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CTV BUILDING

CTV BUILDING COLLAPSE INVESTIGATION

FOR DEPARTMENT OF BUILDING AND HOUSING

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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 load that can be carried without failure.

Base shear – Base shear is an estimate of the expected lateral force that will occur due to seismic ground motion at the base of a structure. [The base shear is a summation of the individual shears occurring at each floor level and is determined from a number of factors including the weight of the building, the site's 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. E.g. the weak beam /strong column approach protects columns.

Catenary – A curve formed by a chain or rope 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.

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

Confined concrete – Concrete which is restrained by transverse reinforcement (i.e. reinforcement at right angles to the principal reinforcement e.g. stirrups around a column or beam's longitudinal reinforcement) from bursting outwards (like hoops on a barrel).

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

Demand – A generic term to describe structural actions caused by gravity, wind, earthquake, and snow, 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 drag.

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

Deflection – Displacement measured from an at-rest or agreed 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.

Design capacity ratios – The ratio of estimated (load) capacity to the (load) demand as used for design purposes.

Design (or response) spectra – Graphical relationship of design response 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. 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 33% and 67% 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.

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

Fixity – Measure of the amount of rotation in a structural member allowed 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 more than usual 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.

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.

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.

Moment demands – The flexural demands on a structural member.

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

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

Non-ductile – Prone to sudden or brittle failure.

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

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

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 – The peak accelerations (or displacements) with the period of vibration of structures due to an earthquake or a design earthquake.

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. Hence the design actions for an earthquake with a return period of 2,500 years is assumed to be 1.5 (or 1.8) times the corresponding values for an earthquake with a return period of 500 years.

Section capacities – The limiting (maximum) actions (bending, shear and axial load) that 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.

Shear – A force applied at right angles to a main axis of a building or 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 – Deforming under constant load. Axial action

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 Aftershock").

A number of possible collapse scenarios were identified. Examination of building remnants, eye-witness reports and a various structural analyses were used to evaluate these scenarios. These ranged from collapse initiated by column failure on the east or south face at high level to collapse initiated by failure of a more heavily loaded internal column at low level. The basic trigger in all scenarios was the failure of one or more non-ductile columns due to the horizontal movement between one floor and the next. The amount of this movement was increased by the plan irregularity of the building. Additional inter-storey movement if failure of the connection between the floor slabs and the North Core prior to the collapse occurred would 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 columns on the east or south face were weakened due to contact with the adjacent Spandrel Panels. In these circumstances it has been difficult to identify a single specific collapse scenario with total confidence.

The most studied scenario, which was consistent with eyewitness reports of an initial tilt to the east, involved initiation by failure of a column on the 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 their ability to sustain load as the building swayed. However, the displacement demands of the 22 February Aftershock were such that column failure was indicated even if there had been no contact with the Spandrel Panels. Loss of one of these columns on the east face would have caused load to shift to the adjacent interior columns. Because these were already carrying high loads at the lower levels of the building, collapse at this level would have been likely.

The low amount of confinement steel in the columns and the relatively large proportion of cover concrete gave the columns little capacity to sustain load 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 and South Wall. The South Wall and the beams and columns attached to that wall then fell northwards onto the collapsed floors and roof.

The low amount of confinement steel in the columns and the relatively large proportion of cover concrete gave the columns little capacity to sustain load once strains in the concrete reached their limit. As a result collapse was sudden and progressed rapidly to other columns. The lack of symmetry of the lateral load-resisting elements placed further demands on the critical columns by causing the building to twist and the displacements to be larger than expected.

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).

Apparently lower than minimum specified concrete strengths may have reduced the load capacities of critical columns and vertical accelerations from the ground motions may have added to the demands on them.

A number of key vulnerabilities were identified some of which affected the structural integrity and performance.

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 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 analysis to assess the demand on and capacity of critical elements.
- Synthesis of information to establish the likely sequence of and reasons for the collapse.

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.

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

The Council's building file was made available including the Drawings of the structure for which the building permit had been issued. These showed that the six level CTV Building was designed in 1986 and building permit approval was granted in September that year. Construction then commenced and continued through 1987.

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 that were installed to connect the floor slabs at Levels 4 to 6 to the lift shaft walls in the North Core in 1991.

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. They found that the geotechnical investigation carried out in 1986 was typical of the time and appropriate for the expected development.

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 has helped to give a better understanding of the condition of the structure and how it collapsed.

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 2011)

BUILDING DESCRIPTION

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").

The reinforced concrete on profiled metal deck floors were supported by reinforced concrete beams which were, in turn, supported principally by circular reinforced concrete columns.

The building was designed as a ductile reinforced concrete structure, with a lightweight roof supported on steel framing above level six. The columns and beams were not designed to be part of the horizontal seismic load-resisting system

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

STRUCTURAL MODIFICATIONS

In 1991, following an engineering review, steel angles ("the Drag Bars") 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 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 material to the collapse on 22 February 2011.

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 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 in 1991. The OIE findings 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 the perimeter columns in the upper floors
- No damage had occurred to the masonry infill wall on the west face (Line A)
- 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.

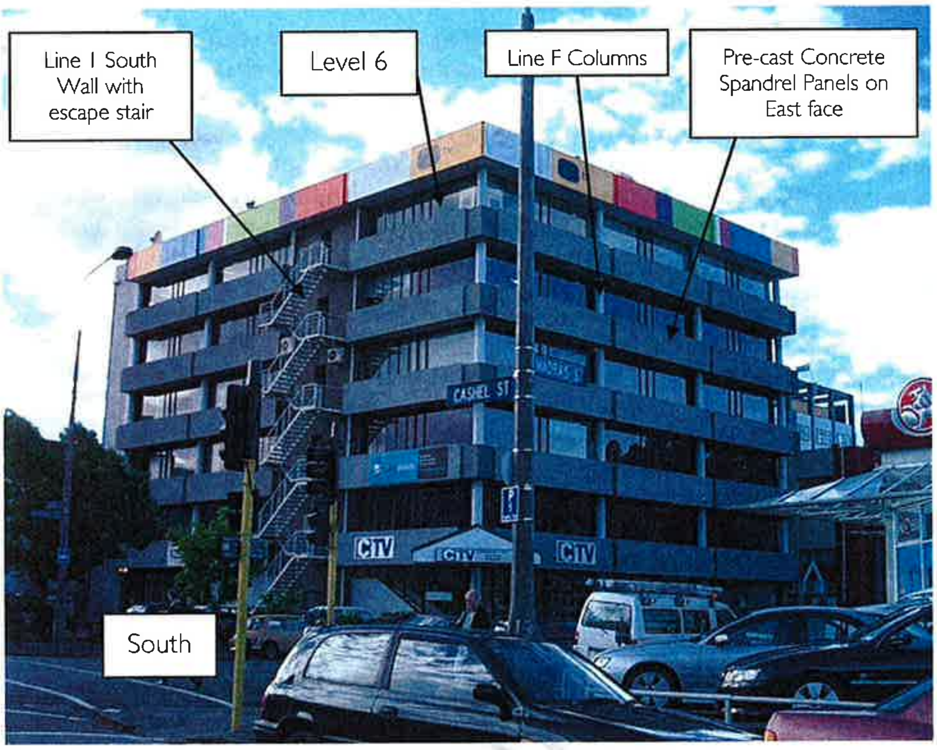

Figure 2 - Canterbury Television Building in 2004 (Photo credits: Phillip Pearson, derivative work: Schwede66) This shows some of the critical features relevant to the collapse such as the Line F columns, the pre-cast concrete Spandrel Panels, and the South Wall.

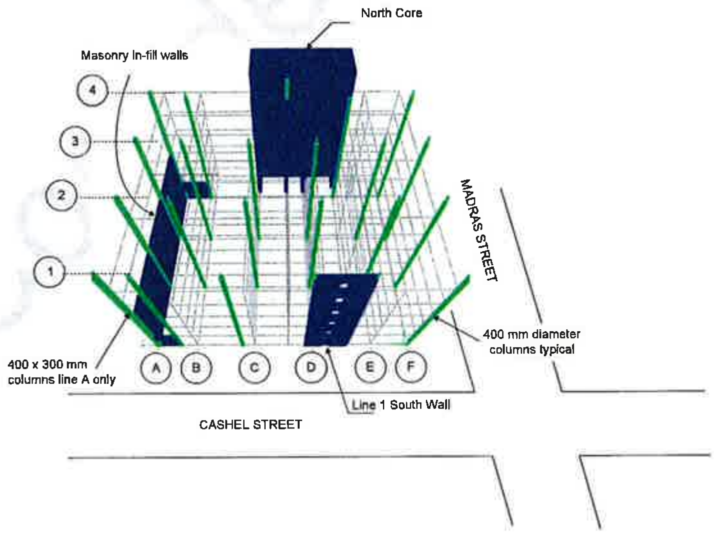

Figure 3 - Building orientation and grid lines referred to in the report. (Note that this diagram is not to scale nor is the building positioned accurately relative to the roads)

Demolition of Neighbouring Building

The building next door to the CTV building began to be demolished almost immediately after the 4 September 2010 Earthquake and continued until a week before the 22 February 2011 Aftershock. The demolition work caused significant vibrations and shuddering in the CTV Building which was a significant concern to the tenants. The view of the investigation team, based on photos of the demolition process, and consideration of the equipment and processes being used, was that the demolition would have been unlikely to have caused any 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 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. 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. 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 DURING 22 FEBRUARY 2011 AFTERSHOCK

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

Figure 4 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.

Eye-witnesses spoken to as part of the investigation saw the building sway and twist violently, the windows shatter, columns rack back and forward, and Spandrel Panels fall to the street. 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 the external structural framing collapsed on top of the floor slabs.

COLLAPSE EVALUATION

Approach and Limitations

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

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

Figure 4 - 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 4 September, 26 December and 22 February earthquakes.

Non-linear time-history analyses ("NTHA") were undertaken using actual records of the 4 September 2010 Earthquake and the 22 February 2011 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; and 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

Results of analyses were compared with the estimated capacities of these elements to assess possible collapse scenarios and to reconcile the results of the analyses with the condition of the building after the 4 September Earthquake.

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.

Overall, the approach for the analysis was to:

- Use established techniques to estimate structural properties and building responses.
- Use material properties which are in the middle of the range measured.
- Examine the effects of using ground motions (or response spectra derived from them) from several nearby recording stations.
- Apply these ground motions or response spectra in the first instance 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.

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.

GROUND SHAKING RECORDS FOR ANALYSES

The nearest strong motion recordings of the three Canterbury earthquakes of 4 September 2010, 26 December 2010 and 22 February 2011 were downloaded from the GeoNet ftp site.

The instruments were located at the following sites:

- Botanical Gardens (CBGS)
- Cathedral College (CCCC)
- Christchurch Hospital (CHHC)
- Rest Home Colombo Street North (REHS)
- Westpac Building (503A)
- Police Station (501A)

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 also no evidence of liquefaction adjacent to the CTV Building.

CRITICAL DEMAND / CAPACITY ISSUES

The following vulnerabilities were identified as potentially being of significance to the collapse:

- Non-ductile reinforcement details in the columns.
- Lack of ductile detailing in beam-column connections.
- Potential interaction between Spandrel Panels and perimeter columns.
- Lack of symmetry in plan of the designated earthquake-resisting North Core and South Wall.
- Vertical and plan irregularity due to apparent lack of separation achieved between the frame and masonry infill walls on the west face.
- Possibility of lower than specified concrete strength in critical columns.

- Limited capacity of connections between the floors and North Core.

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.

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.

COLLAPSE INITIATORS EXAMINED

Five potential collapse initiation scenarios were identified for evaluation:

1. Column failure on Line F or Line 1. This involved collapse initiation as a result of column failure on one of these lines, probably in an upper level, with or without the influence of spandrel interaction. A Line F initiation was noted as being consistent with 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 Level 1 or 2, under the combined effects of axial load (gravity and vertical earthquake) and inter-storey displacement. Low concrete strength could have helped make this scenario critical

3. Column failure due to diaphragm disconnection 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 this level.

4. Column failure due to diaphragm disconnection at Level 4 or Level 5. This scenario has similar characteristics to Scenario 3. It would require the failure of the drag bars installed and the worst effects would be at higher levels. Column axial loads would be lower than for Scenario 3.

5. Column failure due to diaphragm disconnection at Level 6. This scenario has similar characteristics to Scenario 3 and 4 and requires failure of drag bars. The worst effect would be on the columns between Level 5 and Level 6 which were the most lightly loaded.

It was not possible to estimate the displacements involved in diaphragm disconnection so that Scenarios 3 to 5 were not evaluated quantitatively. However, it is clear that significant separation at any level could easily change the location of the column that initiated failure.

Refer to Figure 3 for gridline locations and floor levels.

Figure 9 outlines the key considerations involved in evaluating these scenarios

Critical Column Identification

Drift demands were generally lower at the lower levels of the structure than at the upper levels. However the drift capacities reduced at the lower levels. Critical columns were identified by examining the ratio of demand to capacity at various levels and locations within the complete structure. This process resulted in the identification of two "indicator" columns – one in the upper levels of Line F and one at the ground floor on Level 1 on Line 2 for monitoring using the NTHA.

These columns were chosen because, if it is assumed that all other variables are equal, analyses indicated that the ratio of demand to capacity is greatest in these columns. In fact it must be recognised that the possible existence of low concrete strength, and/or greater than assumed interaction with a Spandrel Panel could mean that a column in another location could have initiated failure.

The frames on Lines 2 and F were also assessed using 2-D displacement compatibility analysis. This checked the frames as a whole using column moment - curvature derived properties and varying Spandrel Panel gaps for a displacement profile consistent with a drift of 0.75% at Level 5 along Line F. This drift level was approximately equivalent to 60% of the performance expectation of the standards of the day.

KEY DATA AND RESULTS

Elastic Response Spectra Analysis

Figure 5 shows the response spectra used in the ERSA. The graph indicates the response in terms of horizontal acceleration for varying structural natural periods of vibration. Low-rise buildings generally have low periods and tall buildings having higher periods. The fundamental vibration modes of the CTV Building corresponded to values around 1.0 second .

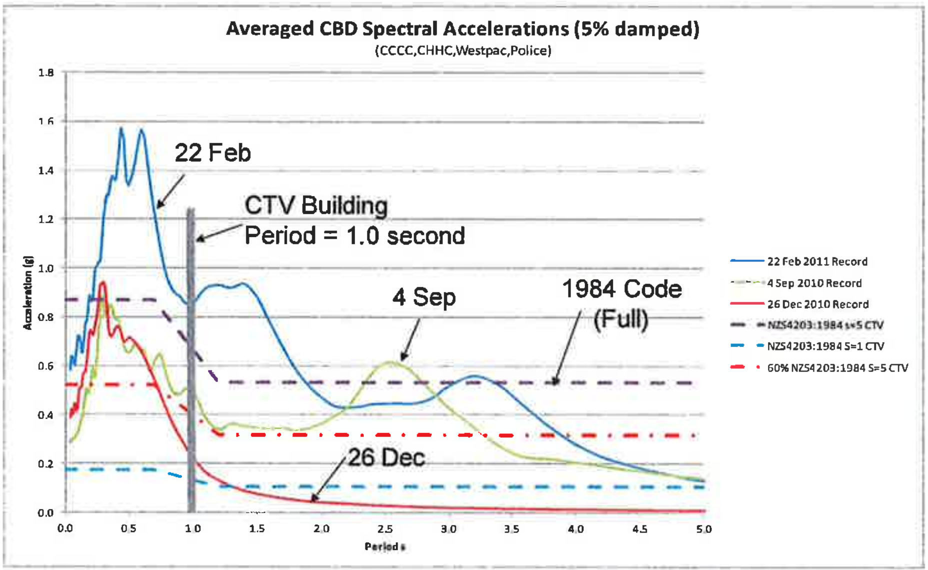

Figure 5 – Response spectra records for 4 September Earthquake, 26 December Boxing day Aftershock and the 22 February, 2011 Aftershock. Also shown (dashed lines) are the spectra for the CTV building according to NZS 4203:1984. The lower dashed line is the design spectra for ductile design that the North Core and South Wall were required to have design capacity in excess of. The upper most dashed line is 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 graphs give an indication of the relative intensities of ground shaking records on 4 September, 26 December and 22 February (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 design capacity and exhibiting structural damage such as yielding of the reinforcing steel or concrete spalling. This is because for design of members, strength reduction or safety factors are applied when using that level of loading.

It can be seen that at a period of 1.0 seconds, the demand of the February record exceeds the full response expectations of the standard. The demand of the September record was around 60 per cent of that value. At that level it was well above the level at which significant damage would have been expected in the South Wall, and the infill masonry on Line A. It also could have caused drifts along Line F sufficient to initiate collapse of those columns.

The report of the OIE after the September Earthquake reported that only minor cracking had occurred to the North Core and South Wall. The OIE believed no yielding had occurred in the reinforcing steel of those elements. No damage to the masonry infill wall on the west face was also reported by Eyewitness 16 who had been preparing the wall for recladding immediately prior to the 22 February Aftershock. He reported that no gaps were evident between the masonry and the columns. The computer analyses found that if the full September earthquake record had been applied to the models they predicted severe damage to the masonry infill wall. This indicates that the real building response to the September ground motion was less than that indicated by the use of the full record in the computer model. It also indicates that the response of the building to the February Aftershock was also less than that predicted by the computer models using the full records.

Column F2 Level 3 – Demand versus Capacity

Figure 6 and Figure 7 show output from the NTHA. The vertical axis shows the amount of inter-storey displacement (drift is the ratio of that displacement to the height between the floors) at that location. The horizontal axis is the time from start of shaking (as input into the analysis). The wavy lines plot the drift over time as the building model responds to the ground shaking record and moves, and are based on application of the full ground shaking record. This drift is a key measure of demand on the column.

The horizontal lines represent the estimated capacity of this column to sustain the drift without failing according to various criteria (assuming average concrete strength). The band between the horizontal lines reflects the difference between no contact with the Spandrel Panels (higher value) and full contact with the Spandrel Panels. In fact this band would be wider if allowance was made for the effect of variable concrete strength and vertical earthquake forces in the column. The areas where the drift has exceeded the estimated capacity are shown shaded.

The key points to note are that for the 4 September Earthquake, the maximum displacement demands are about half those calculated for the 22 February Aftershock. Although there are two places where the 4 September displacements are shaded, there are no cases where they exceed the maximum assessed capacity of the columns when no Spandrel Panel interaction occurred in the upper level columns. On the other hand, the 22 February demands have many "excursions" shown shaded and three that exceed the maximum value.

Initiation of reinforcing yielding in the F/2 column at Level 3 was calculated to occur at drifts of around 0.6%.

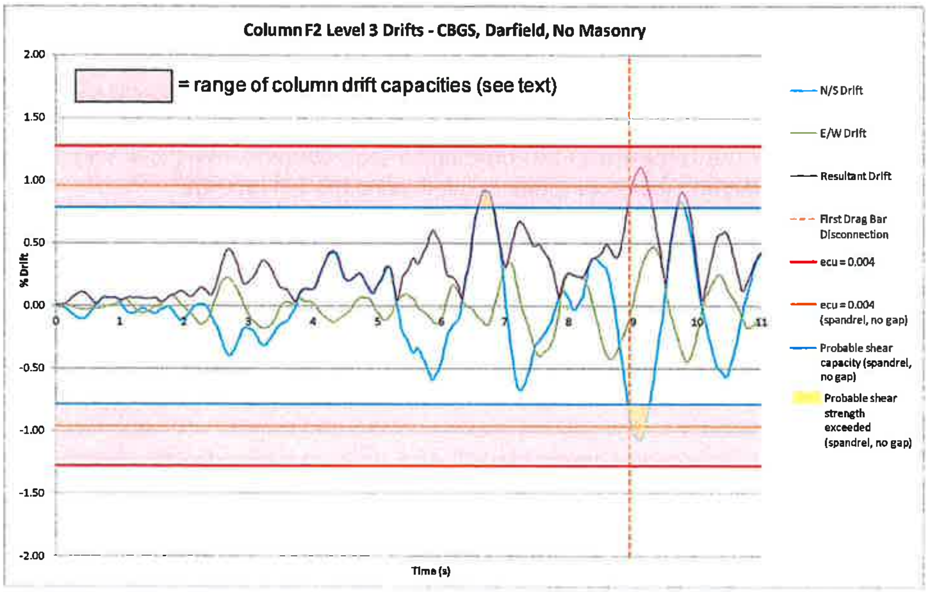

Figure 6 - NTHA drift demands and capacities plotted for column F/2 at level 3 on Line F for the September Earthquake. This indicates that drift demands were predicted by the model to have exceeded the lower bound drifts at which collapse was found to be able to initiate with interaction from Spandrel Panels. Yielding of the column reinforcing may have occurred at a drift of around 0.6%. Disconnection of the slab from the North Core may have occurred at around 1.0% drift demand.

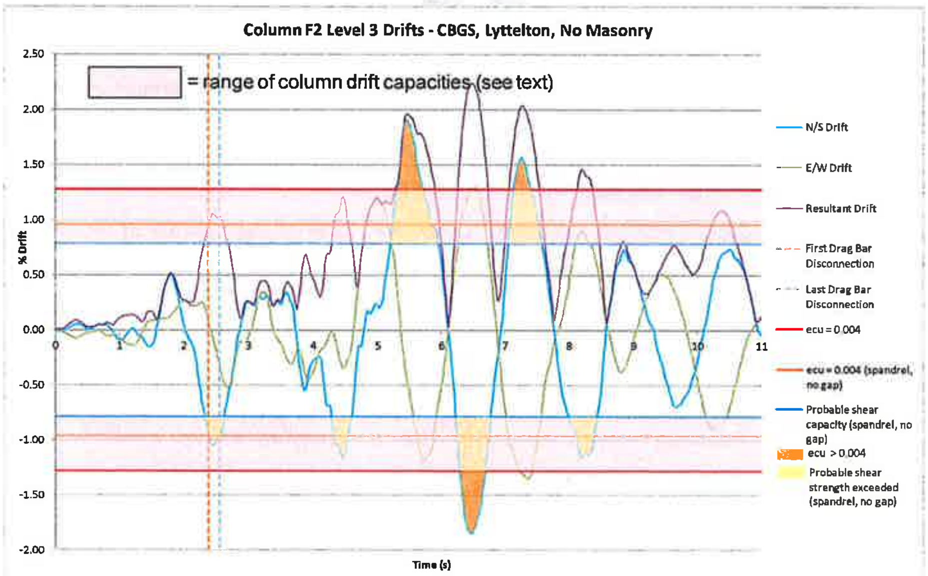

Figure 7 - NTHA drift demands and capacities plotted for column F/2 at level 3 on Line F for the 22 February Aftershock. This shows that drift demands were predicted by the model to have exceeded many times the drift range of 0.75% to 1.3%, at which collapse was found to be able to initiate. The lower bound drift being with interaction from Spandrel Panels and the upper bound without Spandrel interaction at the upper levels. Disconnection of the slab from the North Core is indicated to have occurred at around 1.0% drift demand early in the record.

Such comparisons provide valuable insights into the relativity of demand and capacity, but for reasons described above must be interpreted with care. A number of points are worth noting:

- The demands represent values derived from the full ground shaking record. If it happened that the building response was less than calculated, the plotted displacements would be less. This could be due to the CTV site not experiencing the full ground motions recorded at other nearby sites or because the response of the building was not as great as the analysis determined.

- The vertical lines indicate when the drag bar forces would reach capacity according to the analysis. Calculation of this force is subject to considerable uncertainty. In considering the implications of the plots on Figures 6 and 7 the position of the vertical lines was noted but not taken as definitive. Drag bar failure could well have occurred later than shown or not at all. Even at this level (about 1.0% drift) the 22 February displacement demands were potentially sufficient to fail the column if there was full interaction with the Spandrel Panels

These comparisons give some indication of the challenges of determining which column or mechanism initiated failure. However, the plots indicate that the demands of the 22 February Aftershock were more than enough to cause column failure, whereas the demands of 4 September were less.

Similar plots to Figure 6 and Figure 7 were made for column D2 on Level 1 (ground floor). Displacements (for the full record) were well below the assessed capacity of this column for 4 September and only marginally exceeded the capacity for the 22 February analysis. This is a broad indication that this column was less likely to have been the initiator of the collapse. However, this possibility cannot be ruled out because it may have had lower than average concrete strength and/or suffered more from the effects of the considerable vertical forces generated in the 22 February Aftershock.

Drift Demand Capacity Comparison

Table 1shows a comparison of calculated drifts and capacities for indicator Column F2 at Level 3 to 4 for the maximum drift demand for 4 September, and 22 February for the full record . Also shown are 1986 standard design limits for the CTV Building

- The "1986 Ultimate" drift is the maximum drift demand calculated for the CTV Building indicator columns by the ERSA using the elastic design spectra and standard methods applicable in 1986.

- The "1986 Dependable Strength" drift is the computed drift demand for the CTV indicator columns at the time the reinforcing steel first yields. For non-ductile detailing to be allowed in the columns, under 1986 standards, the shear walls had to be stiff enough to prevent column yield or the dependable strength being exceeded at this level of drift.

CTV Drift Comparisons for Critical Indicator Columns		
Column F2 L3-4		
Demand or Capacity	**Event / Condition**	**Column drifts (% of floor height) Column**
		F2L3
Demand	22 Feb	2.0
	26 Dec	0.5
	4 Sep	1.0
	1986 Ultimate	1.3
	1986 Dependable Strength	0.7
	2010 Ultimate	2.3
Capacity	Failure (No spandrel effect)	1.3
	Failure (Full spandrel effect)	0.8 - 1.0
	Nominal (No spandrel effect)	0.6 -1.0
	Nominal(Full spandrel effect)	0.5 - 0.8

Table 1 - Column F/2 Level 3 drift demand versus capacity using full record.

The 2010 Design Requirement is also shown to indicate the level of drift demand that current design requirements would place on the CTV Building indicator columns. As such it is a measure of the difference between 1986 design requirements and those of current standards – which now require at least limited ductile detailing for all columns irrespective of drift demand.

It is important to recognise that the expectation of design standards in construction is that even at the attainment of the maximum drift levels there should still be a low probability of collapse occurring.

LIKELY 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 total certainty. However, the results of the analysis, taken together with the examination of the building remnants, eyewitness accounts and inspection of photos taken after the collapse point to the following as being a likely collapse scenario.

It appears that collapse was initiated by the failure of one or more columns on the east or south face of the building. These columns are believed to have made contact with the precast concrete Spandrel Panels placed between them, reducing their ability to cope with building displacement. Loss of these columns immediately put large additional gravity loads on the adjacent interior columns which were most highly loaded at ground level.

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 section. . The columns thus had little capacity to sustain load and absorb 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. The beams and columns attached to the South Wall then fell northwards onto the collapsed floors and roof.

Figure 10 and Figure 11 show how the Spandrel Panels are may have affected the columns on the east and south face. Figure 12 and Figure 13 illustrate both the progression of collapse after Line F columns failure initiation and also the case of isolated failure of ground floor columns on Line 2 or 3 and the subsequent collapse of the floor slabs and frames.

Apparent lower than specified concrete strengths may have reduced the load capacities of critical columns and vertical accelerations from the ground motions may have added to the demands on them. The lack of symmetry of the lateral load-resisting elements is likely to have placed further demands on the critical columns by causing the building to twist and displacements to be larger than expected. Failure of diaphragm connections between floors and the North Core, if it occurred, may have resulted in additional displacement demands on the critical columns.

DESIGN AND CONSTRUCTION ISSUES

Structural analyses carried out as part of the investigation confirm that the North Core and the South Wall were reinforced to meet the seismic loadings (NZS 4203:1984) and reinforced concrete design requirements current in 1986 (NZS 3101:1982). However the large difference in the strength and stiffness of the walls meant that the building did not meet the aims of the standard concerning symmetry and regularity of seismic resisting structures. The recommendation of the Standard of the time was that the torsional component of shear in a critical element be not greater than 75% of the translational component of shear. The South Wall would not have conformed with that recommendation.

The investigation identified the following areas of apparent non-compliance with relevant Standards or specifications for the building structure:

- The separation gap between the Spandrel Panels and the columns was smaller than required by standards of the day and was not highlighted in the construction documents (drawings and specification) as being critical to structural performance.
- The circular columns did not have enough spiral reinforcing to meet the requirements of the concrete structures design standard of the time (NZS 3101:1982).
- Tests on 26 columns after the collapse found that the concrete in some columns was significantly weaker than expected (Figure 8). Traces of silt were found in one of the columns that had low test strength.
- 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.

- Some of the beams on the north face of the building were found not to have had their reinforcing steel connected properly into the west face of the North Core.

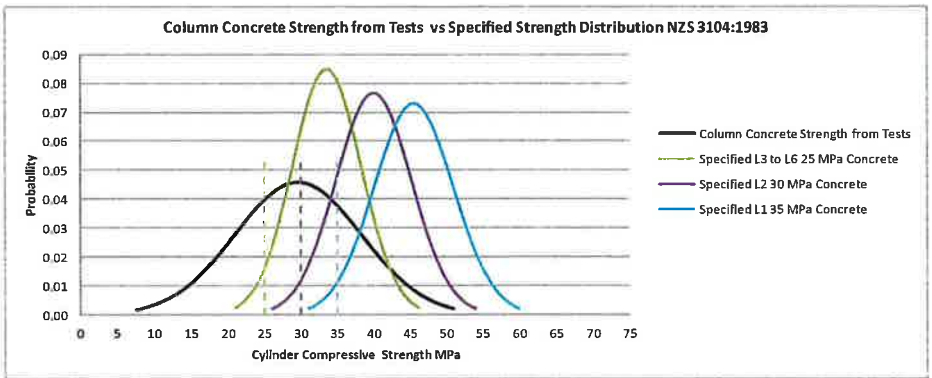

Figure 8 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS3104:1983. This indicates that the concrete in a significant proportion of the columns would have had strengths less than the minimum specified.

When compared to the current standards for new buildings (NZS 1170:5, 2004 NZS 3010: 2006) the CTV building may have only achieved 30% NBS (new building standard) based on drift capacity assessment criteria. This figure applies to the pre-September 2010 condition and is based on detailed analyses carried out as part of this investigation. This assessment is based on the principles of the New Zealand Society for Earthquake Engineering Guideline recommendations (NZSEE 2006)

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

CONCLUSIONS

The investigation found that the damage to the structure observed and/or reported after the 4 September 2010 Earthquake and the 26 December 2010 Boxing Day aftershock did not indicate significant weakening of the structure.

The estimated response of the building using the full 4 September Earthquake ground shaking records and the assessed effects on critical elements don't appear to be fully consistent with observations following the 4 September Earthquake. Analyses using the 22 February Aftershock ground motion records indicate displacement demands on critical elements to be well in excess of their capacities. It was also found however that significantly less than the full record was required to develop critical collapse initiation conditions along Line F, particularly if Spandrel Panels prevented free movement of the columns.

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

- The strength of shaking indicated by the February Aftershock ground motion records and spectra were easily sufficient to cause displacements which were higher than anticipated based on the computer analyses. However the computer models also indicated that there was sufficient shaking to have severely damaged and possibly collapsed the building in the September Earthquake.

- The vertical irregularity produced by the influence of the masonry walls on the west face up to Level 3.
- The plan irregularity of the earthquake-resisting elements which further increased the inter-storey drifts on the east and south faces.
- The apparent lack of sufficient separations between the perimeter columns and the Spandrel Panels reduced the capacity of the columns to sustain the lateral building displacements.
- The low amounts of spiral reinforcing in the columns.
- Low concrete strength likely in some of the critical columns.
- The effects of vertical earthquake accelerations.

Surveys of the site after the collapse found no evidence of 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:

1. Geometrically irregular structures that depend on a primary structure may not perform as well as structural analyses indicate.
2. Buildings designed before 1995 with non-ductile columns may be unacceptably vulnerable. They should be checked and a retrospective retrofit programme considered.
3. Existing buildings with part-height pre-cast concrete panels (or similar elements) between columns may be at risk if separation gaps are not sufficient. Such buildings should be identified and remedial action taken.
4. Buildings with connections between floor slabs and shear walls 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.
5. There is a need for improved confidence in construction quality. Measures need to be implemented which achieve this. 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.

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.

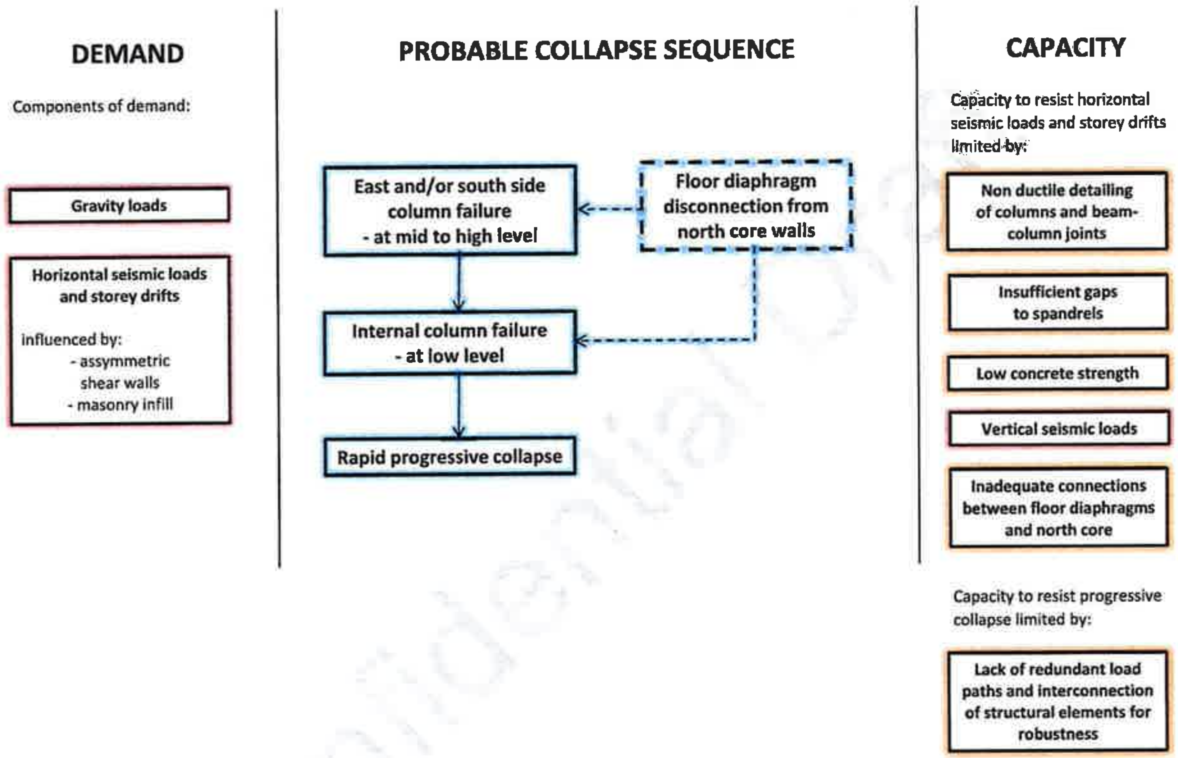

Figure 9 Collapse sequence flowchart

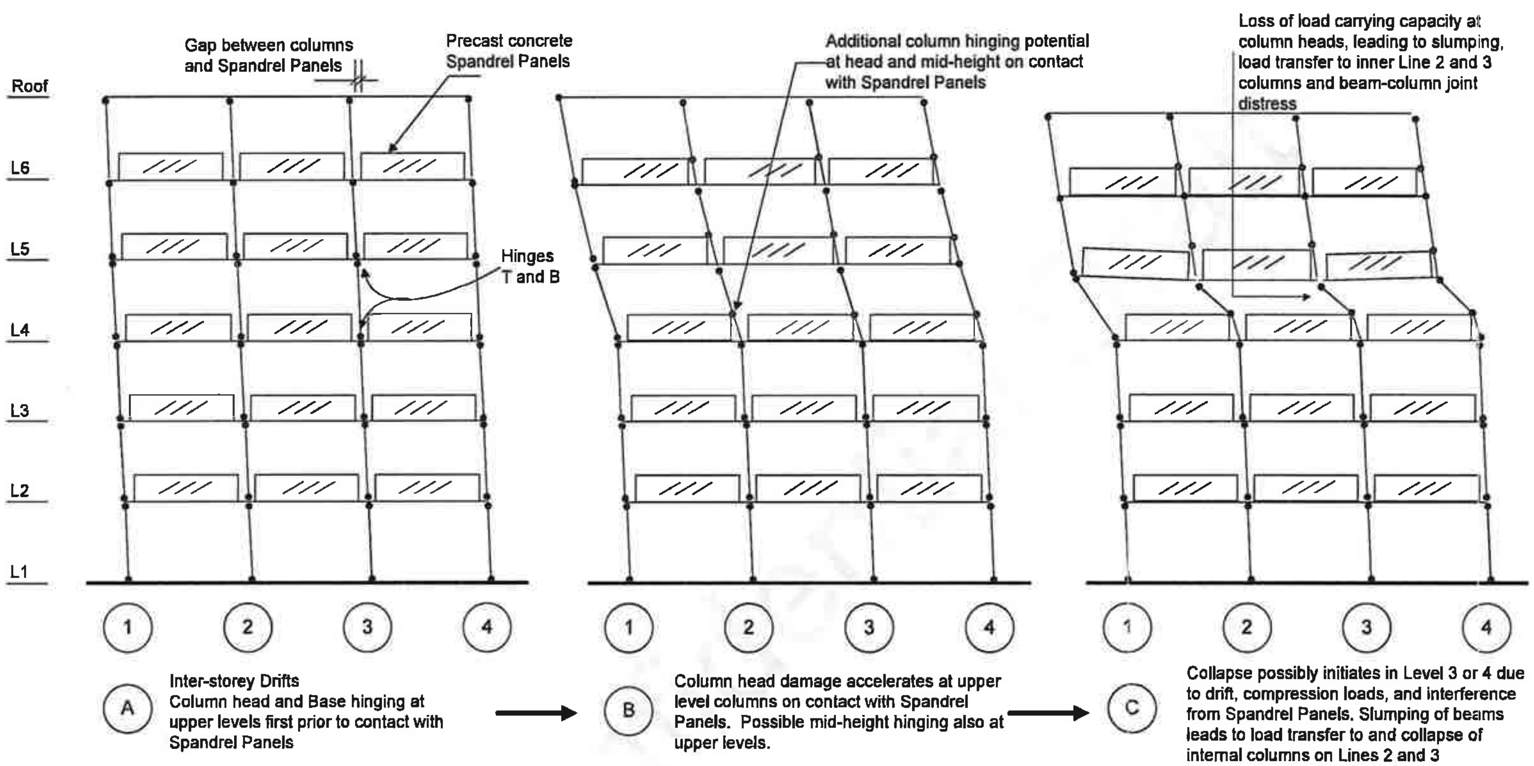

Figure 10 - Possible collapse sequence along Line F as Inter-storey drifts reach critical levels and columns begin to fail from lack of displacement capability or from additional damage caused through contact with precast concrete Spandrel Panels. Displacements and damage are greatest in the upper levels, but inelastic drift capacity less in the lower levels. Also reduction 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.

