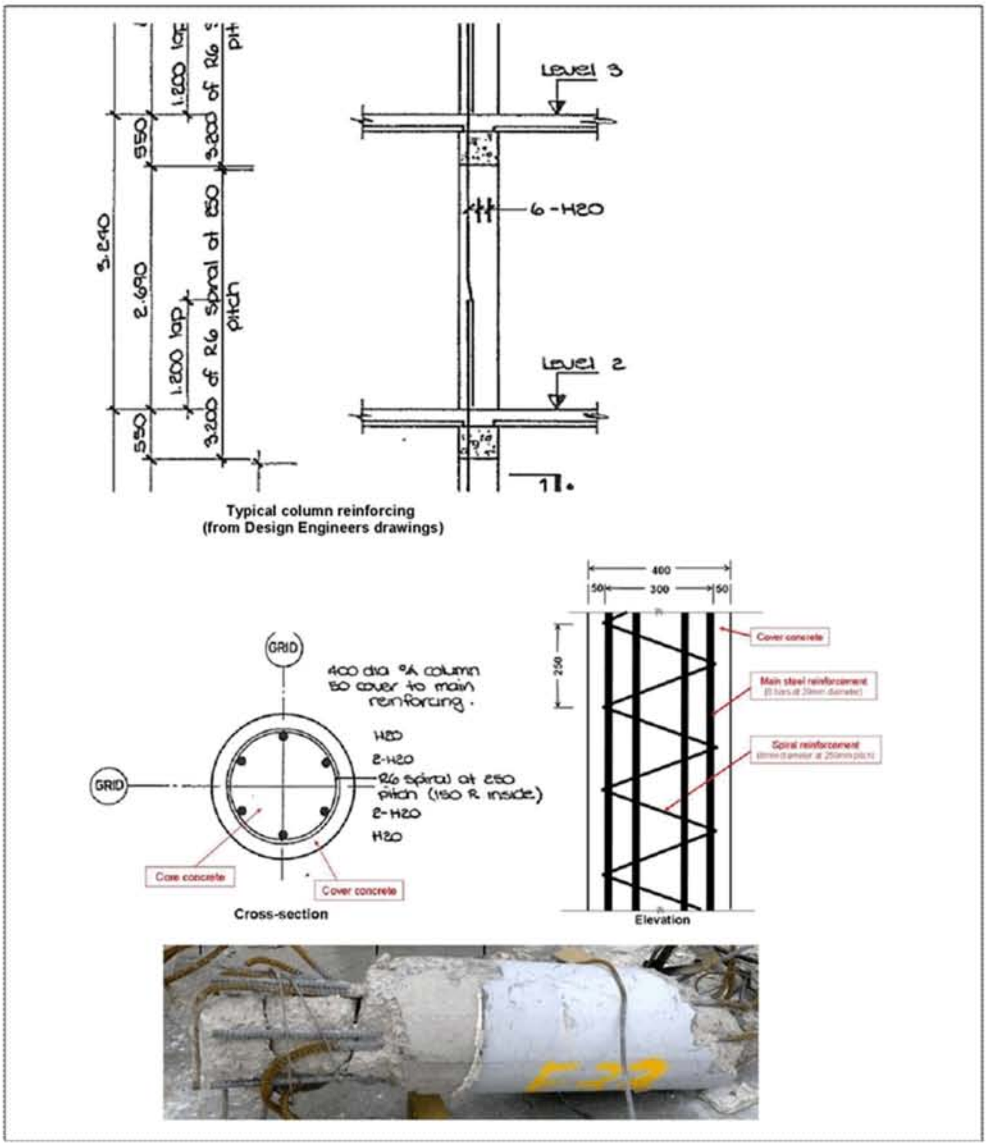

Figure 7 - Typical 400mm diameter column.

The possibility of diaphragm slab separation from the North Core walls prior to column failure was investigated. This was because analyses showed potentially high forces in the connections between the floors and the North Core when the full earthquake record was applied to the computer models. Separation was not able to be justified by review of the physical collapse evidence and localised analysis. It was however found that collapse was able to have initiated at drifts less than that necessary to cause slab separation from the North Core.

Spandrel Panels

A plan and a cross-section of the typical column and Spandrel Panel arrangement are shown in Figure 8.

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.

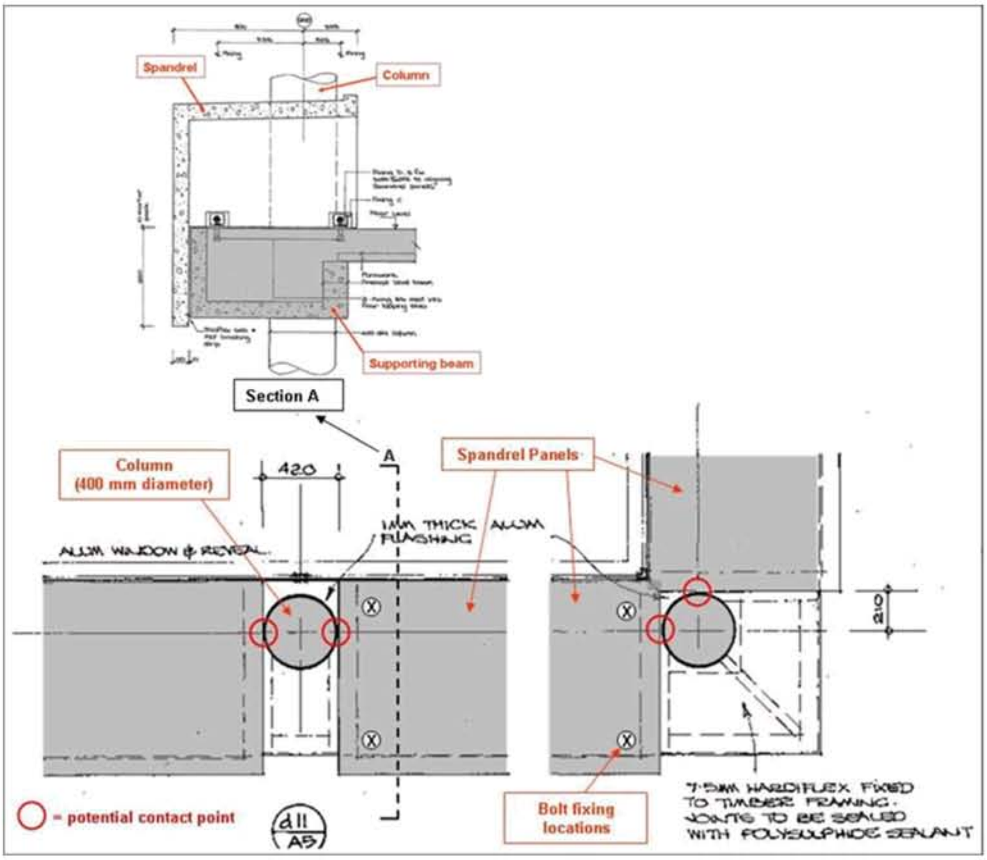

Figure 8 - Spandrel Panel detail.

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.

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 9 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.

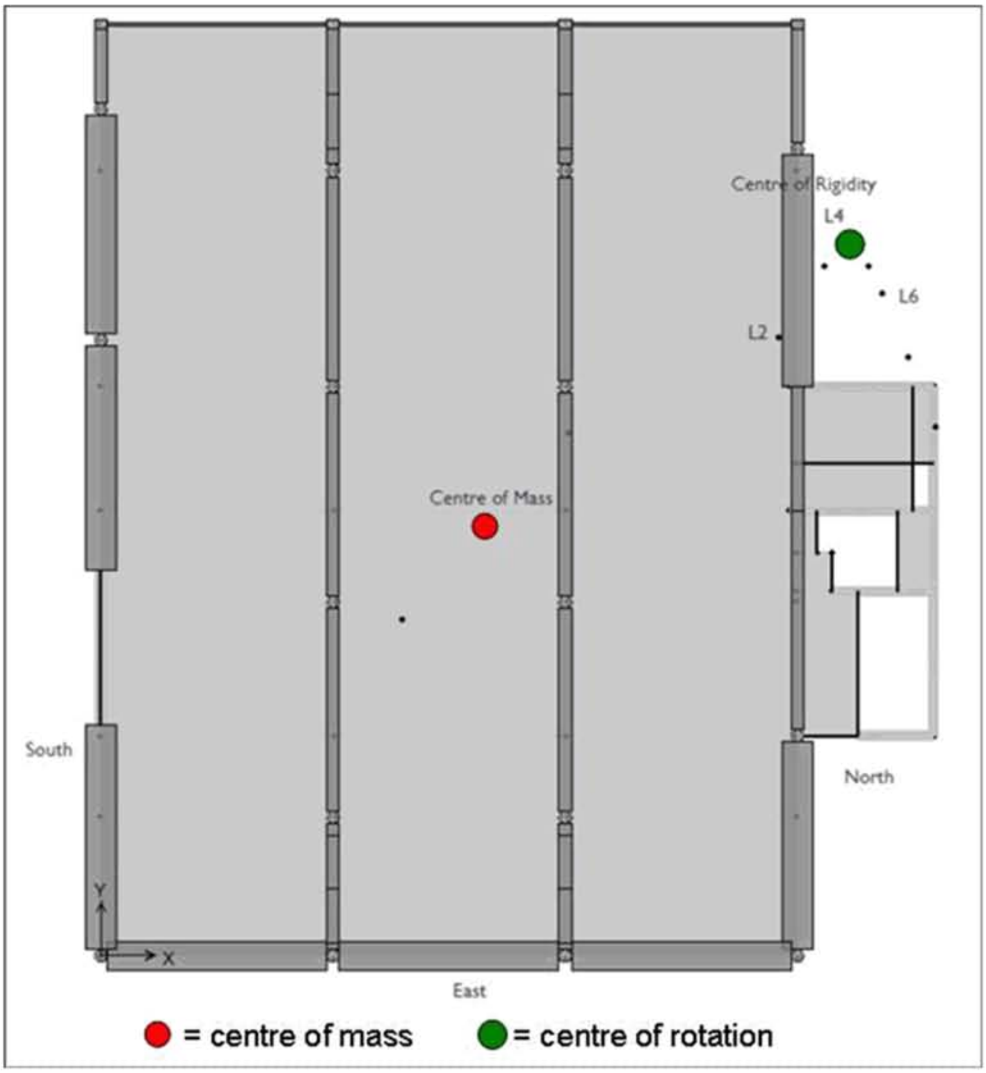

Figure 9 - Plan irregularity.

Diaphragm Connection

Figure 10 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 designed in October 1991 and subsequently installed at Levels 4, 5 and 6. 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.

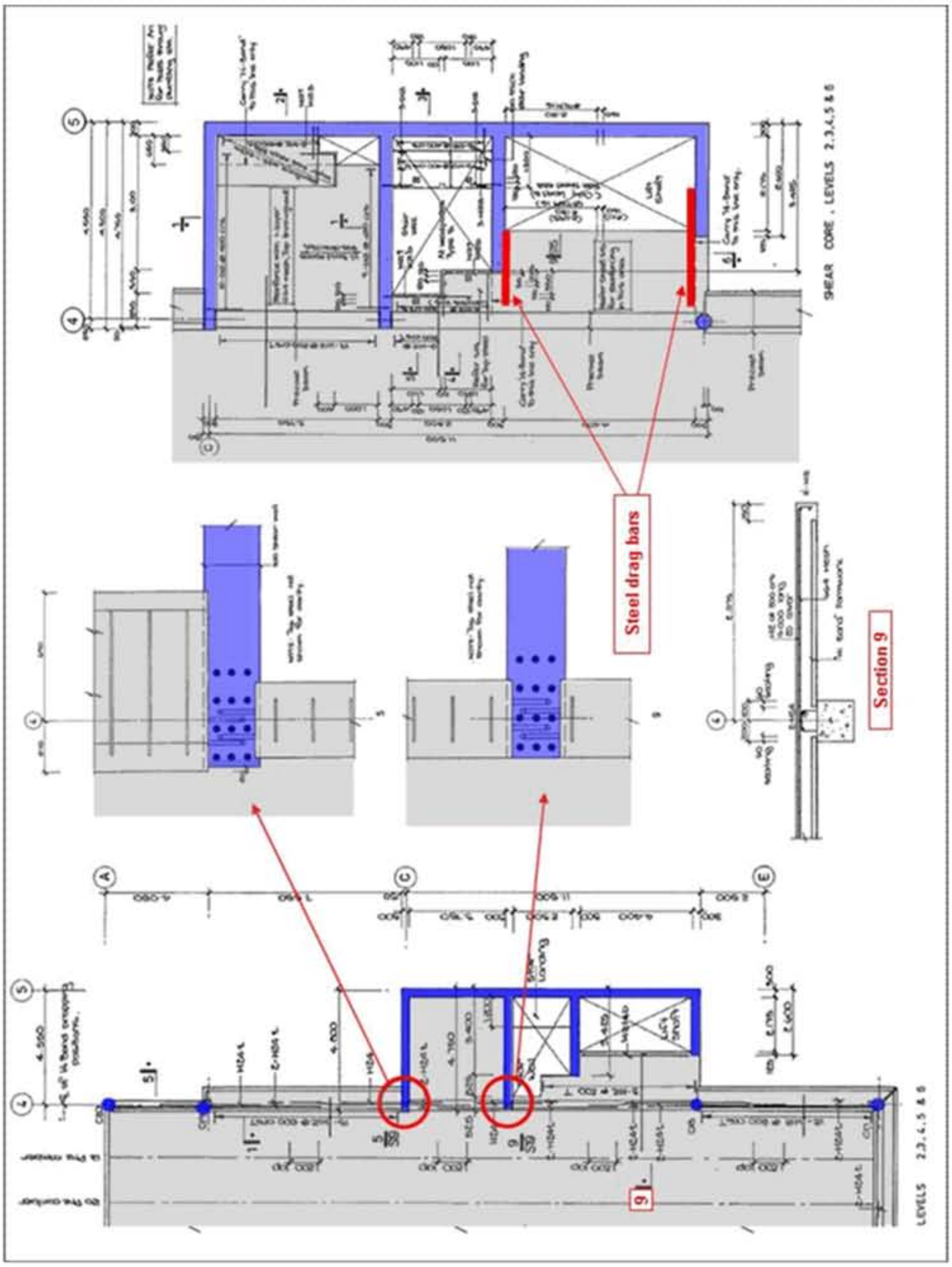

Figure 10 - Diaphragm connections at North Core.

COLLAPSE INITIATORS EXAMINED

Four potential collapse initiation scenarios were identified for evaluation as follows:

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 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 disconnection from the North Core at Level 2 or Level 3. This scenario requires that 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 disconnection at Level 4, 5 or 6. This scenario has similar characteristics to Scenario 3 but involves failure of the Drag Bars and adjacent slab connections to the North Core. The worst effects would be at the higher levels of the North Core. Possible compounding factors in this scenario are the effects on the diaphragm slab connection to the North Core, of east–west foundation rocking, and also of uplift of that slab/wall connection due to northwards displacement due to northwards displacement.

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

Figure 16 outlines the key considerations involved in evaluating these scenarios.

CRITICAL COLUMN IDENTIFICATION

Analyses showed that drift (ie lateral displacement) 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 expected 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.

KEY DATA AND RESULTS

Elastic Response Spectra Analysis

Figure 11 shows the response spectra used in the ERSA. 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 to 1.3 seconds.

The graphs give an indication of the relative intensities of ground shaking records in the September Earthquake, December Aftershock and February Aftershock (solid lines). The response spectra used for design in 1986, when the CTV Building was designed. (dashed lines) The upper dashed line represents "full" design level expectation of the standards which represents the fully elastic response spectra loading that the structure was expected to be able to match in terms of equivalent inelastic or ultimate displacement without collapsing.

The lower dashed line represents the level that the seismic resisting North Core and South Wall were required to resist prior to developing their axial / flexural dependable strength. For design of members, strength reduction or safety factors are applied when using that level of loading.

In ERSA the loadings spectra are applied in directions such as east-west or north-south separately. The response of the building to the same spectra may be different in each direction. The loading standard also required the design loadings to be scaled for each direction which can mean the design spectra curves are different in each direction.