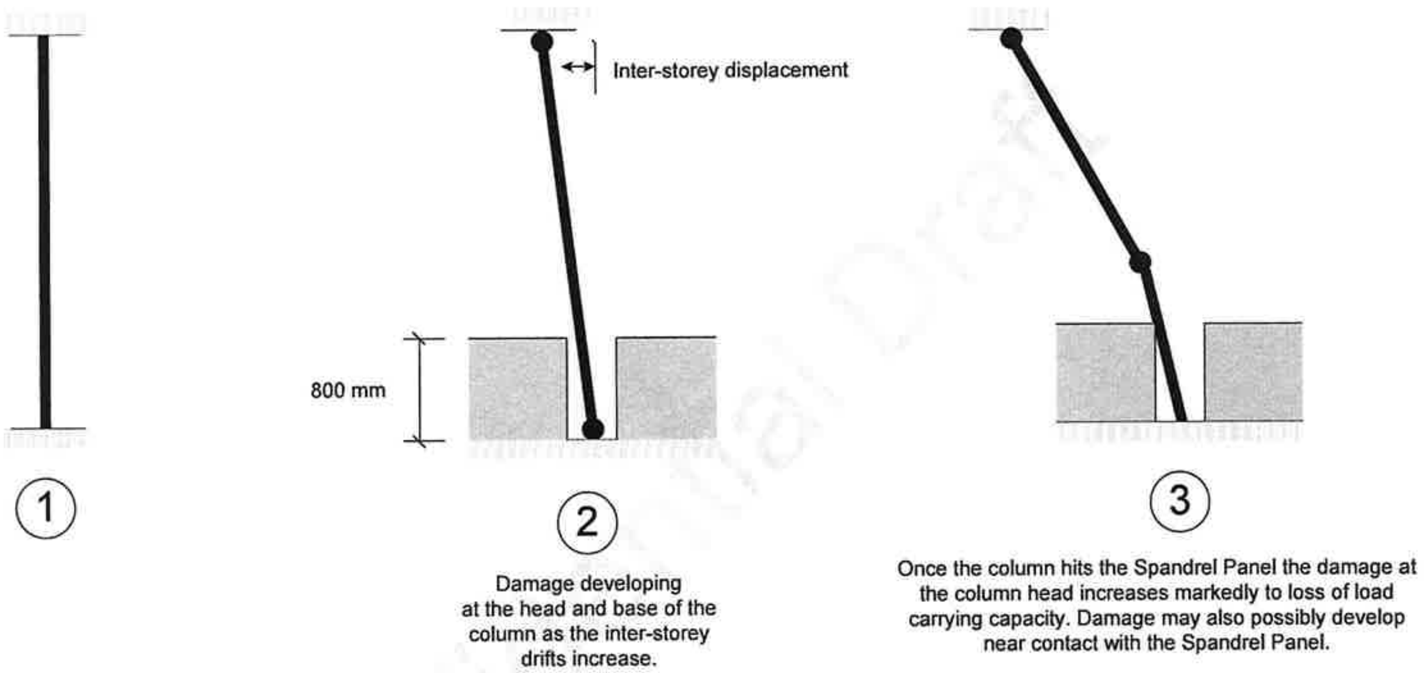

Figure 11 - Development of flexural/compressive column head damage in Line F columns and the damage acceleration effects from interference by the Spandrel Panels.

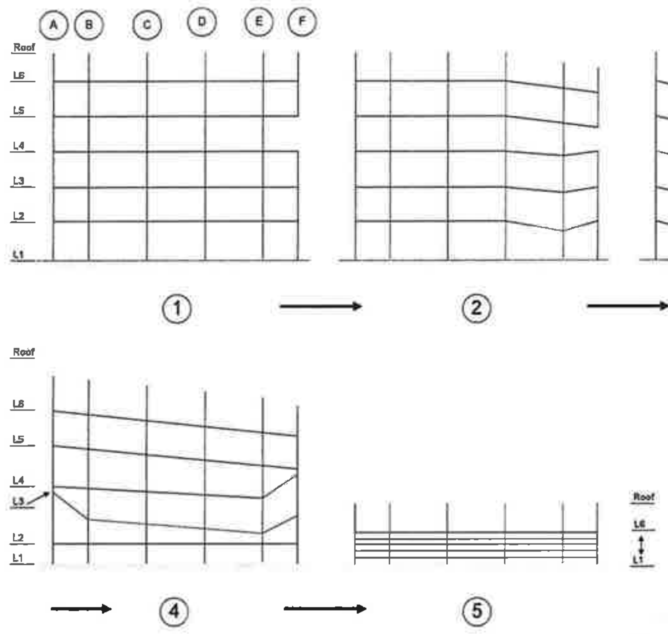

Figure 12 - Possible progression of collapse from loss of column capacity on Line F is shown sequentially as follows: (1) Collapse of Line F columns above Level 4 leads to extra floor area being supported off columns on Line E; (2) The Line E columns begin to collapse under the extra load; (3) As the 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 12.

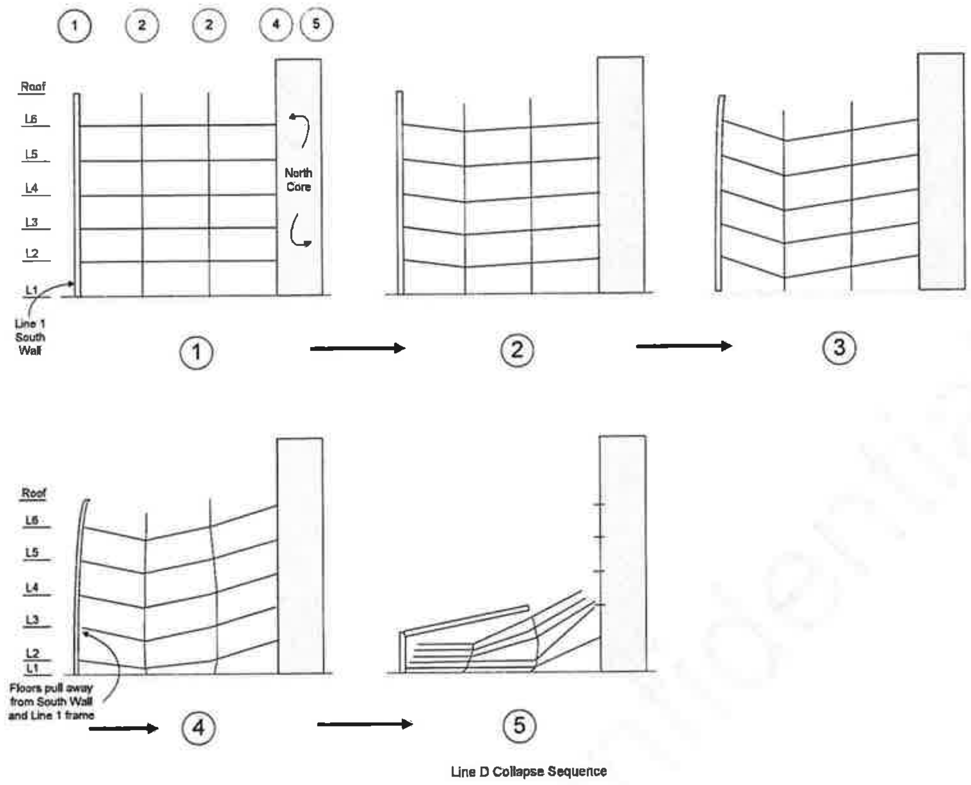

Figure 13 - Possible progression of the collapse on a north-south section through the building simultaneous with that shown progressing westwards in Figure 12 (1) Initial condition ; (2) Line 2 begins to subside; (3) As Line 2 subsides further the stiffer and stronger North Core pulls the collapsing 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 work ("the Drawings").

- A structural review report dated January, 1990, undertaken after construction of the CTV Building for a prospective purchaser ('the 1990 Review Report").

- The Specification dated 30 September, 1986

- The GNS records on the ground shaking near the CTV Building at the time of its collapse ("the GNS Records").

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 following the collapse 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 at Level 6 and the Drag Bars at Levels 4, 5 and 6 of the north shear 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 2011).

STRUCTURAL ANALYSES

Three dimensional ERSA were undertaken. The basis of and the results of ERSA are reported in the appendices.

Three dimensional NTHA were also undertaken by SSL in conjunction with Compusoft Engineering Ltd ("CSE"). The basis and summary of the results of 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 laid after the collapse, laboratory examination and mechanical testing of materials and components salvaged from the site, identification of the collapse sequence, computer based 3D structural analysis of the structure under earthquake loadings and structural calculations.

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 also housed an education facility at Level 4 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 shallow strip and pad footings and foundation beams.

The primary earthquake resisting structure consisted of fully 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. 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. The lower doorway opening had been partially in-filled with reinforced masonry to window sill height.

The secondary structure was not considered by the design standards at the time to contribute directly to the 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 for which elastic behaviour had been assumed for design loading derived from imposed deformations $v\Delta$ 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 precast Spandrel Panels appear 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 features of the structure that were considered by the authors to be relevant for the seismic analysis included:

- The asymmetrical layout of the bracing walls, with the walls at the north side 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 and south shear walls including consideration of the lift voids in the north side shear wall core.
- Lack of connection of the floor diaphragm to walls 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 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 engagement of the in-fill masonry wall and the main structural frame on Grid A.
- The interaction of the pre-cast concrete Spandrel Panels that contained the perimeter columns on the south, east and north faces of the building. No specific seismic separation gap was specified. Assessment of the combined specified construction tolerances showed that a number of the panels would have reasonably been expected to have been in or near contact with the columns.

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

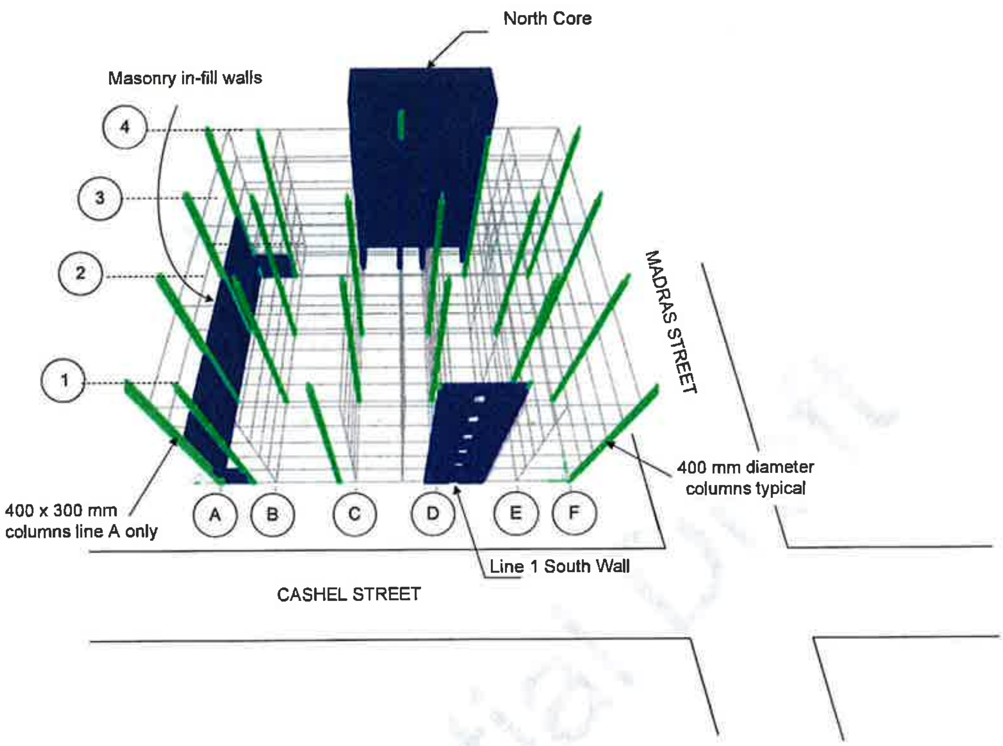

Figure 14 - Building orientation and grid lines used in the report. (Note that this diagram is not to scale nor is the building positioned accurately relative to the roads)

PROCUREMENT PROCESS

The developer gained building permit approval in September 1986.

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 and found to be consistent with normal practice in Christchurch at the time (Sinclair 2011).

It recommended soil spring stiffness values 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 liquefaction on the west side of the adjacent empty site. No liquefaction was observed immediately adjacent to the CTV Building itself (Figure 60) 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 2011).

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.

The structural engineering consultant who undertook the design also supplied a set of drawings, calculations and the structural specification for the building.

The lack of a complete record of the design and subsequent remedial work at the Council made it difficult to quickly assess the building.

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 not considered by the authors to be a significant structural change with respect to the seismic response of the building. It would not have affected the seismic response as 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 specified (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 Aftershock. Review of the design calculations indicates that it was intended to have only a partially filled top course.

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 the same workmen that the joints were filled with the mortar on the outer face.

REMEDIAL WORK AFTER CONSTRUCTION

Consulting Engineer B undertook a review for a prospective purchaser of the building in 1990. Their report showed that they had concerns about how the floor slab diaphragm was attached to the elevator core walls. Their client did not buy the building.

The Site Examination found some structural steel angle drag members bolted into the wall fins 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 these to have been designed and installed in October 1991. No record of these alterations was found on the Council property file.

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 there were the following tenancies within the building at the time of the February 22 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 Aftershock. Some fit out work was reported to be in progress at the time of the February earthquake.
- 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 the above and with floor plans that had been sketched by USAR engineers at the scene of the collapse, based on their interviews with tenants.

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.

One bay of the concrete masonry wall at Level 1 adjacent to the North Core was 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 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 15 to Figure 19), 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 construction joints in the shear walls surrounding the bathrooms and stairway at the north side.
- 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 15).
- Circumferential cracking up the height of the south west corner column at level 6 (Figure 15).
- 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 16).

- 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 16).
- 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 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 21. This is likely to have added to a sense of unease with the building by tenants.

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

Figure 15 - Level 6 400mm diameter columns (Left to right) a) Column 4 D/E outside lift; b) Column on Line 1/A-B with hairline horizontal cracking.

Figure 16 - (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 17 - Damage to wall Linings after 4 September 2010 Earthquake.

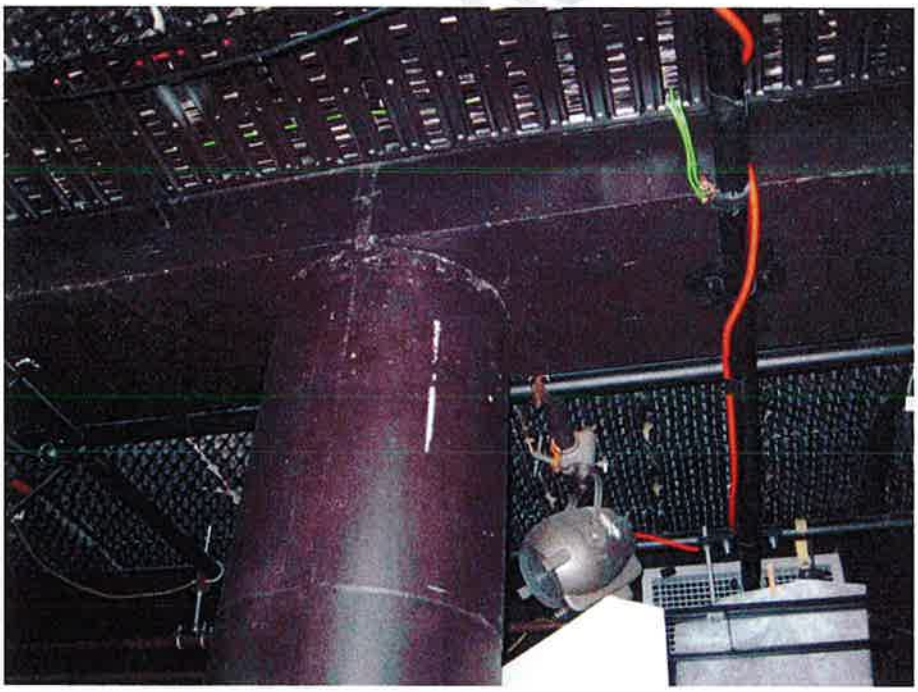

Figure 18 - Level 1 Line 2 or 3 400mm diameter column and beam after 4 September 2010 Earthquake. No visible cracking evident. A horizontal circumferential formwork mark can be seen approximately 600 mm down from the underside of the beam.

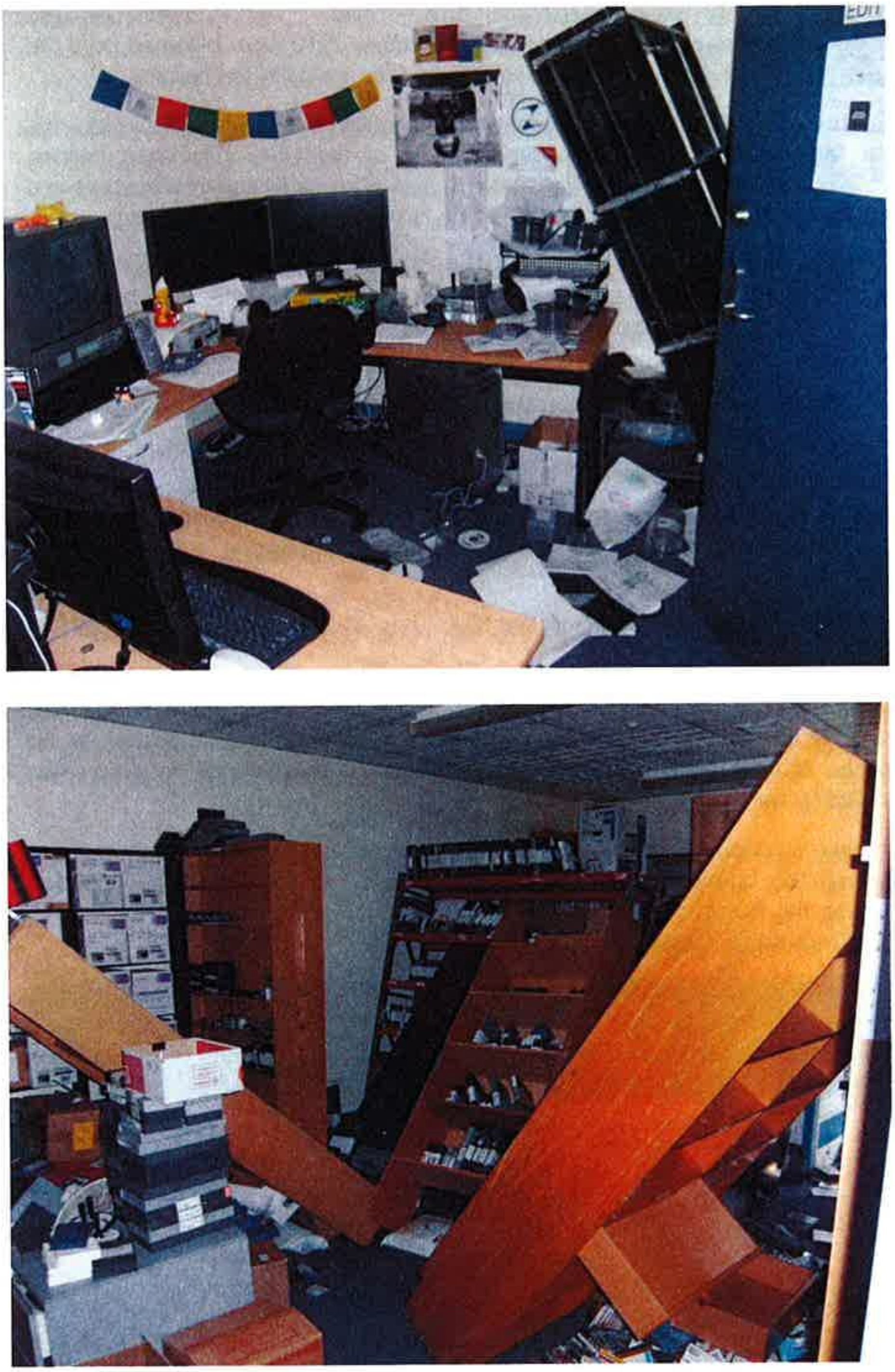

Figure 19 - Damage to office furniture on Level 2 after the 4 September 2010 Earthquake.

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 might fall 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 Boxing Day 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 20).

No obvious 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 Earthquake in September.

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 22 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 26 December aftershock compared to those from the 4 September Earthquake, that there was no evidence of significant structural damage to the building after the 26th December aftershock.

Figure 20 - 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 though not attached to the wall (Line 3/B-C); b) As it was 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 4 September, 2010 Earthquake.

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

Heavy machinery with pneumatic pokers and pincers, and drop hammers, were used to break it up (Figure 21). This caused ongoing 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 21 and not find it to cause any significant structural damage to adjacent buildings.

Figure 21 - 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

Debris began to be moved very shortly after the collapse by heavy machinery that was in the neighbourhood at the time. A fire started shortly after the collapse near the North Core.

The west face along Line A of the building can be seen in Figure 22. No liquefaction is evident on the vacant site between The CTV Building and on the vacant 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, possibly as a consequence of the South Wall falling northwards onto them. Large diagonal cracks can be seen in the Level 2 to 3 masonry infill wall at the south end that had fallen onto the vacant site.

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 24. The unpainted portions of the columns seen in Figure 25 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 10 and Figure 11. The eastwards throw was consistent with column failure initiating along Line F leading to a slumping and eastward tilt of the levels above as shown in Figure 12.

Along the south face on Cashel Street Cashel St (Line 1) the North Core tower stood out clearly as seen in Figure 23. 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 almost horizontal on top of the collapse building. 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.

It appeared that the column attached to the east side of the North Core had collapsed along with the internal columns, pulling the floor slabs away from the North Core and South Wall. The South Wall had then fallen northwards onto the debris.

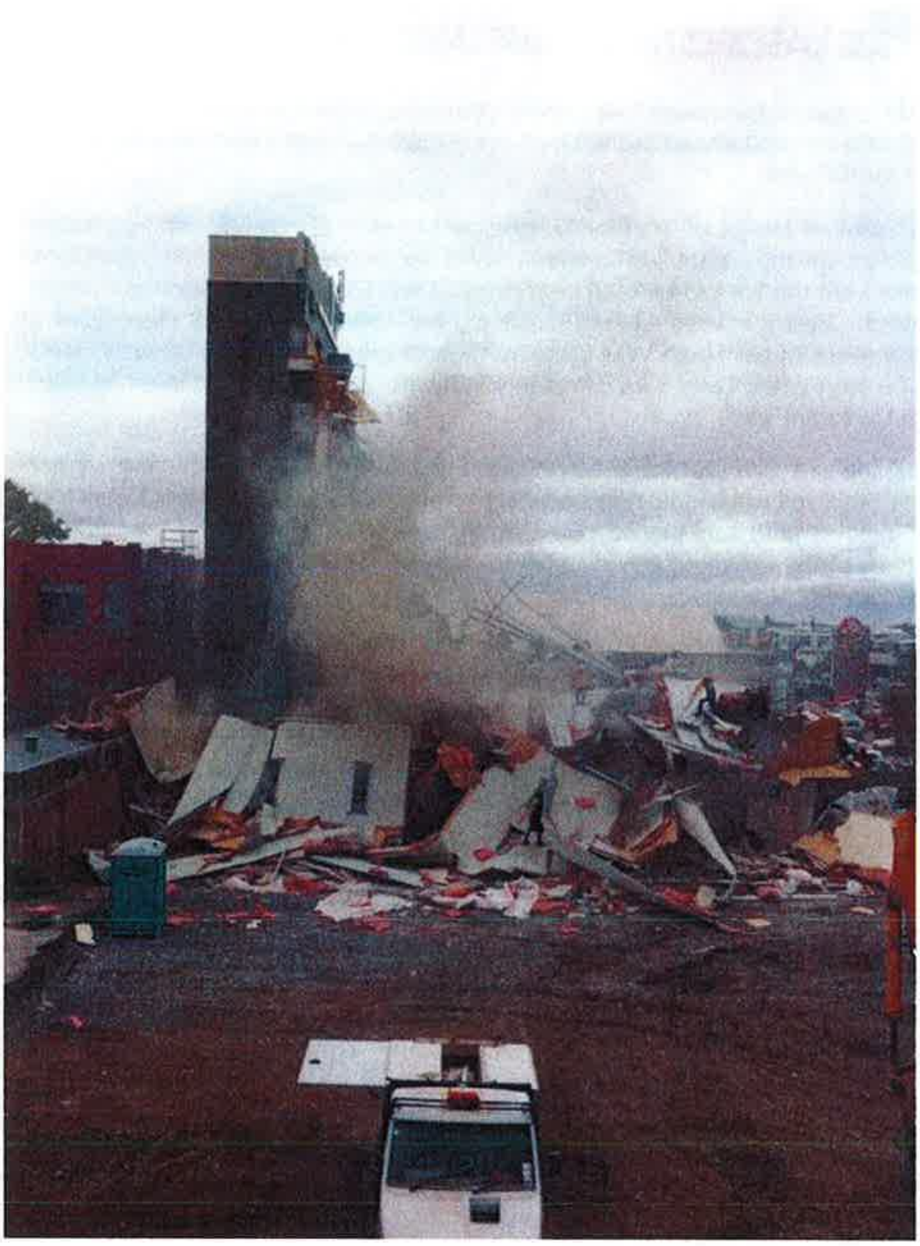

Figure 22 - View of all of the west wall on Line A from Les Mills immediately after collapse before debris removal commenced. No signs of liquefaction can be seen on the vacant site. Smoke from the fire can be seen beginning to arise. Some of the upper light weight external panels between Levels 4 and 6 have fallen northwards, possibly as a consequence of the South Wall falling northwards onto 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 23 - 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 lays almost horizontal on top of the collapse building.

Figure 24 - Corner of Cashel and Madras Streets looking towards North Core. Fractured columns and fallen Spandrel Panels are visible (MSN photo).

Figure 25 – View from 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 26 - North Core with Level 4 and 3 slabs laying diagonally against it.

6 EYEWITNESS ACCOUNTS

INTRODUCTION

Eyewitnesses were formally interviewed and interviews recorded and 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 structure of the building. These have informed the interpretations of the analyses.

Specific observations included:

Feeling vertical jolts or accelerations.

A tilt to the east of the top portion of the building.

East-west movement or twisting.

The upper levels 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.

<u>The subjectivity of time</u>

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).

<u>The subjectivity of sensation.</u>

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 falling from a cliff – 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, and why.

<u>The subjectivity of words.</u>

We 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 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. <u>The building collapsed in on itself.</u>

- "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. <u>A sense of specific levels giving way – then falling straight down.</u>

- "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. <u>Down in a matter of seconds</u>

The majority felt it went in 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 Madras Street site including the North Core, and columns extracted from the CTV area at the Burwood Eco Landfill. Photos 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 ascertain collapse sequence and behaviour. Appendix B has more photos.

IMMEDIATE POST-COLLAPSE CONDITION PHOTOS

Observations

Observations of the immediate collapse debris have been made in a Chapter 5. The following conclusions have been drawn from those observations.

Conclusions

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 had collapsed along with the internal columns, pulling the floor slabs away from the North Core and South Wall. The South Wall had then fallen onto the debris.

- The Level 2 to 3 masonry infill wall on Line A had been severely 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 had been thrown northwards on that side, possibly as a result of the South Wall collapsing onto the fallen floors and roof.
- No liquefaction had occurred on the west, south or east faces of the building.
- The collapse had been confined within the north side of the south face.
- Column head fractures were evident on Line F columns.
- 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 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 12.
- The column head fractures were consistent with the flexural/compressive failure scenario shown in Figure 11.

DEBRIS REMOVAL PHOTOS

Introduction

The debris from the collapse was 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 76). All the floor slabs could be seen laying on top of each other adjacent to the South Wall (Figure 79). 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 28 and Figure 80).

This showed that the floors had all broken away from the Line 1 frame and South Wall close to Line 1 and had fallen to the ground prior to the South Wall and Line 1 frames falling onto the collapsed structure. 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 27 - Concrete Spandrel Panels, perimeter beams and columns on Cashel Street face (Line 1 / B-D).

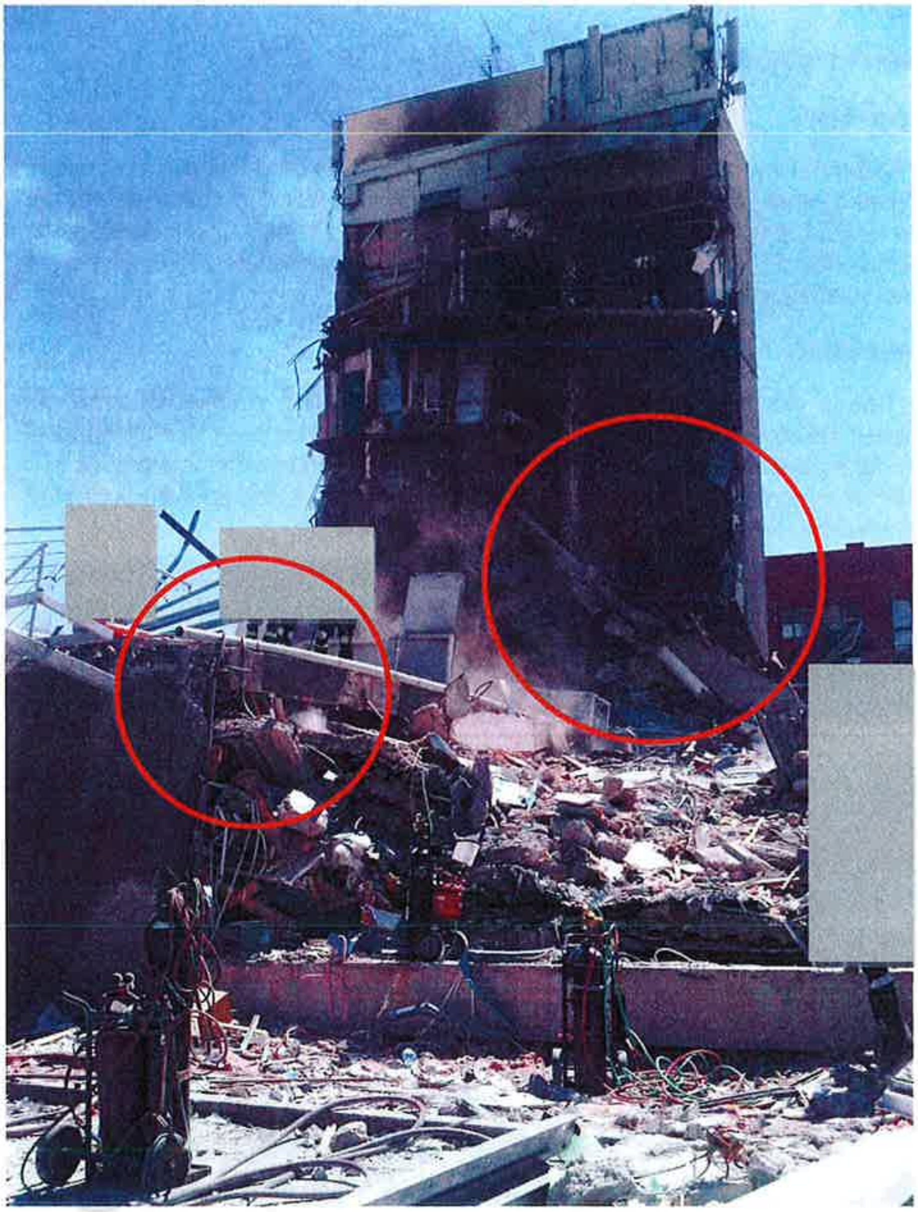

Figure 28 - View from Cashel Street east side of Line I with Line I South Wall lying on debris at left; trapezoidal end profile of floor slabs laying on top of each other in foreground; A portion of floor slab highlighted, appears to be still in contact with the South Wall at Level 2 and may have prevented 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 to be laying diagonally against the North Core (Figure 30). 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 28 and

Figure 29). This seems to have been as a consequence of the collapse of the Line 3 columns and floors as the failure of the Level 6 slab could not have occurred without vertical support from the Line 4-D/E column being available. Otherwise the slab should have rotated about the tips of the Line D and D/E walls.

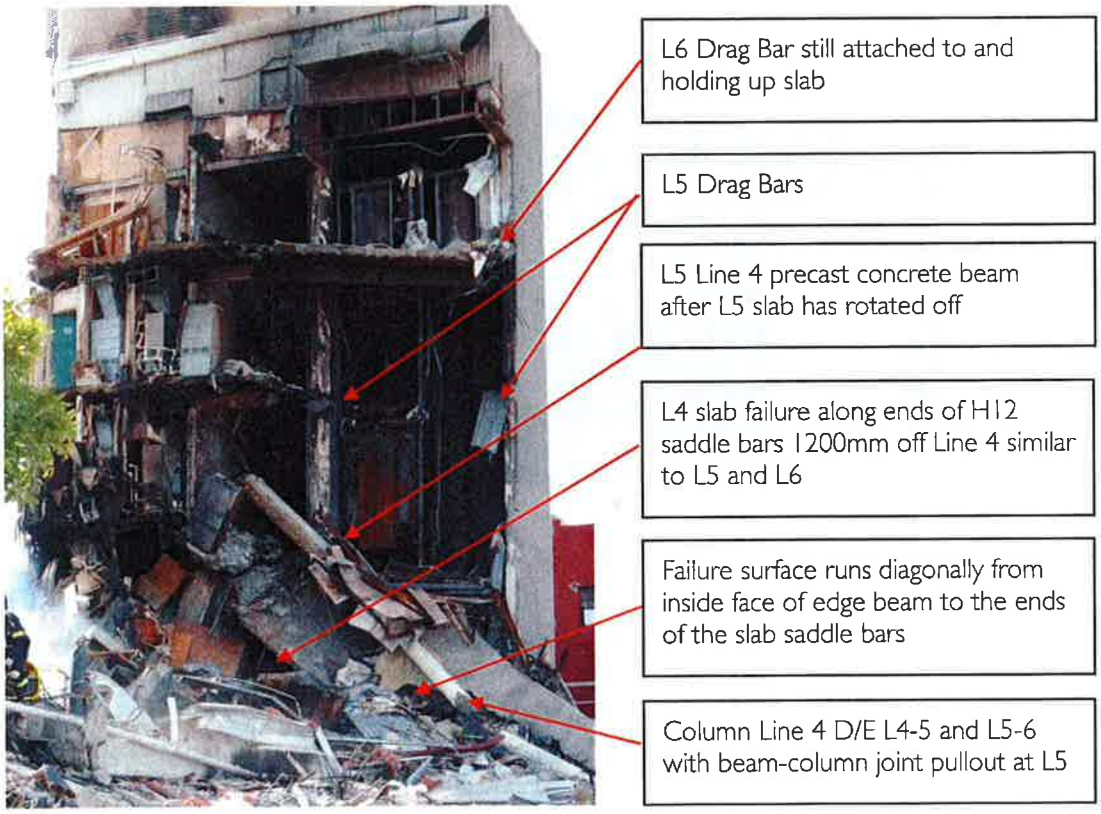

Figure 29 - Failure of slab adjacent to North Core.

Figure 30 – 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 31) and the Line 3 beams laying southwards (Figure 88). This indicated that collapse of the columns on Lines 2 and 3 had occurred before the slabs detached from the North Core and South Wall.

Figure 31 - Line 2 beams highlighted laying rotated northwards. This indicates 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 indicative of Spandrel Panel interference (Figure 32). Column head damage is visible, though little flexural damage could be seen at the base.

In this case it is thought 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. the lack of flexural damage at the base may be indicative of the Spandrel Panel having been close to full contact with the column prior to the Aftershock. Based on the displacement compatibility and push-over analyses this may have prevented base hinging occurring in what is likely to have been an upper level column.

Figure 32 - Line 4 / B column with precast log beam in foreground and shell beam at rear. The column appears to have broken its back on a precast concrete Spandrel Panel and sustained head damage during the collapse.

The South Wall showed fan-like cracking that extended diagonally from the middle of the wall to the outside edges. The cracking also went through the full thickness of the wall. This indicated that the wall had suffered flexural /tensile damage prior to the collapse of the floor slabs (Figure 33).

The east end of the wall had suffered concrete spalling on the outer and inner faces. The spalling damage indicated concrete compressive strains of 0.004 or more had occurred due to in-plane flexural/compressive demands prior to the collapse. The fact that the spalling occurred on the outer south face of the wall indicated that the damage had not occurred due to the wall falling northwards after the collapse of the floors.

Figure 33 - Fan-like flexural cracking on the South Wall in conjunction with compressive spalling of the concrete at the wall east end. This indicates that the South Wall suffered flexural damage prior to the collapse of the floor slabs.

Conclusions

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

- The floor slabs collapsed to the ground prior to the South Wall falling onto the debris.
- 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 northwards and the Line 3 beams lie southwards. The level 3 and 4 slabs at the North Core were found to be lying down diagonally from the North Core. This indicated that the slabs at these levels had broken away from the North Core and South Wall after collapse had 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 following collapse of the Line 3 columns.
- 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

indicated that the Spandrel Panel may have been in contact with the column prior to the 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 2011). 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 from a fire service snorkel. Measurements of the slab and Drag Bar remnants were made as they 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 34). Evidence of fire charring could be seen on the inner surfaces.

The slabs outstands on the west side of the North Core were in the condition seen in the photos immediately after collapse. However the Level 6 slab that had been suspended 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 walls on Lines D and D/E to the North Core. This was consistent with the apparent omission of that 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 35) 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 had been cut of 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 had not 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 therefore appeared that the portion of slab immediately outside the lift well (refer Figure 35) had rotated downwards after the slab beyond that had broken away following collapse of Line 3. As it rotated downwards the slab pivoted about the Drag Bar fixing at the tip of the walls, prying itself off the epoxied threaded anchors fitted into the Drag Bar adjacent to the fixings into the walls (Figure 29 and Figure 104).

Figure 34 - 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.