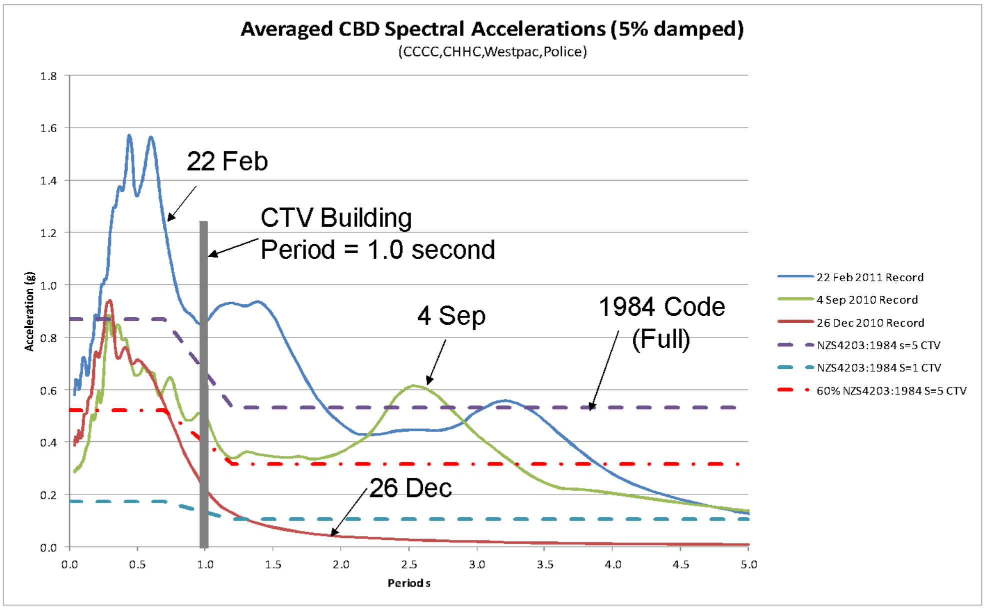

Figure 11 – Response spectra records for the September Earthquake, December Aftershock and the February Aftershock. Also shown (dashed lines) are the design spectra for the CTV building according to NZS 4203:1984. The lower dashed line is the spectra for ductile design that the North Core and South Wall were required to have axial / flexural dependable strength in excess of. The upper most dashed line is the elastic response spectra that the structure was expected to be able to match in terms of ultimate displacement without collapsing.

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 February Aftershock significantly exceeded the full 1984 value required for the design of elastically responding structures. The CTV Building had been designed for ductile response using forces derived from the lowest design spectra shown in Figure 11.

The ERSA indicated that the demand of the February record was approximately 2.2 times the demand of the September record, which was in turn almost 1.7 times the demand of the December Aftershock.

Seismic Detailing Requirements Check

The general structural design standard of the day, NZS 4203:1984, required that the building as a whole, and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life shall be designed to possess ductility. The recommendation of the standard, and possibly its intent, as represented in the commentary to NZS 4203:1984, was that secondary structure was to be designed to possess ductility so that it would be capable of sustaining the vertical loads when subjected to at least 4 times the distortion from the specified loading.

Commentary clauses are generally regarded as informative and not mandatory. Therefore it is understandable that debate has existed over this wording. This recommendation, or intent, depending on one's interpretation was also stated in the earlier version of this Standard, NZS4203:1976. The secondary structure in CTV, which included the columns, did not satisfy this.

NZS 3101:1982, which pre-dated NZS 4203:1984, but post-dated NZS 4203:1976 appeared to interpret this intent to the extent that the deformation under which the secondary structure needed to be detailed to satisfy the additional seismic design requirements of the standard was reduced to 55% of the ultimate drift for a ductile concrete structure. This is the non-ductile detailing limit in Table 1and Table 2.

The commentary to NZS 3101:1982 gave some guidance on what level of cracking would be expected and modelled for seismic analysis. For example 1.0 Ig for columns carrying significant axial compression and 0.5 Ig for beams. If these criteria had been applied then most of the CTV columns would have required the seismic design and detailing provisions of NZS 3101:1982 to have been applied.

One interpretation of the NZS3101:1982 requirements has been set out in Appendix F and applied to the CTV Building.

The drift demand and failure capacity check undertaken in Appendix F as part of the investigation used current column moment curvature analysis that was not readily available to designers in 1986. The software allowed more accurate assessment of the likely maximum drifts that could be sustained by a column prior to it reaching its elastic deformation limit and also its failure capacity.

However even using the more sophisticated approach of the current software and the interpretation used in Appendix F of the requirements with respect to the definition of elastic behaviour it was found that the Line 1 columns on the south face and the Level 5 columns on Line F would appear to have been required to be designed to the seismic design and detailing provisions of NZS 3101:1982.

Column F2 Level 3 – Demand versus Capacity

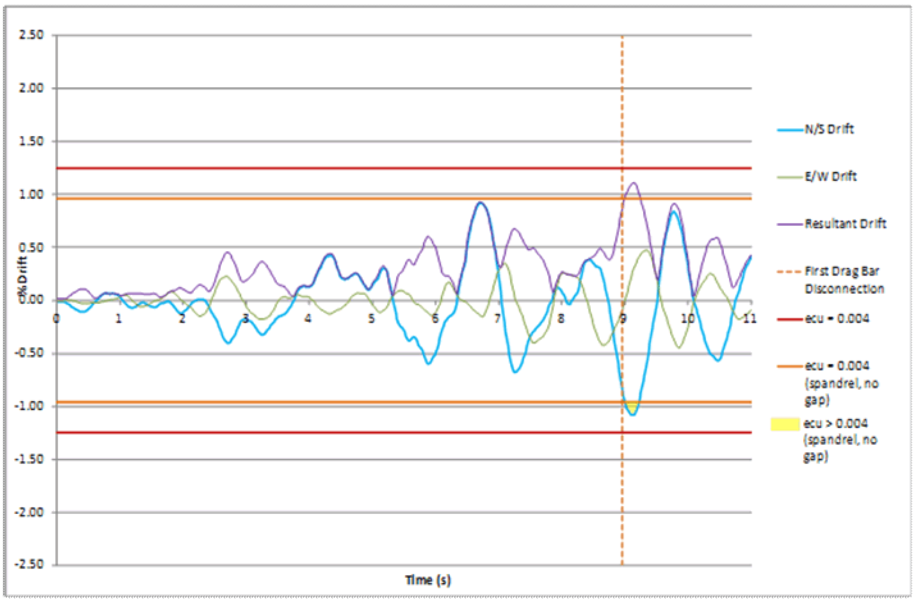

Figure 12 - Comparison of drift demand and capacity – column F2 Level 3 - CBGS, 4 September Darfield Earthquake, no masonry.

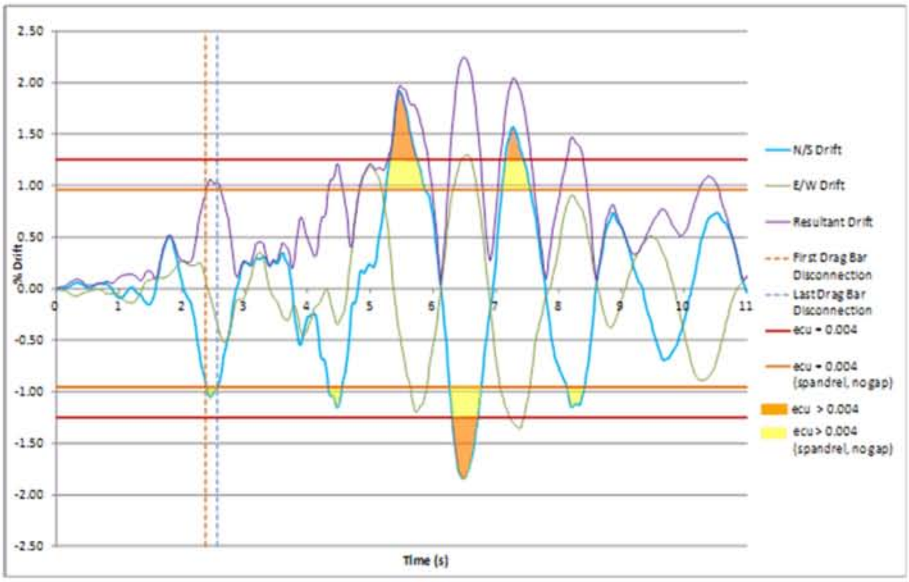

Figure 13 - Comparison of drift demand and capacity – column F2 Level 3 - CBGS, February Aftershock, no masonry.

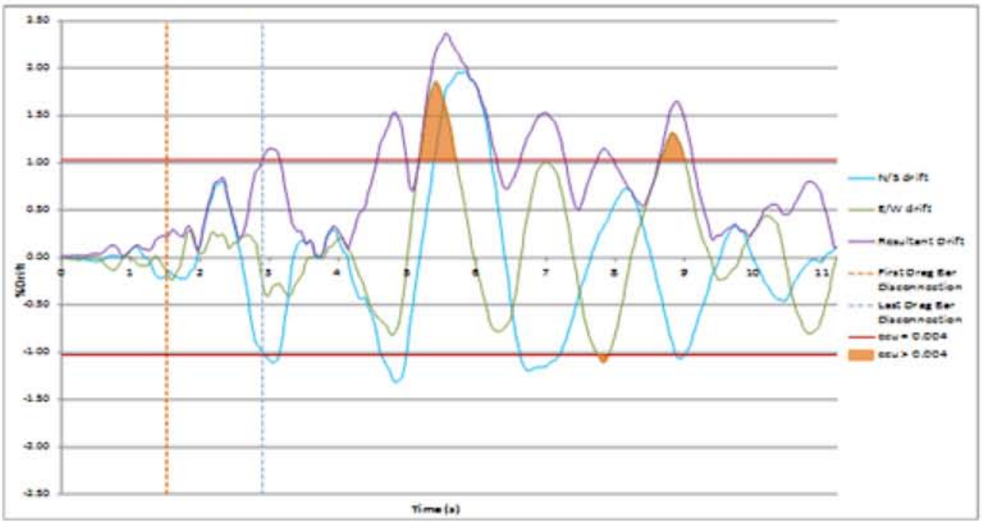

Figure 14 - Comparison of drift demand and capacity – column D2 Level 3 - CHHC, 22 February Lyttelton Aftershock, no masonry.

Figure 12 and Figure 13 show output from the non-linear time history analyses for column F2. Figure 14shows 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 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 thin brown line indicates the resultant drift of the north-south and east-west drifts.

Note that the time shown on the horizontal scale in Figure 12, Figure 13, and Figure 14, is the time from the start of the analysis which is different from the start of the GNS record as shown in Table 8 in Appendix D.

The horizontal lines represent the estimated capacity of this column to sustain the drift without failing according to various criteria (assuming expected concrete strength and without vertical earthquake effects). The band between the horizontal lines in Figure 12 and Figure 13 reflects the difference between "no interaction with the spandrels" (higher value) and "full interaction with the spandrels". The areas where the drift had exceeded the estimated capacity are shown shaded. 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 force 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 yield at compressive strain of 0.002, and the displacement to cause the 0.004 ultimate compression 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 in the September Earthquake, 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 September Earthquake, the maximum drift demands are about half those calculated for the February Aftershock. Although there are two places where the September Earthquake drifts 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 February Aftershock 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 14, with similar conclusions being reached regarding the likely performance of this column in the February Aftershock.

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 February Aftershock were enough to cause column failure, whereas the demands of the September Earthquake were much less.

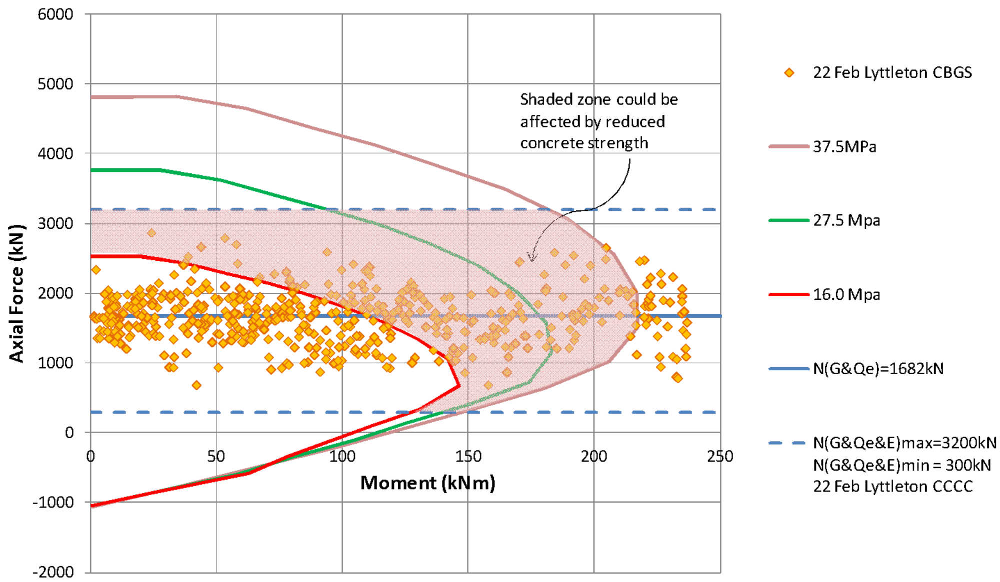

Figure 15 - Column interaction chart showing effect of varying concrete strength and vertical acceleration on column performance.

Effect of Vertical Acceleration

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

Displacements for column D2 on Level 1 (ground floor) (for the full record) were well below the assessed capacity of this column for the September Earthquake and only marginally exceeded the capacity for the February Aftershock 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 February Aftershock.

The effects of vertical acceleration on column drift capacity of the indicator columns at grids F2 and D2, according to the criteria of 0.004 maximum compressive strain was determined from the non-linear pushover analysis of the whole structure (assuming expected concrete strengths and without vertical acceleration effects). The results were also compared to the fixed end moment drift capacities derived using Cumbia software and adjusted for assessed frame effects on Line F.

The NPA found that the drift capacity of the F2 Level 3 column may have been reduced by approximately 12% for a 60% increase in axial load from vertical earthquake effects. The alternative method indicated a reduction of drift of between

0.25% to 0.50% /1000 kN increase in axial demand. Indications were that at greater compression demands the reduction in drift capacity may be higher, however that has not been established accurately.

Similarly the drift capacity of the grid D2 column at Level 3 may have been reduced by approximately 38% for a 100% increase in axial load due to vertical earthquake effects. These axial load variations were the maximum obtained from the NTHA CCCC record for the February Aftershock.

The axial forces in column D2, from the NTHA with vertical accelerations, were found to fluctuate at much higher frequency than the lateral modes of vibration. There was therefore significant variation in the axial force for any given cycle of lateral drift, with an increase in axial force being detrimental and a reduction in axial force being beneficial to the column drift capacity at any particular time step (Figure 15.

Effect of Reduced Concrete Strength

Based upon a comparison of the drift capacities of the columns with concrete of reduced strength from that assumed in the analyses, it appears that a reduction in concrete strength of approximately 13 MPa, could lead to a reduction in column drift capacity of up to 15%.

Drift Demand Capacity Comparison

Table 1 and Table 2 show a comparison of calculated drift demands and capacities for two indicator columns, column F2 at Levels 3 to 4 and column D2 at Levels 3 to 4.

Each table shows the average maximum drift demand for the September Earthquake, and the December and February Aftershocks for the full records as noted. For the February Aftershock, 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 have been allowed provided that the actions induced in the column at this point did not exceed the prescribed 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 gives an indication 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 expected properties based on measured properties and without vertical earthquake effects.

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 1 - Indicative drift demand and capacity values on column at Grid F2 at Level 3.

B. Column on grid D2 at Level 3		
Demand or Capacity	Event / Condition	East-West Column drifts (% of floor height)
	Event / Condition	Full Record
Demand	22 February 2011 (NTHA - CHHC)	1.9
	26 December 2010 (estimate)	0.40
	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 2 - Indicative drift demand and capacity values on column at Grid D2 at Level 3.

POSSIBLE COLLAPSE SCENARIO

Collapse was almost certainly initiated by failure of a 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 would have 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.