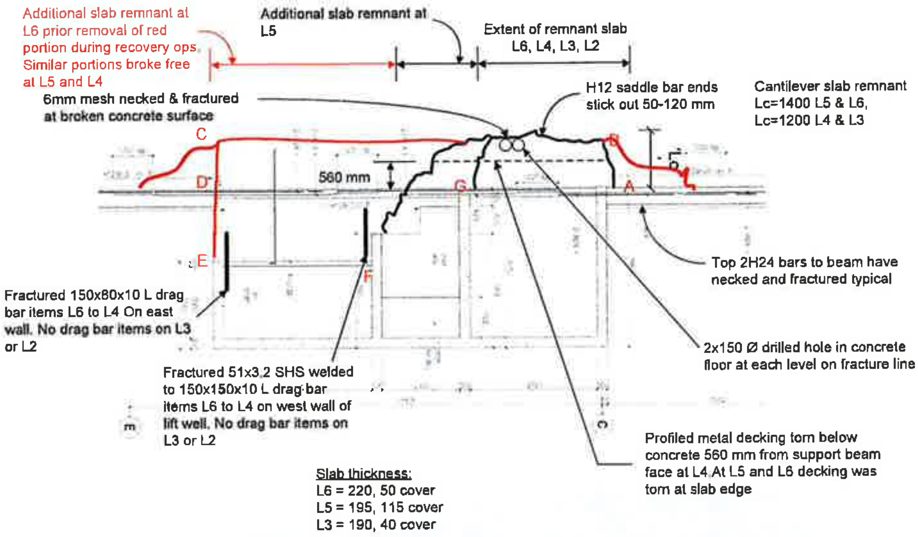

Figure 35 - North Core slab remnant profile based on the Site Examination and review of collapse photos.

South Wall

The South Wall had been cut into single storey height portions during the recovery process. The lower portion of the wall at Level 1 had been broken up during removal (Figure 98). 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 were more but not extensive diagonal cracking in the Level 2 portion of the wall and diagonal cracking in the panels.

The Level 3 to 4 portion of the wall was significant for one way diagonal cracking and damage at the top eastern corner (Figure 99). 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 100 and Figure 83). It was not clear what had caused this damage. However the structural analyses indicated that the level of shear demand in the east panel consistent with 0.75 to 1.3% drifts along Line F, 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 concluded that the damage observed at this location in the wall had been as a result of its fall onto the collapsed building.

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 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 surface raised concerns about whether slippage had 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 along Line I and F.

Other Structural Remnants

Many of the precast concrete beams had smoothly finished interface surfaces between them and the insitu concrete. The drawings were not consistent in showing where these surfaces should be roughened. However normal construction practice was to roughen these to ensure effective interlock between the precast and insitu concrete.

Figure 36 - 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 specified (DENG Detail 5 Dwg S19). Notice the bar imprint on wall at the connection seen in (c) at Level 4 and at Level 3.. This meant that these beams would not have performed as intended.

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

The connection of the Line 4-D/E column into the North Core was found to have only have evidence that three rather than four reinforcing bars had been connected into it from the column (Figure 105). This would have reduced its ability to hold the column up during the collapse, but is not thought to have initiated the collapse.

Reinforcing steel from several pre-cast shell beams on the northwest side of the North Core had not developed into the Line C wall as specified (Figure 36). This would 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 allowed by the concrete construction code of practice NSS 3109, on the northeast corner and less so on the northwest corner.

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

Foundation Excavation

The 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.

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

It was therefore concluded from the levels, foundation pits and examination of the foundation beams that out of plumbness of the north face of the North Core may have occurred during construction. If any settlement had occurred prior to or during the Aftershock this was not evident from the investigations undertaken.

Burwood Eco Landfill Columns

Core testing of two column remnants at Madras Street 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 37).

A number of circular columns examined showed 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.

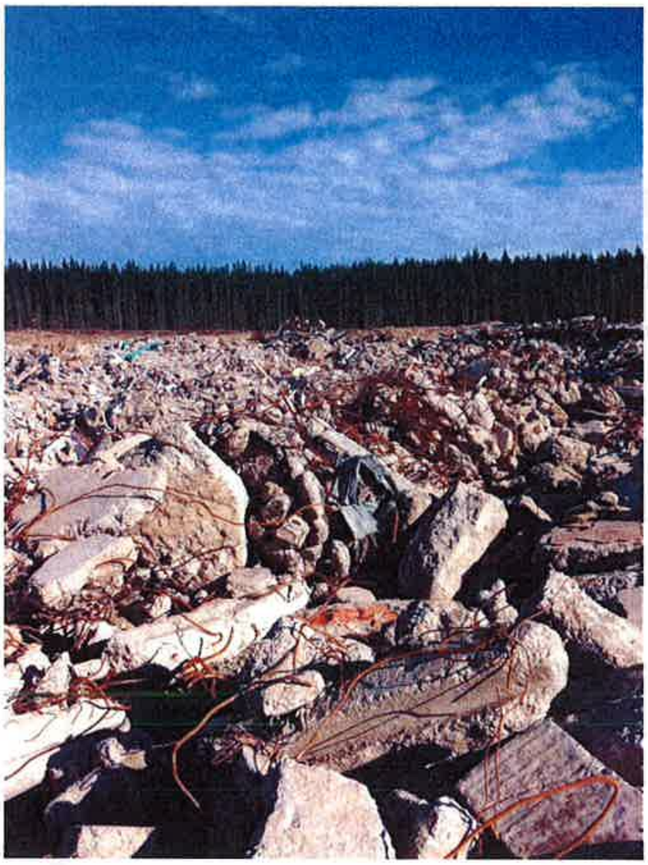

Figure 37 - A portion of the CTV Building debris field at the Burwood Eco Landfill from which columns were extracted for examination and testing.

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 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 33).

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

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

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

There cores were extracted form 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 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.

Based on the limited testing undertaken and comparing the sample mean against the mean of the production specification mean set out in NZS 3104:1983 it could not be concluded that at the time of the collapse the concrete in the slab did not meet the minimum 28-day strength specified of 25 MPa.

However when a 25% allowance for strength-aging was applied it appeared that the suspended slab may have not have achieved the specified 28-day strength of 25 MPa at the time of construction.

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 ConcreteTests 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.

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.

Concrete test properties shown in Table 2were 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).

	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 2 Column concrete test properties statistics

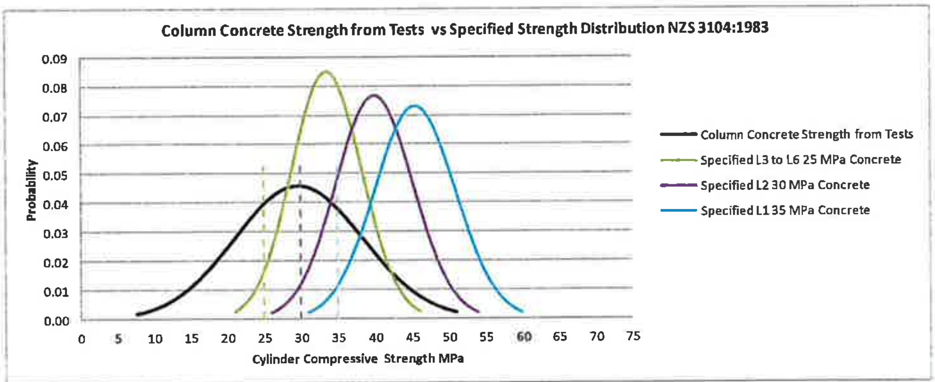

Figure 38 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS3104:1983. This indicates that the concrete in a significant proportion of the columns would have had strengths less than the minimum specified.

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 38.

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 have only achieved a 28-day strength of 17.5 MPa at the time of construction. This is 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.

8 COLLAPSE SCENARIO EVALUATION

INTRODUCTION

The aim of the evaluation was to identify, if possible, the most likely collapse scenario. This section describes selected results of structural analyses and considers those in conjunction with information available from eye-witness accounts, photographs, testing and examination of remnants. The analyses were needed to develop an understanding of the response of the building to earthquake ground motions and the demands this response placed on key structural members. It was recognised that any analyses for the 22 February Aftershock must be interpreted in the light of observed condition of the CTV building after the 4 September Earthquake and Boxing Day Aftershock, and the possibility that these and other events could have affected the structural performance of the building.

The approach taken was to: carry out a number of structural analyses of the whole building to estimate the demands (displacements, actions) placed on the building by the Earthquake and aftershocks; evaluate the capacities of critical elements such as columns; compare the demands with the capacities to identify the structural members most likely to be critical; identify likely collapse scenarios taking account of other information available.

Structural analyses and evaluation included the following:

- An elastic response spectrum analysis of the whole building
- A non-linear time history analyses of the whole building
- A pushover analysis of the whole building
- Displacement compatibility analyses of frames on Line 2 and F.

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 significantly from the mean value assumed in analysis
- The Spandrel Panels on the south and east face of the building could 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 any structural analysis procedures. In this case, particular consideration was given to:

- The possibility that the response of the computer models to the ground motion or response spectra records may differ significantly in nature and scale from that actually experienced by the building.

- The stiffness, strength and non-linear characteristics of structural members assumed for analysis may have differed from actual values. This can result in differences from reality in estimated displacements of the structure and particularly the forces generated within it.
- Estimating the effects on the structure of the very significant vertical ground accelerations is subject to very considerable uncertainty.

Overall, the approach has been to:

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

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 many other possibilities cannot be discounted.

OVERVIEW

Figure 9 presents a diagrammatic summary of the key considerations involved in evaluating possible collapse scenarios. The diagram highlights that at the heart of the evaluation is the comparison of "demand" with "capacity". "Demand" may be thought of as the loads and displacements of the building produced by the combined effects of gravity and earthquake loadings. The "capacity" may then be considered as the strengths of critical members and their ability to displace without critical loss of strength or integrity.

The key factors that influenced the estimation of the nature and scale of the demand on the building are shown on the "Demand" side of the diagram. A different set of key factors influencing the capacity of critical members is shown on the "Capacity" side of the diagram.

Under the collapse heading, the possible "routes to collapse" are shown. These are explained in more detail in later sections, but the common thread is collapse of a critical internal column which triggers progressive collapse. Displacements of the structure, possibly compounded by diaphragm disconnection, are the key drivers that result in demand exceeding capacity.

The following section examines Demand, Capacity and Collapse issues in more detail.

CRITICAL DEMAND/CAPACITY ISSUES

The lack of ductility in the columns made them particularly vulnerable and they have been the focus of the analyses. Columns must support the weight of the building and its contents at all times. When subject to earthquake actions, columns must, in addition, support any vertical loads produced by the ground shaking. Most importantly, they must be able to carry these loads while the building displaces horizontally. The ratio of horizontal deflection between one floor and the next over the inter-storey height is termed "drift". Frequently this is referred to by structural engineers as "inter-storey drift".

The ability of a column to sustain inter-storey drift depends on its stiffness, strength and ductility. There are established methods of estimating the capacity of a particular column to sustain the drift without collapse.

Structural analyses of the building as a whole resulted in a set of structure displacements at every point, and particularly at the top and bottom of every column. This output was used to estimate the drift demand on critical columns. There were two main sets of displacements obtained:

- Those assuming that the masonry wall on Line A stiffened the structure
- Those assuming that the masonry wall on Line A did not stiffen the structure

Both sets of displacements were derived on the basis that the floor slabs remained in contact with the stabilising North Core. The analyses showed high forces at these connections and the appearance of the building following the collapse suggested that there may have been some separation – either before or after the collapse was initiated. The possibility of diaphragm slab separation was investigated but this wasn't able to be justified by review of the physical collapse evidence and localised analysis of the slab diaphragm capacities.

However, it was found that the drifts determined from the analyses were sufficient to exceed the capacities of columns along Line F prior to diaphragm disconnection occurring. The design method set out in NZS 4203:1984 for diaphragm design was found to have limitations that meant that the full seismic resisting capacity of a structure may be limited by the diaphragm connection capacity, which shouldn't be the case.

The Drag Bar connections were shown by analysis in Appendix G to be the most likely location for detachment of the slab to occur due to in-plane rotation of the floors as the building twisted. Review of the physical collapse evidence found that failure had not occurred at the Drag Bar connections to the North Core at levels 4, 5 and 6 prior to slab pulling away. The slabs at level 3 and 4 were seen to have hung up on the North Core with their Line 3 ends resting on the ground after the collapse, as seen in Figure 84. This would not be expected to have occurred if they had first lost their support adjacent to the North Core. It was therefore concluded that the slab failures observed at Levels 4, 5 an 6 had most likely occurred due to the floors losing their support along Lines 2 and 3 as their columns collapsed.

In considering the possibility of diaphragm disconnection, therefore, it was considered sufficient to note that if such disconnection had occurred, it would have added to the drift demands on the critical columns and thus would mean that failure would have occured sooner than in cases where the diaphragm remained connected.

COLLAPSE INITIATORS CONSIDERED

Introduction

Five potential collapse initiation scenarios were identified for evaluation:

1. Column failure on Line F
2. Column failure on Line 2 or 3
3. Column failure due to diaphragm disconnection from North Core at Level 2 or Level 3
4. Column failure due to diaphragm disconnection from North Core at Level 4 or Level 5
5. Column failure due to diaphragm disconnection from North Core at Level 6

Demand Issues

General comment has been made about the variability and uncertainties in the analysis and evaluation process. More specific comments follow on matters affecting the estimation of demand.

Analysis Methods and Limitations

The various analyses provide insights into structural behaviour and response to the earthquake shaking and provide specific values for displacements and actions within the structure. The elastic response spectrum analysis (ERSA) was commonly used in the 1980s on buildings like the CTV Building and is still widely used. As such it provides a perspective similar to that of designers in 1986. This computer analysis method assumes that the stiffness of any part of the structure remains constant and there is no limit to the forces it can sustain. It uses response spectra derived from ground motion records as the basis for determining the earthquake loads in the structure. Vertical accelerations are not usually included. Capacity design principles and displacement compatibility assessments of secondary frames had to be applied to ensure ductile performance of the structure was achieved.

The non-linear time-history analysis (NTHA) method sets limits on the strength of members but allows them to deform beyond their elastic limit. This dynamic analysis method uses ground shaking records directly as input and examines the structural response in time steps through the earthquake record, modifying the structural properties as necessary at each step, and thus involves multiple analyses. Modelling of inelastic behaviour allows more realistic assumptions to be made on structural characteristics. However, the output is critically dependent on the input assumptions and is highly specific to ground motion record chosen.

Pushover analysis ("NPA") is a relatively simple process that allows closer examination of critical elements and the distribution of actions and displacements as the building deforms. Because it allows inelastic member properties to be modelled, it provides insights into the inelastic response of a structure and the likely distribution of displacements and forces within it.

Gravity Loads

Loads due to the weight of the building and its contents must be estimated in any structural analysis. Collapse investigation requires estimation of the actual gravity loads at the time. Normally, the weight of the building can be estimated within reasonable limits and this is not a serious issue, even though estimation of the load due to contents is more difficult.

Earthquake Response of Structure

The analyses assumed that records from nearby sites were applicable. These were applied in full without reduction. The analyses show that the response of the building was strongly influenced by the fact that, in the east-west direction, the North Core was very much stiffer and stronger than the South Wall. This caused displacements to be larger on the south, east and west faces than on the north face of the building. The effect of the masonry wall on Line A was monitored in the analyses.

The combined effect of the asymmetry of the main walls and the influence of the masonry walls on Line A was to increase the inter-storey displacements on the south and east face relative to other locations in the structure.

Comparisons of estimated Drag Bar forces with Drag Bar capacity were used to assess how well the analyses matched the real situations in the 4 September and 22 February events. Because of the possible uncertainties in the levels of actions from the analyses, such comparisons were taken as indicative only.

Inspection of the remnants showed that many construction joints were smooth and not roughened as is normal practice. Cracking along construction joints in walls was reported by the OIE after the September earthquake.

This may have increased displacements but such increases were not allowed for in the analyses. It was noted that any weakness of construction joints above and below beam-column joints could reduce the integrity of the joints. No specific allowance for this effect was made in assessing the capacities of the beam-column joints or columns.

CAPACITY ISSUES

Introduction

Assessment of member capacities in existing buildings presents considerable challenges. The following comments highlight the most important considerations and sources of variability and uncertainty.

Column Drift Capacity

Two different methods were used to assess the drift capacities of the critical columns. These were the Push-over Analysis which used constant stiffness columns section properties with rigid plastic hinges at their heads and bases. This was an elastic-perfectly plastic model.

The other was a prescribed displacement compatibility analysis using effective inelastic section properties at prescribed displacements derived from column moment-curvature software Cumbia and using a displacement profile along Line F and 2 equivalent to development of 0.75% drift at Level 5 along Line F.

This provided some measure of cross-check and gave closely matching results. Capacities were assessed to identify:

- The drift at which the reinforcing steel first yields
- The drift at which the column section would fail (i.e. reach specified strain limits)

The yield limit is of value in comparing observed damage with the results of the structural analyses. This limit was also used in comparing capacities of columns with the requirements of design practices in 1986. Non-ductile detailing was permitted if drifts under the design loading were below those required to cause yield in any column and remained elastic.

Estimation of the drift to fail a column involves assumptions on the limit of strain in the concrete. A value of 0.004 was assumed and this is considered to be realistic and recommended by NZSEE guidelines.. However, values up to 0.007 could possibly be justified. Even at the higher strain level, the drift to cause failure would not increase in proportion for most of the lower level columns. This is because the greater part of the drift capacity was in the elastic deformation of the column as a whole for the more heavily loaded ones, and the post-elastic behaviour was concentrated in "hinges" at the top and bottom of the column.

Comparison of drift demand with capacity was further compounded by:

- The critical effect of assumed concrete strength and maximum strain limit in the estimation of drift capacity.
- The effect of load on the columns - the higher the load, the lower the total and inelastic drift they could sustain. Thus columns in the upper levels could sustain more drift than those more heavily loaded columns at the ground floor level. Most of the columns had the same amount of reinforcing steel.
- Vertical ground accelerations can increase or reduce the loads on columns and thus increase or reduce drift capacity.

On top of these considerations was the potentially severe influence of the Spandrel Panels on column capacity. Observations after the 4 September Earthquake and inspection of structure remnants after 22 February Aftershock indicated that there had been contact between the columns on the north, east and south faces and the Spandrel Panels. Such contact was found by the displacement compatibility analyses to have have greatly accelerated the loss of drift capacity.

Overall structural analysis models did not allow for this interaction but were used to estimate the likely drift demands at these positions. Separate analyses of the frames on Line F were carried out with the effect of Spandrel Panel interaction modelled to provide insights into the effects and the level of forces likely to be generated.

The level of interaction between a column and an adjacent spandrel depended on the gap that existed between Spandrel Panel and column. Because it was not possible to know what the gaps were, various levels of interaction between columns and Spandrel Panels were considered.

In assessing Spandrel Panel interaction it was recognised that the actions generated may have been limited by the capacity of the bolts that connected the spandrels to the floor slab. This raised questions as to the ability of the bolts to act together because one bolt would engage before the others depending on the gap between the bolt and the spandrel. The detail in fact showed no gap – washers were welded in place to provide engagement of all bolts simultaneously. The capacity of the fixings specified was also found to be sufficient to restrain the columns enough at the expected critical drift levels at which collapse was thought to have initiated.

Engagement of the column with the Spandrel Panel involved some flexibility because the vertical section of the Spandrel Panels was offset from the column line. Analyses took this into account.

In summary, though, it is not possible to determine the exact role of the Spandrel Panels in the collapse. Nevertheless, it was possible to conclude that:

- Forensic observations of column remnants suggested that there had been Spandrel Panel engagement in some cases.
- The displacement demands of the 22 February Aftershock were sufficient, based on application of the full ground motions, to fail a critical column without Spandrel Panel interaction.
- Maximum possible Spandrel Panel interaction (minimal gap between column and spandrel) reduced the drift capacities of the indicator columns significantly. In other words the effect of any Spandrel Panel interaction would have been to bring about failure either sooner or at a lower level of structural response to the ground motion than would otherwise have been the case.

Critical Column Identification

Drift demands were generally lower at the lower levels of the structure than at the upper levels. Thus in identifying critical columns it was necessary to examine the ratio of demand to capacity at various levels and locations within the complete structure. This process resulted in the identification of two "indicator" columns – one in the upper levels of Line F and one at the ground floor on Level 1 on Line D.

These columns were chosen because, if it is assumed that all other variables were equal, analyses indicated that the ratio of demand to capacity is greatest in these columns. In fact it must be recognised that the possible existence of low concrete strength, and/or greater than assumed interaction with a Spandrel Panel could mean that a column in another location could have initiated failure.

Further information on the analyses is given in Appendix D and F.

Diaphragm Connection Capacities

Estimation of the actions from the NTHA and ERSA on the Drag Bars attaching the floor slabs to the North Core was subject to some uncertainty. Failure of the Drag Bars could have initiated an "unzipping" effect along the line of the connection.

Holes had been cored in the floor slabs at each level adjacent to the North Core. It is believed that on the basis of the small number and size of these holes in the length of slab connection that these holes were not material to initiation of the collapse.

The effect was analysed in Appendix G. This found that the critical failure location for diaphragm disconnection from the North Core occurred along the tips of the north-south walls. The Drag Bars were weaker than the reinforced slab at the location where the slab was found to have failed. This indicated that the slab had not failed due to excessive in-plane diaphragm actions but due to loss of vertical support as the columns on Line 3 progressively collapsed.

Beam-column Joint Capacities

While the focus was to examine the capacity of the columns, it was recognised that the beam-column joints were vulnerable. The joints did not have sufficient shear and confining steel which is necessary for such joints to maintain integrity when subject to earthquake actions. The design intention was that these joints, as with the columns, would be protected by the stiff walls of the North Core and the South Wall. The standard of the day allowed this non-ductile detailing within defined limits.

The lack of reinforcement made the failure limits of the beam-column joints difficult to estimate. In particular, the shear capacity would have been highly dependent on the level of load in the columns. One example was calculated as an indication of likely relative capacity. This showed that it was possible for the beam-column joints to be damaged at prior to failure of the columns. However, the consequences of failure of a beam-column joint would not necessarily result in collapse of the structure. Hence it was concluded that focus on the column capacities would provide an appropriate basis for investigating the reasons for the collapse of the building.

Line A Wall Strength / Stiffness Capacities

Considerable efforts were made to assess the degree to which the three levels of masonry on the west side of the building affected the response of the building. The basis of modelling the effect of the infill wall for the NTHA is described in Appendix D and the basis for the ERSA in Appendix E.

The mathematical models used were in line with those commonly used in structural analysis for design purposes. However, it was found that, for the 4 September Earthquake, the analysis indicated severe damage in the plane of the walls on Line A if the masonry was fully restrained by the concrete header beams and columns. Photographs of the walls and statements by Eyewitness 16 found no damage or spalling.

The ERSA indicated that at Line F drifts of 0.35%, the nominal bending capacity at the base of the South Wall would have been reached. At the same time the shear demands on the Line A wall if fully constrained by the concrete beams and columns around it, would have been in the order of 75% of the nominal shear capacity limits in the reinforced masonry standard NZS 4230:2004. At this level of shear very little damage would be expected to have been observed in the masonry. This therefore indicated that the drift demands along Line F in the September earthquake may have been as low as 0.35%.

In the stronger shaking of 22 February the ERSA indicated that the Line A masonry wall would have been substantially damaged by the time drifts along Line F reached the critical levels of 0.75% to 1.3%. Major diagonal cracking was observed in the Level 2 wall masonry immediately after the collapse as seen in Figure 22. So its influence on making the torsional component of drift along Line F may have reduced, but this may have been out-weighed by its reduced ability to restrain north-south drifts along Line F and 1 relatively greater than elsewhere in the building. It is therefore difficult to accurately quantify its effect on the collapse.

This suggests that the masonry walls, at least for the 4 September level of shaking, were considerably stiffer than assumed in the NTHA analysis and that the response of the structure to the ground motion may have been significantly less than that indicated by the ERSA and NTHA using full ground motion and spectral acceleration records.

Other Influences on Structural Capacity

Other possible influences on the structural capacity were considered:

- Reinforcement at the bottom of beams on Line 4 was found not to have been anchored into the North Core wall as intended. At Levels 3 to 6 this steel was bent up within the cover concrete, reducing the strength of the connection between the beam and the walls. It is possible that this would have weakened the diaphragm connection to the North Core.
- Smooth construction joints were observed in a significant number of remnants. It is possible that this reduced the strength capacity of some joints and increased inter-storey drifts, such as those in the wall on Line 1 (South Wall) and the beam-column joints. It can only be a matter of speculation as to the extent of this.

KEY RESULTS

Elastic Response Spectra Analysis

Figure 39 shows the response spectra used in the ERSA. The graph indicates the response in terms of horizontal acceleration for varying structural natural periods of vibration. Low-rise buildings generally have low periods and tall buildings having higher periods. The fundamental vibration modes of the CTV Building corresponded to values around 1.0 second.

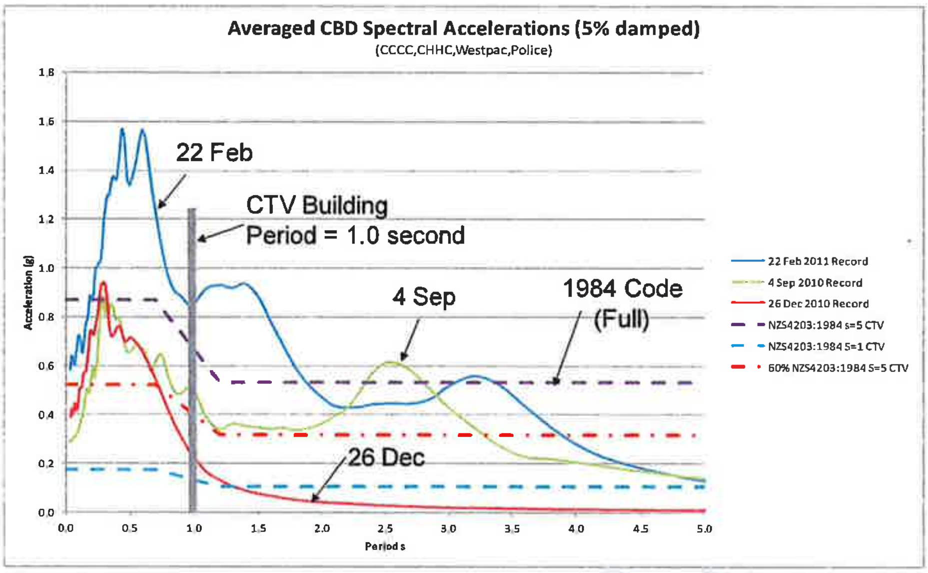

Figure 39 – Response spectra records for 4 September Earthquake, 26 December Boxing day Aftershock and the 22 February, 2011 Aftershock. Also shown (dashed lines) are the spectra for the CTV building according to NZS 4203:1984. The lower dashed line is the design spectra for ductile design that the North Core and South Wall were required to have design capacity in excess of. The upper most dashed line is 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 graphs give an indication of the relative intensities of ground shaking records on 4 September, 26 December and 22 February (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 design capacity and exhibiting structural damage such as yielding of the reinforcing steel or concrete spalling. This is because for design of members, strength reduction or safety factors are applied when using that level of loading.

It can be seen that at a period of 1.0 seconds, the demand of the February record exceeds the full response expectations of the standard. The demand of the September record was around 60 per cent of that value. At that level it was well above the level at which significant damage would have been expected in the South Wall, and the infill masonry on Line A. It also could have caused drifts along Line F sufficient to initiate collapse of those columns.

The report of the OIE after the September Earthquake reported that only minor cracking had occurred to the North Core and South Wall. The OIE believed no yielding had occurred in the reinforcing steel of those elements. No damage to the masonry infill wall on the west face was also reported by Eyewitness 16 who had been preparing the wall for recladding immediately prior to the 22 February Aftershock. He reported that no gaps were evident between the masonry and the columns. The computer analyses found that if the full September earthquake record had been applied to the models they predicted severe damage to the masonry infill wall. This indicates that the real building response to the September ground motion was less than that indicated by the use of the full record in the computer model. It also indicates that the response of the building to the February Aftershock was also less than that predicted by the computer models using the full records.

Column F2 Level 3 – Demand versus Capacity

Figure 40 and Figure 41 show output from the NTHA. The vertical axis shows the amount of inter-storey displacement (drift is the ratio of that displacement to the height between the floors) at that location. The horizontal axis is the time from start of shaking (as input into the analysis). The wavy lines plot the drift over time as the building model responds to the ground shaking record and moves, and are based on application of the full ground shaking record. This drift is a key measure of demand on the column.

The horizontal lines represent the estimated capacity of this column to sustain the drift without failing according to various criteria (assuming average concrete strength). The band between the horizontal lines reflects the difference between no contact with the Spandrel Panels (higher value) and full contact with the Spandrel Panels. In fact this band would be wider if allowance was made for the effect of variable concrete strength and vertical earthquake forces in the column. The areas where the drift has exceeded the estimated capacity are shown shaded.

The key points to note are that for the 4 September Earthquake, the maximum displacement demands are about half those calculated for the 22 February Aftershock. Although there are two places where the 4 September displacements are shaded, there are no cases where they exceed the maximum assessed capacity of the columns when no Spandrel Panel interaction occurred in the upper level columns. On the other hand, the 22 February demands have many "excursions" shown shaded and three that exceed the maximum value.

Initiation of reinforcing yielding in the F/2 column at Level 3 was calculated to occur at drifts of around 0.6%.

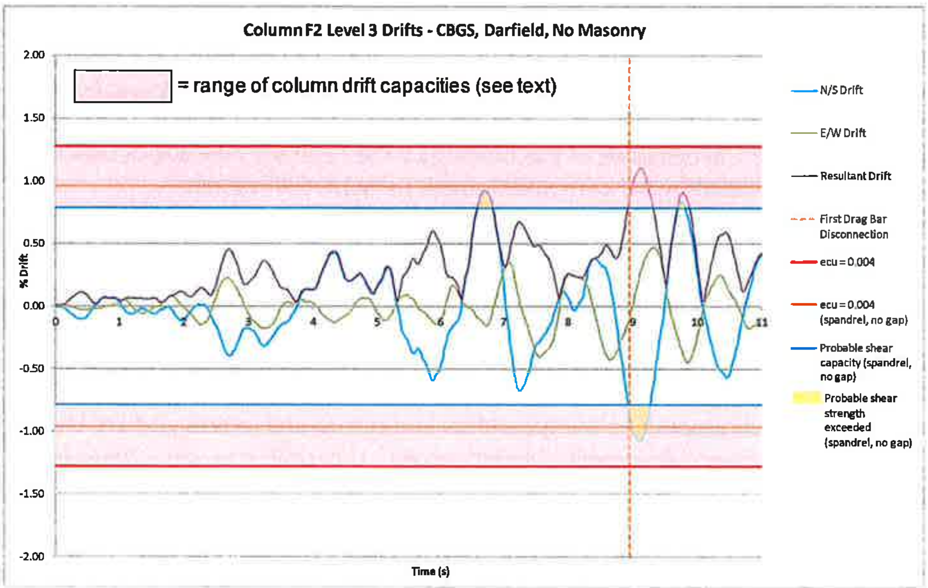

Figure 40 - NTHA drift demands and capacities plotted for column F/2 at Level 3 on Line F for the September Earthquake. This indicates that drift demands were predicted by the model to have exceeded the lower bound drifts at which collapse was found to be able to initiate with interaction from Spandrel Panels. Yielding of the column reinforcing was calculated to have occurred at a drift of around 0.6%. Disconnection of the slab from the North Core was also predicted to have occurred at around 1.0% drift demand.

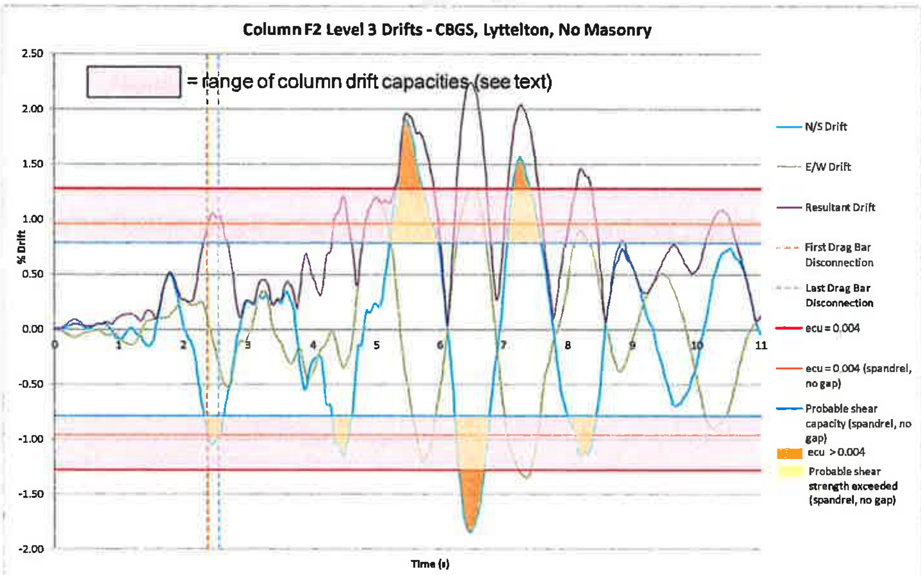

Figure 41 - NTHA drift demands and capacities plotted for column F/2 at Level 3 on Line F for the 22 February Aftershock. This shows that drift demands were predicted by the model to have exceeded many times the lower and upper bound drifts of 0.75% to 1.3%, at which collapse was found to be able to initiate. The lower bound drift being with interaction from Spandrel Panels and the upper bound without Spandrel interaction.. Disconnection of the slab from the North Core was also predicted to have occurred at around 1.0% drift demand early in the record..

Such comparisons provide valuable insights into the relativity of demand and capacity, but for reasons described above must be interpreted with care. A number of points are worth noting:

- The demands represent values derived from the full ground shaking record. If it happened that the building response was less than calculated, the plotted displacements would be less. This could be due to the CTV site not experiencing the full ground motions recorded at other nearby sites or because the response of the building was not as great as the analysis determined. Note that a reduction of about one-half on the 4 September displacements would mean they did not exceed the minimum assessed yield capacity of around 0.6%.

- The column drift capacities shown are based on monotonic considerations. This means that no account was taken in the analysis for the well known degrading effects of cyclic drift demands. It is recognised therefore that these drift capacities represent an upper bound assessment of the column cyclic performance capabilities.

- The vertical lines indicate when the Drag Bar (diaphragm connections to the lift shaft walls) would reach capacity according to the analysis. Calculation of this force is subject to considerable uncertainty, but if it is taken as correct and if the Drag Bars did not fail before column collapse, then the full displacements would need to be reduced by about half. Even at this level (about 1.0% drift) the 22 February displacement demands were sufficient to fail the column if there was full interaction with the Spandrel Panels

These comparisons give some indication of the challenges of determining which column or mechanism initiated failure. However, the plots indicate that the demands of the 22 February Aftershock were more than enough to cause column failure, whereas the demands of 4 September were less.

Similar plots to Figure 40 and Figure 41 were made for column D2 on Level 1 (ground floor). Displacements (for the full record) were well below the assessed capacity of this column for 4 September and only marginally exceeded the capacity for the 22 February analysis. This is a broad indication that this column was less likely to have been the initiator of the collapse. However, this possibility cannot be ruled out because it may have had lower than average concrete strength and/or suffered more from the effects of the considerable vertical forces generated in the 22 February Aftershock.

Drift Demand Capacity Comparison

Table 3 shows a comparison of calculated drifts and capacities for indicator Column F2 at Level 3 to 4.

CTV Drift Comparisons for Critical Indicator Columns

Column F2 L3-4

Demand or Capacity	Event / Condition	Column drifts (% of floor height) Column F2L3
Demand	22 Feb	2.0
	26 Dec	0.5
	4 Sep	1.0
	1986 Ultimate	1.3
	1986 Dependable Strength	0.7
	2010 Ultimate	2.3
Capacity	Failure (No spandrel effect)	1.3
	Failure (Full spandrel effect)	0.8 - 1.0
	Nominal (No spandrel effect)	0.6 -1.0
	Nominal(Full spandrel effect)	0.5 - 0.8

Table 3 - Column F/2 Level 3 drift demand versus capacity using full record.

The table shows the maximum drift demand for 4 September, and 22 February for the full record (column 1). Also shown are two 1986 standard design limits for the CTV: Building

- The "1986 Ultimate" drift is the maximum drift demand calculated for the CTV Building indicator columns by the ERSA using the elastic design spectra and standard methods applicable in 1986.
- The "1986 Dependable Strength" drift is the computed drift demand for the CTV indicator columns at the time the reinforcing steel first yields. For non-ductile detailing to be allowed in the columns, under 1986 standards, the shear walls had to be stiff enough to prevent column yield or the dependable strength being exceeded at this level of drift.

The 2010 Design Requirement is also shown to indicate the level of drift demand that current design requirements would place on the CTV Building indicator columns. As such it is a measure of the difference between 1986 design requirements and those of current standards – which now require at least limited ductile detailing for all columns irrespective of drift demand.

It is important to recognise that the expectation of design standards in construction is that even at the attainment of the maximum drift levels there should still be a low probability of collapse occurring.

DISCUSSION AND CONCLUSIONS

Reasons for the Collapse

Based on their investigations, discussions with eyewitnesses, analyses, calculations and consideration of relevant facts and information made available to them, the authors believe that the following contributed to the collapse of the CTV Building:

1. The most likely collapse scenario is one of premature collapse due to a "short-column" effect from impact of the Line F columns on the precast concrete spandrel cladding panels at Inter-storey drifts of around 0.75% to 1.3%. The resulting damage to the Line F columns caused a loss of vertical load carrying capacity that then overloaded the Line 2 and 3 interior columns. This in turn led to progressive overload of their adjacent columns until total collapse of the CTV Building had occurred.

2. An alternative collapse scenario initiates with overload of a Line 2 or 3 column at Level 1 or 2 under combined effects of vertical acceleration and lateral displacement demands, then progresses in much the same way as the most likely collapse scenario. Displacement compatibility analyses on the Line 2 frame at drifts consistent with lower bound 0.75% drifts occurring at the same time along Line F, found this to be unlikely. This was because flexural and shear initiated failures in column heads and bases would only be expected in the level 5 columns if the concrete strength was less than the lower 20 percentile strength found from the column concrete tests.

3. A third collapse scenario considered involved initiation of collapse with detachment of floor Drag Bars from the North Core at Levels 4 to Level 5 due to overload from horizontal rotation of the floor slabs relative to the North Core. This would have led to displacements along Line F that would have also led to column failure and progressive collapse as for the most likely scenario. However this scenario is less favoured because review of the collapse debris and calculations indicated that the slab at Levels 4 to 6 at the lift core had fallen away as a consequence of the collapse of the Line 3 columns followed by column D/E-4.

4. A fourth collapse scenario was similar to the third except that it considered initiation to have occurred with detachment of the Level 2 and 3 diaphragms, which did not have Drag Bars installed to them in 1991, from the North Core. This was thought to have led to additional lateral displacements and collapse of the lower level internal columns. However the demands on the diaphragm connections, at displacements consistent with lower bound 0.75% drift along Line F, were found to be less than the capacity of the connections. The Level 3 slab was found to have the highest in-plane diaphragm demands yet it was observed in the collapse evidence to have hung up on the North Core. This indicated that it had collapsed due to loss of support at Line 2 rather than due to diaphragm induced effects. By implication of it being subject to lower in-plane diaphragm actions than the level 3 slab, the Level 2 slab was also therefore considered to have collapsed due to loss of gravity support at Line 2.