Figure 17 shows the situation for this scenario with no spandrel interaction (A) and with spandrel interaction (B and C) and Figure 18. Figure 20 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 19 shows the case along Line 2 for the scenario involving initiation on Line F.

Concrete strengths lower than the expected value 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 inter-storey displacement "drifts" 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 drift demands on the critical columns.

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 drift at specified loads 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 loadings 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 loadings 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, elastic performance of the secondary members in the CTV Building was required to be demonstrated at 55% of the design maximum or ultimate earthquake drifts. For the CTV indicator columns the applicable drift for this check is the "1986 Non-ductile detailing" figure in Table 1 and Table 2. The CTV Building columns were required to be designed and detailed for ductility in both cases.

In a similar way, because they were 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 quantity and maximum spacing requirements for shear reinforcement in columns, such as those in the CTV Building. The reinforcement in the CTV columns did not meet these requirements.

Satisfying these requirements would be expected to have improved the ductility of the columns, the reliability of splices in column reinforcing and would have limited the potential for buckling of column reinforcing steel once the cover concrete spalled.

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 needed unless specific requirements for an absolute minimum gap with tighter tolerance was specified. The drawings showed a nominal 420 mm clearance between adjacent Spandrel Panels with no specific reference to a seismic separation gap to the columns.

Plan Asymmetry and Vertical Irregularity

The main seismic resisting elements (ie the concrete North Core and South Wall) 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. These two main stabilising elements, had significantly dissimilar stiffness and strength, and were outside recommended design limits for static analysis. There were no specific restrictions however on geometric irregularity if ERSA was used. However, specific warnings remained in the Loadings Standard about the ability to reliably 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 suggested 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 the Drag Bars on Levels 4, 5 and 6.

The Drag Bars appeared to have been 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 and to withstand the high connection forces indicated by the NTHA. 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 recommended.

Documentation

In addition to the above:

- The top course masonry infill on Line A was shown as fully grouted. However to achieve the seismic separation desired the vertical reinforcing in the top course would have needed some ability to deflect and deform to minimise shear transfer between the header beam and the masonry wall. This would not have been achievable if constructed as shown on the Drawings.

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

- The requirement to prepare the construction joints in the North Core or South Wall as Type B construction joints in accordance with NZS 3109:1990 was not indicated on the Drawings or in the Specification. This would have required treatment to produce a roughened surface sufficient to allow shear friction to develop across the joint. It would be expected that Type B joints would have been required to achieve the design intent. Smooth construction joints may have allowed slippage that increased drift demands on the Group 2 frames.

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 site investigation report prepared for the Design Engineer at the time of the design was reviewed by Tonkin and Taylor ("T&T") as part of this investigation. T&T considered that the geotechnical site investigation carried out in 1986 was typical of the time and appropriate for the expected development.

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 some discolouration that may or may have been due to silt, had low density and did not exhibit the degree of aggregate breakage expected for concrete of the specified strength.
>
> 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 four 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 highlighted 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.

CONCLUSIONS

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.

The investigation found no evidence to indicate that the damage to the structure observed and/or reported after the September Earthquake and the December Aftershock had caused any significant weakening of the structure with respect to the mode of collapse in the February Aftershock.

Although there is some scope for interpretation of the reported building condition, the estimated response of the building using the September Earthquake ground shaking records and the assessed effects on critical elements are not inconsistent with observations following the September Earthquake. The analyses and observations were found not to be very sensitive to the level of demand assumed. The results and conclusions would remain largely unchanged at a lower level of demand in September and February.

Analyses using the full February Aftershock ground motion records indicate drift demands on critical column elements to have been 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.
- 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.

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 and Masonry Infill Walls

Existing buildings with part-height pre-cast concrete panels (or similar elements) between columns may be at risk if separation gaps are not sufficient and maintained. 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 and Construction 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 authors recommend 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.

Disclaimer: This Executive Summary summarises the key points of this report and is not intended to be a substitute for the report in its entirety. The Executive Summary should be read in conjunction with the whole report and the reader should not act in reliance of the matters contained in the Executive Summary alone.

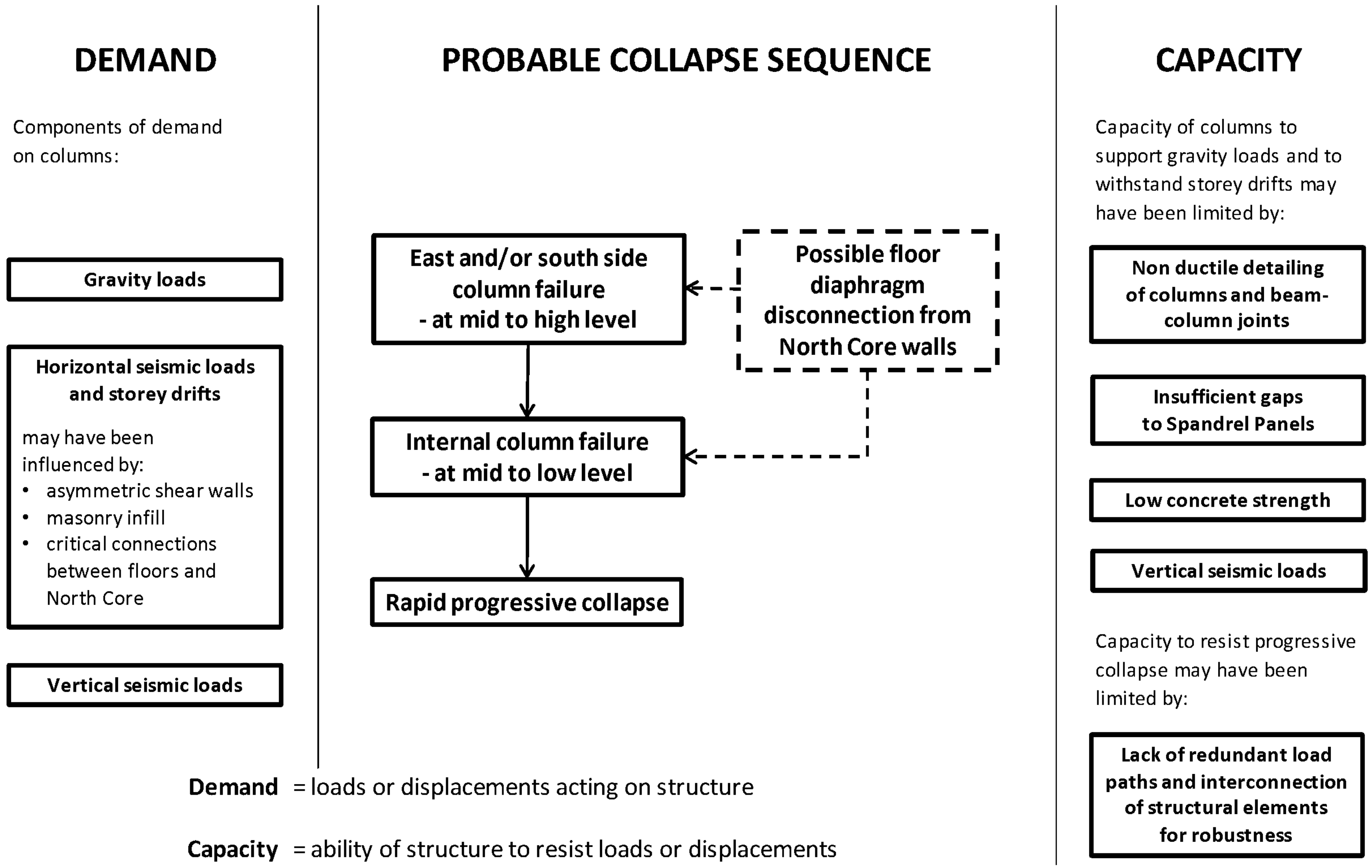

Figure 16 Collapse sequence flowchart

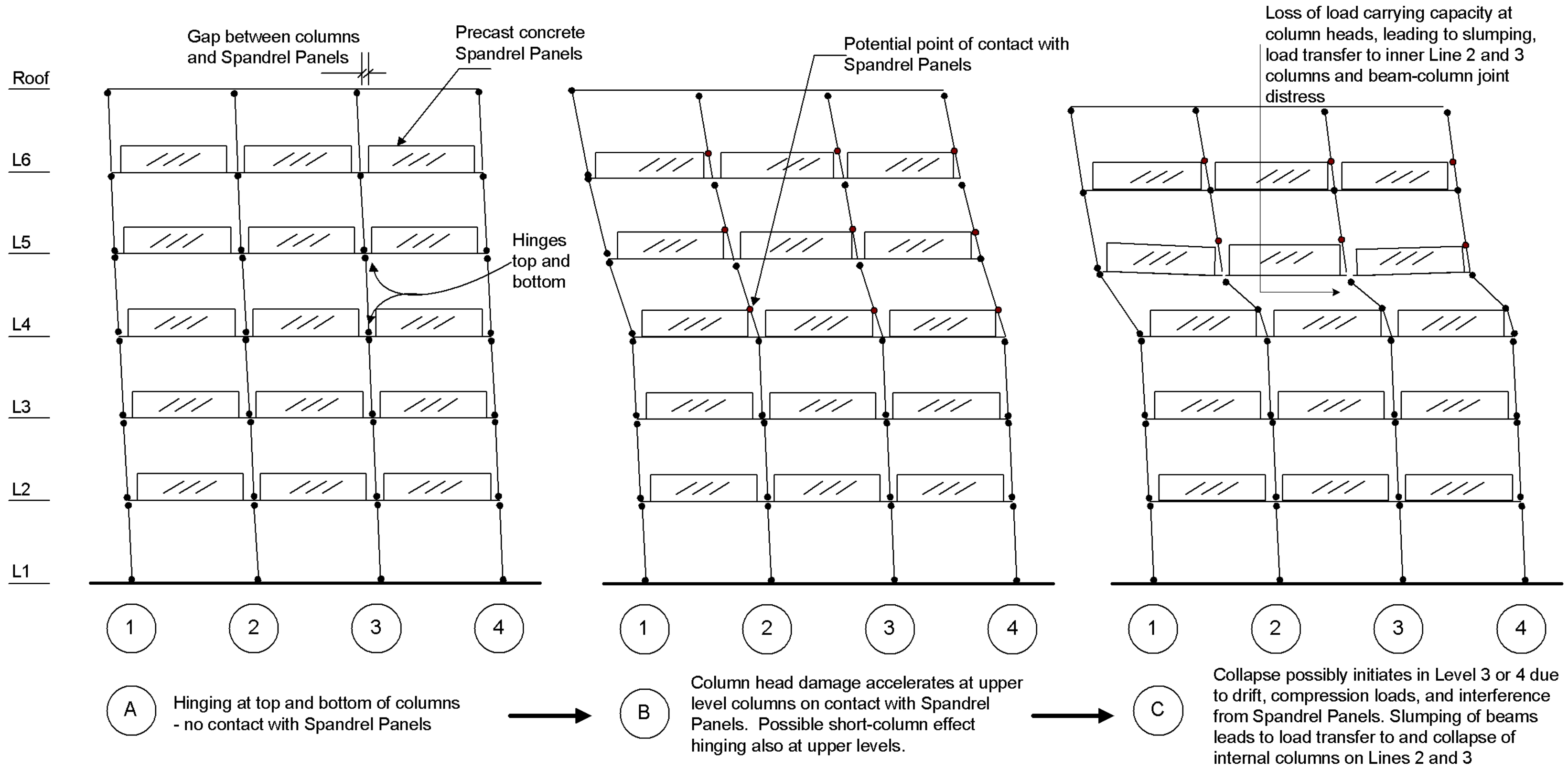

Figure 17 - Possible collapse sequence along Line F as Inter-storey drifts reach critical levels and columns begin to fail from lack of displacement capability or from additional damage caused through contact with precast concrete Spandrel Panels. Displacements and damage are greatest in the upper levels, but inelastic drift capacity less in the lower levels. Also change in torsional stiffness at Level 4 due to the Line A masonry infill wall stopping at that level may have contributed to collapse appearing to initiate above Level 4.

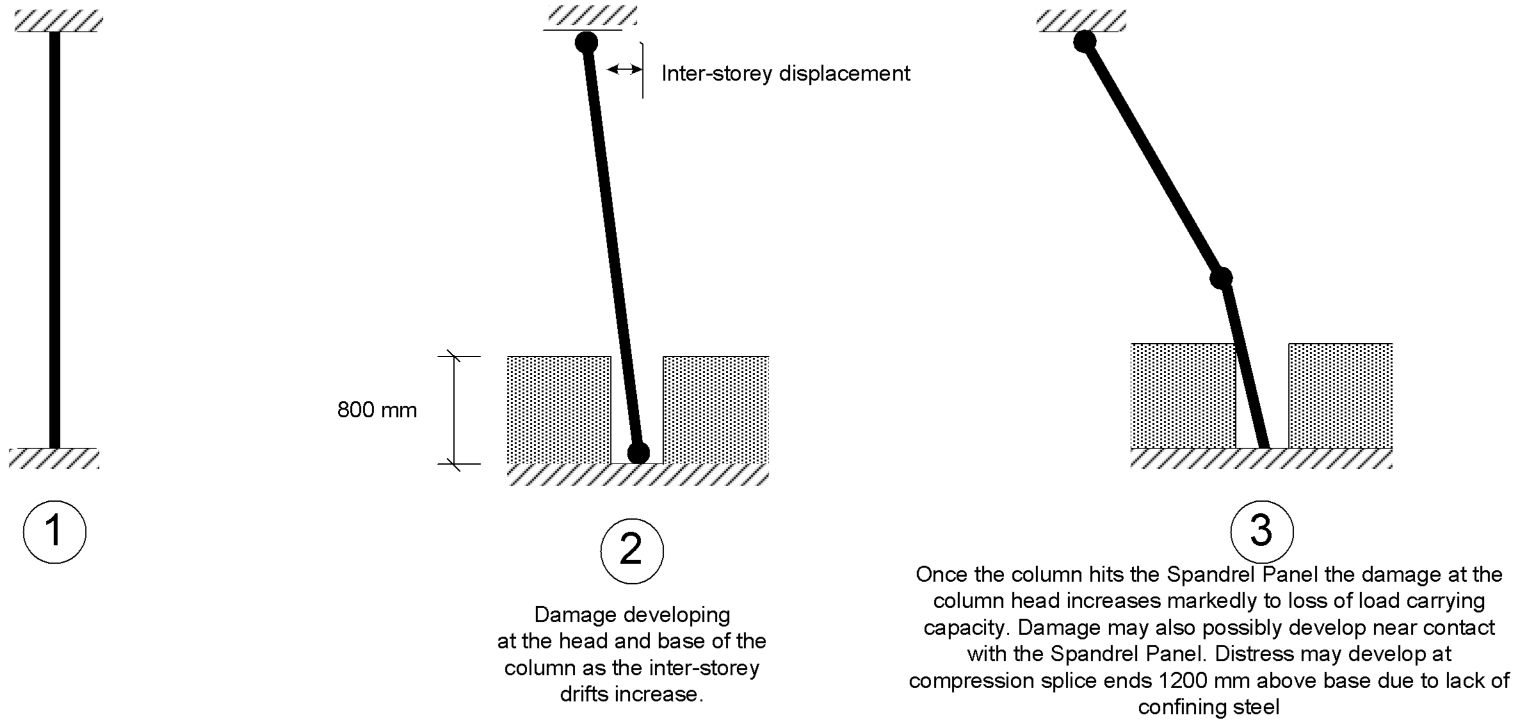

Figure 18 - Development of flexural/compressive column head damage in Line F columns and the damage acceleration effects from interference by the Spandrel Panels.

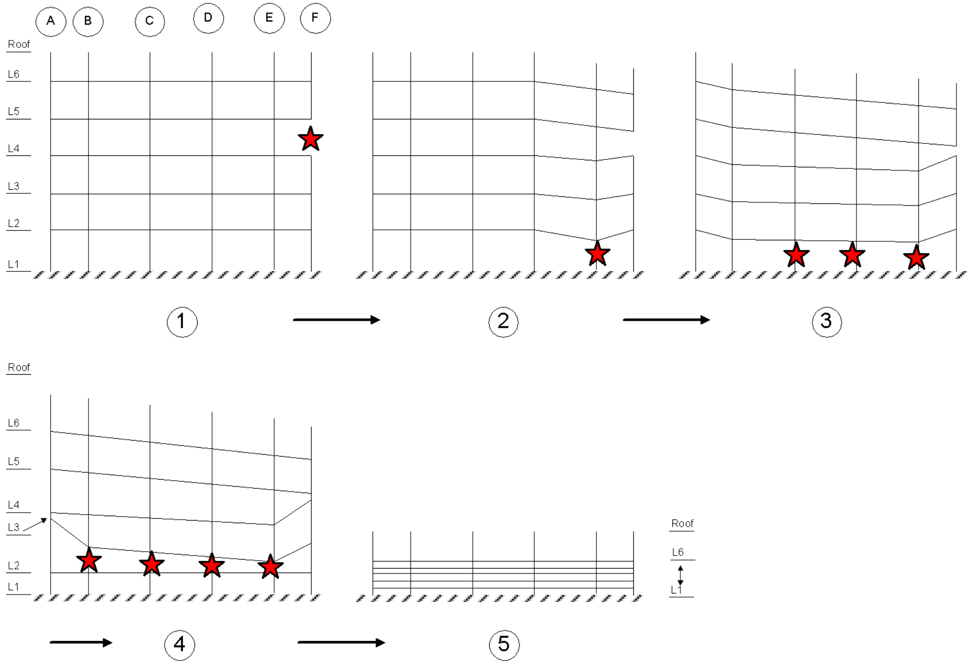

Figure 19 - Possible progression of collapse from loss of column capacity on Line F is shown sequentially as follows: (1) Collapse of Line F columns above Level 4 leads to extra floor area being supported off columns on Line E; (2) The Line E columns begin to collapse under the extra load; (3) As the Line E columns sink additional floor area becomes supported on the Line D columns which in turn begin to collapse, causing an eastward tilt in the upper levels; (4) The upper levels then hit the Level 4 Line F; (5) The collapse completes with all floors laying on top of each other. Note that collapse is also spreading in the north-south direction simultaneously to this as shown in Figure 19.

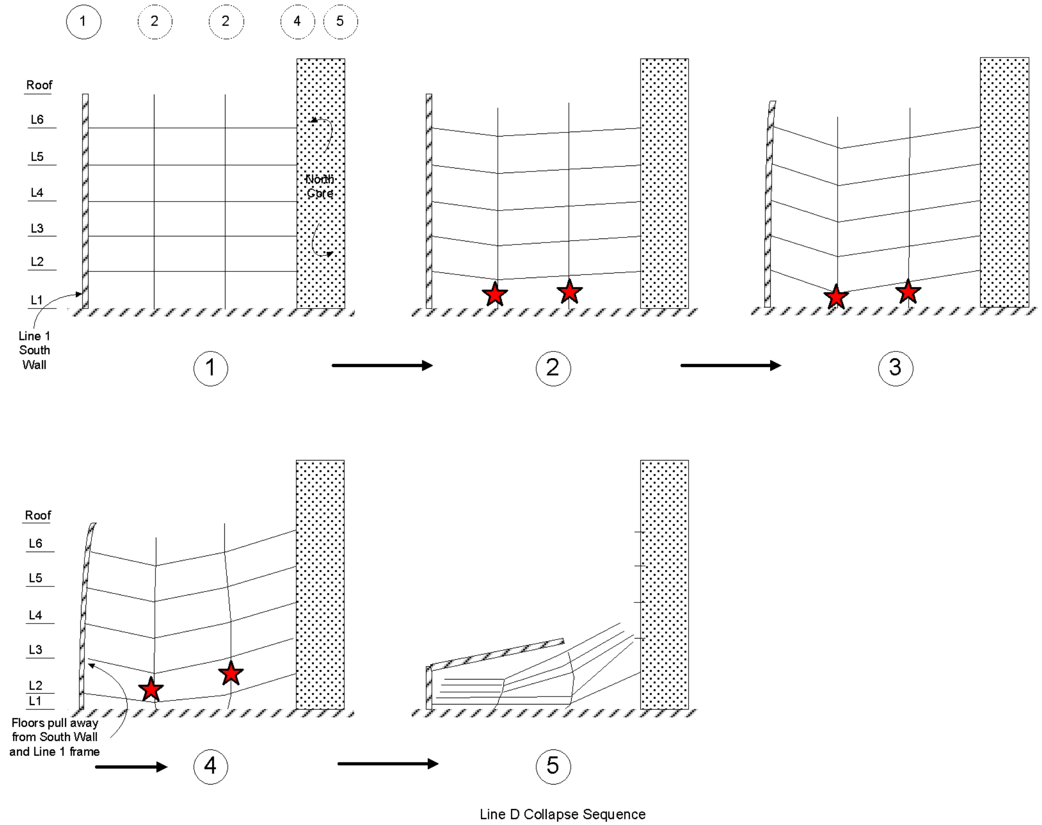

Figure 20 - Possible progression of the collapse on a north-south section through the building simultaneous with that shown progressing westwards in Figure 19 (1) Initial condition ; (2) Line 2 begins to subside; (3) As Line 2 subsides further the stiffer and stronger North Core pulls the collapsing floors towards it; (4) The South Wall is pulled northwards; (5) The slabs pull away from the North Core eventually lying diagonally against it and the South Wall is pulled down onto the collapsed building.

1 INTRODUCTION

OBJECTIVES AND SCOPE

The Department of Building and Housing ("the DBH") appointed the authors to prepare an independent structural report under the direction of an Expert Panel to identify the causes of the CTV Building ("the CTV Building") collapse in accordance with the Terms of Reference.

TERMS OF REFERENCE

The Canterbury region suffered a severe earthquake on 4 September 2010 and an aftershock on Boxing Day. This was followed by another, more damaging aftershock on 22 February 2011. The magnitude 6.3 aftershock on 22 February 2011 caused significant damage to Christchurch, particularly the CBD, eastern, and southern suburbs, the Port Hills, and Lyttelton.

The high intensity of ground shaking led to a number of collapsed or seriously damaged buildings and a large number of people killed or seriously injured. It is important for New Zealanders that the reasons for the damage to buildings generally in the CBD, and to some particular buildings, are definitively established.

Matters for Investigation

The buildings specified for detailed analysis include the: Pyne Gould Corporation; CTV; Forsyth Barr and Hotel Grand Chancellor buildings. Others may be specified for detailed analysis as information comes to hand during the investigation.

The purpose of this technical investigation into the performance of buildings in the Christchurch CBD during the 22 February Aftershock, is to establish and report on:

- The original design and construction of the building.
- The impact of any alterations to the building.
- How the building performed in the 4 September 2010 earthquake, and the Boxing Day aftershock, in particular the impact on the building.
- What assessments were made about the building's stability/safety following the 4 September earthquake, and the Boxing Day aftershock - including the issuing of green stickers and any further structural assessments?
- Why this building collapsed.

The investigation will take into consideration:

- The design codes, construction methods, and building controls in force at the time the buildings were designed and constructed and changes over time as they applied to these buildings.

- Knowledge that a competent structural / geotechnical engineer could reasonably be expected to have of the seismic hazard and ground conditions when these buildings were designed.
- Changes over time to knowledge in these areas; and
- Any policies or requirements of any agency to upgrade the structural performance of the buildings.

The investigation will use records of building design and construction, and will also obtain and invite evidence in the form of photographs, video recordings and first-hand accounts of the state or the performance, of the buildings prior to, during, and after the 22 February 2011 aftershock.

Matters Outside the Scope of the Investigation

The investigation and report is to establish, where possible, the cause or causes of building failures. It is not intended to address issues of culpability or liability arising from the collapse of the building. These matters are outside the scope of the investigation.

2 INVESTIGATION METHODOLOGY

The investigation into the collapse of the CTV Building included:

INFORMATION GATHERING

The following documents were made available to the authors:

- Summary listing of consents for the property.
- Building consent drawings ("the Drawings").
- The Site Investigation Report dated 18 June 1986.
- The structural design calculations dated 1986.
- A structural review report dated January, 1990, undertaken after construction of the CTV Building for a prospective purchaser ('the 1990 Review Report").
- The structural specification dated 30 September, 1986 ("the Specification").
- The GNS records on the ground shaking near the CTV Building at the time of its collapse ("the GNS Records").
- Post-September Earthquake damage assessment reports

WITNESS INTERVIEWS

Interviews were conducted with members of the public who had been in the building at the time or saw the CTV Building collapse. Interviews were also held with people involved in the design, construction and ownership of the building.

SITE EXAMINATION AND MATERIALS TESTING

HCL visited the CTV Building site with a DBH engineer on 12 March 2011 and examined the debris remaining from the collapse at that time ("the Site Examination"). Photos taken by others prior to debris being moved and prior to the Site Examination have also been reviewed and considered. The authors were advised that the condition of the debris remaining on site at the time of the Site Examination was in most cases the same as it had been immediately after the collapse, except that it had been moved. The slab remnant at Level 6 and the Drag Bars at Levels 4, 5 and 6 of the North Core had been removed for safety reasons prior to the Site Examination.

Portions of reinforcing steel and concrete cores at critical failure locations were selected for testing during the Site Examination by HCL for laboratory testing of mechanical properties ("the Materials Testing").

Concrete testing was performed by Opus International Consultants Ltd Christchurch Laboratories Ltd ("Opus"). Mechanical testing of reinforcing steel was performed by SAI Global Ltd Christchurch laboratory ("SAI Global"), and hardness testing of Drag Bar anchor threaded rods was performed by Materials and Testing Laboratories Ltd, Auckland ("MTL").

The results of the Site Examination and Materials Testing are summarised in Appendix L of this report and reported in detail in the separate HCL report (Hyland 2012).

STRUCTURAL ANALYSES

Three dimensional elastic response spectra analyses (ERSA) were undertaken. The basis of and the results of ERSA are reported in the appendices.

Three dimensional nonlinear time history analyses (NTHA) were also undertaken by SSL in conjunction with Compusoft Engineering Ltd ("CSE"). The basis and summary of the results of the NTHA are reported and the findings summarised in Appendix D.

DETERMINATION OF COLLAPSE SEQUENCE

To determine the cause of the collapse required careful observation of the way the collapse debris lay after the collapse, laboratory examination and mechanical testing of materials and components salvaged from the site, identification of the possible collapse sequence, computer based 3D structural analysis of the structure under earthquake loadings, structural calculations and assessment of critical components.

3 DESCRIPTION OF THE CTV BUILDING

CTV BUILDING LOCATION

The CTV Building was located at 249 Madras Street, on the corner of Madras and Cashell Streets, in the Christchurch central business district.

OUTLINE DESCRIPTION, KEY FEATURES AND PHOTOS

The CTV Building had six levels including ground floor as Level 1. It was designed as an office building but at the time of the February Aftershock it also housed an education facility at Level 4, a medical clinic on Level 5 and CTV television and radio in part of the ground floor and at Level 2. The remainder of the ground floor was used as a car park.

This investigation followed the convention for designating floors as Levels used on the structural Drawings, i.e. floor Level 1 was the ground floor.

The gross floor dimensions were approximately 31m x 23m. The building had a lightweight roof supported on steel rafters and concrete columns above Level 6. The suspended floors were constructed with 200mm thick Hi-Bond concrete slabs on precast concrete beams and in-situ concrete columns and walls. The column grid was typically 7.5 x 7.0 m.

The foundations comprised strip and pad footings and foundation beams.

The primary earthquake resisting structure consisted of ductile concrete shear walls at the north and south sides of the building. At the north side the walls were arranged in a C shape around two lift shafts, a stairway and bathrooms areas (North Core). At the south side was a considerably smaller planar coupled shear wall, with coupling beams above door openings at each level that provided access out to a steel escape stair (South Wall). The lower doorway opening had been partially in-filled with reinforced masonry to window sill height.

The secondary structure was able to be considered according to the design standards at the time to not contribute directly to the calculated lateral design resistance of the building for earthquake loadings. It consisted of moment resisting frames of precast log beams supported on 400 mm diameter and 400 mm x 300 mm rectangular reinforced concrete columns. These appear to have been detailed as Group 2 elements in accordance with NZS 3101:1982 assuming elastic behaviour for design loading derived from imposed deformations specified in NZS 4203:1984.

The CTV Building would have been constructed under Council Building By-laws, which likely adopted New Zealand Standard Specifications 1900 series as the model building bylaws either in their entirety, or with some minor changes to suit the Christchurch geographic situation.

The authors were advised that the precast Spandrel Panels appeared to have been designed to comply with NZSS 1900 Chapter 5. At times the requirements for fire design could be in conflict with the other standards. Clause 5.13.6 set out the requirements for the separation of storeys by a floor having a fire resistance rating ("the FRR") of 90 minutes and the spandrel or apron had to be a minimum of 900 mm and provide the same FRR (90 minutes). There was not the same opportunity then to use timber framing and composite materials, so the spandrels were usually in concrete or reinforced concrete masonry. The designer could also use horizontal separation distances to increase the percentage of window openings and FRR.

The Spandrel Panels would also have performed a role in sun control and in providing an architectural feature to the building.

The features of the structure that were considered by the authors to be relevant for the investigation included:

- The asymmetrical layout of the bracing walls, with the walls at the North Core being substantially stiffer than the South Wall in the east-west direction, making the system highly irregular in plan.
- The connections of the floor diaphragms to the North Core walls including consideration of the lift and stair voids in the North Core.
- Lack of connection of the floor diaphragm to walls on Lines D and D/E at Levels 2 and 3.
- The presence of a column directly under the core wall at the north-east corner, attracting axial compression and tension actions under seismic loading.
- The detailing of the edge beams as wide precast shell beams, with a significant volume of lightly reinforced core concrete and an eccentric landing onto the columns.
- The use of draped mesh reinforcement in the profiled metal deck floors.
- The relatively small dimensions of the columns and the short engagement of beam bar anchorages into those columns.
- The light and widely spaced spiral reinforcement in the 400mm diameter circular columns, and the widely spaced ties in the 400 x 300mm rectangular columns and in the beam-column joint zones.
- The possible interaction of the in-fill masonry wall with the main structural frame on Grid A.
- The possible interaction of the pre-cast concrete Spandrel Panels that contained the perimeter columns on the south, east and north faces of the building. A specific seismic separation gap was not specified on the Drawings.

The site was inspected after it had been cleared of most of the debris, and the tower was inspected by elevated platform.

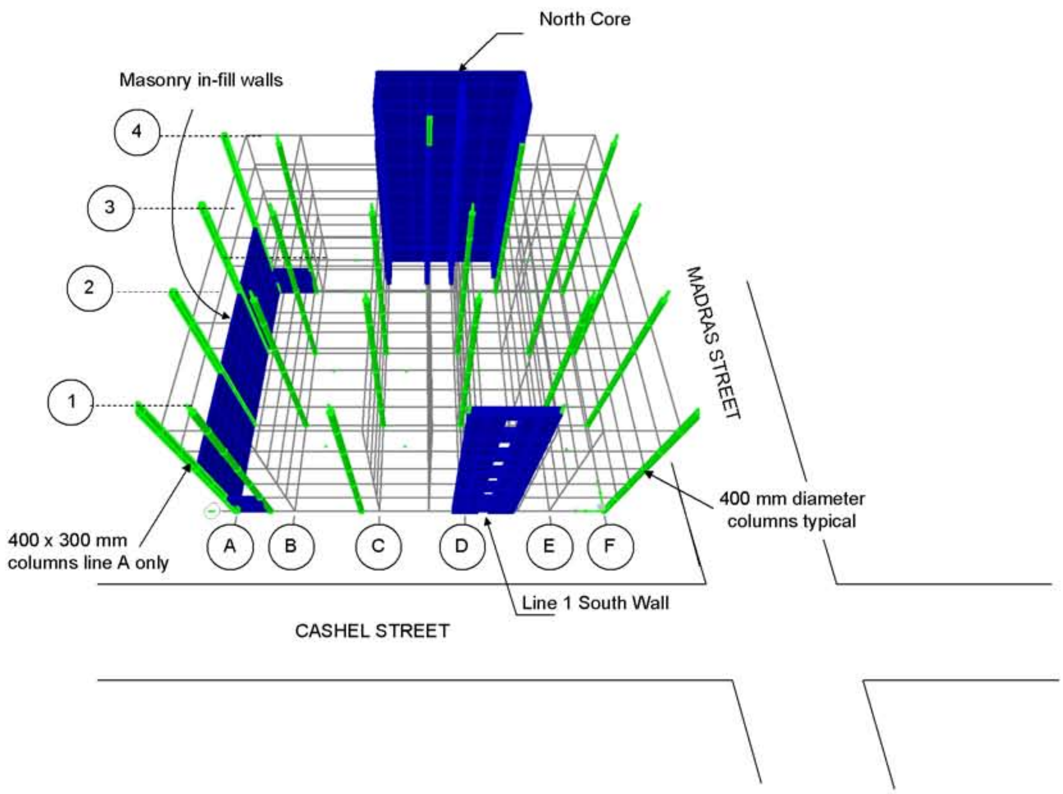

Figure 21 - Building orientation and grid lines used in the report. (Note that this diagram is not to scale and the building is set back further than indicated from Cashel Street.)