5. A fifth collapse scenario initiating with north-south rocking of the North Core leading to failure of the floor slab at approximately 1200 mm out from the south side of the North Core. Once detachment occurred it would be expected to lead to short-column failure of columns along Grid F and I and collapse similar to the other short-column modes.. However the

displacements on the North Core and associated uplift on Line 4, consistent with lower bound drifts on Line F of 0.75% were only sufficient to cause displacements less than those associated with serviceability design. Therefore it was likely that one of the other four collapse scenarios would have initiated collapse prior to this being critical.

The authors believe that based on their investigation the following specific deficiencies in critical components contributed to the collapse:

- The specified gap between the precast concrete spandrel cladding units and the perimeter concrete columns did not allow for adequate construction tolerances nor was there a requirement for a minimum seismic gap to be maintained.
- The 400 mm diameter concrete columns and beam–column joints were not designed and detailed for seismic requirements. This meant that the columns lacked redundancy in axial, flexure and shear capacity to cope with over load demands.
- Based on statistical analysis of the column concrete test results a significant proportion of the columns were likely to have had concrete strengths less than what had been specified. The distribution of concrete strengths was also less than would have been expected when account is made for the increase in strength with time expected for concrete of that age. This reduced the redundancy of load carrying capacity of the columns.

The structure was susceptible to progressive collapse due to a number of factors including:

- Non-seismic detailing to the slab, beams, columns and beam-column joints meant that these elements broke away from each other once columns began to lose load carrying capability. There was very little ability to redistribute load by secondary structural mechanisms such as catenary action once collapse initiated.

- The large proportion of cover concrete and the low level of confining steel in the columns meant that there was little ability to cope with axial overload in a ductile manner.

THE FIVE SCENARIOS

Preferred Collapse Scenario

In the authors' opinion Scenario 1, Line F was the most likely initiator of collapse starting in the upper levels where drifts were higher.

Scenario 1: Line F or 1 Column Collapse Initiation

In this scenario collapse may have initiated in the Line 1 and F perimeter columns above Level 2 at drifts between 0.75% and 1.3% at Level 3, 4 or 5. This would have then led to overload of the Line 2 and 3 columns at Level 1, at the Madras Street end of the building.

The Line 1 and F column lines were found to experience similar and the highest inter-storey drifts in the structure due to the effect of the infill masonry wall on Line A shifting westward the centre of rigidity about which the centre of mass of the structure would have rotated (Figure 42).

Very little inelastic deformation was indicated by the ERSA and NTHA to have occurred in the structure except in the South Wall and Line A masonry infill wall up to the point of 0.75% inter-storey drifts being developed at Levels 3, 4 or 5.

The level at which the Line F columns initiated collapse was found from the displacement compatibility analyses and the pushover analysis to be dependent on the size of the gaps between the columns and the precast Spandrel Panels. With gaps of 3 mm then overload of the columns could have occurred at Levels 2 to 5. With 5 mm average gaps this was limited to Levels 4 and 5. With 10 mm gaps no interaction with the Spandrel Panels would have occurred at 0.75% inter-storey drifts but columns head overload could still have occurred at the Level 4 and 5 at drifts of around 1.3% or less, and yield around 0.6%.

The columns drifts were constrained at the floor levels by the stiffness and response of the South Wall and North Core, and also it appears, the masonry infill wall on Grid A up to the underside of Level 4. This would have prevented the soft-storey mechanisms occurring that are normally associated with column hinging occurring. The significantly higher strength and stiffness of the North Core relative to the South Wall meant that its response would have dominated the building response. Even after bending damage developed in the South Wall limiting its contribution against the twisting motion the North Core was able to continue to resist the motion.

Increased inter-storey drifts and shears were indicated by the ERSA to occur at Level 4 possibly due to termination of the Line A masonry infill wall at the underside of Level 4. This caused a significant reduction in the torsional and north-south translational stiffness above that level.

These higher shear demands on the South Wall at this level compared to the one below were identified to occur for east-west earthquake loadings Figure 126. There may have also been slippage on the smooth construction joints found in the wall during the Site Examination and noted in Figure 100 and Figure 101. These factors may have increased the inter-storey drifts between Level 4 and 5 relative to the floors below it, increasing the likelihood of column failure initiating between Level 4 and 5.

With loss of load carrying capacity on Line F, the interior columns on Line 2 and 3 would then have become overloaded. As they gave way the slab and beams they supported would have pulled downwards on the Line 1 South Wall and frame pulling away the slab from the frames on Line 1 and Line 2. The beams connected into the columns at Grid A would then have pulled down and inwards on the columns pulling out the beam-columns joints in places. Levels 5, 6 and the roof would have then dropped as a distinct unit, but perhaps with a slight lean towards Madras Street collapsing the structure below to the ground.

A number of the eyewitnesses reported seeing the building collapse start in the upper third of the building. Eyewitness 6 reported a slight tilt to the east of the upper floors as the collapse progressed downwards, and the debris observed in

Madras Street immediately after the collapse and before any have been moved in the rescue showed a slight throw eastward.

All these observations and the structural analyses support Line F column failure including Spandrel Panel interference effects, being a likely point of initiation of the collapse.

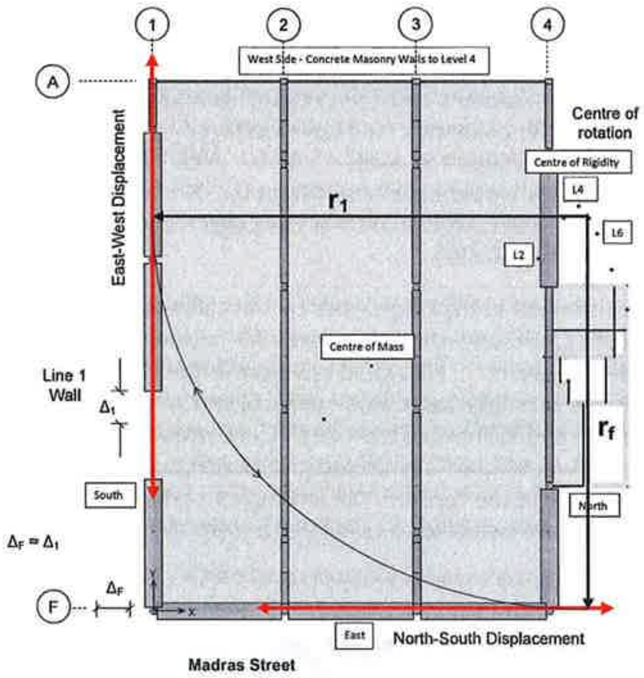

Figure 42 - Torsional behaviour of the building increased due to the effect of the masonry infill wall on Line A. This shows that the building had a tendency to twist about the centre of rigidity that was moved towards the Line A wall because of its stiffness. The centre of rigidity was furthest west at Level 4. This resulted in the columns along Line 1 and F experiencing similar and the highest levels of inter-storey drift as the building responded to the Earthquake and Aftershock. This made the columns on these lines the most susceptible to being damaged and initiating collapse during the Aftershock. Line 1 is thought to have had more protection against progressive collapse occurring due to some of the beams also being supported off the South Wall which was observed to have collapsed after the rest of the building.

Scenario 2: Isolated Line 2 or 3 Column Collapse Initiation

In this scenario collapse may have initiated by failure of one of the most highly loaded secondary frame columns. This was considered as part of the displacement compatibility analysis. This was considered to have been a possibility due to the evidence of low strength concrete in a number of columns tested, and the report of significant vertical accelerations during the collapse. However it is considered to have been less likely for the reasons discussed as follows.

The most highly loaded secondary frame columns were on Lines C and D, and 2 and 3.

It was envisaged that following column failure the floors would have sunk and the slabs would have been forced into catenary type behaviour, causing combined shear, flexural and direct tensile failure of the slabs into the frames and walls. The structure then would have progressively collapsed inwards onto itself.

The concrete specified for the columns at Level 3 to 6 was to have 28-day strength of 25 MPa. Those at Level 2 were to have 28-day strength of 30 MPa. At Level 1 this increased further to 28-day strength of 35 MPa. The 28-day strength was approximately the lower 5 percentile strength of the concrete produced to a mix specification.

However the concrete testing of column remnants found the concrete to have lower 5 percentile strength of 15.3 MPa. This was significantly less than the lowest 28-day strength concrete specified for any columns in the building. For this reason lower strength concrete was considered in the column collapse scenarios.

A check of the gravity actions on these columns at Level 1 in accordance with 1986 Codes, and assuming the specified concrete strength of f'c=35 MPa, showed they would have been working at the upper Code design limit for axial compression actions. (Refer Figure 43.)

If lower strength concrete occurred then the columns would have relied on there being lower than specified in-service live loads and the safety margins inherent within the design assumptions, to have maintained their integrity under gravity compression actions prior to the earthquake.

The displacement compatibility analysis using the displacements of the Line 2 frame consistent with 0.75% drifts occurring on Line F, found that collapse initiation was unlikely to occur on Line 2 prior to initiation having occurred on Line F.

Column base and head flexural hinging is likely to have occurred at Level 5 in columns D/2 and C/2, if the concrete strength was less than the lower 20 percentile of that found from tests (22.3 MPa). However the columns would still have possessed sufficient displacement ductility in reserve to cope with that without immediately initiating collapse. Columns B/2 and E/2 were less highly loaded and were found, along with columns C/2 and D/2, to have shear demands exceeding their capacity if the concrete strength was in the lower 5% of that tested (15.3 MPa).

The 'non-seismic" detailing of reinforcement in the columns (small diameter ties and spiral at wide spacing) offered little in the way of confinement or shear strength. This meant that the columns had little ability to maintain integrity once axial compressive damage began to initiate in the lower floor columns.

In summary this isolated Line 2 or 3 column collapse mechanism is a credible option that cannot be discounted. However it depended on the columns having concrete strengths at the lower end of those tested and/or some amount of vertical acceleration amplifying the column actions.

However it would not be totally consistent with the observation of an eastward tilt as the upper levels fell as a unit, and the slight eastward throw of debris into Madras Street.

The isolated internal column collapse initiation would be more likely to have resulted in an even more concentric debris pile on the site than what was observed.

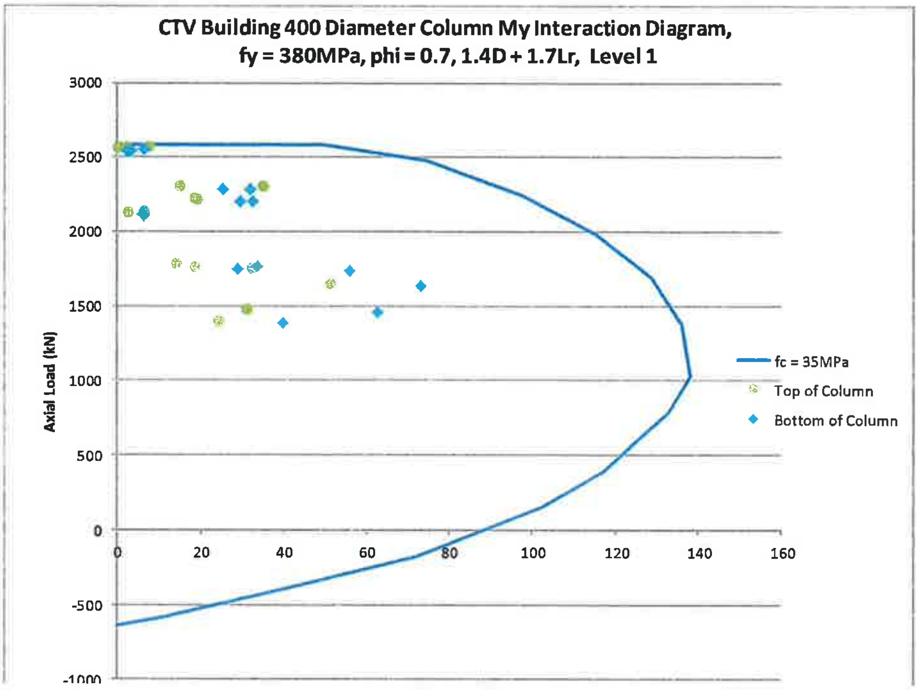

Figure 43 - Column Chart for factored design Gravity Load 1.4D + 1.7 Lr for concrete with specified 28-day strength of 35 MPa. This indicates that a number of the Level 1 columns were getting close to the blue line and nearing the limit to safely hold up the building under general gravity loading conditions within acceptable margins of safety. The phi factor of 0.7 down rates the column strength to 70% of its nominal capacity and the load factors of 1.4 and 1.7 on the dead and live loads respectively factor up the expected loads. For the condition used in the collapse scenarios these safety factors have been reduced to 1.0 to better reflect actual conditions at the time of the collapse.

Scenario 3: Level 2 and 3 Diaphragm Detachment from North Core

In this scenario analysed by Clifton (Clifton 2011) the diaphragms at Level 2 and 3, which did not have Drag Bars installed to them during the post-construction remedial work, were thought to have been able to detach. This was due to high in-plane flexural demands that would occur at loadings recently proposed for diaphragm design (Uma, Zhao et al. 2009).

The effect of this was would have been then to overload Level 1 and 2 columns by imposing greater lateral displacement on them due the loss of restraint provided by the North Core.

The demands imposed on the connection of the diaphragm to the North Core were then checked at the point at which collapse initiating drifts along Line F reached the lower bound 0.75%. This indicated that the floor diaphragm connection had between 2.3 and 3.6 times the capacity required to resist the demands imposed at that point

In conclusion it appears that diaphragm disconnection at Level 2 and 3 was unlikely to have occurred prior to development of collapse initiation on Line F.

Scenario 4: North Core Line D or D/E Drag Bar Detachment at Level 4 and 5

The Non Linear Time History Analyses (NTHA) showed that once the diaphragm ties to walls D/E and D detached then hinging and collapse would follow in many columns through-out the structure. Displacements of approximately 30 mm (0.93% drift) along Line F at the capacity of the Level 4 Drag Bar being reached were predicted by the NTHA.

The greater inter-storey drifts along Line F would then have led to greater flexural hinging at the tops and bases of the perimeter columns, and then at around mid-height as the columns became restrained by the pre-cast concrete Spandrel Panels installed between them.

This level 4 Drag Bar disconnection scenario is not favoured because it appears from the photos of the core immediately after the collapse that the Level 4, 5 and 6 slabs had not failed initially at the Drag Bars. This is discussed in more detail in Appendix G.

Drifts required to initiate collapse along Line F at Level 5 may have been as low as 0.75%. This was less than the drift of approximately 1.0%, found from the NTHA, at which the calculated capacity of the Level 4 Drag Bar connection to the Wall on Line D/E would have been reached.

These analyses do show that the failure of the diaphragm connections to the North Core walls, including the Drag Bars limited the seismic resisting performance of the building.

Scenario 5: North Core Rocking Initiated Failure of Slab

In this scenario rocking of the North Core related to primary North/South response was considered to initiate failure and detachment of the floor slabs from Level 6 downwards:

The pushover analyses, showed considerable uplift of the tips of the North Core developing as the wall displaced to the North.

However at the point at which collapse initiating drifts along Line F reached the lower bound collapse critical drift of 0.75% under the north south acting loadings condition, the northwards displacement at Level 6 was found to be in the order of 82 mm and the upward displacement was only around 12 mm.

An uplift displacement of about 25mm was calculated to be necessary to cause cracking of the slab at the wall tips, and also back 1.2 m from the wall tips at the failure plane location.

This level of vertical displacement was also not significant enough given that a serviceability displacement of span/250 mm of 30 mm would be considered reasonable.

Therefore it does not appear that this cracking condition would have led to a severe failure condition prior to collapse initiating due to Scenarios 1 to 4.

9 DESIGN AND CONSTRUCTION ISSUES

INTRODUCTION

The following design and construction issues have been identified during the investigation. These are issues where the design, the construction or the Standards of the day were found to be potential contributors to the collapse.

DESIGN ISSUES

Building Inter-storey Drift Limits

The building as a whole was required to have sufficient stiffness to not exceed the inter-storey K/SM factored drift displacement and drift limits for Zone B (Christchurch) was 0.0083h or 0.83% (NZS4203:1984 cl 3.8.3.1).

For fully ductile reinforced concrete walls or coupled walls with design capacities (incorporating material strength reduction factors, ϕ) not less than the design actions derived from SM =0.8 seismic loading, a deformation multiplier of K/SM= 2.75 was therefore required (NZS4203:1984 cl 3.8.1.1).

The ERSA and NTHA allowed for some level of foundation rotation, following current practice, rather than full fixity as was allowed at the time of the design (NZS 4203:1984 cl3.8.1.2). Allowing some foundation rotation may have reduced the structural response of the building, but possibly increased the calculated inter-storey deformations between Level 2 and Level 1. However the effect on computed inter-storey deformations would be less pronounced above Level 2.

For Levels 2 to 6 with inter-storey heights of 3.24 m this was 27 mm. For Level 1 with inter-storey height of 3.66 m this was 30 mm. This set the minimum stiffness requirements for the primary seismic resisting structure at S=1 actions factored by K/SM =2.75.

The building as a whole was found to have satisfied the building inter-storey drift requirements.

Drift Capacity of Columns

The concrete structures code of practice for design required the beam and column frames on Lines 1, 2, 3, 4, A and F columns to be designed as Group 2 non-separated elements (NZS 3.5.14.1(b) and cl. 3.5.14.3). A displacement compatibility analysis was required to ensure that the columns could sustain the K/SM drifts imposed on them by the primary seismic resisting frame.

However the displacement compatibility analyses showed that the drift capacity of the Line F columns at dependable strength was less than the K/SM factored inter-storey drift limit of 0.83%. This meant that the columns could not be detailed on the assumption of elastic behaviour and were required to have been designed using the additional seismic design provisions of NZS 3101:1982.

Spandrel Panel Separation

The Spandrel Panels were designated as Group 1 secondary elements by the concrete structures design code of practice (NZS 3101:1982 cl 3.5.14.1). The Spandrel Panels were required to be separated from the columns in such a way as to allow adequate tolerance in their construction and for the K/SM factored seismic inter-storey drifts (NZS 3101:1982 cl 3.5.14.2).

Allowance for construction tolerances in the length of the precast units was not a standardised measure. However the out of position tolerance of the columns and variation in the diameter of the columns have been calculated by the authors using the construction tolerances guidelines BS 5606:1990 (BSI 1990). The method of combinations of tolerances recommended in that guideline was +/- 12 mm at each column face to panel end gap.

Assuming the structure satisfied the 0.83% drift limit, panels 820 mm high above floor level required a 7 mm gap between the panels and the columns at Levels 2 to 6 where inter-storey heights were 3.24 m.

The actual as-built gap to the Spandrel Panels either side of the columns may have ranged between 0 and 22 mm based on the guidelines for assessing combined construction tolerances BS 5606:1990 (Figure 44). This combines the 10 mm off-grid location tolerance of the column; 5 mm oversize allowance on column radius; and half of the 6 mm length tolerance on the precast panels, set in the Specification and the Concrete Construction Standard NZS3109:1987:

Combined tolerance: $10\,mm \pm \sqrt{10^2 + 5^2 + 3^2} = 10 + 12\,mm\ and\ 10 - 10\,mm$

If a site measure was done, as reported by CTV Building construction personnel interviewed, after the Level 1 columns had been cast, and the same steel shuttering forms were used on each level, then this may have reduced a little.

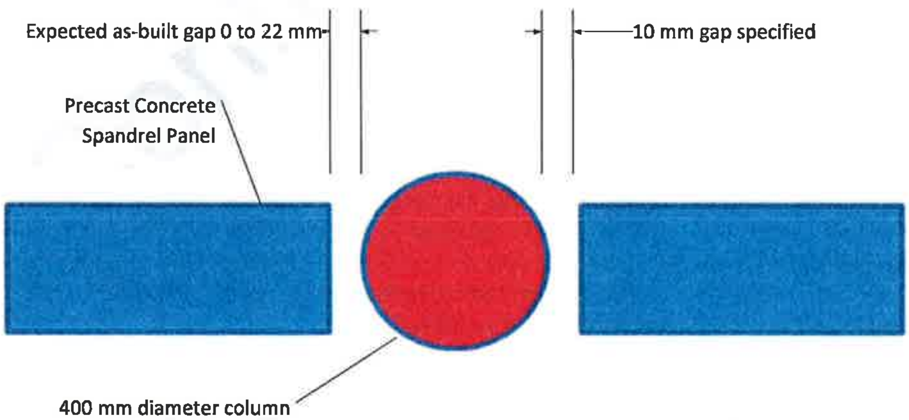

Figure 44 - Expected gaps achieved between Spandrel Panels and columns to achieve a specified gap of 10 mm. This based on BS 6505:1990 guidelines on construction tolerances.

Some of the columns may therefore have had little gap between them and the pre-cast panels. It was found from the displacement compatibility and push over analyses that the effect of the Spandrel Panels interfering with the movement of the columns was to accelerate critical column head flexural/compressive damage. It also could have initiate mid-height hinging of upper level columns which had greater displacement ductility due to the lower axial compression actions on them. Column mid-height and base damage was seen in the beam-column remnant seen in Figure 95.

The displacement compatibility and the pushover analyses also found that column collapse could have initiated without Spandrel Panel interference but at greater inter-storey drifts.

A total gap allowing for seismic drift and construction tolerance of 19 mm would therefore have been recommended if no minimum seismic gap was specified.

A nominal gap of 10 mm was specified between the precast concrete Spandrel Panels on Line 1, 4 and F and the vertical faces of the columns. However the Drawings didn't specify this to be a minimum gap, or that it was required as a seismic separation. This allowed it to be interpreted as an allowance for construction tolerance only.

Beam-column Joints

The beam-column joints on the interior Lines 2 and 3 had very little spiral reinforcing in them to provide confinement and shear strength, and to hold the beams into the joint. This level of detailing is indicative of the joints having been considered to be required to satisfy only the non-seismic design requirements of the concrete structures standard NZS 3101:1982.

The beam column joints on Line 1, 4 and F of the perimeter shell beams were similar. It is conceivable that the lack of continuity steel through these beam column joints meant that the beams were unable to cope with much loss of vertical support as the columns were damaged and failed. Instead of being able to redistribute some of the load along the frame, the beams may have pulled away from the columns, contributing to the progression of the collapse.

Plan Asymmetry and Vertical Irregularity

The main seismic resisting elements were not located symmetrically about the centre of mass. The centre of stiffness of the designated primary seismic resisting elements was significantly eccentric to the centre of mass

The North Core and the South Wall, which was a coupled shear wall, were significantly dissimilar geometrically (NZS 4203:1984 cl 3.1).

The authors were advised that ERSA was used in the original design of the primary seismic resisting structure being the South Wall and the North Core (NZS 4203:1984 cl. 3.4.7.1(c)).

Displacement compatibility analyses of the secondary frames as well as careful interpretation of ERSA results was also required to assess inelastic demands on the structure (NZS 4203:1984 cl C3.4.7.1).

The design calculations that were provided did not include displacement compatibility analysis of the secondary beam and column frames.

Wall on Line A

It seems from the design calculations provided that the Line A masonry infill wall was intended to be separated from the structure as a Group I element from the structure.

Infill walls conforming to the requirements for Group I elements were required to be separated from the structure by twice the K/SM factored inter-storey displacements (NZS 3101:1982 cl. 3.8.4.1(a)). With an upper bound drift limit of 0.83%, at Levels 2 and 3 with inter-storey heights of 3.24 m, this set an upper bound gap of 54 mm. For Level 1 with inter-storey height of 3.66 m this upper bound was 60 mm.

A gap of 25 mm was shown on the Drawings between the masonry infill and the vertical faces of the columns on Line A. The Design Engineers calculations indicated that a partial filling of the top course was intended. However the Drawings did not show any separation between the top of the masonry infill and the underside of the precast concrete beams they were connected to.

The Drawings indicated that the top course was to have no gap between it and the underside of the concrete header beams and be fully grouted. This lack of gap and grouting of the top course would have prevented horizontal slip occurring between the top course and the underside of the beam.

Eyewitness 16 reported that the top courses may have only been partially grouted and some horizontal gaps between the top course and the underside of the beams were observed in places.

Eyewitness 16 also reported that the vertical separation joints between the masonry infill panels and the columns were filled with mortar on the outer face.

No damage to the wall was reported after the 4 September Earthquake by eyewitnesses who had inspected it closely.

In conclusion the authors consider that he masonry infill may have locked up the Line A frame and acted as confined masonry bounded by the precast beams and columns. This would have increased the stiffness and strength of the Line A considerably above what was intended

Diaphragm Connection

No reinforcing steel was specified connecting the lift shaft walls of the North Core into the slabs on DENG Dwg S15 and 16. This omission was picked up after construction during a pre-purchase inspection, and resulted in Drag Bars being installed on Levels 4, 5 and 6.

The quantity of reinforcing mesh in the floor slabs required for shrinkage and diaphragm purposes was marginally less than that required by the Concrete Structures Standard but complied with the recommendations of the floor decking supplier at the time.

The Drag bars that were added were designed following the requirements of the loadings standard of the day NZS 4203:1984. This standard had provisions for the design of diaphragms and their connections. However these provisions have been found from investigation to be insufficient to ensure that the diaphragm connection was sufficient to fully allow for the expected performance of the North Core and South Wall.

This may be a problem with other buildings relying on diaphragm connections to shear walls and designed using the same Standard.

Robustness

The secondary beam and column frames lacked the level of robustness expected of frames designed to cope with the cyclic drift demands of earthquakes. Robustness means the ability of the structure to sustain damage without causing progressive collapse of the building as a whole.

The seismic design provisions of the 1982 concrete structures standard would have improved robustness if they had been applied to the beams and columns.

Documentation

There were a number of design considerations not shown on the Drawings. These included:

Roughening of internal surfaces of some precast shell beams and not others indicated. Refer to the notes on DENG Dwg S18 and sections 1 and 2 on Dwg S22 which don't show roughening compared to similar sections on S20, 21 and 24, which do. The circular formed surfaces at the end of the precast beams where they butt against the beam-column joints would normally be expected to be roughened but weren't shown that way.

The top course of the masonry infill on Line A was shown on DENG Dwg S9 section 6, as fully grouted. However the design calculations indicate that it was intended that this was to be only partially filled to allow some horizontal slip between the top course and the underside of the header beams.

No starter bars were shown extending out of the precast beams on Line 1 and 4 and into the slab (Beams 18 and 22 on DENG Dwg S18). Such bars would be expected to help tie the slab into the perimeter.

The required concrete 28-day Strengths were not shown on the Drawings, but were stated in the Specification.

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

Percentage New Building Standard Assessment

Basis of Assessment

The percentage of New Building Standard ("% NBS") is a measure of conformance of the performance a structure with the current building standards. It usually involves a preliminary assessment, which may lead to higher levels of engineering investigation and analysis.

The % NBS of the CTV Building prior to the Aftershock was assessed in two stages. The first used the Initial Evaluation Procedure ("IEP") of the 2006 New Zealand Society for Earthquake Engineering guidelines. The second stage used the results of the ERSA and the Push-over Analysis to compare the drift capacity of the structure with the 1986 and 2010 standards.

Initial Desktop Assessment

The IEP was completed on the basis of a desk top study for on the reported as-built condition of the structure prior to the 4 September, 2010 Earthquake (Webb 2011). The IEP indicated a large range of potential performance with a lower bound of 44%NBS. The structure was identified as "significantly" irregular in plan though it was recognised that the building should have had been designed for that irregularity due it being designed in 1986. Although greased vertical start bars and separations had been specified in order to reduce the moment capacity and stiffness of the block work, the size and mass of this wall may have added considerable stiffness under lateral loads. The lack of a specified minimum gap between the pre-cast concrete Spandrel Panels and the perimeter columns on Line 1, 4 and F meant that short column effects were possible. The lack of Drag Bars at Levels 2 and 3 were also cause for concern.

Assessment Based upon Indicator Column Drift Analysis

Based on the analyses undertaken as part of this investigation the authors concluded that the CTV Building would have had a %NBS in the order of 40% to 50%. The lower figure is based on significant spandrel interaction and the higher figure on no Spandrel Panel interaction.

CONSTRUCTION ISSUES

Concrete Strength

The concrete strength found in the columns was less than expected. In conjunction with the relatively small size of the columns and lack of sufficient confining reinforcing, this would have made the columns particularly susceptible to collapse.

It is important to clearly communicate the specified requirements in a manner that is easily interpreted by on site construction personnel. Placing the concrete strengths on the Drawings is the best way for this to be communicated.

Concrete strength is known to be influenced by the manner in which it is placed. In this case it was reported that the concrete columns were formed using steel shutters which tend to provide a good environment for concrete placement as water is less likely to leak out. A curing membrane was reportedly sprayed on to the column surfaces after the shutters were removed, so curing should have been adequate. No areas of "boney" concrete where the aggregate lacked adequate cement, were found in the columns examined.

Figure 45 - Concrete from column on Line 4-D/E (C18) showing discolouration from silt.

Cores taken from the Line 4-D/E columns were however found to have traces of silt in them (Figure 45). This may indicate that aggregates and sand used in some of the column concrete had not been adequately washed to remove the silt.

This may have prevented the concrete in some of the columns fully developing its specified strength.

Construction Joints

Construction joints occur at the interface between one concrete pour and another. To ensure good transfer of stresses and to avoid undesirable slip or movement across construction joints it is important to ensure that the surfaces are roughened using prescribed methods in the concrete construction code of practice NZS 3109.

Bent –up Bars

Where precast components are required to be tied into insitu concrete such as shear walls or columns, there is often the potential for reinforcing steel to have been located sufficiently out of position so as to make it difficult to install the precast item correctly.

Care needs to taken in such circumstances to contact the design engineer and determine a way to develop a solution that will satisfy the design and construction difficulties encountered in those situations. If this is not done potentially dangerous situations may arise that could compromise the capacity of the structure.

Separation of Elements

Where separations are required for seismic purposes it is important that these are carefully constructed to ensure the minimum gap is achieved and maintained during the life of the building. Relatively small differences in gap can lead to the performance of buildings being seriously compromised in earthquakes.

Conflicting requirements for seismic separation and fire sealing need to be carefully managed to ensure both hazards are adequately allowed for. Construction personnel may need at times to identify where deficiencies in design documentation coordination and specification between earthquake and fire engineering consultants may be in conflict.

CONSTRUCTION SUPERVISION AND MONITORING

The apparent deficiencies in concrete strength, construction joints, bent-up bars and separation of Spandrel Panels and the infill masonry wall on Line A, is a reminder of the importance of the need for confidence that:

The building has been constructed according to the drawings and specification.

The design intent has been interpreted correctly and followed through.

This requires effective quality assurance measures to be developed and implemented during construction. This includes having appropriately trained and qualified personnel undertaking the work, supervision by the builder, approvals and audits by the BCA, and construction monitoring by the design engineer and architect.

10 CONCLUSIONS

The investigation found that the damage to the structure observed and/or reported after the 4 September 2010 Earthquake and the 26 December 2010 Boxing Day aftershock did not indicate significant weakening of the structure.

The estimated response of the building using the full 4 September Earthquake ground shaking records and the assessed effects on critical elements don't appear to be fully consistent with observations following the 4 September Earthquake. Analyses using the 22 February Aftershock ground motion records indicate displacement demands on critical elements to be well in excess of their capacities. It was also found however that significantly less than the full record was required to develop critical collapse initiation conditions along Line F, particularly if Spandrel Panels prevented free movement of the columns.

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

- The strength of shaking indicated by the February Aftershock ground motion records and spectra were easily sufficient to cause displacements which were higher than anticipated based on the computer analyses. However the computer models also indicated that there was sufficient shaking to have severely damaged and probably collapsed the building in the September Earthquake.
- The vertical irregularity produced by the influence of the masonry walls on the west face up to Level 3.
- The plan irregularity of the earthquake-resisting elements which further increased the inter-storey drifts on the east and south faces.
- The lack of sufficient separations between the perimeter columns and the Spandrel Panels reduced the capacity of the columns to sustain the lateral building displacements
- The low amounts of spiral reinforcing in the columns
- Low concrete strength likely in some of the critical columns
- The effects of vertical earthquake accelerations

Surveys of the site after the collapse found no evidence of vertical or horizontal movement of the foundations. There was no evidence of liquefaction.

11 RECOMMENDATIONS

The performance of the CTV Building during the 22 February 2010 Aftershock has highlighted the potential vulnerability in large earthquakes of the following:

1. Geometrically irregular structures that depend on a primary structure may not perform as well as structural analyses indicate.
2. Buildings designed before 1995 with non-ductile columns may be unacceptably vulnerable. They should be checked and a retrospective retrofit programme considered.
3. Existing buildings with part-height pre-cast concrete panels (or similar elements) between columns may be at risk if separation gaps are not sufficient. Such buildings should be identified and remedial action taken.
4. Buildings with connections between floor slabs and shear walls 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.
5. There is a need for improved confidence in construction quality. Measures need to be implemented which achieve this. 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.

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APPENDIX A – EYEWITNESS SUMMARIES

INTERVIEWS WITH EYEWITNESSES

Interviews were undertaken with those who were willing to speak of their experiences and what they observed. The names of the witnesses are not revealed for privacy reasons. Their locations are shown on the Eyewitness location map (Figure 46).

Some were inside the building at the time; others were in the street or in other buildings next door with a clear line of site to portions of the CTV Building as it collapsed.

The information gathered from the interviews has been collated into common categories and summarised to identify consistent observations for further technical analysis.

EYEWITNESS LOCATIONS

Eyewitnesses inside the CTV Building

1. Level 6: East side of the southwest corner.

2. Level 1: Ran south out from Reception on the East Side of the building.

3. Level 4: North at the right edge of the building.

4. Level 6: Sitting on the side wall next to the demolition site; farthest away from the front area.

Eyewitnesses outside the CTV Building

5. Les Mills building.

6. IRD building.

7. IRD building.

8. In front of CTV driveway on Cashel Street.

9. Unrestricted view from roof of Les Mills building.

10 & 11 Blackwell Motors on Madras Street side opposite CTV.

12 & 13 IRD building.

14. On east side of CTV on Madras Street just past Samoan Church.

15. In front of CTV driveway on Cashel Street.

16. Working on the re-cladding on the CTV at south west corner of CTV building.

EYEWITNESS LOCATION MAP

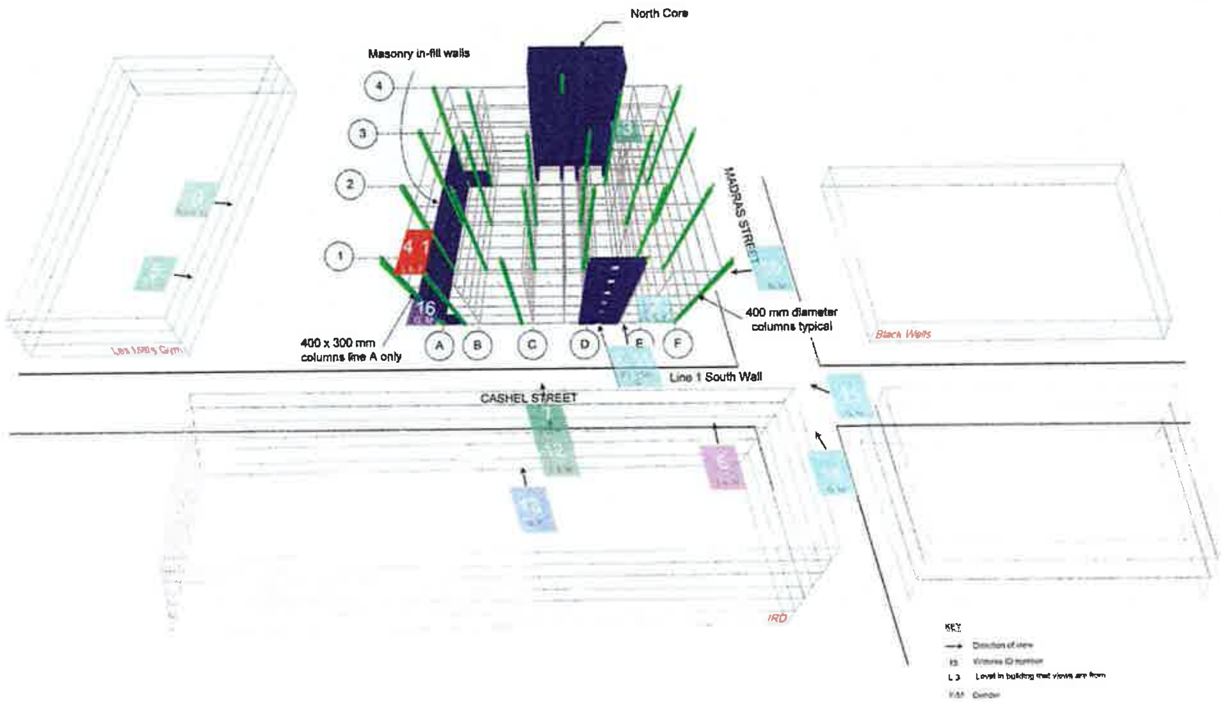

Figure 46 – CTV Building layout and eyewitness locations.