PROCUREMENT PROCESS

The developer gained building permit approval in September 1986. The owner of the property at the times of the September Earthquake and the subsequent February Aftershock ("the Building Owner") gained title for the property in March 1991.

SITE INVESTIGATIONS (SOILS, SEISMOLOGY)

The original site investigation report dated 18 June 1986 was reviewed for the authors by geotechnical engineers Tonkin and Taylor Ltd (T&T)and found to be consistent with normal practice in Christchurch at the time (Sinclair 2011). The original report estimated small settlements for foundations based upon long term soil stiffness values.

T&T recommended higher soil spring stiffness values more appropriate for modelling the soil-structure interaction effects for seismic analysis.

Liquefaction is not considered to have contributed to the collapse. There was a report of localised liquefaction in the west side of the adjacent empty site. No liquefaction was observed immediately adjacent to the CTV Building itself (Figure 71) or in the streets around the site on the south and east sides.

Pits dug at the north face of the lift and stair core walls found no evidence of soft soil, settlement, uplift or liquefaction (Hyland 2012).

DESIGN, DRAWINGS AND SPECIFICATIONS

The Christchurch City Council property file, including the building consent drawings and subsequent tenancy fit-outs was made available and reviewed. No structural design calculations or structural specification for the original design were found on the Council file. Also, no drawings, design calculations or specification for the remedial structural work after construction involving the installation of Drag Bars to the North Core, were found on the Council file.

The Design Engineer supplied a set of structural drawings, calculations and the Specification for the building, as well as the site investigation report.

Police, Fire Service, public witness photos and TVNZ news video files were received showing the collapsed structure and the deconstruction process that followed the collapse.

VARIATIONS DURING CONSTRUCTION

One drawing in the set of Drawings, Drawing S26, had been amended to show a reinforced concrete masonry wall in place of the consented precast concrete panel wall at the ground floor entry off Madras Street. This change was considered by the authors to be not significant with respect to the seismic response of the building as the wall it was not documented as being connected to the Level 2 floor diaphragm.

Pre-cast concrete Spandrel Panels were found in the post-collapse debris from Line 1 between Lines B to D indicating that these had been installed rather than the timber framed panels drawn (DENG Dwg S25).

The top course in the infill concrete masonry wall on Line A apparently was not fully grout filled as shown on the Drawings, according to workmen working on the wall immediately prior to the February Aftershock. Review of the design calculations indicates that it may have been intended to have a partially filled top course to allow movement to occur.

The concrete masonry infill walls up to Level 4 on Grid A on the west side were drawn as panels separated from the main structure by vertical joints filled with a flexible sealant. However, it was reported by one of the same workmen that the joints were filled with mortar on the outer face.

REMEDIAL WORK AFTER CONSTRUCTION

An Independent Consulting Engineer undertook a review for a prospective purchaser of the building in January 1990. Their report showed that they had concerns about how the floor slab diaphragms were attached to the lift shaft walls in the North Core. The Independent Consulting Engineer advised that these concerns were conveyed to, and acknowledged by, the Design Engineer. The Independent Consulting Engineer's client did not buy the building.

The Site Examination found structural steel angle Drag Bars bolted into the lift shaft walls and the floor slab at the three upper levels 4, 5 and 6, but not the two lower floors Level 2 and Level 3.

Drawings and calculations provided by the Design Engineer showed the Drag Bars to have been designed in and a quote for installation received in October 1991. No record of these alterations was found on the Council property file. There was no record of the remedial work on the Council file. Council have advised that it appears no application was ever made to Council by the Building Owner in respect of that work.

POST-OCCUPANCY TENANCY ALTERATIONS

From the Christchurch City property file it appears there have been several tenancy changes during the life of the building. Interviews with tenants confirmed that the following tenancies were within the building at the time of the February Aftershock:

- Level 6 – Offices in western half. The east side was vacant.
- Level 5 – Medical clinic.
- Level 4 - Language school.
- Level 3 – Vacant office. Had been a travel school but they had moved out in December 2010 so was vacant at the time of the February Aftershock. Some fit out work was reported to be in progress at the time of the February Aftershock.
- Levels 2 and 1 – Television and radio studios.

The latest tenancy floor plans were searched out and reviewed. The consented floor plans appeared to be generally consistent with floor plans that had been sketched by USAR engineers at the scene of the collapse, based on their interviews with tenants. The latest fit out drawings at Level 4 appeared to have been for a tenant prior to the language school, and showed mainly offices and interview rooms.

The building changed use from its original commercial office use to an education facility, a medical centre and studios resulting in increased design live loads according to the design Standards. However, for the purposes of this investigation into seismic performance the vacant areas were considered by the authors to have compensated for the additional live load that would have been applicable for design.

The change of use to education facility at Level 3 in 2001 was noted in correspondence on the Council file. At that time Section 46 (2) of the Building Act 1991 appeared to require that the building would, in its new use, comply with the structural provisions of the building code as nearly as is reasonably practicable to the same extent as if it were a new building. No structural engineering report relating to that change of use was found on the file. In relation to the other education tenancy that occupied Level 4, no building consent application or structural engineering report relating to a change of use was found in the Council file. No record of any structural changes made to the building, as a consequence of the change of use in either case, was found.

One bay of the concrete masonry wall at Level 1 adjacent to the North Core was shown removed and reconstructed to a new curved alignment as part of an alteration to the Madras Street entry area. Other masonry infill walls appear to have been consented for one of the previous tenancy alterations at Level 1. However these walls were subsequently removed as part of the fit-out work for the latest television and radio tenancy at Levels 1 and 2. As a result it was decided that the masonry walls at the west side only would be modelled in the analyses carried out for this investigation.

As part of the fit-out work to accommodate the television and radio studios a new internal stairway was constructed between Levels 1 and 2 near the south-east entry, involving the creation of a large penetration through the Level 2 floor slab.

A small area of Lundia compacting storage was shown on the Council fit-out drawings for level 5. However it is uncertain whether this existed at the time of the February Aftershock. The potential additional weight from this was considered not to be significant, taking into account the vacant tenancies at the other levels.

4 EARTHQUAKE AND OTHER EFFECTS PRIOR TO 22 FEBRUARY 2011

EFFECTS OF 4 SEPTEMBER 2010 EARTHQUAKE

The CTV Building gained a green placard from the Level 1 rapid assessment that was carried out on 5 September 2010, one day after the earthquake. The Level 1 rapid assessment form noted that the exterior only was inspected and that no damage was observed.

On 7 September 2010 a follow-up Level 2 rapid assessment was carried out. The Level 2 rapid assessment form noted that "the building was looked at by three senior Christchurch City Council building officials, the building manager was interviewed and no issues were sighted by users of the building." It was noted on the Level 2 form that the existing placard type was Green, and the new posting chosen was "Inspected, Green, G2." The G2 damage intensity was defined on the form as involving "light damage", which was "low risk". The G2 usability category was defined as "Occupiable, with repairs required."

Subsequently, a damage report dated 6 October 2010 was prepared by Consulting Engineer A who had been engaged by the building owner.

The damage report identified minor structural damage and non-structural damage in several areas, and included selected photographs (Figure 22 to Figure 27), as follows:

- Fine diagonal cracks up to 0.2mm wide in the first two storeys of the south coupled shear wall.
- Fine diagonal cracks up to 0.2mm wide and horizontal cracks up to 0.3mm wide at some construction joints in the North Core shear walls.
- Fine circumferential cracking to the north-east corner column immediately above the spandrel at level 4.
- Circumferential cracking up the height of the column connected to the North Core wall D/E at level 6 (Figure 22).
- Circumferential cracking up the height of the south west corner column at level 6 (Figure 23).
- Fine diagonal cracking to the level 2 beam on the north face in the eastern end bay.
- Spalling of the plaster finish to the ends of the spandrels adjacent to the south coupled shear wall at most levels.
- Spalling of plaster finishes from the inside face of the ground floor concrete masonry wall on Line 4 at the south side of the stair (Figure 24).
- It was noted in the damage report that "at the west end of the building in the garage at ground storey there are concrete block infill panels

between the structural columns. These block infill panels are separated by a flexible sealant from the columns. They do not appear to have suffered any damage."

- Cracking of the floor slab at level 4 where it connects into the South Wall (Figure 24). It is not clear whether this was caused by the earthquake.
- Non-structural damage, including cracks to wall and ceiling linings and to windows, and permanent deformation of door openings.

Work was in the process of being carried out to repair some of the above damage at the time of the February Aftershock, including epoxy grouting up of cracks in concrete columns and beams.

Tenants interviewed described the building as feeling more "flexible" after the February earthquake. Demolition of the neighbouring building commenced after this event and continued until the week before the February aftershock. Shudders were often felt through the CTV Building, especially when the adjacent concrete foundation structure was demolished with wrecking balls and concrete pokers, as can be seen in Figure 29. This is likely to have added to a sense of unease with the CTV Building by tenants.

Tenants recalled and the OIE reported many cracks in and damage to partitions and adjacent to structural walls and columns. Eyewitness 1 recalled there being very visible floor to ceiling cracks at the corners of the walls butting into the lift door walls, on each side of the lifts at Level 6. It is possible that this indicated that some slippage had occurred between the Drag Bars and the underside of the profile metal deck concrete slab or the lift shaft walls. The holes in the Drag Bars around the anchor bolts could be expected to have had some tolerance for movement due to the typical over sizing of holes by 2 or 3 mm in line with normal construction practice. This slippage, if it occurred, could not be verified.

The authors conclude from the above that there was no evidence reported of significant change to the building's seismic resisting capacity although it is acknowledged that no inspection of the connection of the floor slabs into the North Core, including the Drag Bars, or the connection of the column at grid 4 D/E to the North Core was reported.

The cracking to the Level 6 columns may have been an indication of interaction of the columns with the Spandrel Panels in the September Earthquake.

Figure 22 - Level 6 400mm diameter columns Column 4 D/E outside lift with horizontal cracking.

Figure 23 - Level 6 400mm diameter Column on Line 1/A-B with horizontal cracking.

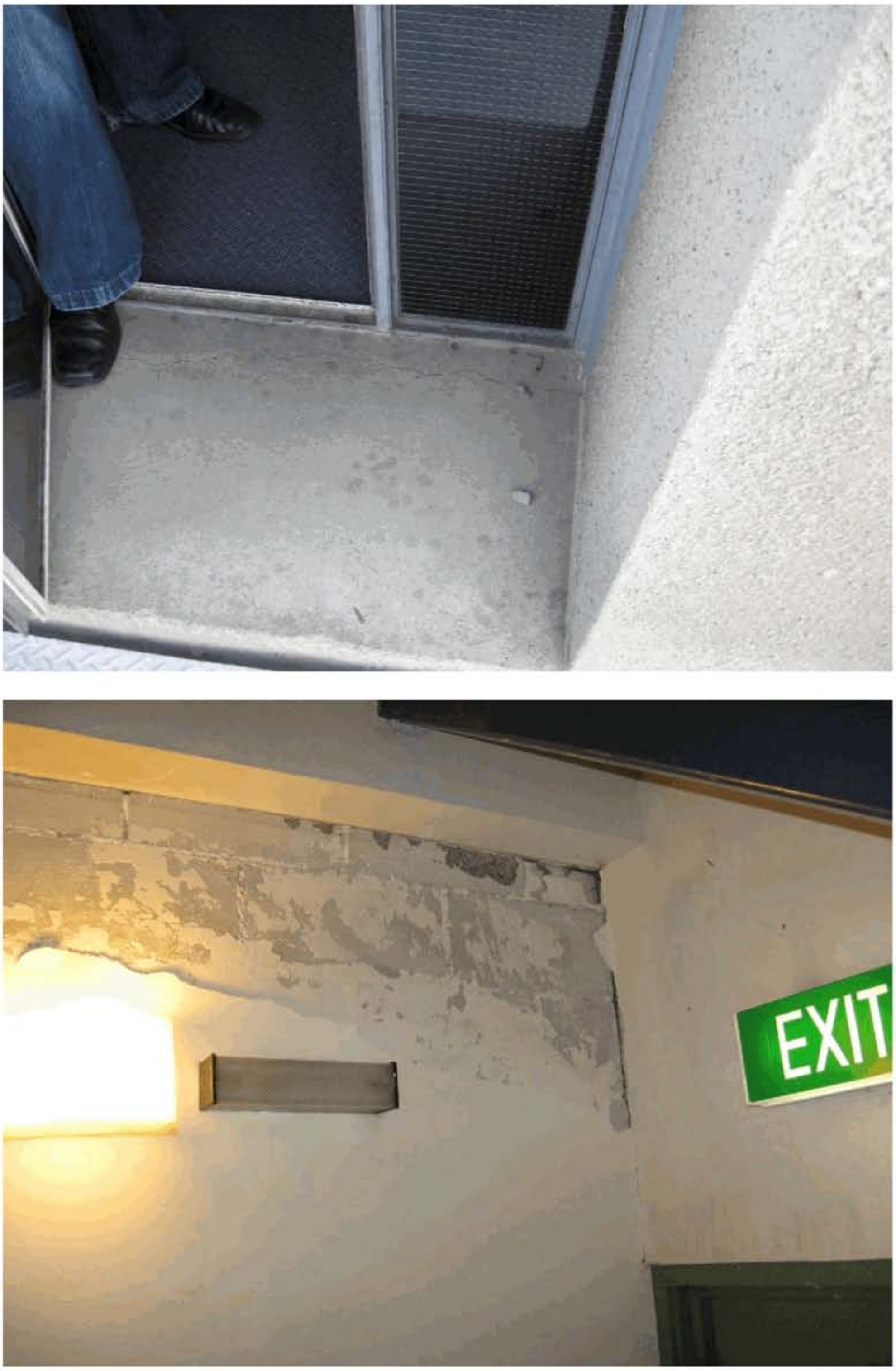

Figure 24 - (Top to bottom) (a) Fine cracking in floor at junction with South Wall; (b) Spalling of plaster finishes on internal masonry in-fill wall on Line 4 in front of stair well.

Figure 25 - Damage to wall Linings after 4 September 2010 Earthquake.

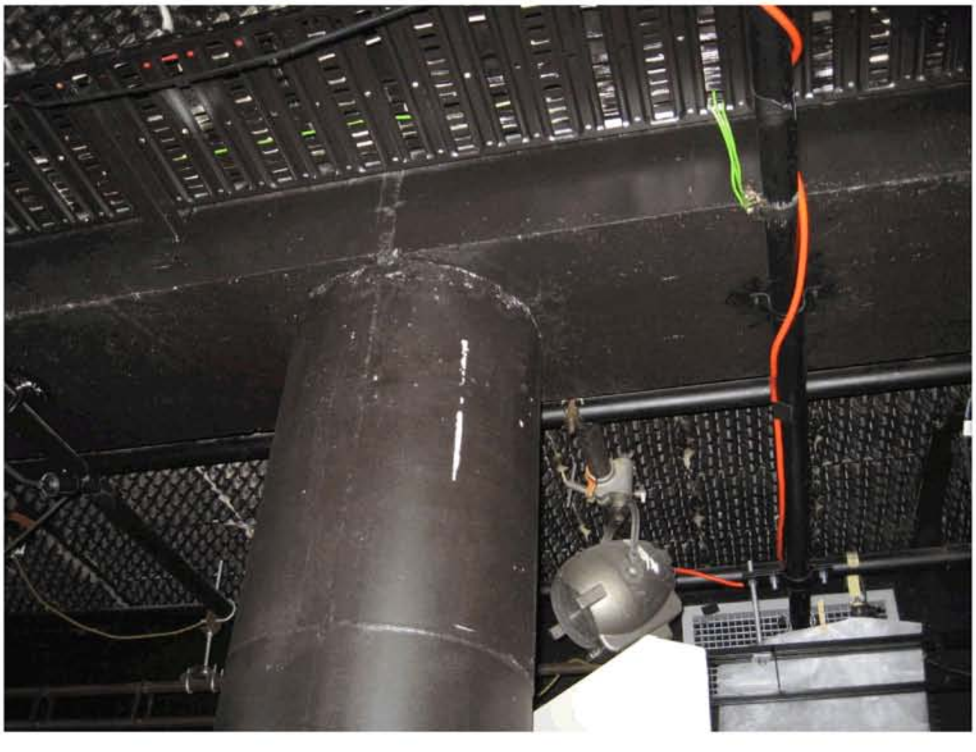

Figure 26 – Internal 400mm diameter column and beam after 4 September 2010 Earthquake. No visible cracking evident. A horizontal circumferential formwork mark can be seen approximately 600 mm down from the underside of the beam indicates it was a Level 1 column which were constructed using extensions to the formwork shutters.

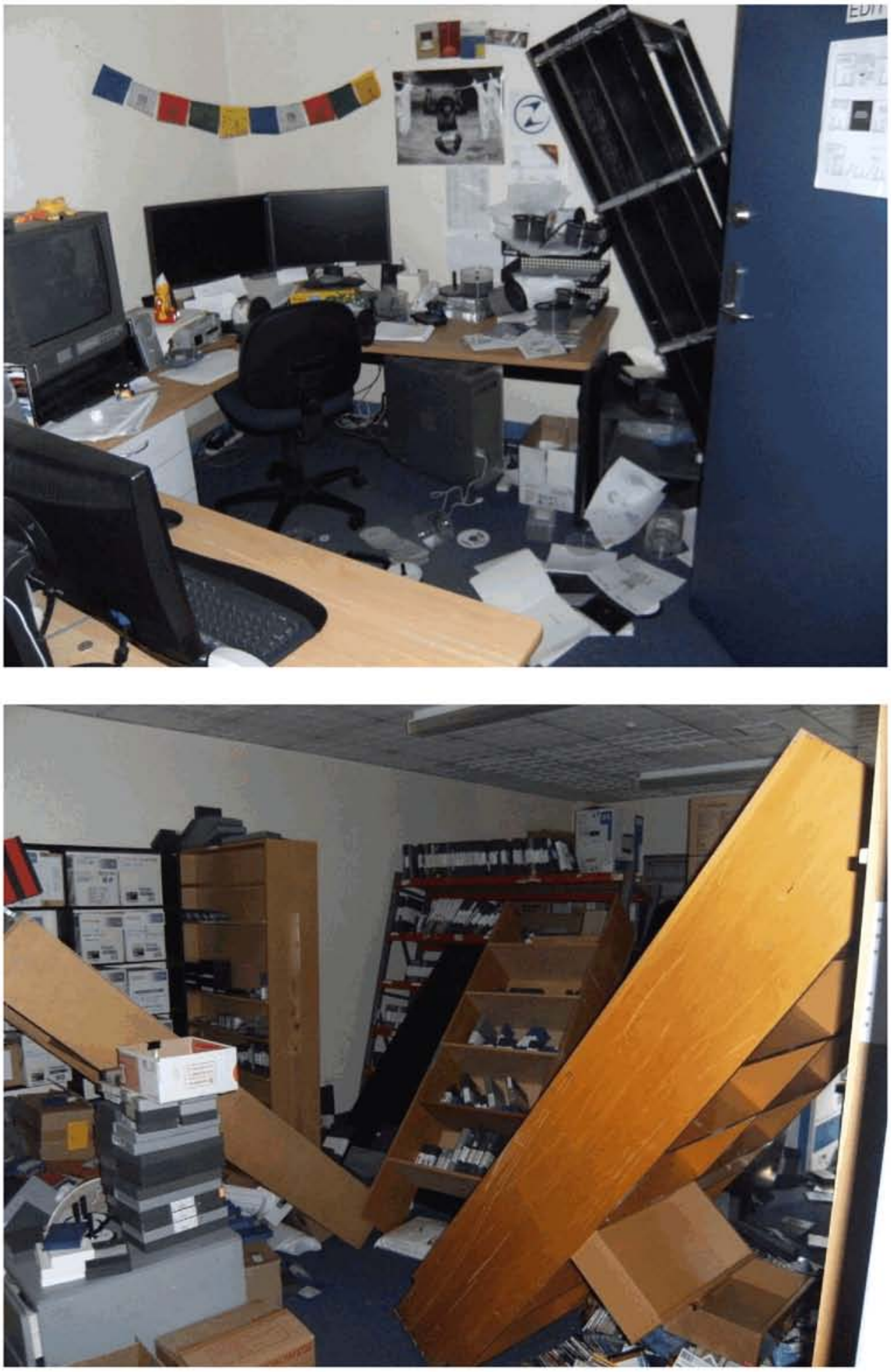

Figure 27 - Damage to office furniture on Level 2 after the 4 September 2010 Earthquake.

EFFECTS OF 26 DECEMBER 2010 AFTERSHOCK

A 'Christchurch EQ Rapid Assessment Form – Level 1', and a 'USAR Damaged Building Reconnaissance Report' dated 27 December 2010 were obtained from the Council files. Both rapid assessments were from the outside of the building.

The first page of the Rapid Assessment form identified a broken pane of glass that may have been at risk of falling onto a balcony. The second page of the USAR Damaged Building Reconnaissance report showed the broken glass pane had been re-inspected and recommended temporary hazard tape and no further engineering assessment.

A detailed description with photos, of the interior damage that occurred in the December Aftershock on Level 6 was obtained from the tenant. The damage on Level 6 was described by the tenant as more severe than in the September Earthquake.

Filing cabinets were knocked over in the south direction in offices on the west wall of the building. Pictures fell from the walls. Less damage was reported in the offices further into the building (Figure 28).

No obvious further damage was reported to have occurred to partition walls. Damage was not sufficient for an insurance claim to be made or for partitioning to be repaired on Level 6.

The column on Line 4 D/E by the lifts had visible wavy cracking which it also had after the September Earthquake.

The tenant contacted the Council for an inspection, however the tenant was apparently advised by the landlord that the building had been inspected by his engineer, and the damage was considered minor, so the Council inspection was cancelled by the tenant.

A student interviewed from Level 4 advised that a person thought to be an engineer inspected the building within the fortnight before the February Aftershock. However the name or company that that person worked for is unknown. No damage was obvious to the student at the time.

The authors conclude on the basis of the above, and consideration of the relative size of the calculated displacements from the December Aftershock compared to those from the September Earthquake, that there was no evidence of significant change to the building's seismic resisting capacity after the December Aftershock.

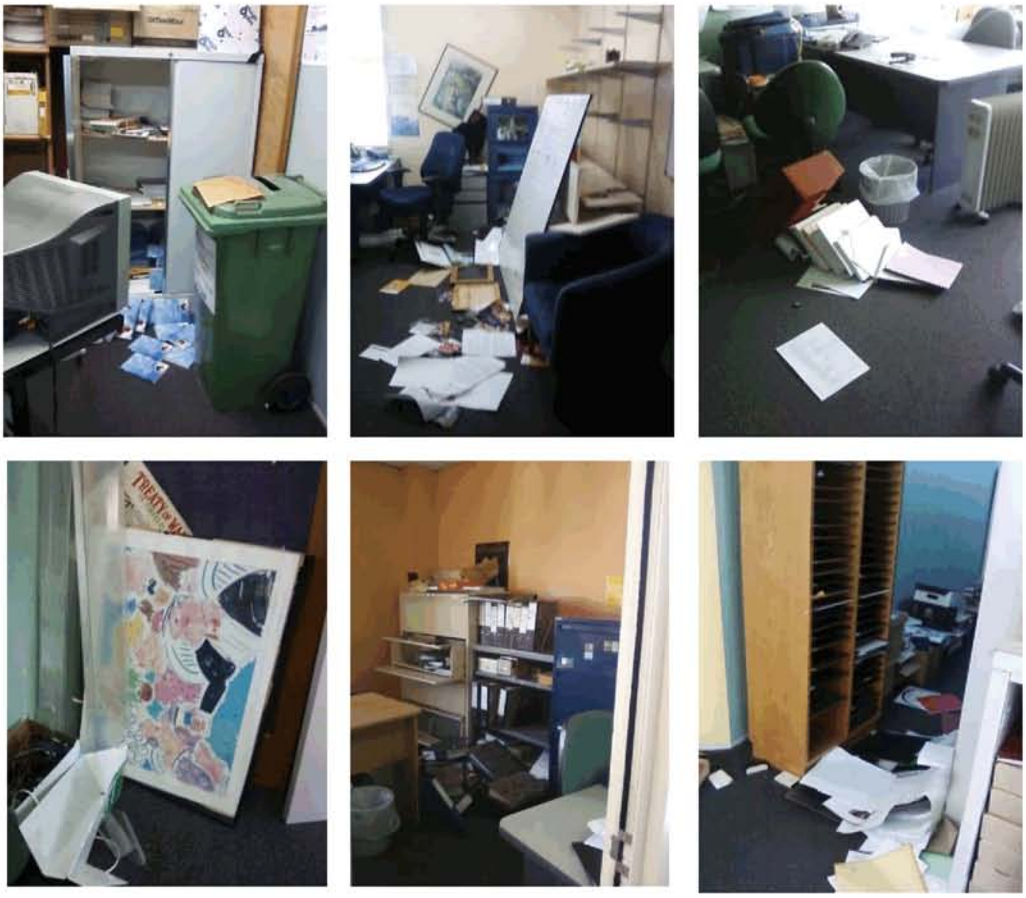

Figure 28 - Damage after 26 December, 2010 Aftershock on Level 6 of CTV Building (clockwise from top left) a) Cabinet door had opened but hadn't fallen over (Line 3/B-C); b) As it was found, except that the filing cabinet had been stood back up(Line 2?A-B); c) Oil heater had been righted. Two filing cabinets had fallen to the floor; (Line 1/B-C) d) The cubby-hole unit had not emptied of papers in the earthquake on 4th September. However in December it had fallen against the corridor wall towards Cashel Street. It had been righted before the photo was taken.(Line 2/B-C) E) The shelves and filing cabinets had gone down, but had been righted before the photo was taken (Line 4/A-B). F) The painting had fallen from the wall.(Line 1/A-B).

EFFECTS OF DEMOLITION OF NEIGHBOURING BUILDING

Demolition of a reinforced concrete building and preparation of the site for a car park, commenced on the adjacent site immediately after the September Earthquake.

Work on the adjacent site continued until the February Aftershock and collapse of the CTV Building on 22 February, 2011.

Heavy machinery with pneumatic pokers and pincers, and drop hammers, were used to break it up (Figure 29). This caused on-going and disturbing vibrations to occupants in the CTV Building.

The authors consider it unlikely that structural damage was caused by the demolition sufficient to affect the earthquake resistance of the CTV Building. This is because it is common practice to use such equipment for demolition work like that seen in Figure 29 and not to cause any significant structural damage to adjacent buildings.

Figure 29 - Heavy machinery demolishing the building adjacent to the CTV Building after the 4 September Earthquake. The boundary wall is still in place covering the Line A infill masonry wall of the CTV Building.

5 COLLAPSE ON 22 FEBRUARY 2011

The February Aftershock caused the sudden and almost total collapse of the CTV Building. The North Core only remained standing after the collapse.

Debris began to be moved very shortly after the collapse by heavy machinery that was in the neighbourhood at the time. It was reported that a fire started shortly after the collapse near the North Core and continued for several days.

The west face along Line A of the building can be seen in Figure 30. No liquefaction is evident on the vacant adjacent site. Smoke from the fire can be seen beginning to arise. Some of the upper light weight external panels between Levels 4 and 6 have fallen northwards from their original position, possibly as a consequence of the South Wall falling north towards them. Large diagonal cracks can be seen in the Level 2 to 3 masonry infill wall that had fallen onto the vacant site at the south end.

Along the east face on Madras Street (line F) fractured columns with spear shaped heads projected out of the debris adjacent to precast concrete Spandrel Panels that had tumbled onto cars parked in the street, as seen in Figure 32. The unpainted portions of the columns seen in Figure 33 showed where the Spandrel Panels had been located.

The slight eastwards throw of the debris along Madras Street was consistent with the report of Eyewitness 6 of a slight tilt to the east of the upper levels before they fell straight down. The column head fractures are consistent with flexural/compressive failure scenario shown in Figure 17 and Figure 18.

Viewed from the south face on Cashel Street Cashel St (Line 1) the North Core stood out clearly as seen in Figure 31. The cars in the car park on the south face were largely undamaged. The white escape stair that was attached to the South Wall can be seen still attached to the wall as it lay on top of the collapse debris. A portion of the floor slab in front of the lift doors at level 6 remained suspended in mid-air without column support, and a similar portion of slab at level 5 hung down precariously.

The column that had been attached to the east side of the North Core appeared to have collapsed along with the internal columns, pulling the floor slabs away from the North Core. The South Wall had then fallen northwards onto the top of the debris after the slabs attached to it pulled it downwards as they collapsed and detached from it.

Figure 30 - View of the entire west wall on Line A from Les Mills immediately after collapse before debris removal commenced. No signs of liquefaction can be seen on the adjacent vacant site. Smoke from the fire can be seen beginning to rise. Some of the upper light weight external panels between Levels 4 and 6 have fallen northwards, possibly as a consequence of the South Wall falling north towards them. Large diagonal cracks can be seen in the Level 2 to 3 masonry infill wall at the right hand end that has fallen onto the vacant site. Roof steelwork can be seen in mid-picture.

Figure 31 - Cashel St. south face with North Core tower in background immediately after collapse and prior to the fire starting. The cars in the car park on the south face were largely undamaged. The white escape stair that was attached to the South Wall can be seen still attached to the wall as it lay on top of the collapse debris.

Figure 32 - Corner of Cashel and Madras Streets looking towards North Core. Fractured columns and fallen Spandrel Panels are visible (MSN photo).

Figure 33 – View looking west across Madras Street. A Line F/3 column is highlighted showing conical fracture in the painted portion, above unpainted portion which had been enclosed by Spandrel Panels.

Figure 34 - North Core with Level 4 and 3 slabs laying diagonally against it.

6 EYEWITNESS ACCOUNTS

INTRODUCTION

Eyewitnesses were formally interviewed and interviews recorded, transcribed and summarised to identify consistent observations of the collapse.

The experiences of those who survived the collapse of the building, combined with those viewing it from different angles from outside, have given helpful clues as to what actually happened to the CTV Building. These have been taken into account in the interpretations of the analyses.

Specific observations included:

- Feeling vertical jolts.
- A tilt to the east of the top portion of the building.
- East-west movement or twisting.
- The upper level(s) falling as a unit onto the floors below.
- The building falling in on itself.
- It all coming down in seconds.

INTERPRETATION OF EYEWITNESS OBSERVATIONS

In reflecting on the interview findings, the authors have taken into account three very important human responses to crisis.

The subjectivity of time

Time can stretch or shrink or be lost altogether for some people in times of crisis. This is why gaining multiple perspectives is important.

Although time distortion is commonly reported during a traumatic experience, there is little research addressing the phenomenon. However the study referenced below has investigated the role of the effect on time perception in a very stressful experience by indexing novice tandem sky-divers' levels of fear and excitement before the sky-dive and soon after landing. Estimations of how long skydivers thought their experience lasted were obtained after landing. Whereas increased fear was associated with the perception of time passing slowly, increased excitement was associated with the perception of time passing quickly (Campbell and Bryant 2007).