CTV
CTV
9
7
6
12
5
3
2
1
8
15
11
View from south-east

continued

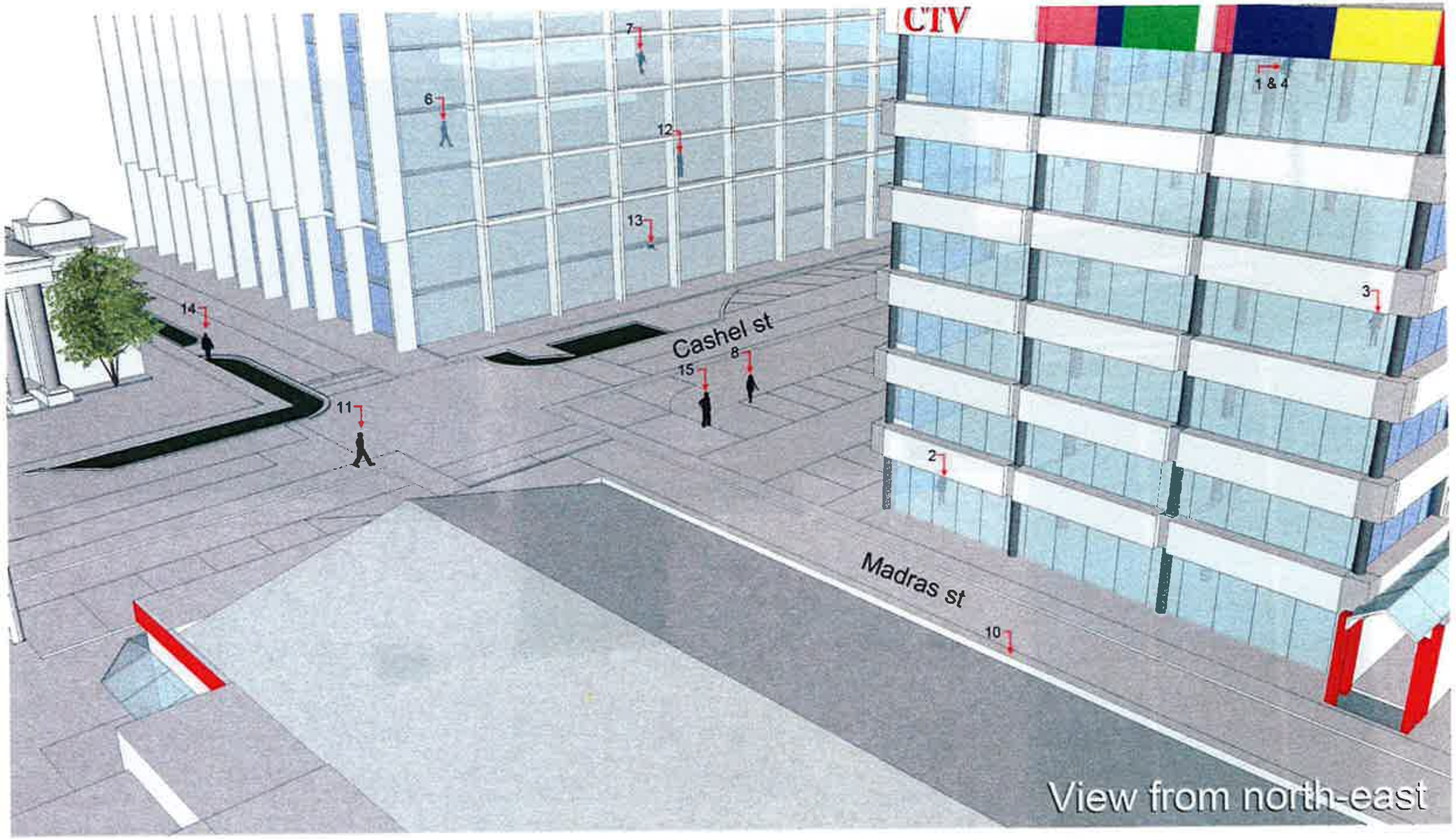
CTV
1 & 4
7
6
12
13
3
14
Cashel st
8
15
11
2
Madras st
10
View from north-east

Figure 47 - Eyewitnesses located on perspective views around the CTV Building.

INTERVIEW SUMMARIES

Eyewitness 1

Eyewitness 1 was in the eastern side of a room on the southwest corner of Level 6 at the time of the earthquake. (See Figure 46 and Figure 48)

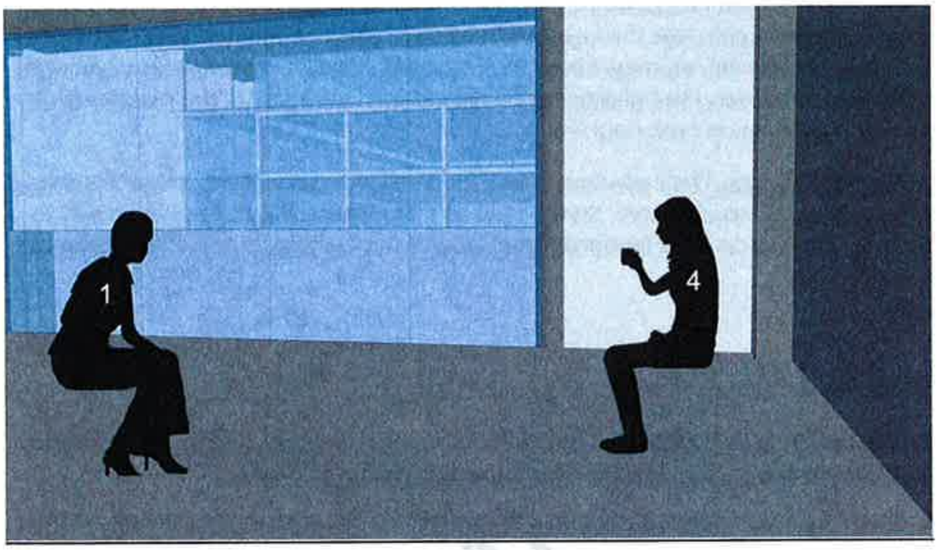

Figure 48 - Eyewitness 1 and 4 locations in southwest corner room on Level 6.

She described the quake as a sudden violent lurch – a continuous movement. "Then the building just went joo-joo-joo-joo, and just did not stop. I just felt like we'd gone really far forward and then just kept moving all the time continuously." This she described as not "after" a first jolt, but it being the whole jolt. When it was over, she was on the floor and the ceiling was on her "so what part would have come down first? It would have to be below us – as we just "came down", like floating down. "The whole ceiling collapsed in on us and most of us; in fact all of us I think were pinned to the floor."

- *Direction of fall.*

Where she was there was not a sensation of the floor falling down, more a "sense" of tipping. After the lurch she was "pushed back a little". "A feeling like I was moving in that (east) direction – and then there was just lots of movement, and during that movement the glass exploded on the Cashel Street side. People and furniture also slid towards the eastern wall. One of her colleagues also felt that the whole building was tipping over but she commented that he was standing and she remained seated and felt it differently. For her it was more a sensation of continuous movement and slight tipping.

- *Time frame.*

She said "I am being generous in saying the building was down in less than 30 seconds. Some of my colleagues say it was much quicker than that."

• *Pre-earthquake observations about the building.*

Continual vibration during next door demolition. This eyewitness commented on the demolition that had been going on next door, since September 4th. Some staff had found the continual vibration in the building distressing, particularly in light of nerves around the aftershocks. She referred to a huge vibration on the day when the last part of the demolition occurred. "One day there must have been a wall that either backed on or was semi attached to the back of our building – when that came down a huge vibration went right through the building." She commented that when the demolition ended she returned from the Christmas holidays thinking the vibrations would end. However, the building still continued to vibrate from "the machinery or whatever was going on next door."

Cracks in the lift area. This eyewitness reported what she described as major cracking in the corners by the elevator. "It was cracked from the ceiling all the way down to the floor. This was on the Hereford Street side of the building, at the intersection of the walls.

Eyewitness 2

Eyewitness 2 was on Level One, Reception – running out south from the front door (east side) straight across Madras Street towards Blackwell Motors.

She described the noise and impact of the quake as like a jet plane landing on the roof. "The whole, all the glass, everything was going. The noise was unbelievable. I ran for my life thinking the building was going to get me on the way. I knew it was breaking up. I ran for the doors, everything was coming at me; you know all the windows coming in. I just got through the door. There was no one else on the ground floor at the time… all our other staff were on the first floor and they did not stand a chance. I knew I was the only one that got out, because I knew what was coming down around my ears as I was running."

• *Direction of fall.*

When this eyewitness turned around she was on the corner of Madras and Cashel. She did not actually see the building fall; by the time she got there the building was down. "The building had just pancaked – six floors were down to next to nothing." Inside it had felt "like being pushed around all over the place".

• *What the ground shaking felt like.*

During the aftershocks, when she had made it to Latimer Square, she described the ground as like "jelly". The road was "going up and down… horrific."

• *Time frame.*

"Fortunately I was standing by my desk when it happened. I would not have had time to get up from my chair. By the time I ran across the road really fast and turned around, the building was down. A matter of seconds really. Then, there was another big aftershock and a whole wall of the Samoan Church collapsed over into Madras Street."

• *Pre-earthquake observations about the building.*

The eyewitness commented on the drilling that had been going on inside the building before the earthquake. Every now and again we would get a boom-boom and a shake ...no one felt safe in that building. They had already taken a building down next door so I don't know why they were drilling into the side of the building."

The eyewitness also described how in an earlier small earthquake the girls up in the sales office were shaken about it – yet she did not feel it on the ground floor.

She also remarked that she could not remember what the inside staircase (which was right beside her desk) was doing during the earthquake. However she remembered that in a previous 5 earthquake it was like "the whole thing (staircase) was swinging towards me." She could not recall seeing any damage from the two earthquakes before 22 February.

Eyewitness 3

Eyewitness 3 was on Level Four – north on the right edge of the building.

She described her first experience of the earthquake as, a bounce – a jump and then everything moving. She refers also to a second sensation of a definite drop." The analogy I've used in describing how it felt, is being on an ice rink in flat shoes. Completely just spun from one side of the wall to the other. Then you realise that it wasn't just going to shake, and it wasn't going to stop." She remembers moving towards the underneath of her desk. Then everything went black, everything sort of stopped. The sensation of dust, not being able to breathe. ....the weird sensation that you weren't level, on a slope. I put my hand in the air and realised that the ceiling was actually resting on the top of my desk. Then there was a second movement – a definite downward movement, it went like "choooomf – like on a seat when you drop. She could not be 100% sure of the movement between the first drop and this, as it was already moving..... everything seemed to be dropping constantly – very disorientating – but the second drop feeling was a definite. You suddenly thought "whoa!"and things went downward more."

• *Direction of fall.*

Initially this eyewitness was thrown one way and back again. Against the eastern wall and then thrown back to the west and back east again against her desk. She then got under her desk. "The first initial shake was when it went" – then a feeling of what she called a second drop that where she felt like she was on a "slope". She said "I was pushing with my heels, you felt like you were pushing up hill." First when she was under her desk she had room, but in the second stage "I was sort of on my side."

• *Time frame.*

"It seemed like a long time." But she felt unsure of time – "to be honest, time just – it was very bizarre."

• *Pre-earthquake observations about the building.*

"They demolished a building here behind us – starting pretty much when I started work in the October. "All I do know is we bounced constantly while the digger work was going on. They finished a week before the quake."

Her understanding was that when they took the building down next to the CTV they left a single layer of brick with no bracing. The building that came down only went up to level 3, below level 4.

Eyewitness 4

Eyewitness 4 was inside the CTV against a side wall on Level 6 that comes out to Cashel Street just in front of the IRD (Inland Revenue) building. The side next to where the demolition work was. (Figure 48)

"Usually our meeting would have been in the middle of our premises – but on this particular day we were sitting furthest away from the front area. This decision pretty much saved our lives. I was strategically in a good place because I had no obstruction to access to a door frame. We all eventually came out in the car park. I just felt this "chooo" (vertical feel) a bolt, a "thump" that almost propelled me off my seat. I was like a rocket under the door frame, my colleague and I together as we had rehearsed many times before when the demolition work was really bad. I held on to this flimsy little aluminium doorframe. I was standing up and felt a real sharp jolt from underneath."

- *Direction of fall.*

"I felt a bolt upwards at first, and then it started going sideways. Initially it was really strong with the bolt underneath, like this was very, very fast, real fast, up and down, and then it was swaying, and then it all collapsed, collapsed, collapsed." It started with the usual thump of an aftershock and then accelerated from there. "So there was a thump and I was already under the door, others were still sitting." She felt that she was in line with the doorway as it fell, not sort of falling out of it. "There was a real lion kind of noise, roaring – like cracking. One thing I noticed very quickly was the pink batts coming down on us, so the ceiling must have given pretty soon. The pink batts were the only thing that fell on me. Whatever was collapsing like the other walls caving in, they were just kind of collapsing and nothing really fell on me because everything fell against the frame. Then I remember a little bit of tilting (not steep) to the back from the ground (toward Cashel Street.) It was not much; it wasn't like I had to hold on. I was still standing when we were down 5 floors. I did not have the sensation of freefalling. When it came to a halt I thought we had just come down one floor. When I looked through the open ceiling out I thought I was still high up – then realised oh my God, we're just a metre off the ground…. I was totally surprised that the floor on my side was still in one piece. Nothing had come through." The partition wall she was up against, on the east side of her meeting room side stayed vertical all the way along. On the southwest corner of the floor were the worst injuries. When referring to the tilting of the building – she described it as a slight diagonal lean towards Cashel Street corner demolition site. "My sense is that when the whole building went up and sideways and just went "shhhhoooo" down, leaving the lift shaft still standing. Being in the top of the building where I was saved me. So much more damage happened in the middle part of the building."

- *Time frame.*

"I could not see anything, you know, because the whole walls caved in and – like it was all blocked within seconds, seconds. It was amazing how quickly people stepped into the rubble and got us out, and then the fire broke out in the lift or lift shafts."

• *Pre-earthquake observations about the building.*

This eyewitness mentioned a fear amongst some colleagues that the demolition work was perhaps weakening the building. It was her feeling that it was undermining the building. "This is only my sense, it is not a science."

There used to be two big building complexes next door, and the one adjoining the CTV Building was taken away. Around two weeks before the earthquake they had just freed the area of the building.

"I was right on the outside (of her floor), and when the demolition happened the big diggers, whatever you call them, were pulling that wall. It made a shudder. I don't know for sure – but when they took the building next to us down, I believe it had at least some parts attached to our building." (Lower than her level.)

She described the demolition going on from September to February. On the day of the earthquake they were still coming in with big machinery, flattening it to turn it into car park. "There were constantly machines, and stuff coming down and falling down. Big huge chunks of concrete were just falling to the ground. You could feel it all the time.... Then there were the aftershocks as well. They were horrible as the whole building was just going big sway, big sway."

My sense was "my God, this building is constantly exposed to quite a lot of stress... I thought we're not safe in here...it's not okay, part of it."

She also mentioned that even before the demolition of the building and before the earthquakes when aerobics classes were happening at Les Mills "our building was vibrating." "The outside wall was never very thick I felt."

When asked if she noticed any damage in the building getting worse subsequent to September – she made this comment. "Right at the lift shaft, these big pillars. I noticed like a bigger crack around, I think, the pillar closest to the lift. There was another one – the pillar was intact, but just alongside there was a crack (she moves her hand in an S shape) which just went down." She had hoped when she saw them, that they were just superficial.

Eyewitness 5

Eyewitness 5 was in the Les Mills building next to the CTV Building on the 3rd level (2nd level of the Les Mills gym)

"I was directly opposite (just 10 metres away from) a large window that you could see the CTV Building through. When the earthquake struck I remember turning around and then seeing the CTV come down through the window. I could not see the top of the CTV Building. I saw a portion of it then it all came down. I don't think I could see from edge to edge, but I saw a lot of it…."

- *Direction of fall.*

"I saw the collapse. It was just almost like a level gave way and it just went - whooompf. It was like one of those controlled demos on TV. It was just straight down – and then after when I was down at the site helping out (and as you can see from the TV images) it was really compact, the rubble and that…" The eyewitness found it hard to describe the feeling that its almost like a level was removed and it just all came down. He did not actually see a level collapse – it was just the way it all went down.

- *Time frame.*

"It just fell really quickly. Like ploooop. A couple of seconds. I was on the heavy bags facing away from the window maybe seven, 10 seconds passed as I stabilised myself. I turned around and then another few seconds, then saw the CTV Building come down. The first thing I saw was it coming down." The eyewitness was definite that the CTV was down during that first Aftershock, the first tremor. A big aftershock happened minutes after when he was outside Les Mills, and he saw the scaffolding on the Samoan Church come down.

- *Post–earthquake observations at the site.*

He was standing at the front, Cashel Street side. "Everything was just so compact. I remember I just could not believe it was a five-storey building. It was just so tight, the pile, real compact. It was deep down I think the fire. I think it must have caught like this – there were pink batts around, so it must have caught onto that. It was real smoky because the corrugated iron was on top of it. When the digger pulled back some corrugated iron, you did see flames come up."

"Part of the building was still standing. I remember the CTV sign was down." On the Les Mills side, he also remembered seeing the pink batts, and corrugated iron type stuff, sheeting, along the wall. "There were tons of massive puddles, craters with puddles in the graded part between Les Mills and the CTV. There was also a crack in the street where water was flowing out."

Eyewitness 6

Eyewitness 6 was in the IRD (Inland Revenue) building on the third floor as the earthquake hit. (Figure 49).

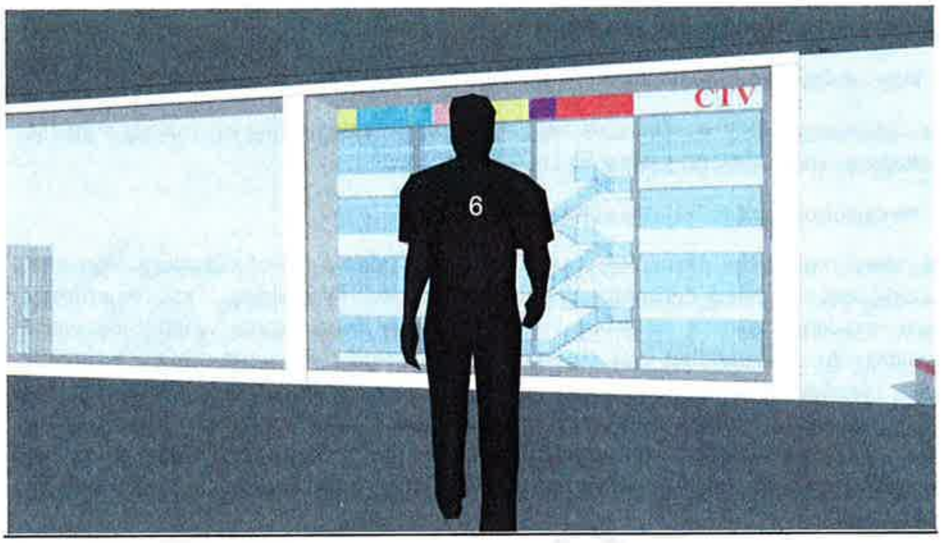

Figure 49 - Perspective of Eyewitness 6 in IRD building

"I was standing looking out the window at the time that it collapsed so I could see the top half of the building. It started to collapse a few seconds into the quake and what I could see was the top started leaning towards the east, and then basically it just collapsed straight down."

- *Direction of the fall.*

"It was just a slight lean, and it went down vertically. Then we had white dust come up so all we could see for a few seconds was white dust against the windows. Then the Samoan Church opposite us fell down." The third floor of the IRD was the fourth level, so he could see at least the three top levels of the CTV Building. He had no recollection of floors falling into other levels and said "it almost looked like it came down in one piece. It looked like there was something coming up which may have been dust. I was focusing on the top of the building and that, from what I can see, it was going down as a unit."

He pointed out that there seemed to be nothing breaking at the time, but said "I cannot swear to that.... It just looked like something happened below and it was coming down. I did not see anything disintegrating in my field of vision, so whatever was happening was happening further down." Then there was the white out – he could not see anything through the windows at that point. Before the white out, he also recalls a momentary dark flash – but could not tell what it was. "Whether it was smoke or dust or lower floors breaking up, I could not tell. That was only momentary."

• *Time frame.*

The time that this eyewitness felt the first ground movement to the time when he saw the CTV Building collapse was described as seconds. "It would have been a few seconds, but time's pretty elastic in those sorts of things. It probably seemed longer than it was, but it was a few seconds.

• *Post- earthquake observations of the site.*

His observations of the site were few, as he was concentrating on making sure his colleagues were safe, and getting to Latimer Square.

• *Pre-earthquake observations about the building.*

He noted that in the preceding weeks there had been a lot of vibrations from the building that was being demolished next door to the CTV Building. First "when they were knocking down a wall, but I think probably even worse when they were breaking up concrete that was set in the ground and they were using a wrecking ball." He described them as being "like point three earthquakes or something like that – we weren't feeling them, but we were feeling the shocks from the wrecking ball. We'd get vibrations in our building quite often. They were breaking up the concrete approximately one week before the quake, and it was going on for two or three days."

Eyewitness 7

Eyewitness 7 was in the IRD (Inland Revenue) building looking out the window on level five. (Figure 50)

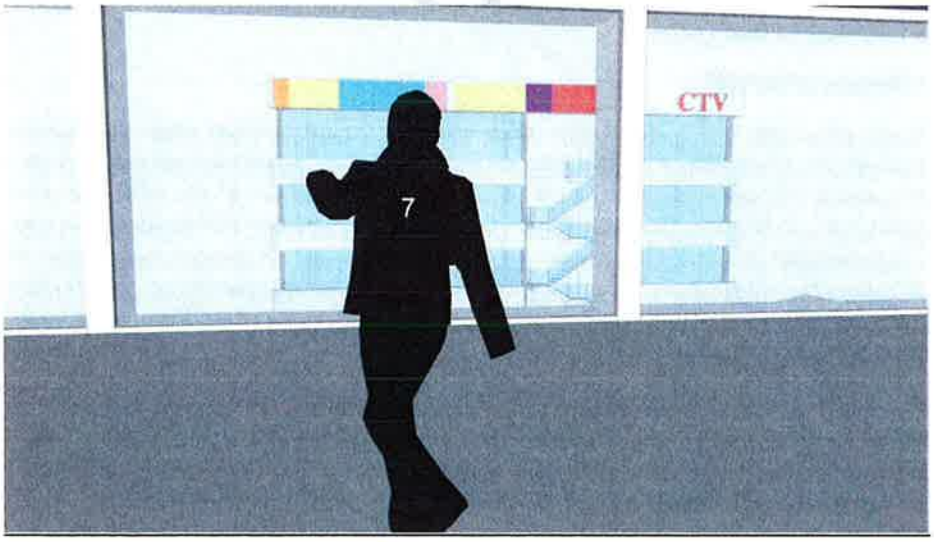

Figure 50 - Perspective of Eyewitness 7 from IRD building.

During the earthquake this eyewitness was under his desk as much as he could be. He was situated right next to the window, and had a full view of the CTV Building. "I could see out to the empty lot and down the side of the building, and I could see the car yard, and yeah, the edge of the entrance way to the CTV reception. I could see the whole lot." He was looking out the window as the CTV Building came down. "I've been in the IRD building plenty of times during aftershocks and I'd never really gone under my desk, I had not felt the need to. But this was quite different. It was super violent, so I was under my desk immediately and it got more and more violent. It was not just a shake. It just kept going with intensity and I was being bounced out of my desk and back again. I don't know how long it went on for but it just stopped, suddenly. There was quite a bit of noise in the office and people upset and so I stood up to call my team together and then looked out the window, and then the CTV Building came down."

• *Direction of fall.*

"A flash of the CTV Building and then it sunk into the ground – you know like the 9/11 buildings, exactly like that. The top floated and was engulfed by a cloud. I probably wasn't even aware that the building had collapsed because it looked like it was engulfed by dust - I realised because we were still in the building for about two or three minutes afterwards that suddenly it was gone. You could just see the lift well…..I don't know how it just sheared off that." His overall impression was that it disappeared – "like sinking into this cloud," "Pretty much as a block."

• *Time frame.*

This eyewitness described seeing the building fall after the quake had stopped, after he stood up from his desk. He felt it did not fall immediately. It was after the earthquake. However he mentions in the same segment of interview that "I've lost some moments in time."

• *Post-earthquake observations about the site.*

The eyewitness remembers the lift well standing, and people helping to lift rubble off with some digging machinery that was on the site. "I ended up over at the lift well at some point where I was fighting a fire. I remember getting to the CTV Building and then suddenly I was on top of the building, so how I got there I do not know, but I was helping get people out for about seven hours or so. "

"I had the expectation that it (the building) would be all over the show. But It fell into a complete square. I mean essentially a seven-storey building had compacted into something that was less than the height of this floor to ceiling." (Referring to the interview room.) Also, "as they were pulling people out, the majority of them were from the 5th floor and they had no idea that the building had collapsed. It was a real shock for them to feel 'how am I on top of this building?' "

• *Pre-earthquake observations about the building.*

"The building next door to the CTV had been damaged in the September earthquake and they were pulling that down. I heard that they were going to turn the land into a car park so they were making it quite flat. So there was a lot of heavy drilling and a lot of demolition ball stuff going on, and often we would be in the IRD building and it would feel like there were tremors – so it it was shaking the IRD building I can only imagine it was having a similar effect on the CTV and the buildings around it because it really felt like the ground was shaking with the work they were doing there."

He recalled no damage to the building being evident after the September and Boxing Day quakes.

Eyewitness 8

Eyewitness 8 was crossing the road to return to the CTV from lunch. As it happened she was standing in front of the CTV driveway. (Figure 51)

Figure 51 - Perspective of Eyewitness 8 in front of CTV Building on Cashel Street.

This eyewitness was halfway over the road with a colleague (in front of the CTV driveway) when it happened. "There was this massive jolt and we grabbed each other. We were looking around us as it started shaking really really bad – and the CTV Building was just sort of swaying back and forward, and the IRD (Inland Revenue) building too – we were looking back and forwards going " which one's going to go" because the IRD windows were coming out just like jelly.... All of a sudden, I think it was the fourth floor (level 5) of the CTV Building just gave way. .."

- *Direction of fall.*

"...The pillars,- like all the glass shattered, and then just - I think the pillars just gave way on the outside. (Moved outwards). And then the fourth floor (what the interviewers call level 5) came down and hit the next floor down. It sort of stopped for a like half a second, and then it dropped again to the next floor down, and it just continued that way down to the ground. But the fifth floor (Level 6), I am pretty sure that stayed intact until it hit the rubble at the bottom." She raced to the rubble at this point – into the car park straight up to the front of the building. "My colleagues car was parked in the second car park across from Madras Street right in front of the building, and it had only really knocked it's bumper off. It came down that straight, it was absolutely crazy. Then I ran around the other side of the building, the back of Madras Street, yelling people's names and stuff. We found a couple of colleagues – but apart from that I do not remember much. But yeah – the fourth floor (Level 5) collapsing...."

• *Time frame.*

The collapse of the building seemed to happen in seconds. . Level 6 dropping as a unit onto Level 5, and then Level 5 onto the ones below. "It must have been the roof that sort of collapsed into level 6 when it hit the rubble."

• *Post-earthquake observations about the site.*

The eyewitness remembers the lift shaft still standing. Also when the building collapsed all the water pipes burst and there was water pouring into the pile of rubble. "Some of it's a blur."

• *Pre-earthquake observations about the building.*

This eyewitness did not bring up any comment about the building before the earthquake.

Eyewitness 9

During the earthquake this eyewitness was on top of the Les Mills checking the air conditioning units – where there is a 360 degrees view of the city. The Les Mills building is four stories so he could not quite see the roof of the CTV, but had a straight view of the collapse. Since a building had been recently knocked down, there was just open space between the CTV and the Les Mills building, so he could see the whole of the west wall. (Figure 52)

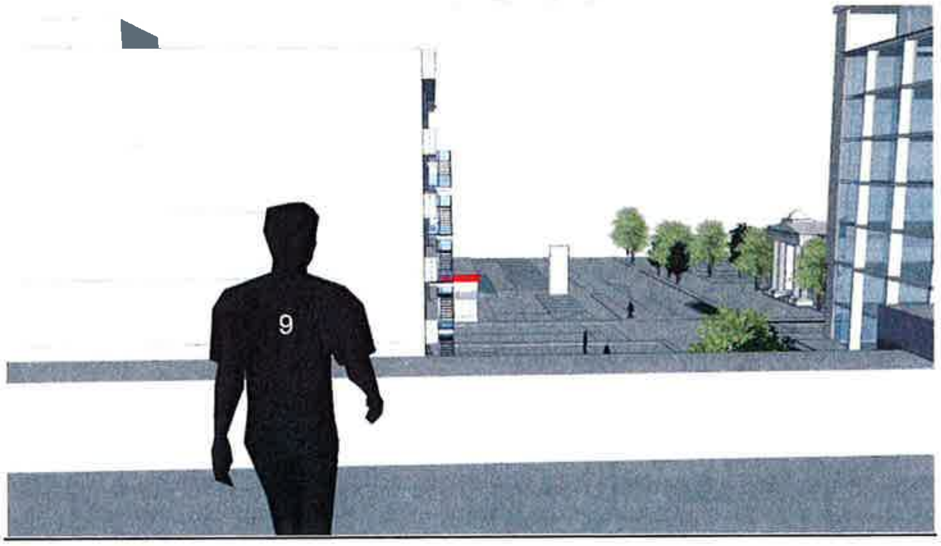

Figure 52 - Perspective of Eyewitness 9 on roof of Les Mills' gym.

• *Direction of fall.*

This eyewitness described it as very harrowing to see the building go down. At the time he was between the air con units. "It was a very vicious punch, as if someone had punched you in your back off your feet, trying to lift you up. I was bashed around a bit and turned around and I was steadying myself on the ends of the units – and I saw the building was there one minute and it just sort of crumpled before my eyes. I can see right down to base level, to ground car park level, and it just folded at the bottom like a pack of cards. The first floor folded in, the second floor followed it milliseconds later, and then it went down like somebody had kicked its legs under it. The fire escape stood up for a few seconds longer. The corner the cherry pickers were working on (south west) just crumpled like a piece of paper. It was just like it were a chair leg and someone had kicked that corner and the whole corner caved, sort of folded under itself and then the next piece. The corner had gone so there was no support."

His experience of the earthquake was one of being lifted, then dropped, then kicked again from all directions. "So if that building did lift up and that got knocked on the next wave, then there was nothing on all that frontage to hold it up. The sheer weight of it brought it down."

"The frontage of the building came away, I presume, because there was nothing attached to the lift after the third floor (Level four) upwards. It just ripped away because you could see the lift shaft, the lift doors, everything. To clarify further – when climbing on the rubble, it wasn't flat, it was at an angle. It spread itself in a line."

The image that sticks with this eyewitness the most is the Cashel Street fire exit stairwell corner disappearing in front of him, and the rest coming down. "It was weird just to see a skeleton for a few seconds; I'd say 5 seconds tops, of the fire escape standing and then sort of crumpling underneath it, because the next floor pulled it down. But it just stood there - and you think that's physically not possible – unreal."

• *Time frame.*

All this happened in seconds "whoof – boomf" and then there was one big cloud of dust and in the corner of the thing there was smoke starting to come up. "I don't think 30 seconds passed by the time it was all over the place, fire alarms, chaos. He did not get the feeling of two shocks. Just the one that went with a bang – and then the sensation of loads of aftershocks. "It is quite possible there were a lot of shocks rippling back." He saw what happened in front of him in the space of a few seconds - then his concern was people.

• *Post-earthquake observations about the site.*

This eyewitness was involved for hours on the site helping get people out. He noticed:

<u>Pieces of the fire escape stairs</u> "You could see sections of the fire escape (Cashel Street end) still left in pieces as it had fallen on to the rubble". He felt that if the building had pancaked (for him meaning 'come down as one') it would have pulled this down, and it would have been twisted metal – "but it wasn't…."

<u>The contained way it landed:</u> "It is amazing that there was not a brick or anything in the car park area, and so many cars parked close by untouched, just covered in dust. The building fell into itself, and was all contained within that area. The building came down within its own space, its own footprint. So if it had come down flat, everything would have spread out. You try and put something down that flat, something's got to go left or right, but it didn't. But as I say, with it coming in on an angle, it was all

still stuck underneath, so it had somewhere to hold, and it just sat on itself – like a big pile of bricks."

The Fire: He did not expect the building to burn as he thought "it's all concrete – it's not going to burn." A big machine had been left there which he used to get the fire people into the lift area with breathing apparatus. The smoke was black and acrid by that time.

Liquefaction: "There was a lot of liquefaction, not around the CTV itself, but where the knocked down building used to be there was a great big hole opened up – and water bubbling up. The liquefaction was all along the front of the Les Mills building, it was pouring in the front door."

Samoan Church coming down: The eyewitness was already helping on the site when the Samoan Church went down.

• *Pre-earthquake observations about the building.*

'The building that was demolished between Les Mills and the CTV was finished on the Friday before the quake. I don't think that would have weakened the structure much. Cherry pickers were in, lads with battens, and they were battening the wall all the way up to put new cladding up the wall. White cladding which was nice. They were doing a good job and they were doing it safely. There were two lads on the end platform that day, and had gone away for dinner. The new cladding crushed it flat."

Eyewitnesses 10 & 11

When the earthquake happened these two colleagues from Blackwell Motors were on the Madras Street side of the CTV Building. Eyewitness 10 was getting into his car parked opposite the CTV and outside Blackwell Motors. Eyewitness 11 was in the Blackwell Motors building. Eyewitness 10 saw from the ground of the building (Figure 53). Eyewitness 11 was looking to the skyline (Figure 54)

Eyewitness 10 had just got into his car parked directly opposite the CTV Building outside his work premises. "I basically shut the door and when the earthquake hit, I actually thought that someone had hit my car initially. Then I realised 'well no' as I watched the parapet from Blackwell Motors fall down beside me. Realising I was not in a good spot, I leapt out of the car to run, then as I turned around to shut the door, I looked across the road and realised 'that building (CTV) is going to come down.' I jumped back in the car again and remember thinking 'this isn't going to be good' then got out of the car again. Then I looked across – it went and it seemed to drop in the reception corner. So it seemed to drop and almost fall around in on itself. Then I remember a couple of our staff (including Eyewitness 11) there disappearing off into the rubble."

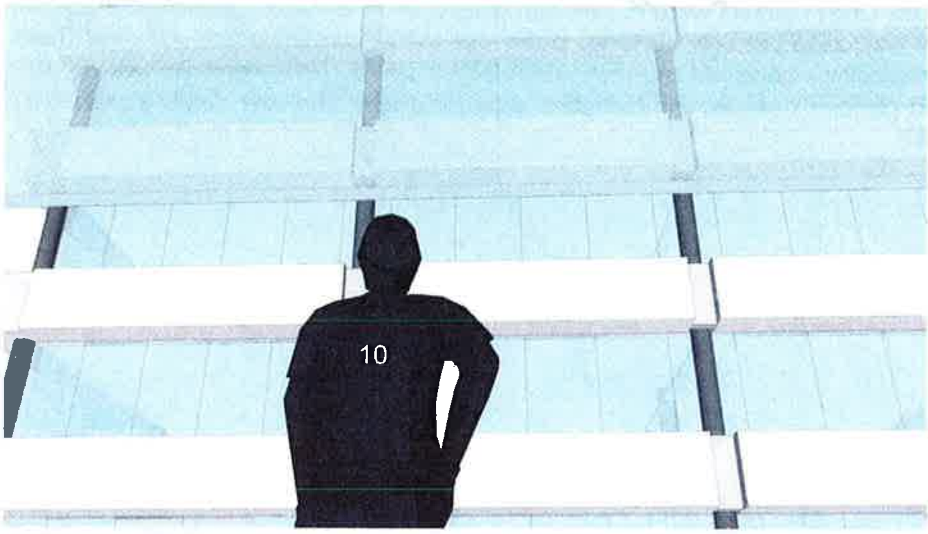

Figure 53 - Perspective of Eyewitness 10 from Madras Street.

Eyewitness 11 ran out of the Blackwood Motors building at the time of the quake. When he was in the yard he remembers seeing the Samoan Church fall apart, it was opposite him on diagonal. To him the quake seemed to go 'boom boom boom' for quite a while. He remembers "there was an aftershock and a dome fell off the top of the church right, and landed in Madras Street. That's when I heard this screaming around the side, and I started running around (wiping the dust out of my eyes by this stage) to see what the screaming was. It was a lady in the middle of the road, and as I came around the corner.. I looked up and I could see the top of the building coming down, and it seemed like it was falling away from the lift tower…but I could only see the top of it. I didn't look at the bottom. "

Figure 54 - Perspective of Eyewitness 11 from corner of Cashel and Madras Streets.

- *Direction of fall.*

According to Eyewitness 10 there was definitely a movement downwards. "I'm pretty confident in saying it was the reception corner. (Madras /Cashel end). From my point of view that bottom corner went first, and then it kind of wrapped itself around and down. So when I was sitting in the car it went 'bang' that way (a bit to the east) and almost fell, sort of down then over. Then obviously as it went down, the rest of it went whooooo through.

Eyewitness 11 saw the skyline of the building coming down and it looked like the whole lot was just" booomf boomf boomf" down. He did not see the bottom.

Although Eyewitness 11 felt like it was kind of a blur in some ways – both eyewitnesses felt that the building did not fall down straight away, but in an aftershock. Eyewitness 10, unlike Eyewitness 11 saw the building from the ground – and it seemed to him like it dropped in the reception corner.

- *Time frame.*

These eyewitnesses seemed to experience time differently. For Eyewitness 10 things went very quickly while he was watching the building, for Eyewitness 11 – there was a sense of slow motion. "I looked and I thought – that buildings coming down – and then it was just coming down slowly. Yeah well it seemed like it was coming down slowly. It probably wasn't but it seemed slow at the time. There was so much going on – I don't know if I actually saw that or I actually imagined that. I think something had obviously come down before I got around the corner."

- *Other observations.*

One of their colleagues, standing in the yard – could see waves, the waves coming from the ground in the tar seal; Coming across from the hill, south to north coming across the car park. Eyewitness 10 also remarked that other than being dusty, his car was unmarked, no damage despite being so close to the collapse. Eyewitness 11 also commented on the swaying of the IRD building during the earthquake.

Eyewitness 12 & 13

When the earthquake happened these two people were in the IRD (Inland Revenue) building. Eyewitness 12 was on the 2nd floor (Level three) of the IRD right against the window with an unobstructed view over Cashel Street (Figure 55). Eyewitness 13 was on the ground floor (Figure 56) . She could not quite see to the top – but could see the width of the fall of the building's western wall .(To the side of the CTV rather than right in front of it.)

Eyewitness 12 "It happened so quickly, but the impression I had was that the wall facing the IRD building, the Cashel Street wall, seemed to fall away first – come towards the IRD building, then very quickly afterwards the rest of the floors just seemed to pancake or concertina down towards the same general direction. Leaving just the back wall that seemed to just be standing there with parts of the floor at various stages." His impression was that when the front of the building came towards him, it was actually coming off. "Not the building, just the front piece, and then everything cascaded behind it… It did not actually spill too far towards IRD, it just virtually went straight down I think. The front must have collapsed because I mean I saw floors up behind it, so that suggested to me that the front had just fallen off. And then I saw the rest of the floors. I could see the top level because it had sort of pancaked out. I could see people on the top level sort of trying to get off it – I do not know how high it was, 5 levels? (There are 5 suspended levels). It probably went down to about two levels in height I guess in real terms, and I remember seeing the back wall. There was a floor. It must have been about the third or fourth floor. (Madras Street side.) There was part of the floor (a piece) still sort of sitting off the back wall but without any supports, suspended, no column underneath it. There was a woman up there, alive, yelling."

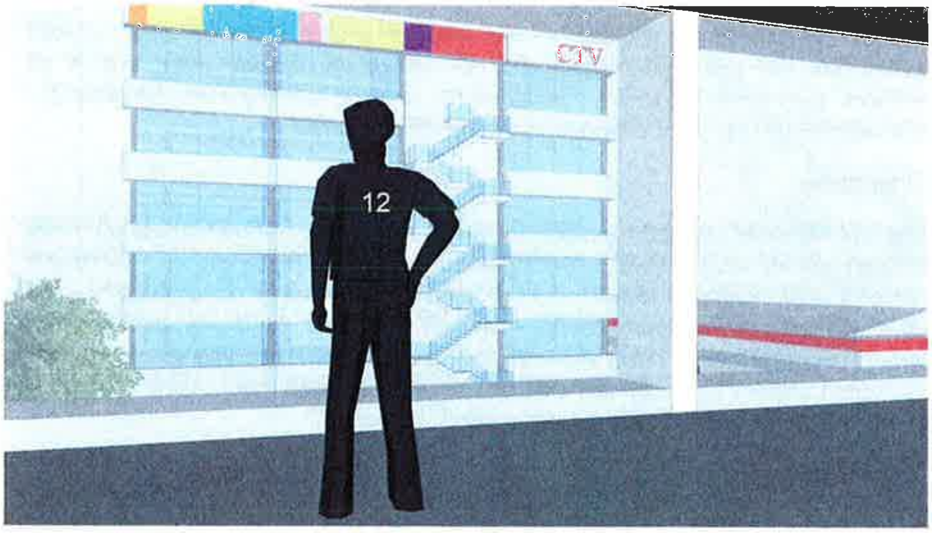

Figure 55 - Perspective of Eyewitness 12 from the IRD Building.

Eyewitness 13

"I was on the ground floor of the IRD building and our natural way out is through the glass doors, so we were all sort of facing the glass opening doors onto Cashel Street. We saw the CTV come in on itself and then drop like a river. By "coming in on itself" she meant "the building did not fall away going out, it sort of went in on itself and crumbled. Came in and just dropped." She described it as coming in from the top, and then "it just sort of came down, like all the rubble and stuff just flowed down and blew a whole lot of white dust. We couldn't see anything after because it just completely went white."

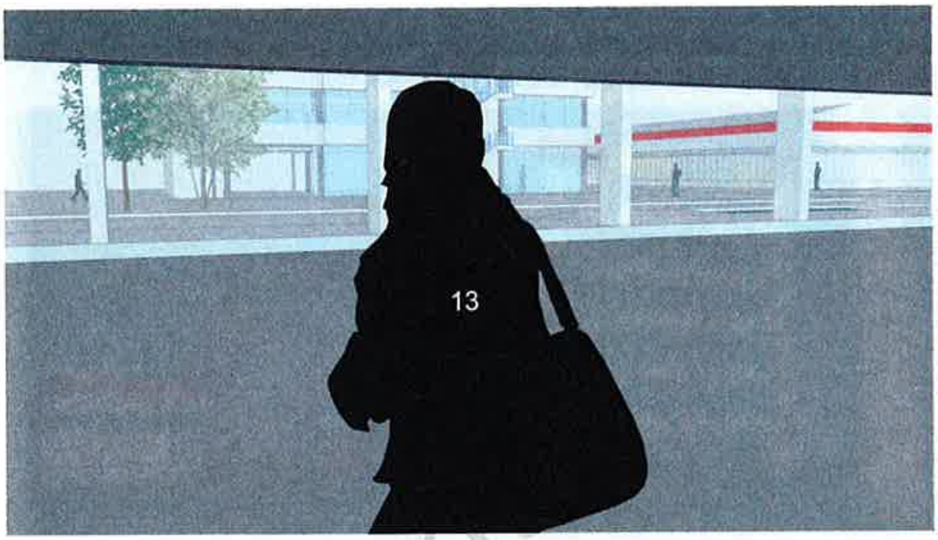

Figure 56 - Perspective of Eyewitness 13 from the IRD Building.

• *Direction of fall.*

Eyewitness 12 was higher up and right against a window, whilst Eyewitness 13 was on the ground floor, near the lifts, so comparatively quite a distance from the front of the IR building. (Around 10 metres back.) Eyewitness 13 saw no movement out from the CTV, and Eyewitness 12 felt that the front had come out towards their building. Both had the impression of it going down very quickly, not spilling out.

Eyewitness 13 was amazed that it collapsed in. "Like you expect it to go out, but it actually came in – and then just fell really hard. Our building was still rocking when that thing was flat." She also commented about where it went in – "to me it was like two-thirds up and it just went in, and just went down. I remember saying to people – it went in on itself, how does it go in on itself?!"

• *Time frame.*

According to Eyewitness 12 on the third level, "the whole thing happened in 20-odd seconds that earthquake. In the time that our building was rocking the whole building collapsed, so it was really really rapid. Down after the first hit. I watched it for 15 – 20 seconds and we were gone, going down to Latimer Square." Eyewitness 13 also found it very rapid.

• *Other observations.*

Eyewitness 13 considered Eyewitness 12's description of the front of the building falling off. "Maybe what he said explains why we got that huge cloud of whiteness? Totally white. Nobody on the ground floor went out those front doors; we all went through the back."

Eyewitness 14

Eyewitness 14 was in his lunch break at Coffee Supreme at 218A Madras Street, which is just south of the intersection with Cashel Street. When the earthquake hit, he was walking up to the intersection on the east side, just coming up past the Samoan Church, with the IRD building on his left. (20–30 metres from the intersection with an unobstructed view of the CTV.) (Figure 57)

Figure 57 - Perspective of Eyewitness 14 from Madras Street.

"I saw people at the intersection kind of hanging on. You could hear rumbling. I heard scaffolding collapsing on the Church. Realising it was serious, I tried to grab one of the parking poles…I froze and looked up north towards the CTV Building. You could see it all shaking, pretty much around the intersection. Twisting back and forth….then the external cladding…glass shatter and everything, it was just falling off. The floors were just sort of collapsing pretty much from this corner (the southeast corner) working its way back. I could see pillars coming out. It pretty much collapsed from the back, like somebody smashing a wedding cake and a deck of cards. A total catastrophic collapse; shocking to see."

• *Direction of fall.*

Eyewitness 14 recalled the upper columns going initially but it was disintegrating at all levels. Some of the pillars fell out to the side, others fell in the direction of the twisting. (He had a strong recollection of the twisting back and forth along Cashel and back up Madras.) "It was pretty much the outsides falling and behind that the whole building was just falling to bits…. I was probably seeing columns coming out from maybe two or three floors up, but you could see the whole building collapsing in on itself."

• *Time frame.*

"It just collapsed in seconds from the first quake. "There was an aftershock 10 minutes later, but the CTV was a complete pile of rubble after the first shake. There was nothing but the lift shaft still standing." He estimated that it took about 3 – 4 seconds to sense the earthquake happening, then 4 – 5 seconds to grab a parking pole. Then he saw it go 2 – 3 seconds after that.

Eyewitness 15

Eyewitness 15 had gone for lunch from the CTV and was just returning, crossing the road on Cashel Street, just in front of the IRD (Inland Revenue) Building. (Figure 58)

Figure 58 - Perspective of Eyewitness 15 from Cashel Street.

"We (Eyewitness 8 and 15) were in the middle of the road when it happened. It was shaking quite violently and then I looked up at the CTV Building. It was standing at this point, and what we saw was the fourth floor (level 5) collapsed first, so it sort of pancaked down which in turn pancaked the rest of the building down, and then the top floor broke on impact when it hit the ground.....the sound was probably the most horrific thing, everything just sort of crumbling in on each other."

• *Direction of fall*

According to what Eyewitness 15 saw, not the top floor but the next floor down was the one that broke first..."dropping into the slab of level 5 which was still intact until it hit the ground. That whole building collapsed apart from the lift shaft. It fell so straight down, that it only knocked the bumper off my car literally parked right at the front door."

• *Time frame.*

"The earthquake had been shaking violently for about 5 seconds – then the fifth level gave way to the rest of it - and I would say the whole thing was down within say 12 seconds. Only 5 seconds warning."

• *Pre-earthquake observations about the building.*

In the corners of the wall on the second level, on the side where the building had just been knocked down, he had noticed tiny gaps in the brickwork where light was coming through. He said that the day before they had been drilling wooden planks into the side of the building, perhaps the previous week as well. "It wasn't high up, it was the first two levels I think. They were 5 or 6 metres long bits. They weren't covering the whole side of the wall. They had a wrecking ball out the day before also."

Eyewitness 16

Eyewitness 16 was working on the CTV Building at the time the earthquake hit. He was facing the building towards Madras on the corner with a view of the corner column on the Cashel Street edge, out the front.) He and his workmate were hard up against the building (Figure 59).

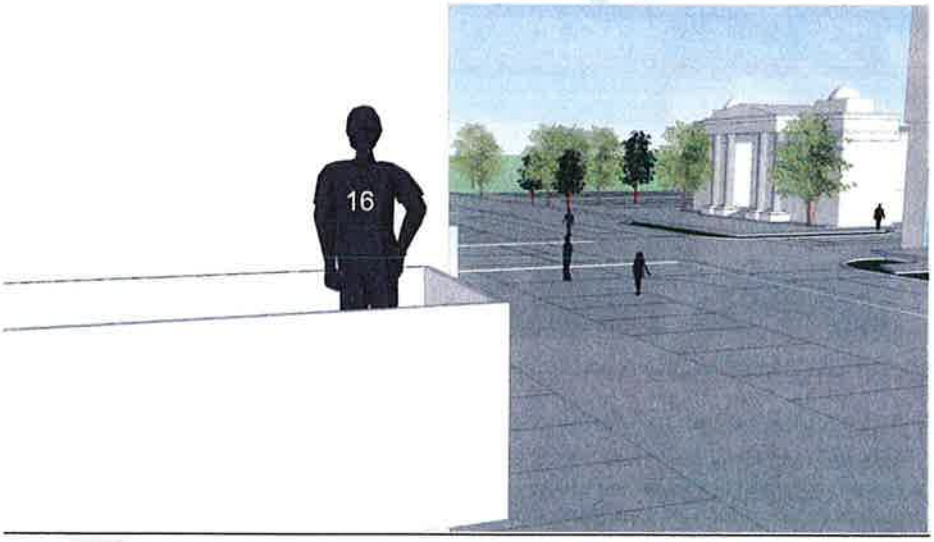

Figure 59 - Perspective of Eyewitness 16 from the elevated work platform at the southeast corner of the CTV Building.

He and his workmate were wall cladding the CTV Building, making it watertight from where the last building had come off it when it was demolished. "We had been working on it for two days... we'd taken the scissor lift about three metres up ...when I turned to my workmate to ask him for a drill to drill out the rivet and then everything started shaking away."

• *Direction of fall.*

"It all seemed to jump upwards". He felt a vertical jolt around about a movement of 200mm. I remember looking up and seeing the building pretty much right above my head, so it had obviously swayed from side to side. I threw my workmate off the machine and as I was jumping I had to push myself out of the way of the falling corner pillar. (Southwest) Just out of the corner of my eye I saw the concrete spit out the corner. The pillar came down and brought the machine down to the ground and buried the wheels. It felt like the building moved in the front."

He described seeing the column fracture. "It buckled out. It had cracked and the two bits held still by the steel had spat out, and obviously as the weight got too much, it broke and came down. This was in the middle of the column, between floors. It 'kicked out' in the direction of Les Mills. I remember I was still looking at the corner of the building at that time - it looked like the like the block in front of me came up and back down again. I turned away to the right to throw my workmate off the end of the machine, then I turned back to make sure nothing else was coming and that is when I saw the corner – sticking out around 300mm. It let go – and came down when I was jumping out."

In summary, this Eyewitness was at level 2, and saw it breaking up between level 3 and level 4 columns at the front (southwest) corner. He felt what had happened to the building was like this: "The bottom couple of floors had come out, and the rest of it had come straight down."

• *Time frame.*

All this happened in seconds. He himself was seconds from disaster – saved most likely from his scissor lift holding the debris off when he was sitting beside it.

• *Observations after the quake.*

"The thing that made this side look worst was because it had the security stairway on the outside of the building going up – the emergency exit. That was down, and because there were cars and all sorts there, it made it look like there was lots of debris here, but you could actually physically get to the bottom of the building when we were getting people out."

• *Pre-earthquake observations about the building.*

- Prior to the earthquake, Eyewitness 16, was concerned that people should not have been in it when they were working with the wrecking ball. He noticed that the building was making weird noises.

- He had been working up and down that wall. At about the third level the iron stopped. "Obviously the building that was beside it before had a flashing that went up behind the iron and then it had the rest of the building – but because that had been taken down, all of this was just concrete block façade. We were going to be tucking the iron underneath that. We'd placed 50 x 50 mm timber battens along there and were dyna-bolting about every 400 mm. The wood was so that we had something to screw the iron to instead of having to dyna-bolt every point and put plugs in them. They were 10mm dyna-bolts, and some were 40mm, and 90mm for the random hollow bricks where obviously the grout had not come all the way through. We were using the smaller bolts, just so it was grabbing and the iron wasn't going to come off. The battens went right along horizontally."

- He had not been involved in pulling down the wall that was from the old building away from the CTV. All of that was done, and cleared off the site, before his work started. "It was basically just a work site that had chip stone in it, and it must have been the ground foundations they were working on at the front of the section… pulling out big chunks of concrete that were still left in the ground. We watched them smashing with the big wreckers at lunchtime on the day before. They were doing all sorts of banging on the ground with a digger. It had a big like T-bar that went on the end packing down what they had taken out the day before. They took the pile of concrete debris away and poured crusher steel, or whatever it is, to fill the holes in and used the big arm to pack it down."

We asked some additional questions.

- *"When you were putting the battens on, when you looked at that block work, did it look like it had anything fixed to it in the past?"*

"No. It was roughly mortared as if it was the internal of a brick wall. The building side had obviously been there first. They'd put in the columns and put the bricks on the internal side, because we had to scrape the whole wall off with all the excess mortar that was hanging out of the joints so the battens would sit on it flat."

- *"Any wires sticking out or any sort of tie-backs?"*

"No the only things that you really noticed was that all of these columns were out probably 20mm proud of all these internal block walls."

"..Across the top, underneath each of those beams, you say there were some hollow blocks, but on top was there a gap?"

"No-ah, a couple of floors had gaps actually. I couldn't tell you offhand which ones they were…"

- *"If you looked at the columns and saw the block work, did you see any gap between block work and the column?"*

"No. It was all mortared."

- *"Are you sure it was mortar and not a flexible sealant?"*

"It looked like mortar because we scraped it all. That wall went to the beginning of level 4. It had wall cladding all the way along there. Three levels of block work."

- *"Can you remember what the shape of the column was?"*

"They were square with squared corners. It wasn't like the days now of precast. It looked like it had been boxed up where you could see the joins where the concrete had come out of the edges – as if it were boxed in with wood. There might have been slight gaps. You could see the inch sort of lines in the concrete where the joins were. Just a mould they'd made. That was one thing I did not think I was going to see."

APPENDIX B – PHOTOS OF COLLAPSED BUILDING

B.1 IMMEDIATELY AFTER COLLAPSE

The state of the structure immediately after collapse has been derived from photos supplied by the public and others. Debris began to be moved very shortly after the collapse by heavy machinery that was next door to the building at the time.

West Wall (Line A)

Figure 60 - West side of building with North Core partially obscured by smoke, prior to heavy machinery removing debris. No liquefaction evident.

Figure 61 - Southwest corner (Grid A/1) with corner column still standing. Collapsed work platform under wall panels can be seen at the right on which Eyewitness 16 was working.

Cashel St (South, Line 1)

Figure 62 - Cashel St. face with North Core tower in background prior to fire starting.

Figure 63 - Western end of south face (Line 1). Collapsed Line shear wall with escape stair to the right.

Figure 64 – View southeast corner of the CTV Building looking northwest. The South Wall collapsed onto the top of the debris from Level 2 can be seen, identifiable by the white fire escape stair still attached to it.

Figure 65 Madras St with precast Spandrel Panels fallen onto cars.

Madras Street (East, Line F)

Figure 66 - Corner of Cashel and Madras Streets looking towards North Core. Fractured columns and fallen Spandrel Panels are visible (MSN photo).

Figure 67 – View from southeast corner of CTV Building along Madras Street. This shows Line F Spandrel Panels fallen onto cars parked in the street indicating a tilt to the east during collapse.

Figure 68 – View from 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.

B.2 DEBRIS REMOVAL SEQUENCE

The debris from the collapse was removed from site and taken to a secure designated area at the Burwood Eco Landfill. The photos show stages in the sequence of debris removal. The identities of personnel have been blocked out.

Overhead Views

Figure 69 - Aerial view from southeast with debris being removed by heavy machinery. Fire has blackened the North Core. The Samoan church is damaged in the foreground (Dominion Post).

Figure 70 - Aerial view from northwest with heavy machinery removing debris. Water puddles on the vacant site appear to have been due to fire fighting (NZ Herald).

Figure 71 - Spandrel Panels and beams at Cashel Street Line 1 and on Line 4 in background standing vertical. Roof steelwork debris is visible.

Figure 72 - Concrete Spandrel Panels, perimeter beams and columns on Cashel Street face (Line 1 / B-D).

Figure 73 - Line 4 / B-C Spandrel Panels against tower wall, showing (left to right) a) View from north face; b) View from west showing timber framing for wall linings.

Figure 74 - Debris being cleared from Madras Street face.

Figure 76 - View from Cashel Street with debris being cleared away from west wall.

Figure 77 – Precast concrete beam being lifted from the debris near the South Wall.

Figure 78 - Perimeter 400 mm diameter column with spalled base and bar lapping zone at left unpainted portion that would have been located at Spandrel Panel infill areas.

Figure 79 - View from Cashel Street east side of Line 1 with Line 1 South Wall lying on debris at left; trapezoidal 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. Remnants of North Core slabs and the collapsed column on Line 4 D/E can be seen at the rear. The Level 6 slab in front of the lift well can be seen still suspended in mid-air by its Drag Bars connected at Walls D and D/E, even after loss of support from column 4 D/E.

Figure 80 - Portion of Line I South Wall being lifted out by crane.. The Level 6 slab in front of North Core has been removed for safety reasons.

Figure 81 – Upper portion of South Wall being prepared for removal. A portion of slab can be seen highlighted in the foreground in contact with the South Wall on this side at Level 2. It may have forced the wall to pivot against it, preventing it breaking over at its base at Level 1.

Figure 82 - Line 1 South Wall at Level 1 showing masonry in-fill at door opening, in-plane flexural fan-like cracking and spalling of concrete at right (east) end. A portion of profile slab can be seen end on through the opening

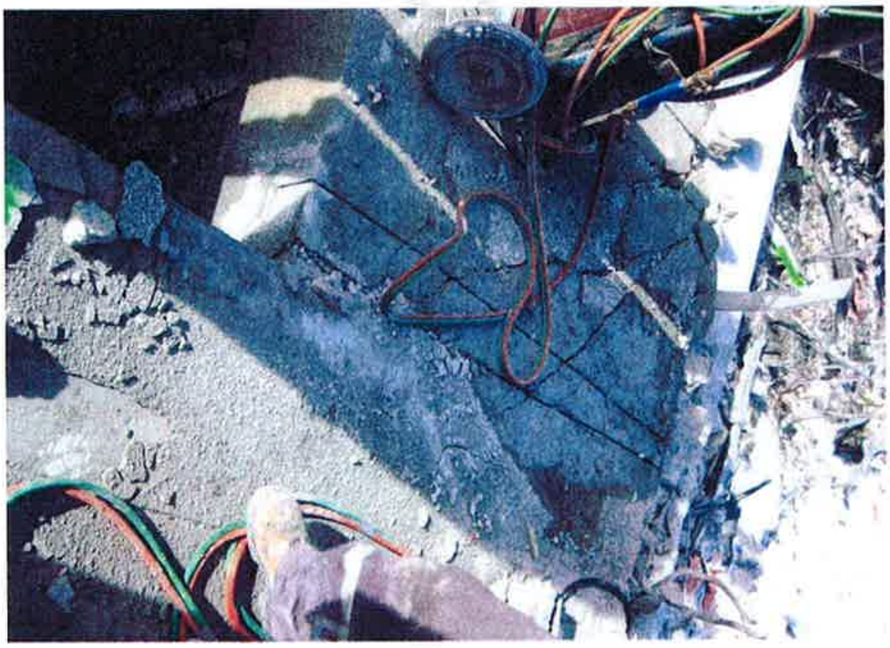

Figure 83 – Line 1 South wall at Level 4 showing severe diagonal cracking in east panel.

Figure 84 – 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.

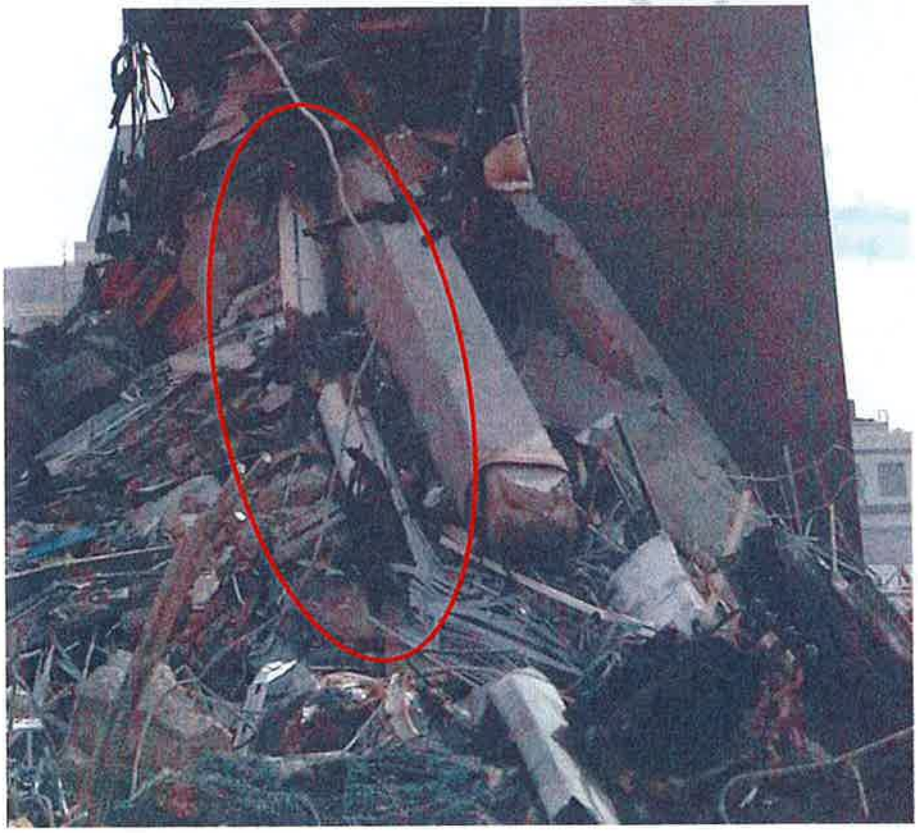

Figure 85 - North Core column 4 D/E highlighted amongst the debris. Hinging can be seen above and below the beam column joint.

Figure 86 – View from southwest of North Core. The Level 4 slab can be seen lying diagonally against the North Core.

Figure 87 - Line 2 beams lying rotated northwards.

Figure 88 - Line 3 beams lying rotated southwards

Figure 89 - Perimeter columns at beam-column joint with shell beam on right side.

Figure 90 - Line 4 / B column with B22 precast log beam in foreground and B23 shell beam at rear. No hinging is apparent at the base of the column compared to the perimeter column Item E33.

Figure 91 - North Core slabs remaining to be removed.

Figure 92 - North Core slabs removed.

Figure 93 - All debris removed leaving the Level 1 slab on grade and remnants of the North Core.

APPENDIX C - SUMMARY OF SITE EXAMINATION AND MATERIALS TESTING RESULTS

INTRODUCTION

The following summarises observations and material properties from the Site Examination and Materials Testing. A more detailed account is found in the Site Examination and Materials Testing Report (Hyland 2011).

PROFILED METAL DECK AND CONCRETE SUSPENDED SLAB

The profiled metal deck that formed the 200 mm thick slab had de-bonded from the underside of the concrete in many cases during the collapse. This is not unexpected as it is recognised by engineers that profiled metal decking does not rely on chemical adhesion with the concrete to develop the properties of composite profiled metal deck concrete slabs.

The steel decking had pulled away from the supporting beams in all cases except at the pre-cast beam support on Line 4 at the North Core. In that case the steel decking appeared to have fractured in tension.

A portion of the decking was tensile tested and found to exceed the minimum specified yield stress of 550 MPa

PRE-CAST CONCRETE SHELL BEAMS

The pre-cast concrete shell beams were found to have no reinforcement in the in-situ in fill concrete.

There was no roughening of the precast surface on the inside of the shell beams to encourage composite behaviour of the shell and the in-fill concrete. Composite behaviour between the shell and the infill concrete would have increased the ability of the beams to resist the demands placed on them.

The slab on the shell beam on Line 4 that connected into the shear core wall had fractured along the inside edge of the beam.

The bottom reinforcing steel in the shell beams had not been developed fully into the Grid C core wall on Line 4 as specified, except at Level 2. The bars had been bent back into the concrete infill in the shell beam (Figure 94).

Figure 94 - 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 specified (DENG Detail 5 Dwg S19). Notice the bar imprint on wall at the connection seen in (c) at Level 4 and at Level 3.. This meant that these beams would not have performed as intended.

400 MM DIAMETER COLUMNS

The exterior 400 mm diameter column (Item E33) had flexural failure at the floor level lap joint of the vertical reinforcing steel, and compression-flexural fracture at the upper end of the column (Figure 95)

The lap joint in the exterior columns was concealed by the external Spandrel Panels and interior linings.

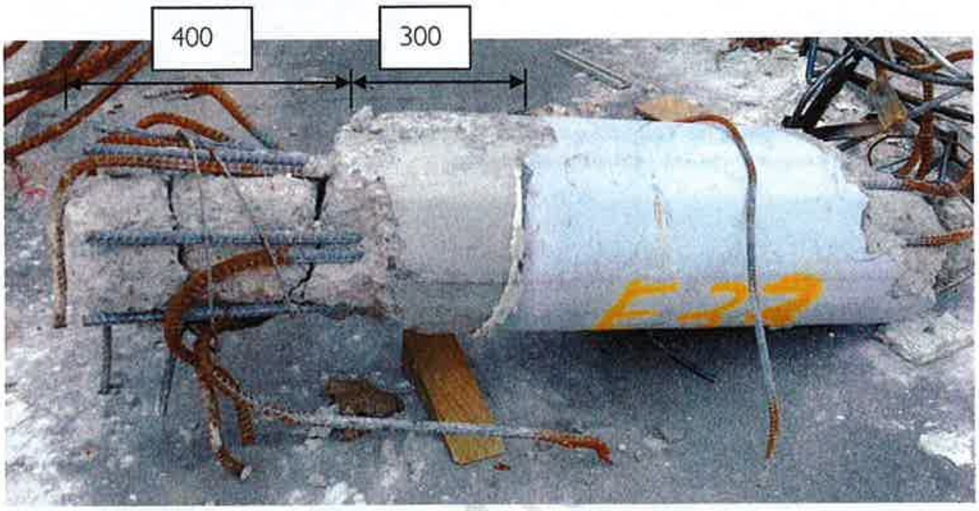

Figure 95 - 400 Diameter Exterior Column Item E33. (DENG C5 or C11, Dwg S15). Left end is bottom of column at floor level with concrete spalling over lapped vertical reinforcing. Horizontal cracking in core confined by R6 spiral which had fractured. The unpainted portion was protected by Spandrel Panels.. Right-hand end fracture occurred below beam-column joint.

INTERNAL PRE-CAST LOG BEAMS ON LINE 2 AND 3

The ends of the pre-cast internal log beams that supported the 200 mm thick profiled metal deck slab, had smooth formed un-roughened ends at the interface with the beam–column joint zone. This would have reduced beam-column joint shear capacity weakening its ability to hold together during earthquakes (Figure 96).

EXTERNAL PRE-CAST LOG BEAM ON LINE 1 AND 4

The ends of the pre-cast log beams supported by the corner columns on Grid A had a smooth un-roughened end where it connected into the columns. This would have reduced the beam-column joint shear capacity weakening its ability to hold together during earthquakes.

No starter bars connected the log beam into the 200 mm slab that was supported on the shell beams. This is unusual (Figure 97).

Figure 96 - Interior Pre-cast Log Beams from Line 2 and 3 (DENG Section 3 Dwg S15) showing smooth concrete formed for beam-column joint, and bottom hooked bars that have pulled out of beam-column joints without any obvious straightening;

Figure 97 - Item E18 Pre-cast edge beam north-west corner (DENG B22 Dwg S18 (from left to right) (a) Smooth form finish at attachment to column 4A (DENG Detail 1 Dwg S19); (b) No starters (reinforcing bars) from pre-cast beam into slab to prevent the profiled metal deck slab pulling away (DENG Section 4 Dwg S15). If roughened these joints may have slowed down development of progressive collapse.

LINE I SOUTH WALL

The Line 1 South Wall that extended from Level 1 on the ground to the roof had been broken up into single story components during de-construction.

Level 1 to 2 (Item E1)

This panel showed flexural cracking patterns typical of cantilever shear walls rather than coupled shear walls (Figure 98). This was likely due to the effect of the Level 1 doorway having been infilled with reinforced concrete masonry.

Reinforcing steel taken from the east end of the wall was found to have yielded and elongated prior to the collapse of the building.

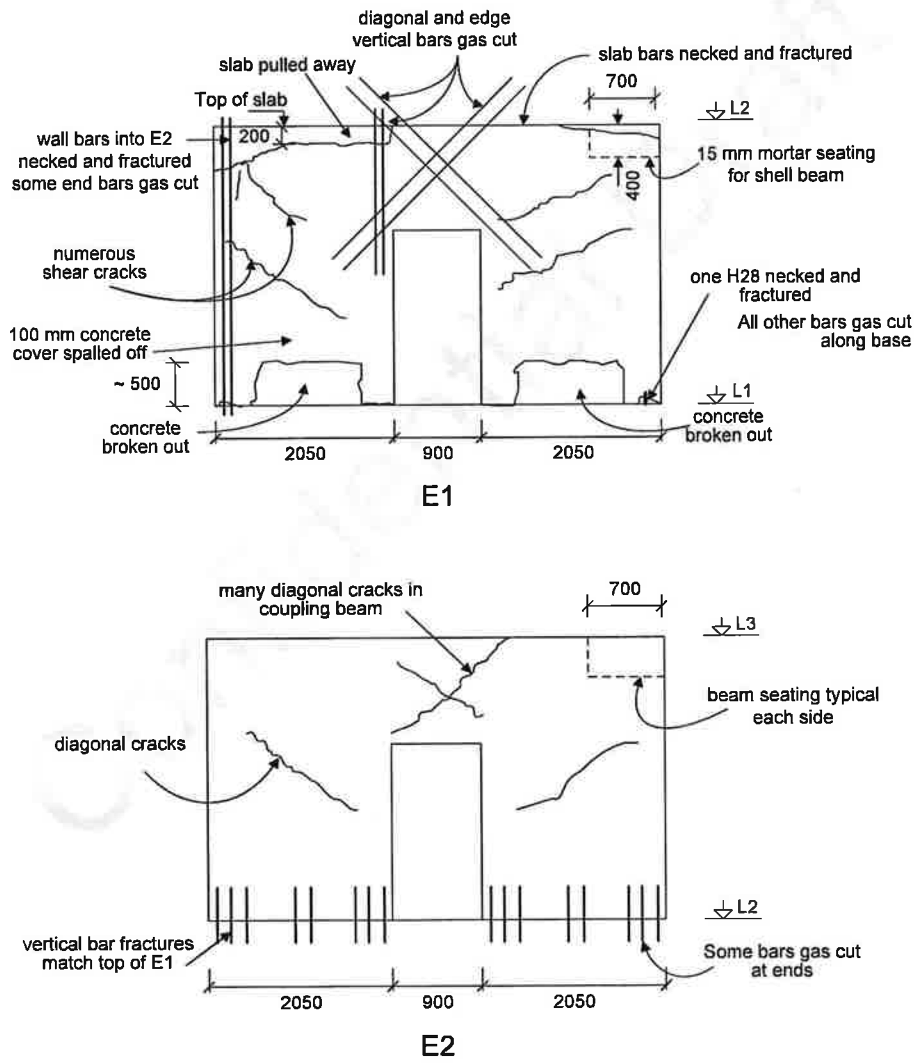

Figure 98 - Line 1 South Wall remnants (top) E1 Level 1 to 2; (Bot) E2 Level 2 to 3.

Level 2 to 3 (Item E2)

This panel had diagonal cracking in the piers consistent with cantilever wall behaviour and two way diagonal cracking in the door head coupling beam (Figure 98).

Level 3 to 4 (Item E3)

This panel had dominant uni-directional diagonal cracking running from the bottom west corner to the top east end (Figure 99).

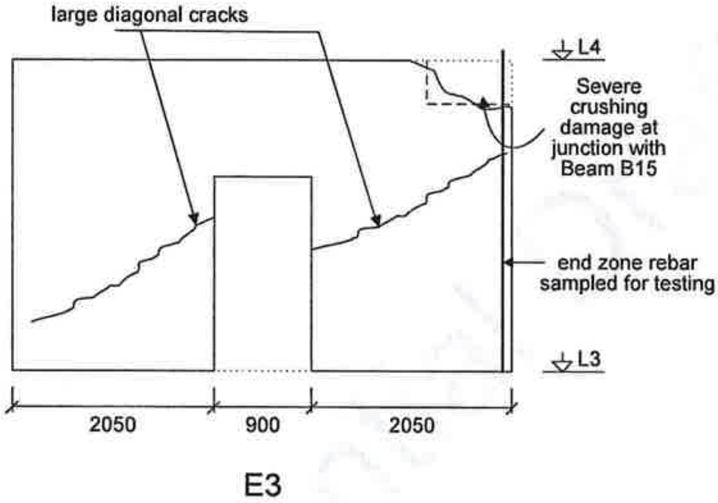

Figure 99 - Line I South Wall remnant E3, Level 3 to 4.

Level 4 to 5 (Item E4)

Severe two-way diagonal cracking in east pier and loss of cover to vertical reinforcing steel on east edge.

Smooth mortar construction joints rather than roughened at junctions with pre-cast shell beams B15 and B16 (Figure 100).

The cracking may have been caused on impact with the ground during the collapse, as the calculated shear capacity appears to have been adequate for the loadings considered to have occurred at the time of the collapse..

Level 5 to 6 (Item E5)

Weak concrete in west pier adjacent to top of doorway that was able to be dislodged by boot (Figure 100).

The top surface of wall was smooth rather than a roughened construction joint for slab seating.

This may have led to increased lateral displacements due ti possible slippage on these joints.

Bars from wall into attached pre-cast beam had fractured.

No obvious cracking had occurred in the wall or the door head coupling beam.

Level 6 to Roof (Item E5A)

No obvious cracking had occurred in the wall piers or door head coupling beam. This indicates that the South wall could have sustained more damage before it would have become unsafe.

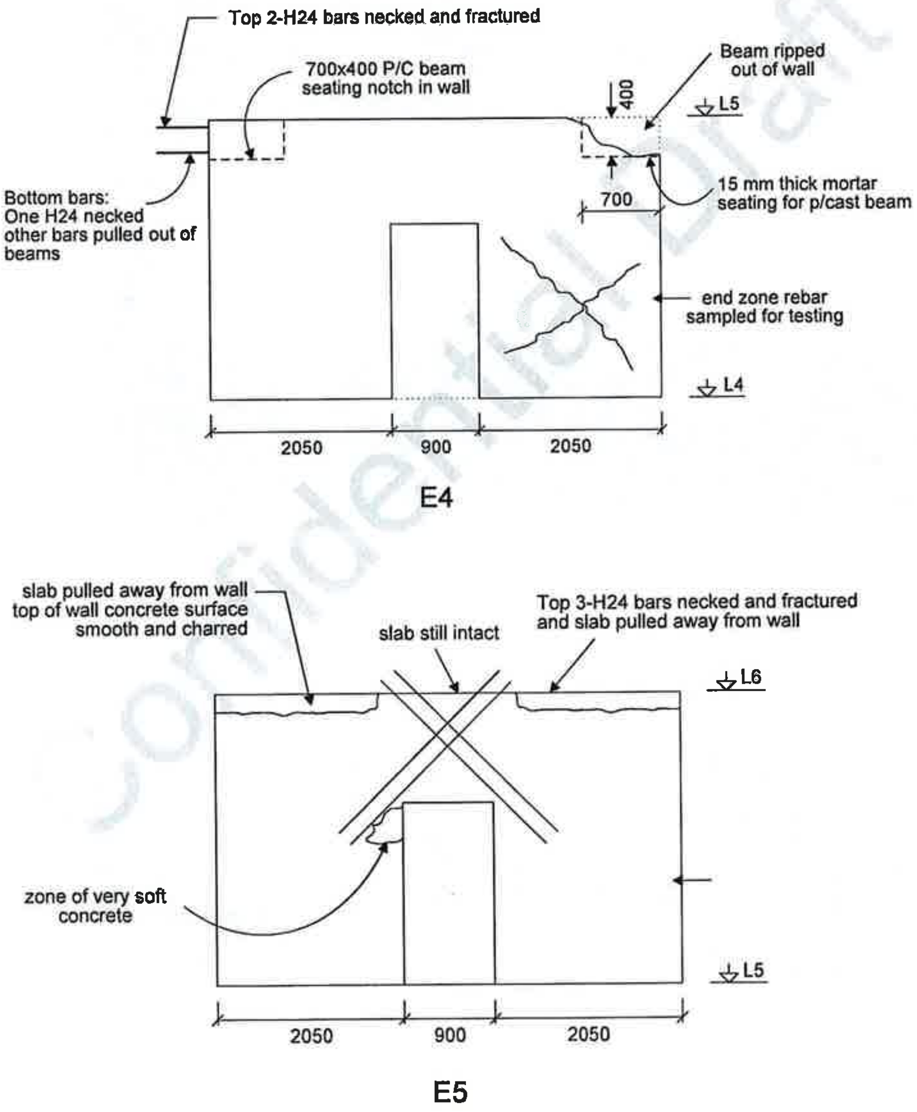

Figure 100 - Line I South Wall remnants E4 Level 4 to 5 and E4 level 5 to 6.

Figure 101 - Line 1 South Wall Level 5 to Level 6 (Item E5) (clockwise from top left) (a) Crumbly concrete at door edge of west pier able to be dislodged by boot; (b) Smooth and charred construction joint on top west surface looking east; (c) Charred construction joint above west pier. Door sill on left; (d); Top east corner with fractured top 3-H24 bars. Floor 664 mesh exposed.

NORTH CORE WALLS

Only fine cracking was found on the North Core walls after collapse (Figure 102).

No obvious cracking on Line D/E wall.

Horizontal flexural cracking on west and north west face of Line 5 wall and north end of Line C wall at Line C/5.

Fine two-way diagonal cracking on the inside faces of Level 1 to 2 Line 5 and D walls in North Core.

The North Core therefore did not appear to have been overloaded or suffered any significant permanent deformation during the Aftershock.

Figure 102 - 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.

SLAB AND BEAM REMNANTS ON LINE 4 OF NORTH CORE

The extent of the slabs at the time of examination was measured (Figure 29).

Portions of the level 6 and Level 5 slabs that were still attached immediately after the Aftershock were removed during deconstruction for safety reasons. The slab at level 2 had also been broken back. The rest of the slab was in the condition it was left after the event.

Level 6 Slab

The slab had a vertical fracture face that coincided with the ends of the H12 saddle bars from the support beam on Line 4 (Figure 103).