The subjectivity of sensation.

For example, if someone has no sensation of falling, it suggests a "slow" fall. In real terms there is a "rush" that is experienced with a fast fall, for example, like sky-diving – and no sensation at all when falling slow in a lift or an elevator. So people's sensations can say a lot about the way the building fell.

The subjectivity of words.

The authors took care to find out what people meant by certain words they chose in their description of the collapse of the building. For example, "Pancaking" to one person, can mean a different thing to another.

COMMON OBSERVATIONS ABOUT THE FEBRUARY AFTERSHOCK AND COLLAPSE

1. A first jolt, thump, jump, kick from underneath that felt like being pushed or kicked upwards.

    - "Sudden violent lurch then continuous movement."
    - "A bounce – a jump - then everything moving."
    - "A bolt like a thump – real sharp jolt from underneath that moved you upwards."
    - "Super violent. I was bounced."
    - "Massive jolt, then bad shaking."
    - "A hit, bang. Then shaking."
    - "Vertical jolt, a sense of jump upwards."
    - "A vicious punch."
    - "Felt being lifted, then dropped, then kicked again on all levels."

2. Sense of tipping, swaying, moving in an east-west direction

    - "A sense of tipping."
    - "A sideways movement."
    - "Top leaned to towards east. Collapsed straight down. Just a slight lean."
    - "Swaying."
    - "Seemed to drop in the reception corner."
    - "A sense of slope after dropping from the jolt – then very fast collapse."
    - "Twisted back and forth."
    - "A slight tilt to the back from the ground towards Cashel Street."
    - "It was just a slight lean and it went down vertically."
    - "A sense of slope after dropping from the jolt – then very fast collapse."

3. The building collapsed in on itself.

- "It fell in on itself."
- "It fell straight down in on itself."
- "Went in on itself."
- "Collapsed in on itself."
- "Everything was so compact, a tight pile."
- "It fell into a complete square…. compacted into something that was less than the height of floor to ceiling."
- "The building just came down in a pile. The lift well was still standing."

4. A sense of specific levels giving way – then falling straight down.

- "Like a level gave way – then whoomf."
- "It looked like level 5 gave way – stopped for half a second then dropped to next floor, then continued all the way down."
- "Level 6 dropped as a unit onto level 5, then level 5 onto the ones below."
- "The way it fell - it was almost like a level was removed and it all just came down."
- "Next floor from top floor dropped first, whole building collapsed apart from the lift shaft."
- "Floors collapsed from south east corner working its way back. Upper columns went, then disintegration at all levels."
- "Folded in on the bottom. With the corner gone, no support."
- "Seemed to drop in reception corner, then fell around and in on itself, falling away from the lift tower."
- "The bottom couple of floors had come out, and the rest of it had come straight down."

5. Down in a matter of seconds

The majority felt the collapse only took seconds. Those who said it was slower (two) also indicated that they had either lost time or time seemed bizarre. As one Eyewitness said "time is pretty elastic in these sorts of things."

For one who sensed the building going down slowly, the building looked "like the top floated and was engulfed by a cloud".

- “Crumbled in seconds.”
- “Happened in seconds.”
- “Whole walls caved in and down in seconds.”
- “In as little as 12 seconds from the earthquake hitting.”
- “From 15 – 20 seconds.”
- “Down in 30 seconds or quicker.”
- “Started to collapse a few seconds into the quake.”
- “Seemed to happen in seconds.”
- “It dropped like a river.”
- “Came down very quick.”
- “Only 5 seconds warning from the time the earth quake hit.”
- “The CTV was down during the first earthquake.” (not a later aftershock)
- “All happened in seconds.”
- “Very quick.”

7 EXAMINATION OF COLLAPSED BUILDING

INTRODUCTION

The examination of the collapsed building involved physical examination of the North Core, collapsed structural remnants at the Madras Street site and columns extracted from the CTV area at the Burwood Eco Landfill. Testing of the properties of material samples collected during the examination was also undertaken. A detailed description of this part of the investigation is described in the Site Examination and Materials Tests report by HCL (Hyland 2012). Photographs of the collapse taken by the public prior to debris being moved and by rescue agencies and the media during the removal of debris were used to help identify the likely collapse sequence and behaviour. Appendix B contains more photos.

IMMEDIATE POST-COLLAPSE CONDITION PHOTOS

Observations of the immediate collapse debris have been made in Chapter 5.

The photos of the building immediately after the collapse indicate the following:

- It appeared that the column attached to the east side of the North Core (grid 4-D/E) had collapsed, pulling the floor slabs supported by it down from the North Core.
- The Line 2 and 3 frames appeared to have collapsed towards the centre of the building. The Line 2 frames lay northwards and the Line 3 frames lay southwards. The slabs appeared to have been pulled down and away from the North Core and South Wall as the Line 2 and 3 frames fell.
- The South Wall had fallen northwards onto the top of the debris
- The Level 2 to 3 masonry infill wall on Line A appeared to have been overloaded in shear prior to or during the collapse.
- Very little debris had fallen into the vacant site on the west face of the building. Some upper level wall debris had been thrown slightly north from its original position on that side. This may have been as a result of the South Wall collapsing onto the fallen floors and roof.
- No liquefaction appeared to have occurred on the site.
- The collapse had been confined within the building footprint on the south face.
- The slight eastwards throw of the debris along Madras Street was consistent with the report of Eyewitness 6 of a slight tilt to the east of the upper levels before they fell almost straight down. The eastwards throw was also consistent with column failure initiating along Line F leading to a slumping and eastward tilt of the levels above as shown in Figure 19.

- Column fractures were evident on Line F columns. The column fractures were consistent with the flexural/compressive failure scenario shown in Figure 18.

DEBRIS REMOVAL PHOTOS

Introduction

Most of the debris from the collapse had been removed from site and taken to a secure designated area at the Burwood Eco Landfill. Photos were examined to identify the order and manner in which structural components had fallen. This helped in the development of collapse scenarios and review of analytical results. More photos are shown in Appendix B.

Observations

The Line 1 frames attached to the South Wall had fallen northwards onto the collapsed structure (Figure 87). All the floor slabs appeared to laying on top of each other adjacent to the South Wall (Figure 90). A portion of floor slab appeared to still be in contact with the South Wall and may have prevented the South Wall from breaking over at Level 1(Figure 36 and Figure 91).

This indicated that the floors had broken away from the Line 1 frame and South Wall close to Line 1 and had fallen to the ground with the South Wall and Line 1 frame then toppling down on top of them. A portion of slab from one of the levels may have held up sufficiently against the South Wall at Level 2 to cause the South Wall to pivot about it as it fell northwards.

Figure 35 - Concrete Spandrel Panels, perimeter beams and columns on Cashel Street face (Line 1 / B-D).

Figure 36 - View from Cashel Street, east side with Line 1 South Wall lying on debris at left; end profile of floor slabs laying on top of each other in foreground; A portion of floor slab highlighted, appears to be still in contact with the South Wall at Level 2 and may have prevented the South Wall breaking over at ground level. The collapsed column on Line 4-D/E at the North Core in the background is also highlighted.

The Levels 3 and 4 slabs could be seen laying diagonally against the North Core (Figure 38). This indicated that the Level 3 and 4 slabs lost their vertical support on Line 3 prior to them breaking away from the North Core. A portion of the slabs that had been outside the lift well on the North Core appeared to have fallen away as the column connected to the North Core at Line 4-D/E collapsed (Figure 36 and

Figure 37). The failure of the Level 6 slab adjacent to the North Core indicated that vertical support from the Line 4-D/E column had been being available at that stage. Otherwise the Level 6 slab should have rotated about the tips of the Line D and D/E walls.

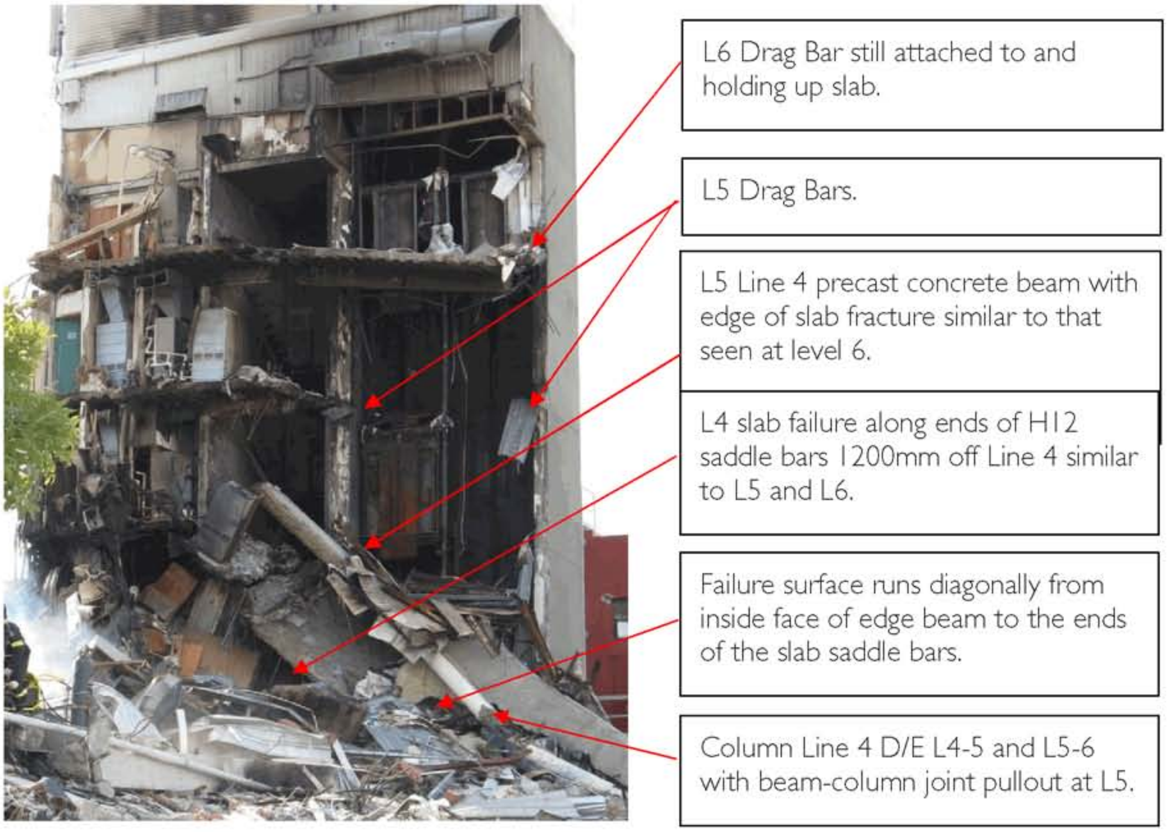

Figure 37 - Failure of slabs adjacent to the North Core.

Figure 38 – View of North Core showing Level 4 slab lying diagonally on top of Level 3 slab. This indicates that the Level 3 and 4 slabs lost their vertical support on Line 3 prior to breaking away from the North Core.

The Line 2 beams were found to be laying northwards (Figure 39) and the Line 3 beams laying southwards (Figure 99). This appears to indicate that collapse of the columns on Lines 2 and 3 had occurred before the slabs detached from the North Core and South Wall.

Figure 39 - Line 2 beams highlighted laying rotated northwards. This appears to indicate that the Line 2 columns collapsed prior to the slabs breaking away from the Line 1 South Wall and frame.

A perimeter column still attached to a Line 4 perimeter shell beam from the northwest face of the building was found showing severe damage that was perhaps indicative of Spandrel Panel interference (Figure 40). Column head damage is visible, though little flexural damage could be seen at the base.

In this case it is possible that the damage may have occurred during the progression of the collapse of the floors rather than in the initiation of the collapse. This is because the analyses showed that the inter-storey east-west drifts along Line 4 were not as high as those along Line 1 and north-south along Line F. It could be that the lack of flexural damage at the base was indicative of the Spandrel Panel having been close to full contact with the column prior to the February Aftershock. This may have prevented base hinging occurring in what is likely to have been an upper level column.

Figure 40 - Line 4 / B column with precast log beam in foreground and shell beam at rear. The column may have broken its back on a precast concrete Spandrel Panel and sustained head damage during the collapse.

The South Wall showed angled fan-like cracking that extended diagonally from the middle of the wall to the outside edges. The cracking also seemed to extend through the full thickness of the wall. This appeared to indicate that the wall had suffered flexural /tensile damage prior to the collapse of the floor slabs (Figure 41).

The east end of the South Wall had suffered concrete spalling on the outer and inner faces. The spalling on the outer south face of the wall may have been an indication of flexural/compressive damage prior to the wall falling northwards during the collapse of the floors.

Figure 41 – Angled fan-like flexural cracking on the South Wall in conjunction with spalling on the outside face at the east end. This indicates that the South Wall may have suffered flexural / compressive damage prior to the collapse.

Conclusions

Review of the photos taken by rescue agencies as the debris was removed indicated the following:

- The floor slabs appeared to have collapsed to the ground and the South Wall then toppled over on top of them.
- A portion of collapsed floor slab appeared to hold up the South Wall at Level 2, preventing it breaking over at ground Level 1.
- The South Wall appeared to have sustained in-plane flexural damage prior to the collapse of the floor slabs.
- The Line 2 beams were found to lie rotated northwards and the Line 3 beams lay rotated southwards. The level 3 and 4 slabs at the North Core were found to be lying diagonally from the North Core. This indicated that the slabs at these levels may have broken away from the North Core and South Wall as collapse occurred along Lines 2 and 3.
- The collapse of the column attached to the North Core at Line 4-D/E appeared to have occurred after the Level 6 slab had pulled away.
- What appeared to be Spandrel Panel induced damage was found in a column that also showed column head damage. The lack of damage at the base of the column may have indicated that the Spandrel Panel may have

been in close contact with the column prior to the February Aftershock occurring.

PHYSICAL EXAMINATION

Introduction

The Madras Street site was examined, material samples collected and tested by HCL following the completion of the rescue and recovery operations. Columns at the Burwood Eco-landfill were also extracted and tested by HCL. The CTV Building Site Examination and Materials Tests report describes the findings in detail (Hyland 2012). A summary of the results is in Appendix C and the conclusions are summarised as follows.

Madras Street Site Examination

The Madras Street site was examined over a number of days from 12 March 2011.

Site Condition

No evidence of liquefaction around the perimeter of the building was found.

Structural remnants had been labelled and placed in a pile at the southeast corner of the site by rescue and recovery agencies for review. This assisted the investigation greatly.

North Core

The North Core was inspected on two occasions using a man cage suspended from a crane the first time and the second time from a Fire Service snorkel. Measurements of the slab and Drag Bar remnants were made as they were found at that time.

The North Core was found to have only minor cracking near its base and otherwise its walls appeared largely undamaged (Figure 42). Evidence of fire charring could be seen on the inner surfaces.

The slab outstands on the southwest side of the North Core (Line C to C/D) were in the condition seen in the photos immediately after collapse. However the large Level 6 slab remnant that had remained supported by the Drag Bars had been removed for safety reasons, as had a number of the Drag Bar outstands.

There was little or no reinforcing steel found to have connected the ends of the North Core walls on Lines D and D/E to the floor slabs. This was consistent with the apparent omission of designated reinforcing on the Drawings.

The slab failure surfaces diagram in the Site Examination and Materials Tests report therefore has been modified in this report (Figure 43) to account for observations from the photos of the condition of the structure immediately after the collapse and as found during the debris removal.

The Drag bars were all found to have maintained their connection to the walls they had been fixed to. The ends of them had bent down or they had been cut off prior to the Site Examination. The epoxied threaded anchors that had attached the Drag bars to the slabs at those levels remained upright where they occurred within the wall and were bent over approximately 30 degrees to the vertical on the bent down portion of the Drag Bars.

This indicated that the slabs at Levels 4 and 5 may not have broken away from the North Core walls due to in-plane diaphragm actions as this would have caused all the threaded anchors to have sheared off or bent over. It could be that the portion of slab immediately outside the lift well (refer Figure 43) had rotated downwards after the slab beyond that had broken away as Line 3 collapsed. For the slab failure to occur before the remnant rotated down, vertical support would still have been required from column 4-D/E. As it rotated downwards the slab appears to have pivoted about the tip of the walls, prying itself off the epoxied threaded anchors fitted into the portion of the Drag Bar adjacent to the walls (Figure 37 and Figure 115).