664 mesh in the slab had fractured in a ductile manner which is the way it was intended to.

The profiled metal deck steel decking had fractured in tension adjacent to the edge of the fractured slab edge.

Level 5 Slab

The fractured edge of the slab was similar to that at level 6.

Reinforcing was located in the bottom of the slab rather than as specified near the top surface.

Cracks were found running from cores drilled in the slab for pipes.

Level 4 Slab

The imprint of the bent back bottom bars from the pre-cast shell beams (Figure 94) was visible in the cover concrete of the wall.

The profiled metal deck decking of the fractured slab was still clamped to the support beam on Line 4 and fractured in tension.

Level 3 Slab

Similar to Level 4

Level 2 Slab

Bottom bars of pre-cast shell beam had been developed into the core wall on this level only, and beam-column joint type diagonal cracking was seen on the end of the wall. This was consistent with cyclic demands having occurred there during the Aftershock.

Figure 103 - Line 4 Core Wall Slab Remnant at Level 6 amenity area (clockwise from top left) (a) Slab edge on stairwell wall looking west with H12 saddle bar exposed and ends of mesh below it; (b) Vertical concrete fracture surface with reinforcing mesh fractured; (c) Slab looking west with cores cut in floor for amenities; (d) Fractured mesh angled downwards; (e) Fractured slab edge looking east. Torn metal decking aligned approximately with concrete fracture edge; mesh at varying height within slab; (f) Cores for amenities at fracture edge can be seen and are a small proportion of the total fracture surface length.

SLAB DIAPHRAGM CONNECTIONS TO NORTH CORE WING WALLS ON GRID D AND D/E

After the original construction of the building had been completed, Drag Bars were fixed into the slab and into the walls at Levels 4, 5 and 6 on Lines D and D/E with epoxy grouted threaded rods.

Level 2 Connection of Slab to Walls

No reinforcing steel connected the slab to the east wing wall D/E.

A 20mm hole was found in the west wing wall D where a reinforcing bar had pulled out.

Level 3 Connection of Slab to Walls

An H12 bar was found fractured at the end of the west wall D.

No reinforcing steel was found to have connected the east wing wall D/E to the slab.

Level 4 Connection of Slab to Walls

The Drag Bars on both the west and east wing walls had partially fractured in bending and tension. The epoxy grouted 20 mm threaded rods that were fixed vertically into the slab and into the Drag Bar on the west wall appeared to have pulled out in tension. This occurred as the slab between Lines D and D/E rotated downwards, pivoting about its Drag Bar supports at the ends of the lift shaft walls.

The 20 mm diameter Drag Bar threaded rods were hardness tested by MTL and found to have Rockwell Hardness HRB. This conformed with the minimum requirements of AS 4291.1:2000 (SAA 2000) for Property Class 5.8 threaded rods.

Level 5 and 6 Connection of Slab to Walls

Similar to what was seen at Level 4 (Figure 104).

Figure 104 - Level 5 Lift Well Wing Walls Grid D and D/E (anti-clockwise from top) (a) the Drag Bar consisted of a 150x150x10 L steel angle with a 51 x 3.2 SHS welded to it; 4-M24 anchors were epoxied into the wall and 6–M20 threaded anchor rods 350 mm long were epoxied into the slab at the profiled metal deck rib. 3-M20 threaded rods remained upright on the Grid D Drag Bar. The 51x3.2 SHS had fractured in bending and tension at the bolt hole adjacent to last bolt into wall and twisted with the slab; This shows that the slab that had been fixed to the Drag Bar had rotated downwards as the column on Line 4 D/E collapsed, (b) Epoxy grout can be seen around the threaded anchor rod that had been in the slab; (c) The drag Bar is bent downwards and holes where 3-M20 threaded anchor rods had been can be seen; d) On Wall D/E a 150x75x10 L steel Drag Bar was still fixed into the wall D/E with 5-M24 threaded rod anchors. The end of the Drag Bar had been gas cut during deconstruction.

CONNECTION OF COLUMN D/E 4 TO NORTH CORE AT LEVEL 7

The column had pulled away from its connection to the North Core wall D/E. Four H20 bars were specified on the drawings to be bent into the wall (Figure 105). However only three 20 to 24 mm diameter holes were found in the location where the column bars had pulled out. This indicates that one bar had not been placed as specified. Though not considered to have initiated the collapse, if all these bars had been present they may have prevented the collapse of column D/E 4. Even so this would not have prevented the collapse of the other columns in the building.

Figure 105 - Lift Well Wing Wall D/E: Column D/E 4 Connection (DENG Dwg S14); 3 x 20 to 24 mm diameter holes can be seen where reinforcing bars from column have pulled out. The drawing shows that 4-H20 bars were required to be bent in to the wall.

LEVELS AND POSITIONAL SURVEY

The floor slab, slab overlay and foundation beams were found to have levels consistent with original construction tolerances and practice.

No evidence of long term foundation settlement or settlement induced by the Aftershock could therefore be inferred.

The core walls on Line 5 were surveyed for verticality by sighting on the eastern and western corners of the north face of the wall. It was found that there was a northwards out-of-vertical measurement of 91 mm over 18.53 m between Level 1 and Level 7 at the northeast corner, and 68 mm over 18.53 m at the northwest corner.

This is greater than the plumbness limit of 25 mm for structures greater than 12m high in NZS 3109.

OTIS, the company that maintained the lifts at the CTV Building advised that they had no records of the inside faces of the walls being out-of vertical alignment after construction.

REINFORCING STEEL PROPERTIES

Reinforcing steel samples were extracted from the Line 1 South Wall and tested to determine tensile properties, production uniformity and work hardening during the Aftershock.

The reinforcing steel from the South Wall was found to conform to the standards of the day.

The H28 steel extracted from the lower portion of the South Wall item E1 was found to have elongated 3.3 % more than the other 16 to 28 mm bars extracted. It also had an elevated yield stress and ultimate tensile strength. This is known to occur in constructional steels that have been work hardened and have subsequently strain aged (Hyland, Ferguson et al. 2003).

This is evidence that the bar appeared to have "work-hardened: during the Aftershock and prior to the collapse of the building.

The chemical analysis of the 16 to 28 mm bars found that they had chemical compositions consistent with them being from the same or similar production runs.

The suspended slabs were reinforced with hard drawn steel 664 mesh sheets with wires spaced at 150 mm cross centres. The 664 steel mesh from the suspended slab was sampled and tested.

The 664 steel mesh was found to conform to the standards of the day.

CONCRETE PROPERTIES

Cores were extracted from columns, beams, slabs and walls for compressive strength testing. The chord modulus of elasticity was also determined for the South Wall and North Core concrete.

The sample means of the test results for a particular member were adjusted up by a factor of 8% where required, to allow for test orientation effects where testing had been done transverse to the direction of casting (Figure 106). This was in accordance with the recommendations of the Concrete Society Technical Report 11 (GBCS 1987).

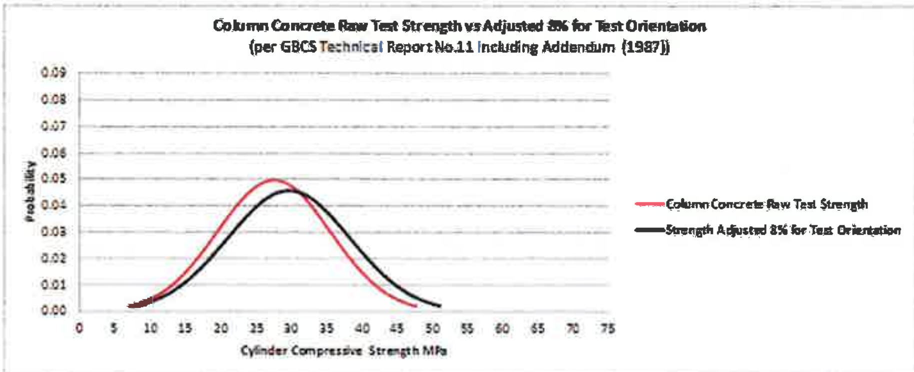

Figure 106 - Column concrete test strengths compared to strengths adjusted 8% for test orientation being transverse to direction of concrete casting. This adjustment in test strength was recommended by the Concrete Society Technical Report 11 "Concrete Core Testing for Strength".

The adjusted sample means were then assessed against the known means of concrete properties with 28-day strengths conforming with 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.

Suspended Slab Concrete Properties

The suspended slab concrete was core tested in two locations. The average strength at test was 24.6 MPa.

In conclusion, at the time of the collapse the concrete in the suspended slab had mean strength not greater than that of concrete with 28-day strength of 25 MPa

This indicates that at the time of the collapse the concrete in the slab may have met the minimum 28-day strength specified of 25 MPa.

The mean strength of the concrete was also not greater than that with 28-day strength of 20 MPa Aged by 25%. This indicates that the slab concrete may have only acheieved 28-day strength of 20 MPa or less at the time of construction. This was less than the 28-day strength of 25 MPa that was specified.

South Wall and North Core Concrete Properties

Concrete cores extracted from one location each in the South Wall and the North Core found an average strength of the walls of 33.8 MPa.

This was adjusted 8% for testing orientation transverse to casting direction, to give 36.5 MPa.

In conclusion, at the time of the collapse the concrete in the shear walls had mean strength not greater than that of concrete with 28-day strength of 35 MPa. This indicates that it would likely have had strength satisfying the minimum specified 28-day strength of 25 MPa.

The concrete in the shear walls also had mean strength not greater than that of concrete with, 28-day strength of 30 MPa Aged by 25% or less. This indicates that it would likely have had strength satisfying the minimum specified 28-day strength of 25 MPa at the time of construction.

The chord modulus of elasticity of the shear wall concrete was found to be an average of 27,600 MPa. This was consistent with what would be expected for concrete with that strength

The calculated average secant modulus of elasticity was 26,100 MPa.

Column Concrete Properties Summary

The concrete column test strengths derived from core and rebound hammer tests are shown in Table 4. These are also shown factored up by 8% to allow for the effect of testing transverse to the direction of casting (GBCS 1987).

The strength of the concrete in the columns was based on the testing of a statistically significant sample of 26 column remnants selected at random from the debris at the Burwood Eco Landfill.

The adjusted sample mean of all columns tested was 29.6 MPa.

This indicates that at the time of the collapse the columns in Levels 1 to 6 had mean concrete strength equivalent to that of concrete with specified 28-day strength of 20 MPa (Figure 107). This is less than the minimum 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 also indicates that at the columns in Levels 1 to 6 would only have achieved a 28-day strength of 17.5 MPa at the time of construction (Figure 108). This is less than the minimum 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.

	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 4 Column concrete test properties statistics

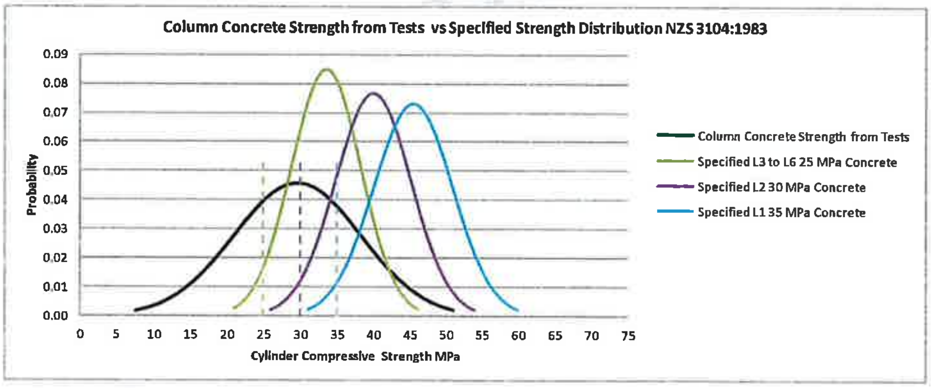

Figure 107 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS3104:1983. This indicates that the concrete in a significant proportion of the columns would have had strengths less than the minimum specified.

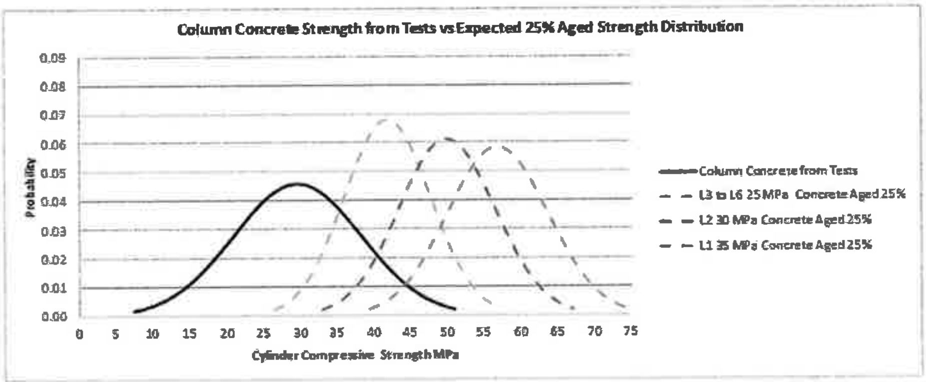

Figure 108 – Column concrete test strengths adjusted for test orientation vs 28-day concrete strength distribution according to NZS 3014:1983 strength-aged by 25%. This shows that the concrete in the columns had significantly lower strength distribution compared to the lowest concrete strength specified when account is made for the strengthening of concrete with age.

APPENDIX D - NON-LINEAR TIME HISTORY ANALYSIS

INTRODUCTION

Non-linear time history analysis (NTHA) were used to evaluate the response of the CTV Building to the ground motions that had been recorded at other sites in the Christchurch CBD for the 4 September 2010 Darfield' Earthquake and 22 February 2011 Lyttleton Aftershock. The NTHA examined the likely response of the building at every step in time; however the results should be interpreted as an approximation only. This is because there are a large number of uncertainties and assumptions involved. The objective with the NTHA has been to model the overall lateral stiffness and strength of the building as accurately as possible.

The main findings from the analysis are described in the following sections. Floor diaphragm connections and columns are a focus, since they are potentially critical failure mechanisms under seismic loading. Irregularity of the building structure and the resulting torsional response is a contributing factor. The fragility of beam-column joints is also discussed.

The load demands on floor diaphragm connections to shear walls were obtained directly from the analysis. For columns, the inter-storey drifts output from the NTHA were considered to represent the earthquake demand, against which various potential failure mechanisms were then assessed. The vertical stiffness of floors and beams was modelled to enable the additional demands from vertical ground accelerations to be quantified.

ANALYSIS OVERVIEW

The three dimensional model shown in Figure A was created using the SAP2000 finite element program. Static pushover analyses and non-linear time history analysis were carried out using this model to evaluate seismic actions on the structure.

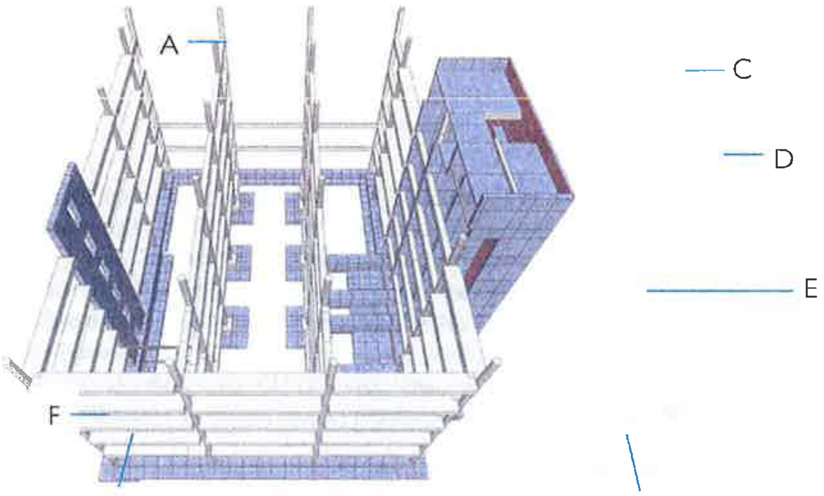

Figure EEEEE - SAP non-linear analysis model viewed from east side

The basis of the non-linear analysis is reported in more detail in the referenced 'Non-Linear Seismic Analysis Report' by Compusoft Engineering, who was engaged by StructureSmith to carry out the analysis. Key points from that report are summarised below.

The analysis of the CTV structure investigated two different structural configurations denoted '*MODEL A*', and '*MODEL B*', as outlined below.

- **MODEL A (no masonry)**
  This was the structure 'as designed' and included the contribution from the primary seismic force resisting system (the concrete shear walls), and the secondary structural elements that were not detailed for separation (the concrete frames) only. The masonry infill walls and precast concrete spandrels were assumed to be effectively isolated from the structure so as not to participate in the seismic response. A variation of this configuration where the precast spandrels engaged the perimeter columns was run as a pushover analysis only to enable assessment of the perimeter columns under that condition.

- **MODEL B (with masonry)**
  The structural form described in MODEL A above, but with the masonry infill walls not effectively isolated from the identified structure and so contributing to the seismic response. This modelled what was considered to be the upper bound effect of the masonry as built, with no gap to the adjacent columns and with an upper bound stiffness and strength based on flexural yielding of each individual masonry panel.

Note: The interpretation of the primary and secondary structure in Models A and B above was based on our review of the original design calculations, which appear to have considered the concrete frames and the masonry infill walls as 'gravity only' secondary elements, with the masonry intended to be separated.

The overall procedure for the non-linear analysis consisted of the following stages:

1. A gravity analysis on the structure using appropriate imposed loading allowances.

2. A nonlinear static pushover analysis of the structure for the two primary directions starting from the end state of the gravity analysis. This enabled the non-linear performance of the individual lateral load resisting structural components to be verified and then combined together in the model to be used for the NTHA.

3. The three adopted ground acceleration records from the 4 September Darfield Earthquake and 22 February Lyttelton Aftershock were aligned to the principal axes of the CTV Building, which are essentially north-south and east-west.

4. Non-linear time history analyses using the three adopted ground acceleration time history records of the 4 September Darfield Earthquake and 22 February Lyttelton Aftershock. This process was carried out for both structural forms *MODEL A* and *MODEL B* for the Darfield Earthquake and then for Model A only for the 22 February Lyttelton Aftershock. All components of the acceleration time history were incorporated simultaneously including north-south, east-west and vertical components.

5. The results were then processed and the performance reviewed.

KEY ASSUMPTIONS

Key assumptions and features in the SAP non-linear model included the following:

1. Material strengths were taken as the average values from tests carried out by Hyland (Hyland 2011). Average concrete strengths for columns were taken as equal to the specified 28-day strength + 2.5MPa. In fact concrete strengths were found to vary considerably and this was taken into account in the assessment of columns following completion of the analysis.

2. Foundations were modelled with non-linear soil spring supports, with compressive stiffnesses evaluated by Tonkin and Taylor Ltd, and with gapping under uplift conditions to model the potential rocking of foundations.

3. Beams and columns were modelled as elastically responding frame elements, with stiffness modifiers determined from moment-curvature relationships. Inelastic behaviour of the beams and columns was incorporated by the way of discrete hinges. These hinges considered stiffness degradation but not strength degradation during hysteretic cycling and had no plastic rotation limits applied. Hinges were located at the face of the connecting member (i.e. at face of beam for column hinges and at face of column for beam

hinges). For columns, rigid-plastic interacting M-M hinges were used, calibrated for the average gravity axial compression action on the column.

4. It has been assumed for the purposes of the NTHA that beam hinge formation is not limited by the capacity of the beams bar end anchorages or beam-column joint shear capacity. Joint demand and capacity would need to be assessed post analysis.

5. Examination of the detailing for the connection between the top of the column at grid 4 D/E and the overhanging core wall indicated that it was not capable of transferring the significant axial forces that would result from moderate seismic demands. For that reason the connection was released in the non-linear model.

6. Physical evidence indicated that the positive moment (bottom) reinforcement of the beam along gridline 4 between grids B and C was not effectively anchored into the north core wall on grid C at levels 3 and 4. No moment capacity was provided in the model at these locations to reflect this finding.

7. The yielding portions of shear walls were modelled using nonlinear layered shell elements which incorporated inelastic material effects at a fibre level. Where there was no significant inelastic demand the walls were modelled using linear elastic shell elements with stiffness modifiers determined from moment-curvature analyses. Modelling of the diagonally reinforced coupling beams in the South Wall used non linear links substituted for the fibre elements to reduce computation times.

8. Floor diaphragm connections to the north core walls on grids D and D/E were identified as an area of potential connection failure. As a consequence of a lack of tie reinforcement it was assumed that there was no tensile or gravity connection between the slab and these walls at levels 2 and 3. At levels 4 to 6 a retrofitted steel angle tie (or 'Drag Bar') provided limited tensile and gravity connection to the slab at the tips of the walls on grids D and D/E. The Drag Bars were modelled using fuse tension links incorporating 2mm initial slip in connections and the calculated elastic stiffness of the steel angle section. At actions equal to the calculated limit state capacities of the Drag Bar and its connections (based on design documentation and tested properties of anchor bolts and slab concrete) the fuse links would disconnect. No limitation was placed on the compressive load capacity. Gravity load transfer at this interface is expected to be limited to a low value by slab reinforcement yielding and so has been taken as zero for the purposes of the seismic analysis. Floor diaphragm connections to other walls were assumed to remain connected for the purposes of the analysis, but were to be assessed post analysis.

9. In-plane stiffness of the floors was modelled as 0.5 Agross for an average slab thickness of 173mm to allow for nominal cracking. For out-of-plane

demands the floors were considered to have effective stiffness corresponding to 0.5Igross at midspan. The effective out-of-plane stiffness adjacent to beam lines was taken as the average of the positive and negative stiffness. This was determined from moment-curvature analyses considering the reinforcement present (it appeared there was no bottom reinforcement from the floor slab into the supporting beams). The effect of the profiled metal deck was not incorporated into the model.

10. In Model B, the upper bound stiffness and strength of the masonry infill was modelled using elastic shell elements, with non-linear link elements connecting each masonry panel to the underside of the floor or beam above at each level. Based on a calculation of the flexural capacity of a typical masonry panel, the non-linear links transferred a maximum of 100kN shear from each 2.3m wide masonry panel at up to 20mm lateral displacement, degrading to zero shear after 35mm lateral displacement. This is less than the shear strength of the masonry that could be developed if the panels were constrained by the beams and columns around them.

11. The NTHA did not include the potential effects of variation of concrete strength or the potential interaction of the precast façade spandrels with perimeter columns directly. The reason the effects of the spandrels and varying concrete strength have not been explicitly modelled is that they are predicted not to alter significantly the overall building response to earthquake shaking. However, they are important factors to be considered in the assessment of individual elements such as columns, with reference to the storey responses obtained from the analysis.

STATIC PUSHOVER ANALYSIS

Static pushover analyses were carried out to verify the lateral stiffness and strength of the various components of the lateral load resisting structure - before they were combined in the full model to be used subsequently for the NTHA.

The pushover curves for model A, with the masonry infill walls effectively separated, are shown in Figures B and C below. In these figures, displacements were recorded at a node located approximately at the centre of mass of level 6, and the base shear components were recorded at the top of the foundation beams.

A feature that can be seen is the significant difference in stiffness and strength between the north core and the South Wall in the east-west direction. This represents a severe plan irregularity in the seismic resisting system.

It can be seen that the plots for the east and west pushovers are almost identical, indicating a similar response in both these directions. By comparison the initial response of the building in the northward direction is stiffer than in the southward direction, which can be attributed to the differences in foundation stiffness under the north core.

More base shear is carried by the north core for a northward push than for a southward push. This is due to the mobilisation of the gravity loads on beams along gridline 4 to resist overturning as the core walls rock and move upward beneath the beams. This behaviour is not as significant in the southward direction because of the restraining effect of the foundations to downward loads on gridline 4.

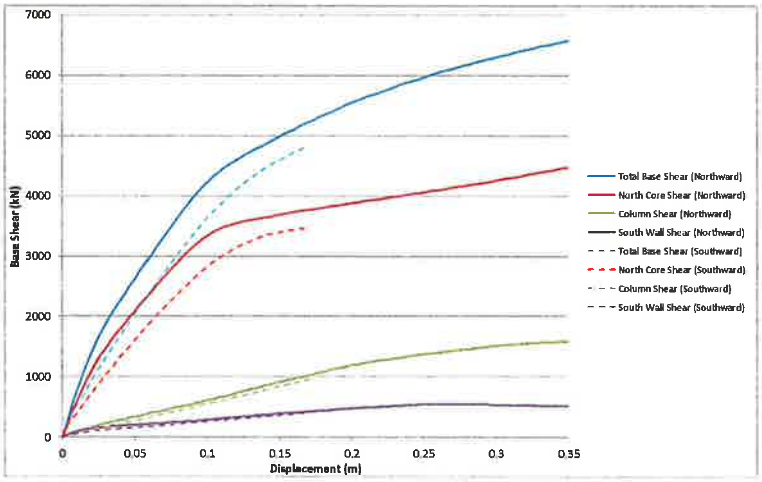

Figure B - Push-over curves, north-south, no masonry.

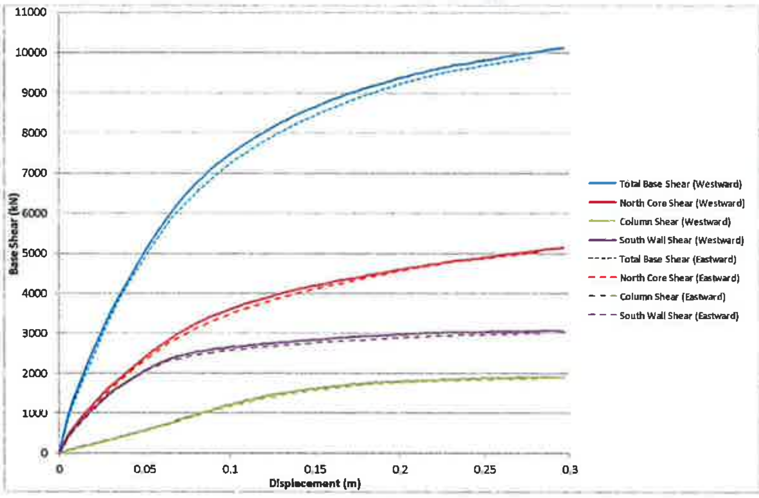

Figure C - Pushover curves, east-west, no masonry.

In figure D below are plotted the pushover curves for Model A (no masonry), shown by the dashed lines and Model B (with masonry) shown by the solid lines. The stiffening effect of the masonry can be seen. Also, the strength degradation of the masonry is evident in the eastward pushover curve, where the solid and the dashed purple lines converge near the right hand side. Note that all the other pushover curves would similarly converge if they were extended out to greater lateral displacements.

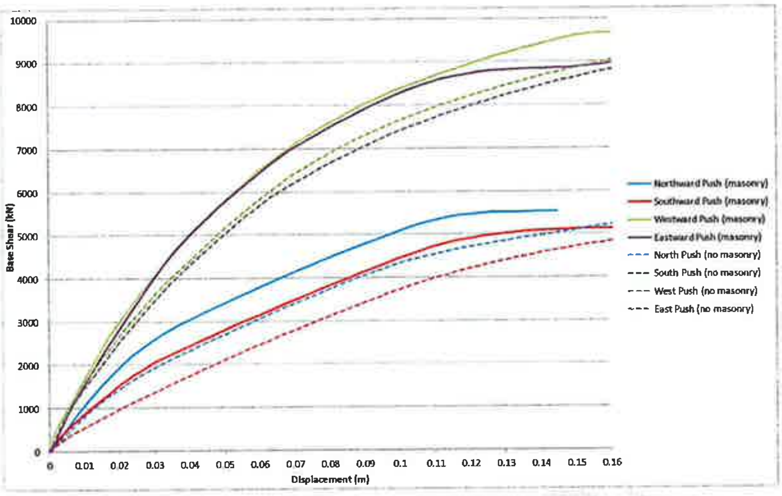

Figure D - Pushover curves, total base shear, with and without masonry.

STRUCTURAL MODELS AND EARTHQUAKE RECORDS

The NTHA analysis runs that have been carried out are shown in Table A below:

Event	4 Sep Darfield	4 Sep Darfield	22 Feb Lyttelton
Structural Model	Model A – no masonry infill	Model B – with masonry infill	Model A – no masonry infill
Earthquake record:			
CBGS	☑	☑	☑
CCCC			☑
CHHC			☑

Table E - Summary of NTHA cases.

The above analysis runs were all carried out with the diaphragm Drag Bar fuse elements described above. This enabled the comparison of results for both structural models, A and B, using the same earthquake record; and also the comparison of results for three different earthquake records using the structural model A.

A further NTHA was then carried out using Model A with the CBGS 22 February Lyttelton Aftershock record, and with the Drag Bars at levels 4 to 6 remaining connected, i.e. not fused, to enable the upper bound diaphragm connection forces to be quantified.

Most of the NTHA's were carried out with all the earthquake direction components acting simultaneously, i.e. north-south, east-west and vertical. To assess the effect of vertical accelerations separate NTHA's were undertaken using only the vertical components of the ground accelerations from the CBGS and CCCC 22 February Lyttelton Aftershock records.

The analyses for the 4 September Darfield Earthquake and 22 February Lyttelton Aftershock both assumed an undamaged structural state at the start of the earthquake record.

The input ground motions used were those recorded at other sites in the Christchurch CBD, located between 650m and 1500m from the CTV site. Tonkin and Taylor have advised that the sites where these recorders were located have broadly similar geological profiles to CTV but that the results from the three suitable records (Christchurch Cathedral College CCCC, Christchurch Hospital CHHC and Christchurch Botanic Gardens CBGS) should be averaged when estimating the response at the CTV site.

For the purposes of the NTHA, reduced length ground motion records were used to reduce computation times. Record start and finish times were selected to ensure that all significant shaking is captured by the analysis and these times are presented in Table B. All results reported in this document have been presented relative to the adopted start time for each acceleration time history record.

Station Name	Event	Start Time (sec)	Finish Time (sec)
Christchurch Botanic Gardens (CBGS)	4 Sep Darfield	28.90	40.90
Christchurch Cathedral College (CCCC)	22 Feb Lyttelton	15.04	23.90
Christchurch Hospital (CHHC)	22 Feb Lyttelton	16.00	27.20
Christchurch Botanic Gardens (CBGS)	22 Feb Lyttelton	16.50	25.50

Table F - Adopted earthquake records, start and finish times.

BASE SHEARS

Peak base shears were recorded during the NTHA as shown in Tables C and D. Results have been recorded at the top of the foundation beams and are presented in units of gravitational acceleration (g), with the total seismic weight above that level being approximately 33,272kN.

Direction	Model A Base Shear (g)	Model B Base Shear (g)
Northward	0.13	0.14
Southward	0.16	0.15
Westward	0.21	0.22
Eastward	0.22	0.22

Table G - Peak Base Shear, 4 September Darfield Earthquake, CBGS record.

Direction	CCCC Base Shear (g)	CHHC Base Shear (g)	CBGS Base Shear (g)
Northward	0.28	0.20	0.26
Southward	0.18	0.21	0.22
Westward	0.38	0.31	0.34
Eastward	0.40	0.39	0.39

Table H - Peak Base Shear, 22 February Lyttelton Aftershock, various records as shown.

The peak base shears above are the overall lateral forces that had to be resisted by the seismic resisting system. Comparing the base shears obtained from the NTHA for Model A and Model B for the 4 September Darfield Earthquake event, as shown in Table C, it can be seen that there is little difference. In Model B the masonry was found to have yielded and just started to degrade towards the end of the Darfield record.

In Table D there is seen to be some variation, but also broad equivalence between the base shears from the three adopted seismic records.

Base shears are generally greater in the east-west direction than north-south because of the greater lateral stiffness in that direction.

STORY DRIFTS

As shown in Figures E to H, maximum storey drifts predicted by the NTHA for the 4 September Darfield Earthquake event are around 1.1% (+/-35mm) in the north-south direction, 0.61% (+/-21mm) in the east-west direction along grid 1 and 0.3% (+/-10mm) in the east-west direction along grid 4. Storey drifts were less along grid 4 because of the greater stiffness of the north core walls in the east-west direction when compared with the South Wall.

It is noted that the predicted drifts for Darfield would have been sufficient to cause interaction with the masonry infill walls and for the precast spandrels to interact with perimeter columns at the east and the south sides if the effective separation gap was less than the 10mm nominal gap shown on the drawings.

Comparison of the results for storey drifts for the masonry and non-masonry models for the 4 September Darfield Earthquake event can also be seen in Figures E to H. This shows that the masonry walls have minimal effect on the drifts in the east-west direction, which is not surprising given that the masonry walls are at right angles to that direction. In the north-south direction the masonry walls have a stiffening effect.

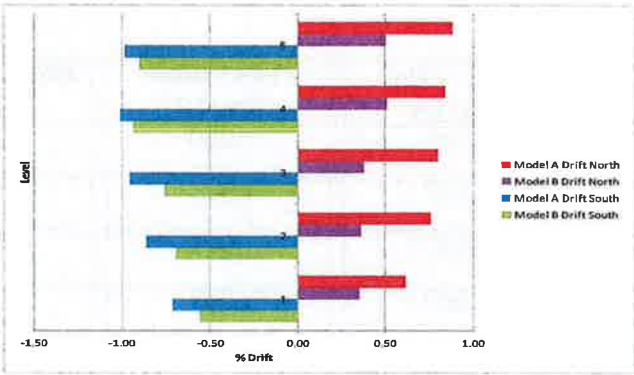

Figure E - Frame A north/south maximum storey drifts – 4 September Darfield Earthquake.

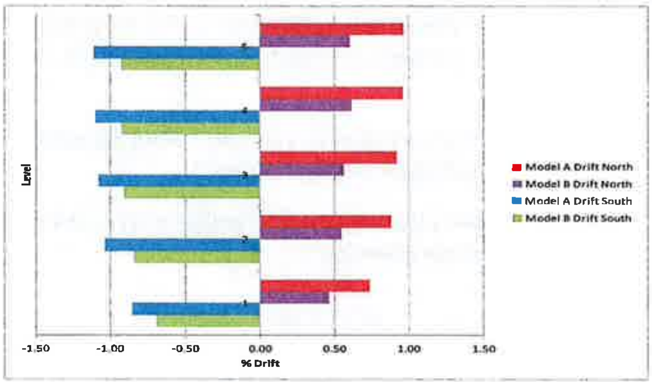

Figure F - Frame F north/south maximum storey drifts – 4 September Darfield Earthquake.

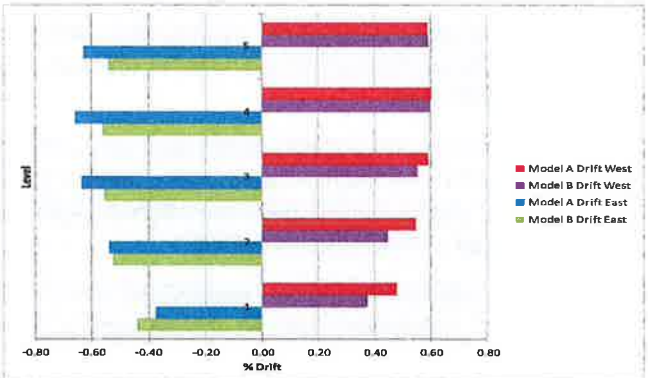

Figure G - Frame I east/west maximum storey drifts – 4 September Darfield Earthquake.

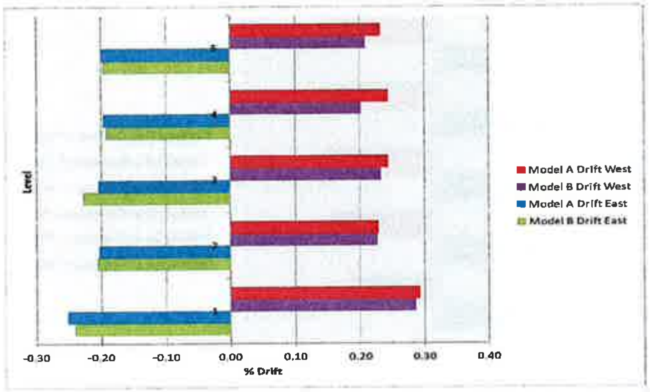

Figure H - Frame 4 east/west maximum storey drifts - 4 September Darfield Earthquake.

It is normal when carrying out time history analysis to use several different earthquake records and to average the results. Comparison of the results for storey drifts for the three different earthquake records adopted for the 22 February Lyttelton Aftershock can be seen in Figures I to L. Storey drifts are shown to be somewhat higher in the north-south direction for the CHHC record and higher in the east-west direction for the CCCC record.

For the 22 February Lyttelton Aftershock maximum storey drifts predicted by the NTHA are shown in figures I to L to be up around 3% (+/-100mm) in the north-south direction and also in the east-west direction along grid 1.

In the east-west direction along grid 4 the storey drifts are predicted to be around 1% (+/-32mm). This variation in drift between grids 1 and 4 reflects the severe plan irregularity and the torsional behaviour discussed earlier.

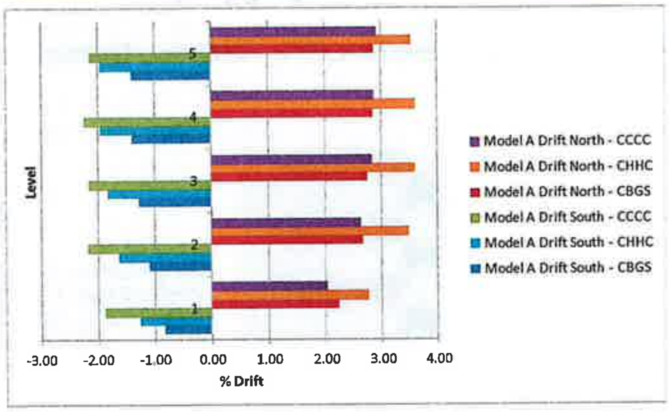

Figure I - Frame A north/south maximum storey drifts – 22 February Lyttelton Aftershock.

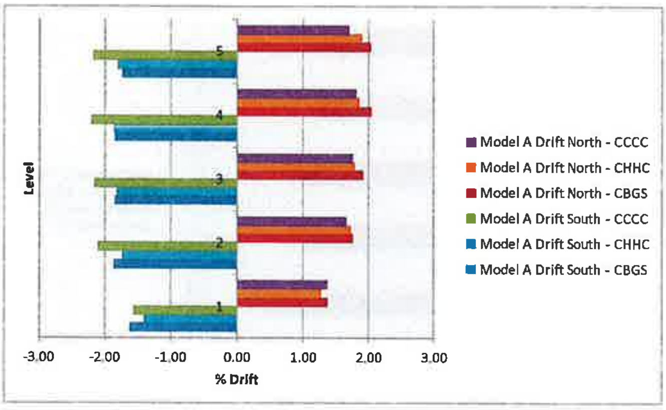

Figure J - Frame F north/south maximum storey drifts - 22 February Lyttelton Aftershock.