Figure 42 - North Core cracking (clockwise from top left) (a) No obvious cracking on Line D/E wall; (b) Horizontal flexural cracking on west and north west face of Line 5 wall and north end of Line C wall at Line C/5; (c) Fine two-way diagonal cracking on the inside faces of Level 1 to 2 Line 5; (d) and D walls in North Core.

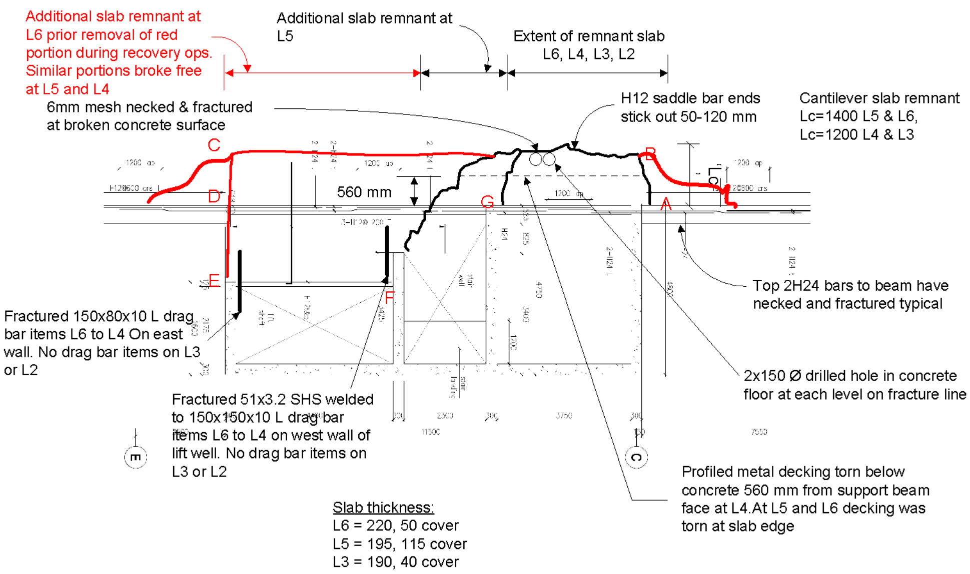

Figure 43 - North Core slab remnant profile based on the Site Examination and review of collapse photos.

South Wall

The South Wall had been cut into single storey height portions during the deconstruction process. The lower portion of the wall at Level 1 had been broken up during removal (Figure 109). There was very little cracking in the door head coupling beam of the level 1 portion of the wall. This may have been due to the influence of the masonry infill to the doorway and the depth of the coupling beam.

There was more but not extensive diagonal cracking in the Level 2 portion of the wall and diagonal cracking in the coupling beams.

The Level 3 to 4 portion of the wall was significant for one way diagonal cracking and damage at the top eastern corner (Figure 110). This corresponded to severe diagonal two way cracking through the lower portion of the eastern panel of the level 4 to 5 portion of the wall (Figure 111 and Figure 94). It was not clear what had caused this damage. The structural analyses indicated that the level of shear demand at which collapse was considered likely to have occurred, was unlikely to have been sufficient to develop that level of cracking prior to the collapse. It was therefore considered that the severe damage observed at this location in the wall may have been as a result of its fall onto the collapse debris.

The level 5 to 6 portion of the wall had a portion of very weak powdery concrete at the west top edge of the doorway. This was likely due to concrete segregation or lack of compaction below the diagonal reinforcing steel that extended through the door head coupling beam. It was not considered material to the collapse. Very little cracking was found in this portion of the wall. The construction joints at the top of this portion of wall were found to be smooth and charred. This indicated that the Level 6 slab had pulled away from the wall at the construction joint prior to the collapse of the wall. The surface had been charred by the fire that started after the collapse indicating that the slab had not been removed from this location during the recovery process.

The smooth construction joint surfaces raised concerns about whether slippage may have occurred along the construction joints in the South Wall leading to greater inter-storey drifts than were calculated by the structural analyses. This may have reduced the level of earthquake loading necessary to develop collapse critical drifts.

Other Structural Remnants

Many of the precast concrete beams had smoothly finished interface surfaces between them and the in-situ concrete. The Specification required these surfaces to be roughened.

Figure 44 - Precast shell beam (Item E14) from northern face Grid 4, west side of North Core (DENG B23 Dwg S18). (clockwise from top right) (a) to (b) Fractured slab outstand remnant at east end from which slab concrete cores were extracted. The bottom H24 bars from shell beam have been turned back into the concrete infill rather than embedded in shear wall as detailed (DENG Detail 5 Dwg S19). Notice the bar imprint on wall at the connection seen in (c) at Level 4 and at Level 3.

The lack of roughened surfaces at the beam-column joints may have reduced the beam –column joint strength and resilience.

The connection of the top of the Line 4-D/E column into the North Core appeared to have had only three rather than four H20 reinforcing bars from the column into the wall (Figure 116).

Reinforcing steel from several pre-cast shell beams on the northwest side of the North Core had not been developed into the Line C wall as detailed (Figure 44). This may have increased flexibility and reduced the resilience of the structure. However it was considered not to have initiated the collapse due to the analyses showing that the lowest levels of inter-storey drifts were expected to have occurred on Line 4.

Levels Survey

A levels survey was conducted to determine if there had been any obvious settlement of the building during the earthquake. This found no evidence that this had occurred. It appeared that the variation in floor levels was consistent with accepted construction tolerances for such work at the time.

The North Core was found to be significantly out of plumb or vertical alignment compared to the tolerances expected and allowed by the concrete construction code of practice NZS 3109, on the northeast corner and less so on the northwest corner. The lift installation company was contacted and had no record of any significant out-of-alignment on the inside face.

This raised the question as to whether the North Core had been pushed northwards during the February Aftershock or had settled due to liquefaction effects. This led to some foundation excavation being undertaken.

Foundation Excavation

Foundation excavation was undertaken to determine if any signs of liquefaction had occurred adjacent to the north side of the North Core and if any damage had occurred in the foundation beams that were to have restrained north-south movement.

Pits were dug at the west end of the North Core down to the underside of its footing. No damage was observable in the foundation nor was there any sign of liquefaction material. If any settlement had occurred prior to or during the February Aftershock this was not evident from the investigations undertaken.

The ground slab on the south side of the North Core was lifted and the foundation beams examined. This found no evidence of cracking indicating that the foundation beams had not been overstressed during the February Aftershock.

It was therefore unclear from the levels, foundation pits and examination of the foundation beams how the out of plumbness of the north face of the North Core had occurred.

Burwood Eco Landfill Columns

Core testing of two column remnants at the Site found lower than expected concrete strengths. As a consequence an additional 24 columns were tested from remnants extracted from the debris at the CTV section of the Burwood Eco Landfill (Figure 45 and Figure 46).

A number of circular columns examined showed axial/flexural hinging failures near mid-height as well as hinging at the base or head. Other circular columns were found full height with hinging damage at their heads and bases.

Rectangular columns which had all been located on Line A in the structure, typically exhibited beam-column joint failure as well as other damage.

Figure 45 - A portion of the CTV Building debris field at the Burwood Eco Landfill from which columns were extracted for examination and testing.

Figure 46 - CTV Building columns extracted for examination and testing from the Burwood debris field.

MATERIALS SAMPLING AND TESTING

Reinforcing Steel

Reinforcing steel was extracted from the South Wall remnants from Level 1 to 2, Level 3 to 4 and Level 4 to 5. The locations are shown in detail in the Site Examination and Materials Tests report.

A portion of reinforcing steel removed from the Line 1 South Wall near ground level appeared to have "work hardened" during the February Aftershock and prior to the collapse of the building due it having a higher yield stress and reduced elongation compared to the other reinforcing steel, 16 mm diameter and greater, tested which otherwise had very similar properties. This was consistent with the in-plane flexural damage that was seen in the photos taken during debris removal discussed earlier in this report (Figure 41).

A piece of 664 reinforcing steel mesh was also extracted and tested.

All the reinforcing steel tested appeared to conform to the standards of the day.

Concrete Testing and Statistical Assessment

Concrete testing and results are described in detail in the Site Examination and Materials Testing report (Hyland 2012).

All tests were done on cores without visible cracking prior to testing in accordance with normal core testing practice. However the concrete test results need to be interpreted in light of the fact that the samples were extracted from components that had been damaged in the collapse.

The sample means of the test results for a particular member were assessed against the known means of concrete properties with 28-day strengths conforming to NZS 3104:1983. A lower 0.1% acceptance limit was applied to identify upper bound conformity with a specific strength category. Where the sample size was sufficiently large an upper 0.1% rejection limit was also applied to identify non-conformity with a lower strength category. Sample mean strengths were increased by 8% where testing orientation was transverse to casting direction, in accordance with Concrete Society recommendations (GBCS 1987).

Allowance for Strength-Aging Effect of Concrete

Concrete is widely known in the construction industry to strength-age or increase in strength over time. The amount of strength-aging is dependent on the mix design, batching, placement and curing practices. There is no quantitative relationship currently known for concrete manufactured in Christchurch. However the California Department of Transportation (Caltrans) found that in California concrete with 20 to 25 MPa specified 28-day strength had at least 25% strength –aging over 20 to 30 years. Concrete batching practice typically sought to achieve target strength 20% greater than the specified 28-day cylinder compressive strength. This led to the use of a divisor of 1.5 on the strength-aged specimen test results to approximate the specified 28-day compressive strength (Priestley, Seible et al. 1996).

The statistical relationships of ready –mixed New Zealand concrete 28-day strengths at the time of the CTV Building construction have been inferred from the concrete production standard NZS 3104:1983 Table 5 (SNZ 1983). These are plotted in Figure 50 with the actual test strength distribution.

It is widely understood in the construction industry that concrete properties can be adversely affected by concrete placement techniques, and these effects have not been accounted for in these distributions of concrete properties.

The statistical properties of the same concrete strength-aged have been derived by application of a factor of 1.25 to the mean and standard deviation of the 28-day strength properties.

Wall Concrete

Cores were extracted from portions of the South Wall and the North Core.

When adjusted for being taken transverse to the casting direction the average strength of the two sets was 36.5 MPa which was greater than the specified 28-day strength of 25 MPa.

Slab Concrete

Two sets of three cores were extracted from floor slabs attached to perimeter beams found on the Madras Street site.

The mean strength of the two sets of cores was 24.6 MPa. Based on the testing undertaken and comparing the sample mean against the mean of the production specification mean set out in NZS 3104:1983 it appeared that at the time of the collapse the concrete in the slab may have met the minimum 28-day strength specified of 25 MPa.

Beam Concrete

One core was extracted from a precast interior beam from Line 2 or 3. When adjusted 8% for testing transverse to casting direction, its strength of 27.0 MPa exceeded the specified 28-day strength of 25 MPa.

Column Concrete

Column Concrete Tests on 26 columns were conducted using a combination of core testing and rebound hammer testing calibrated to the cores test results in accordance with ASTM C805. A summary of the column test results including the Rebound Hammer relationship to Cored Compressive Strength is shown in Appendix C of the Site Examination and Materials Testing report (Hyland 2012).

The total number of columns in the CTV Building was 123. This meant that 21% of the columns were tested. The columns had been selected at random from the debris pile by systematically walking over the debris field to identify column remnants which were then extracted.

Therefore a significant proportion of the columns were tested and due to their random selection, the results provide a useful statistical base for analysis of the properties.

	As-Tested	Adjusted 8% for Test Orientation
Sample Size (n)	26	26
Minimum (MPa)	16.0	17.3
Maximum (MPa)	46.6	50.3
Lower 5% (MPa)	14.2	15.3
Mean (MPa)	27.4	29.6
Upper 95% (MPa)	40.6	43.8
Coefficient of Variation (cov)	0.293	0.293
Standard Deviation (MPa)	8.04	8.68

Table 3 - Column concrete test properties statistics

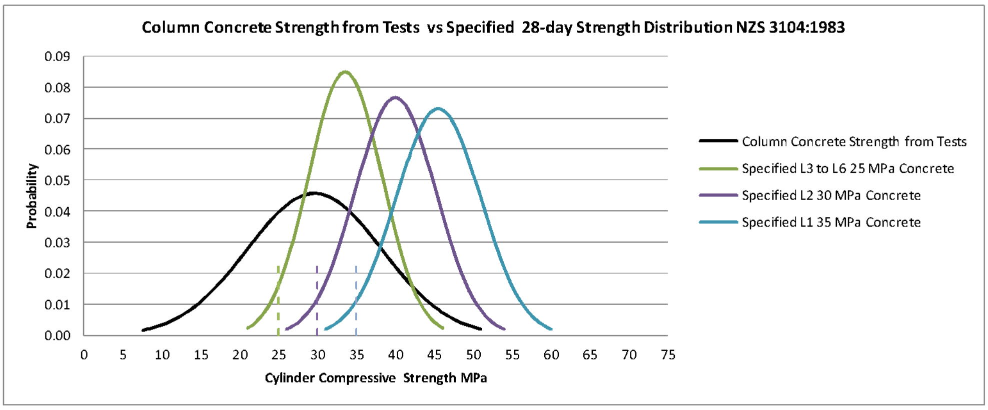

Figure 47 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS3104:1983. This indicates that the concrete in a significant proportion of the columns may have had strengths less than the minimum specified.

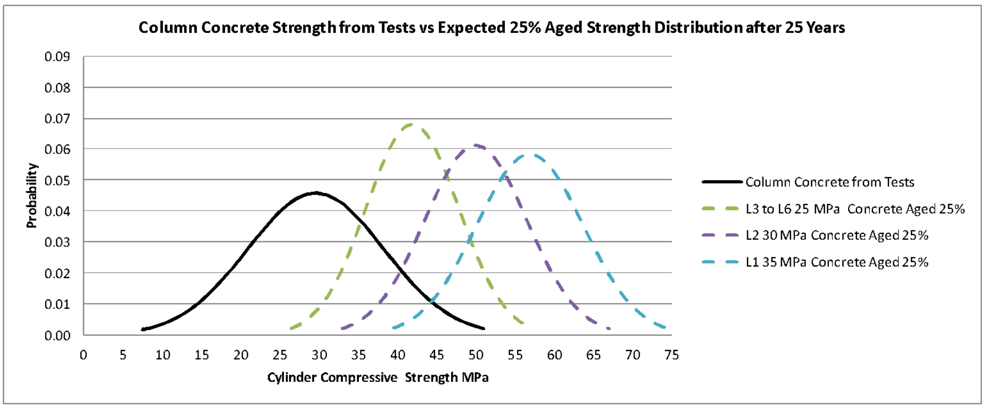

Figure 48 - Column strengths compared to expected strength distributions with 25% allowance for aging after 25 years.

The only column tested that had been visibly affected by fire was column C18 taken from 4-D/E adjacent to the North Core, still in its original position on site after the collapse. Special effort was taken to extract and trim cores to avoid fire-scorched cover concrete in test specimens from that column.

As previously stated the tests were made on members that had been damaged in the collapse. However, all tests were taken away from areas that had been obviously damaged. All cores were inspected visually before testing for cracking in accordance with the requirement for core testing of NZS 3112:1986.

Concrete test properties shown in Table 3 were adjusted by a factor of 8% to account for testing having been undertaken transverse to the direction of casting in accordance with Concrete Society guidelines (GBCS 1987).

Based on the testing undertaken, it appeared that at the time of the collapse the columns in Levels 1 to 6 had mean concrete strength equivalent to that of concrete with 28-day strength of 20 MPa. This was less than the minimum specified concrete 28-day strength of 35 MPa for columns at Level 1; 30 MPa for columns at Level 2; and 25 MPa for columns from Level 3 to Level 6. This is shown graphically in Figure 47.

Based on the testing undertaken and using a 25% allowance for strength aging it appears that the concrete in the columns in Levels 1 to 6 may not have achieved the specified 28-day strength at the time of construction.

The expected distribution of strength is shown compared to the tested distribution of strength in Figure 48.