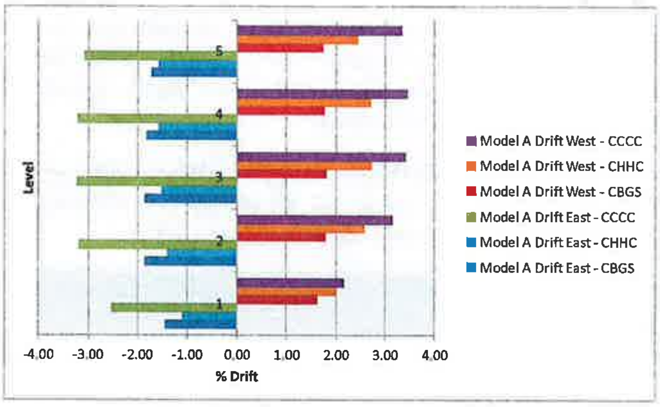

Figure K - Frame I east/west maximum storey drifts – 22 February Lyttelton Aftershock.

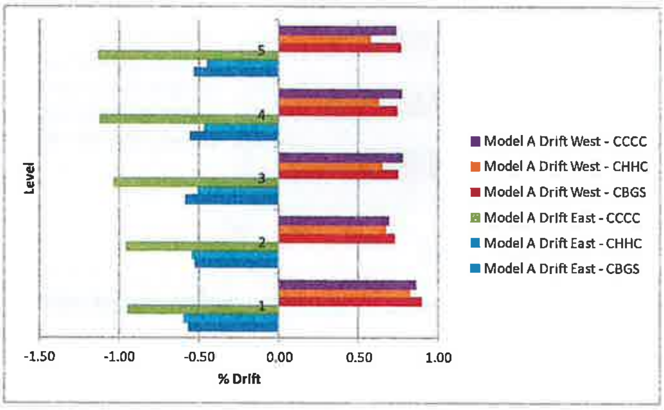

Figure L - Frame 4 east/west maximum storey drifts - 22 February Lyttelton Aftershock.

Overall, taking into account the large number of input variables and assumptions there is considered to be reasonable agreement between the results from the three earthquake records used. Having established broad agreement between the results from the three records, the remainder of the detailed results for the 22 February Lyttelton Aftershock are reported for the CBGS record only, but bearing in mind the potential variations that can occur.

EFFECTS OF MASONRY INFILL WALLS

It has been shown in Figures E to H that the masonry infill walls, if engaged with no effective separation, would generally have caused a reduction in storey drifts for the 4 September Darfield Earthquake CBGS record. This reduction in drift occurs because of the additional torsional resistance of the masonry acting in tandem with the concrete shear walls and the concrete frame.

Figure M below is a plot showing the shear force in a typical 2.3m wide masonry infill panel over the duration of the 4 September Darfield Earthquake CBGS record. It shows that the masonry panel, if fully engaged, would have been participating up to near its bending limited shear capacity throughout. There were nine masonry panels on grid A at each floor level, meaning that the total storey shear contribution from the masonry ranged up to 900kN (where limited by cantilever bending capacity).

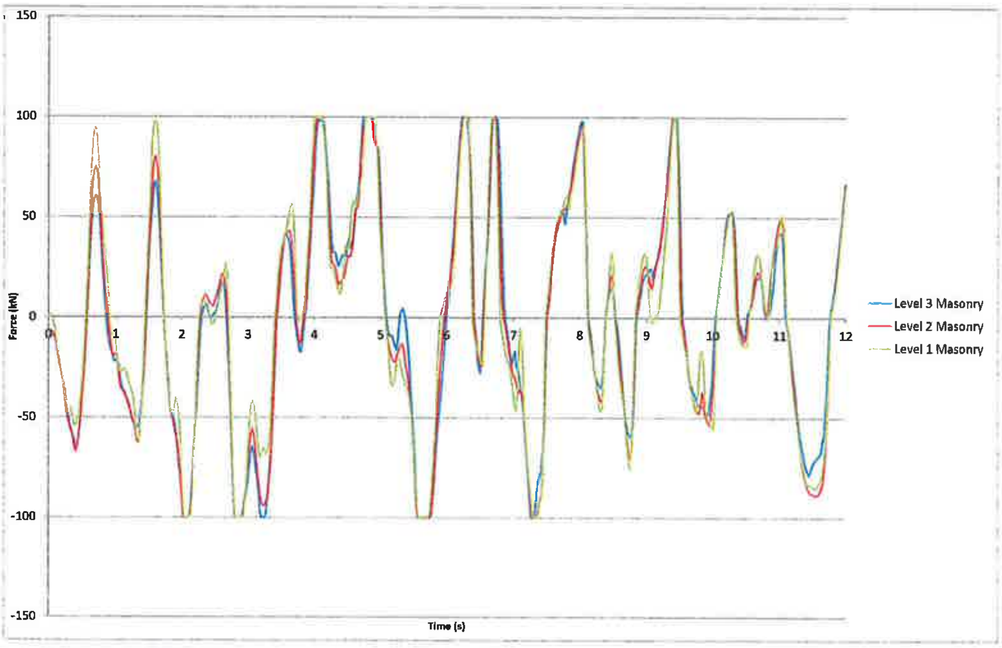

Figure M - Shear Force in typical 2.3m wide masonry infill panel, 4 September Darfield Earthquake CBGS. This shows that he masonry panels reached their shear resistance as limited by flexural capacity on a number of excursions of loading/.

INELASTIC DEMANDS FOR THE 4 SEPTEMBER DARFIELD EARTHQUAKE

The inelastic demands indicated by the NTHA for the 4 September Darfield Earthquake were compared to the damage reported by the OIE, who carried out the post-Darfield damage assessment for the building owner.

The results of the NLTA indicated that inelastic demand from axial actions and bending of the north core walls and the South Wall may have occurred only in the lower part of level 1. From the NTHA for 4 September Darfield Earthquake CBGS, the maximum vertical tensile strain predicted by the analysis in the bottom metre of the grid D wall was 9.7mm/m, and for the South Wall was 6.7mm/m. In other words the steel in the bottom metre of the wall was predicted to stretch by up to 9.7mm, which would lead to cracking in the concrete with the sum of all the crack widths over that bottom metre also adding up to 9.7mm. With Model B the corresponding maximum strains were 4.2mm/m and 6.3mm/m respectively.

First impressions are that these maximum strains suggest a level of damage somewhat higher than the minor 0.3mm wide cracks that were reported by the OIE after the 4 September Darfield Earthquake. The OIE believed no yielding of reinforcing had occurred in the structure. This may be an indication the NTHA model is over-predicting the response, perhaps due to the input ground motion not accurately representing the shaking experienced at the CTV site or the building response to the ground motion being different to that of the computer model..

Figure N below, is a plot of the strain at the base of the southern coupled shear wall at the eastern face, for Models A (no masonry) and B (with masonry).

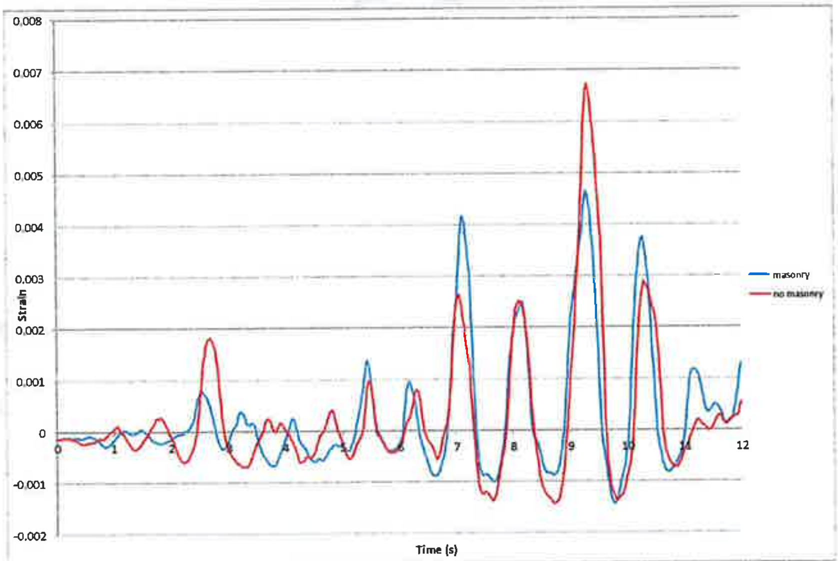

Figure N - Strain at base of South Wall at eastern face, 4 September Darfield Earthquake CBGS.

The NTHA for CBGS 4 September Darfield Earthquake indicated that one of the coupling beams in the South Wall was predicted to yield at level 2, with a maximum strain of 3.5mm/m for Model A and 2.5mm/m for Model B.

The development of column nominal bending capacity was also predicted from the CBGS 4 September Darfield Earthquake NTHA, with column nominal bending capacities developing in the upper level columns on grid F, and then progressing to other locations and to lower levels. Up to 10 columns were predicted to develop their nominal capacities in Model A and three columns in Model B. Again, this appears to predict a level of damage somewhat greater than the hairline cracking reported by the owners inspecting engineer.

ASSESSMENT OF FLOOR DIAPHRAGM CONNECTIONS

The NTHA for CBGS 4 September Darfield Earthquake predicted that the Drag Bars on grids D and D/E would disconnect at Level 4, progressing up to Level 5 and westward to grid D at between seven to nine seconds into the record. The connection forces were found to have only just exceeded the modelled upper bound tensile strength of the Drag Bars, and not all the grid D and D/E Drag Bar connections gave way.

The floors surrounding the lift core and the Drag Bars were not reported as having been inspected following the 4 September Darfield Earthquake and so this aspect of the damage prediction cannot be verified against the actual damage that occurred, if any.

For the 22 February Lyttelton Aftershock CBGS event the NTHA predicted that Drag Bar connections between floors and walls on grids D and D/E all disconnected at between 2.3 and 2.6 seconds into the record. The remaining slab connections to walls C and C/D were also found to be over-stressed once the Drag Bar disconnections occurred, and so the floor diaphragms may have disconnected from the north core completely had this failure mechanism been modelled.

Although the NTHA predicts the disconnection of the floors from the Drag Bars, this needs to be considered in light of the particular structural configuration, and the analysis assumptions and reconciled with observations of the collapse debris on site. The NTHA model indicates that there would have been considerable interaction between the individual walls in the North Core and the connecting floor diaphragms.. Therefore the analysis results appeared to be very sensitive to the assumptions made about the stiffnesses and strength of these connections. Also, in practice the 'disconnection' of the floors from the Drag Bars may have required considerably more elongation and slip than the 2mm to 3mm modelled.

To verify the behaviour of the behaviour of the structure, irrespective of whether or not the diaphragms had disconnected from the North Core, and to enable quantification of the peak diaphragm actions, another NTHA run was completed. Here the Drag Bars remained connected with unlimited tensile capacity at levels 4 to 6. When the results from this analysis were compared with the original analysis (i.e. with the fused Drag Bars), it was found that the differences were small as far as storey drifts were concerned.

It is interesting to see in Figures O and P that the maximum calculated total diaphragm connection force to the North Core exceeded 3500kN in tension at level 3 (0.61g) using the full record without Line masonry. This compares with a total design tension diaphragm connection force of 0.125g that would apply for level 3 from the applicable Loadings Standard NZS4203:1984.

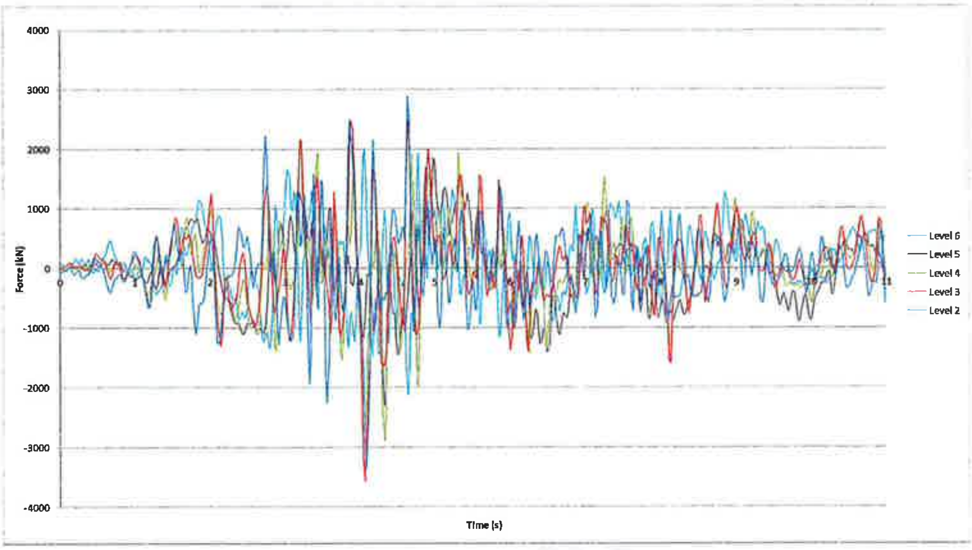

Figure O - North Core total diaphragm N/S actions (no disconnection), CBGS 22 February Lyttelton Aftershock.

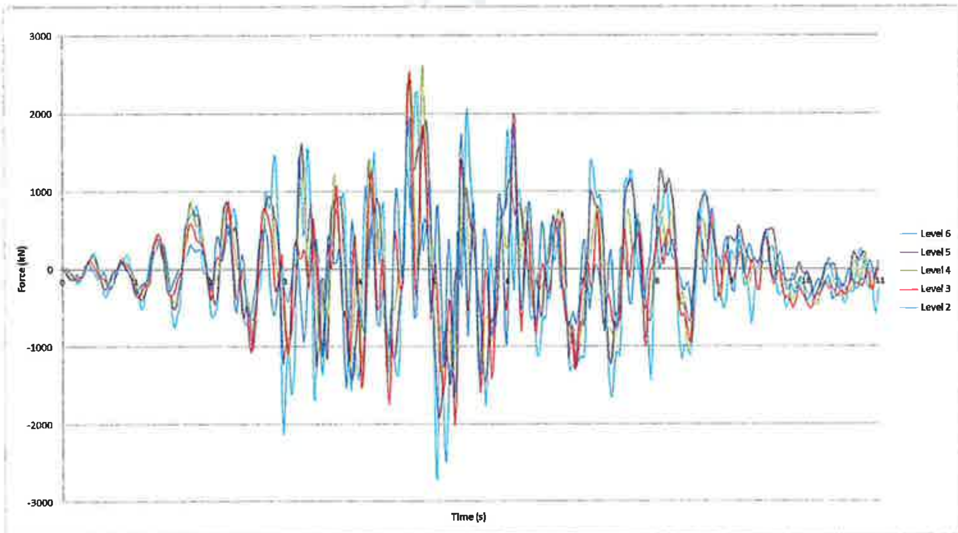

Figure P - North core total diaphragm E/W actions (no disconnection), CBGS 22 February Lyttelton Aftershock.

VERTICAL EARTHQUAKE EFFECTS

Most of the NTHA's were carried out with all the earthquake direction components acting simultaneously, i.e. north-south, east-west and vertical. To assess the effect of vertical accelerations separate NTHA's were undertaken using only the vertical ground acceleration components of the CBGS and CCCC 22 February Lyttelton Aftershock records.

The maximum variation in axial force was obtained during the analysis for a selection of columns under the CBGS and CCCC records. This analysis showed up to +/- 80% variation in axial actions for the most heavily loaded columns. The maximum axial action variation may not occur at the same time as the maximum horizontal actions, however the interaction of vertical and horizontal components is likely to occur at various times and would affect the column behaviour, in particular it would reduce the maximum storey drift that the columns could sustain.

In Figure Q below the variation of axial action, bending moment and shear force is plotted over time for one of the most heavily loaded columns at grid D2 at level 1. This shows the bending moment and the shear force in phase at around one cycle per second, and the vertical component superimposed at a higher frequency. The flat spots on the bending moment and shear force curves represent the times when column hinging is occurring in the model. From this plot it will be appreciated that assessing the demand on, and capacity of the column at any particular instant in time was difficult.

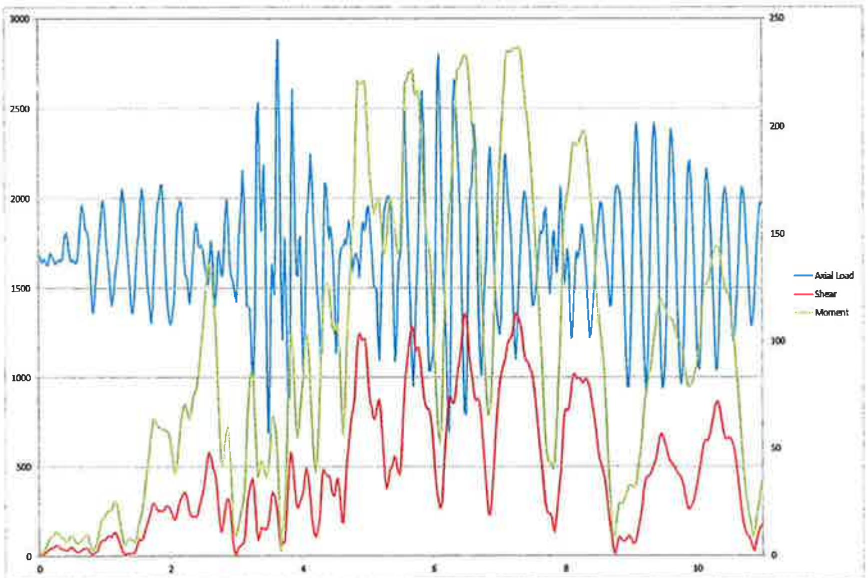

Figure Q - Column Actions D2 Level 1, 22 February Lyttelton Aftershock, CBGS.

To gauge the significance of the vertical accelerations in relation to column strength capacities the M (moment) – N (axial action) interaction diagram shown in Figure R was generated.

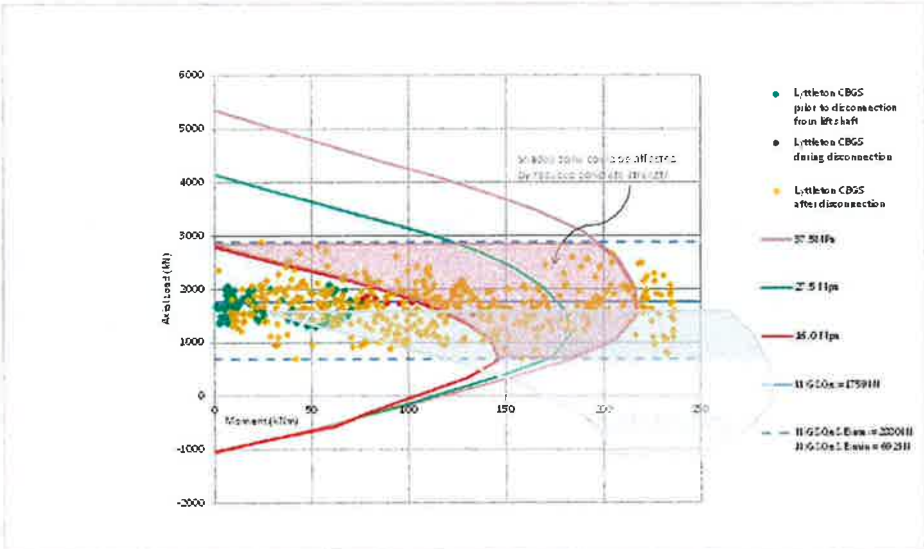

Figure R - Column D2 Level 1 M-N Interaction diagram. (Fy=448MPa, phi=1.0, no masonry.)

Points that can be observed from Figure R include:

- The M-N interaction curves have been drawn for three different concrete strengths 37.5MPa, 27.5MPa and 16MPa. The specified concrete strength for this column was 35MPa, which is the average strength that was used in the NTHA. The lower bound strength found from material testing of column remnants was 16.0MPa.

- The solid horizontal blue line is the gravity axial action on the column, 1759kN. Note column D2 is one of four columns in the building with the highest gravity compression action.

- The dashed horizontal blue lines show the maximum and minimum axial action from the NTHA (CBGS 22 February Lyttelton Aftershock), including vertical earthquake effects.

- The data points marked by green, red and gold diamonds are the moment and axial action in this column at each time step from the NTHA (CBGS 22 February Lyttelton Aftershock record) prior to, during and following disconnection of the floor diaphragms from the Drag Bars.

- There is a slight miss-match where the data points go outside the interaction curve at the right hand side of the chart. This is due to strain hardening effects and because the analysis used M-M hinges, which do not automatically account for the variation in axial action throughout the analysis.

- The shaded area represents M-N combinations from the NTHA that would be affected by reduced concrete strength, i.e. there would be further hinging and potential column failures in this area if we had these reduced concrete strengths.

- The unshaded area represents M-N combinations that are within the admissible range for either concrete strength.

Figure R indicates that vertical earthquake effects alone may not have been enough to fail the columns if they had been constructed at the specified strength. However, vertical earthquake effects in combination with column actions resulting from lateral drift are significant and may have contributed to column failures, particularly if in combination with reduced concrete strengths.

ASSESSMENT OF CRITICAL COLUMNS

General

From the NTHA using the 4 September Darfield Earthquake CBGS record, ten columns were predicted to development of nominal capacity in Model A, and four columns were predicted to develop nominal capacity in Model B (with the masonry infill walls fully engaged). Column hinging was predominantly in the eastern frame, on line F, and initiated in the level 5 columns, progressing to lower levels as the displacement demand increased.

The 22 February Lyttelton Aftershock CBGS NTHA indicated that up to 90 columns yielded and deformed plastically in Model A. Taking into account the non-ductile detailing of the columns, this widespread plastic behaviour points to the potential for column failure. It is significant that the time when widespread column hinging was predicted (around 5.5 seconds into the record) is the same time that storey drifts increased to around 2% (65mm) drift along grid F, as shown in figures U and W.

Potential failure criteria and critical columns

Calculations have been carried out in accordance with the New Zealand Society for Earthquake Engineering (NZSEE) detailed assessment guideline of 2006 to assess the storey drifts that could lead to failure of columns by two criteria, as follows:

The first criteria was taken to be when maximum concrete compression strain exceeded 0.004. The 0.004 maximum compression strain figure was assessed as being a critical condition for these columns because the widely spaced spiral transverse reinforcement was not sufficient to provide effective confinement.

The second criteria is a lower bound shear strength criteria based on the following formula,:

$V_{LB} = 0.85(Vc + Vs + Vn)$

Where Vc = shear resisted by concrete mechanisms

Vs = shear resisted by transverse reinforcement: and

Vn = shear resisted as a result of the axial action

The factor 0.85 was used to obtain an estimate of the lower bound of extensive test data for columns by Priestley et al., as outlined in the NZSEE Detailed Assessment Guidelines 2006.

Inter-storey drifts corresponding to development of nominal capaity of critical columns were found from the non-linear push-over analysis to be around 0.8% to 0.9% (25mm to 30mm) on typical floors and around 1% to 2% (35mm to 40mm) on level 1, which had a greater storey height. This is somewhat higher than the 15mm or so storey drift predicted by analyses of fixed-ended columns at 3.24m high. The difference can be explained by the additional elastic curvature that is available at the column ends from elastic rotations of the incoming beams and floors and additional strain penetration into joint zones. The development of yield in the outermost reinforcing bar was found to occur from the moment curvature analysis at around 0.6% drift.

The following columns were chosen for detailed analysis. Column D2 was chosen because it is one of the most highly loaded columns under gravity compression actions. Column F2 was chosen as a column that experiences high drift near the SE corner, is one of the first columns to yield under lateral displacements and that may have been affected by interaction with adjacent precast Spandrel Panels. Calculations were carried out for both these columns, at all levels, for both the identified failure modes and then the results were plotted for each column at the critical levels, as discussed below.

Analysis results:

The following time history plots of column drifts, Figures S to X, have been developed from the 4 September Darfield Earthquake and 22 February Lyttelton Aftershock NTHA's using the CBGS ground motion record and Model A (no masonry). The potential column and diaphragm connection failure criteria outlined above are superimposed on the figures. Points to note on these figures include the following:

The column failure analyses here consider no masonry infill, average concrete strengths and average gravity compression actions on columns. The potential variations due to variable concrete strength and vertical earthquake accelerations are not shown in these plots - but were calculated to result in reduced drift capacities, particularly if considered together.

Potential beam-column joint failure has also not been included in these Figures.

Figures S and T, for the column at grid D2 Level 1:

1. The east-west drift is key here (the green wavy line), because that is the direction of the floor beams on line 2. The coincident north-south drift and the resultant drift are also shown superimposed for information.

2. The maximum concrete compressive strain limit of 0.004, which indicates ultimate curvature for an unconfined column, is calculated to have been reached at a drift of around 1.02% (the horizontal red lines). 1.02% drift was not exceeded during the 4 September Darfield Earthquake, but it was

exceeded in the 22 February Lyttelton Aftershock, as indicated by the orange shading.

3. The lower bound shear strength limit discussed below is not critical for this column.

4. The times where the floors were calculated to have disconnected from the steel angle Drag Bars on the lift shaft walls at levels 4 to 6 are shown by the dashed vertical lines

Figures U to X, for the column at grid F2:

1. The north-south drift is key here (the blue wavy line) because that is the direction of potential interaction with the precast spandrels. The coincident east-west drift and the resultant drift are also shown superimposed for information.

2. At level 3 (Figures U and V) the maximum concrete compressive strain limit of 0.004 is calculated to have been reached at a drift of around 1.28% (the red horizontal lines) with no spandrel interaction, or at 0.96% (the orange horizontal lines) in the case of a Spandrel Panel adjacent to the column with no initial gap. 1.28% drift was not exceeded during the 4 September Darfield Earthquake, but it was exceeded in the 22 February Lyttelton Aftershock, as indicated by the orange shading.

3. The lower bound shear strength limit is not critical for this column without spandrel interaction. However, with a spandrel adjacent to the column with no initial gap the resulting short column behaviour leads to higher column shears and the potential for shear failure in a column with less than minimum shear reinforcement. This indicates a reduced drift capacity of 0.79% (the blue horizontal lines). This lower bound shear drift capacity of 0.79% was exceeded in the 4 September Darfield Earthquake and the 22 February Lyttelton Aftershock where indicated by the yellow shading.

4. The level 3 column is critical for the 0.004 compressive strain criteria. For the probable shear strength criteria, with a spandrel adjacent to the column with no initial gap the drift capacity is approximately 0.97% at level 2, 0.79% at level 3 (as plotted), but further reduced to 0.7% at level 4 and 0.67% at level 5.

5. The times where the floors were calculated to have disconnected from the steel angle Drag Bars on the lift shaft walls at levels 4 to 6 are shown by the dashed vertical lines

6. Figures W and X show the comparable situation for the column at grid F2, level 5

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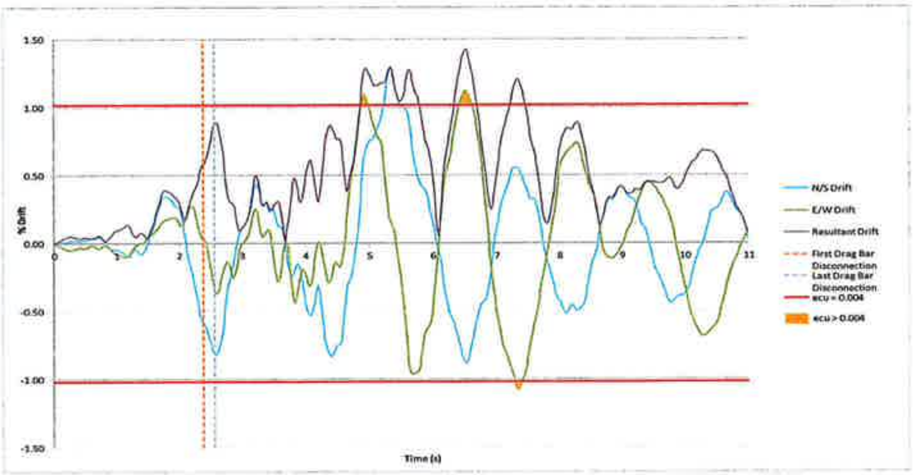


Figure S - Column D2 Level 1 Drifts - CBGS, 22 February Lyttelton Aftershock, no masonry.

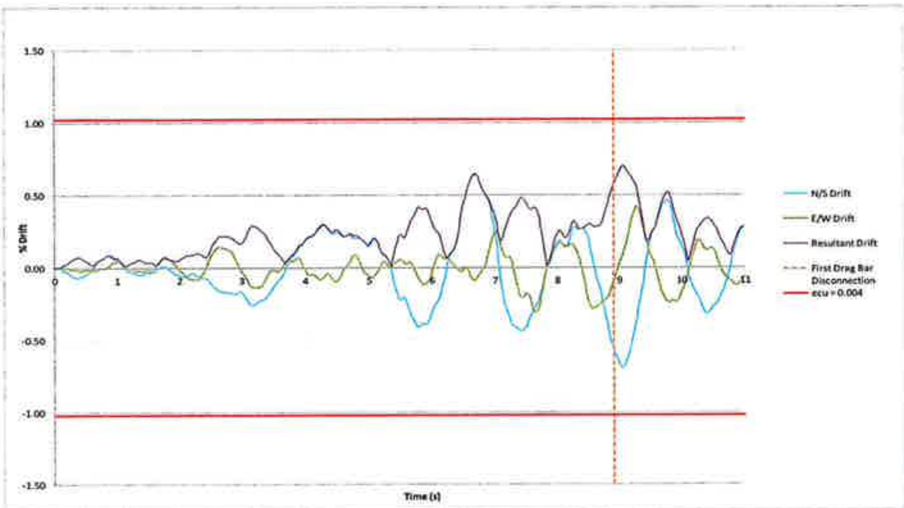


Figure T - Column D2 Level 1 Drifts - CBGS, 4 September Darfield Earthquake, no masonry.

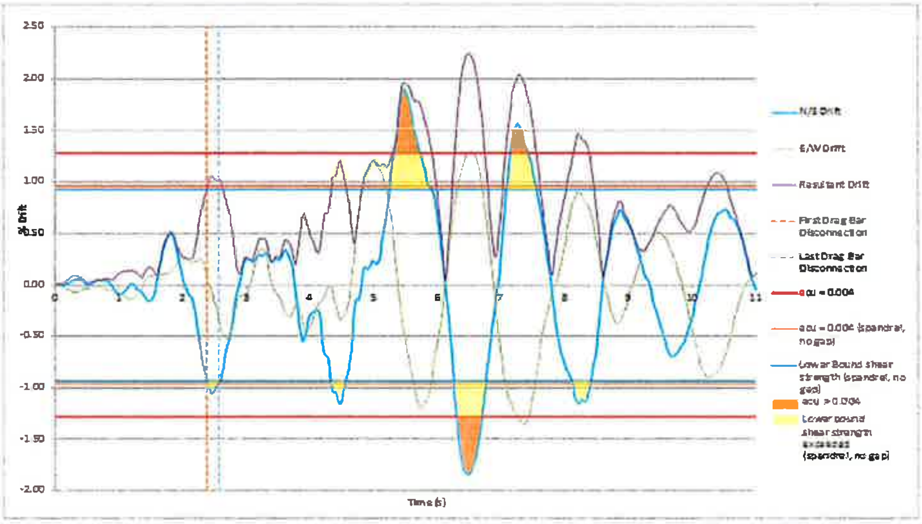


Figure U - Column F2 Level 3 Drifts - CBGS, 22 February Lyttelton Aftershock, no masonry.

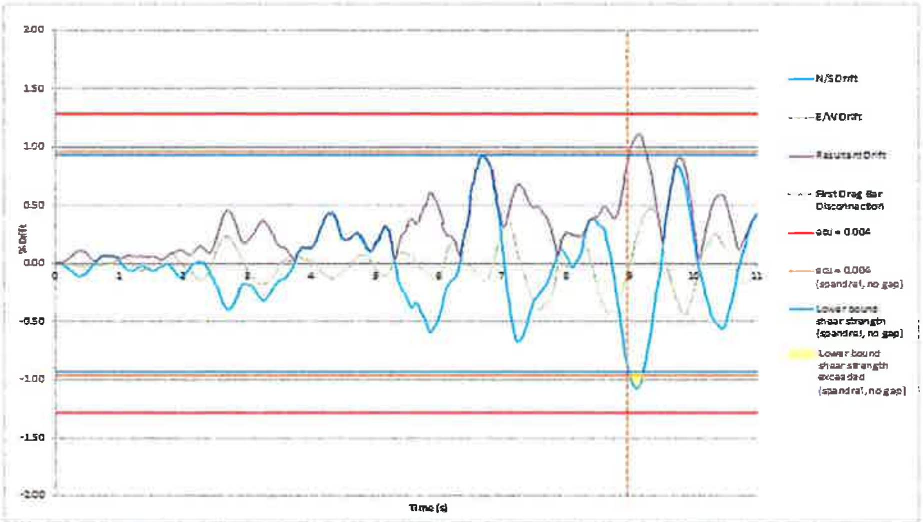


Figure V - Column F2 Level 3 Drifts - CBGS, 4 September Darfield Earthquake, no masonry.

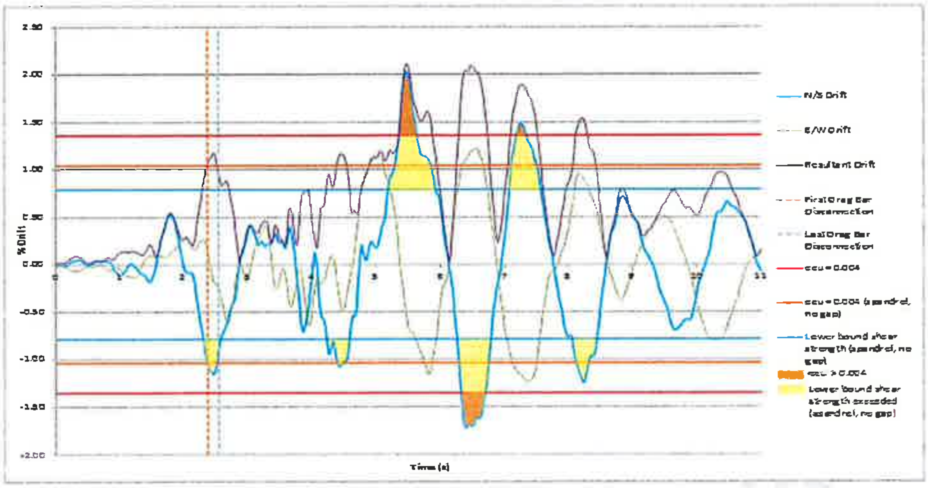


Figure W - Column F2 Level 5 Drifts - CBGS, 4 September Darfield Earthquake, no masonry.

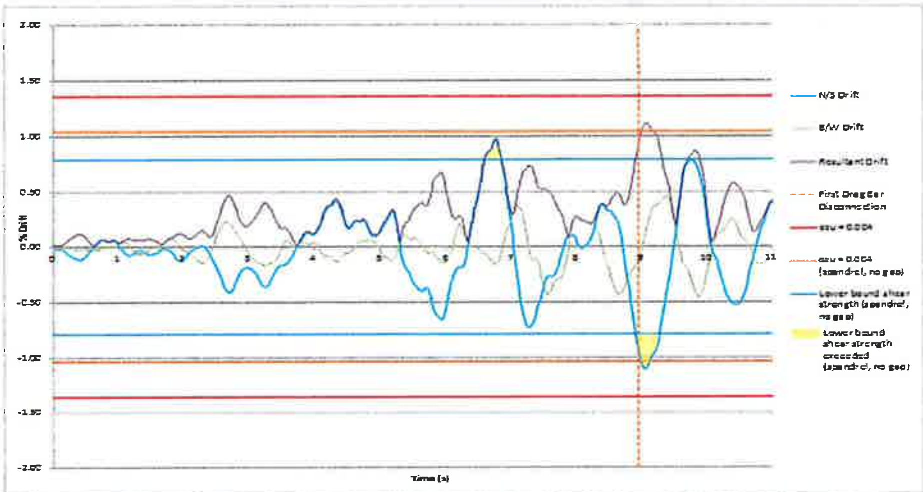


Figure X - Column F2 Level 5 Drifts - CBGS, 4 September Darfield Earthquake, no masonry.

Overall the comparisons show that it is more likely that failure initiated in the high level Line F columns (more shaded areas where demand exceeds capacity) In fact line I columns would be similar because they were shown to undergo similar drifts and they also had precast Spandrel Panels between.

ASSESSMENT OF BEAM-COLUMN JOINTS

Beams columns joints have been assessed as potentially another critical weakness. Two aspects of the joints were considered possible causes of premature failure, these being the joint shear capacity and the potential pull-out of inadequate hook anchorages. The corner column joint were considered to be most critical since they have beams incoming from two orthogonal directions, with bar anchorages from each beam placing demands on the column joint zone.

The assessment method is uncertain and varies greatly with axial action and concrete strength.

Given the greater uncertainties with analysis of the joints, and given the results that had come out of the column analyses, it was decided that limiting the analysis to columns would be sufficient for the purposes of this investigation.

CONCLUSIONS

NTHA has been used to evaluate the response of the CTV Building to ground motions recorded at nearby sites in the Christchurch CBD on 4 September 2010 Darfield Earthquake and 22 February Lyttelton Aftershock. The results are subject to considerable uncertainty due to possible variations in the ground motion at the CTV site, real building response to ground motions, various assumptions made in the analysis, concrete strengths, Spandrel Panel gaps and other variables; however the analysis has indicated the following:

1. Maximum storey drifts around 1% for the 4 September Darfield Earthquake and around 3% for the 22 February Lyttelton Aftershock.

2. A highly irregular seismic resisting structure, with drifts for the 22 February Lyttelton Aftershock at the east, south and west sides being two to three times the drifts at the north side of the building.

3. The masonry infill walls, if fully engaged in the seismic response, were seen to introduce additional plan irregularity and vertical irregularity to the system. However, at the same time they also generated additional torsional resistance in tandem with the concrete shear walls and the concrete frame. The overall effect of the masonry was generally to reduce storey drifts, which were seen as the major factor leading to column collapse. It also would have increased the torsional response of the building so that the columns on Lines 1 and F experienced the greatest level of drift in the building.

4. It has been difficult to reconcile the damage predicted by the analysis with reports of damage by others after the Darfield Earthquake. The analysis generally indicated a higher level of damage than what was reported.

5. The analysis has given insights into the relative likelihood of various failure mechanisms.

6. The primary seismic resisting system (i.e. the concrete shear walls) did not fail prior to the collapse of other parts.

7. Some of the floor diaphragm connections were predicted to fail during the 4 September Darfield Earthquake, and all of the floor diaphragm connections were predicted to fail early on during the 22 February Lyttelton Aftershock.

8. Assuming average concrete strengths, and without the effect of vertical accelerations or Spandrel Panel interference, the columns were predicted to survive the 4 September Darfield Earthquake, but were predicted to fail in the 22 February Lyttelton Aftershock by exceeding the drift capacity that would lead to ultimate compressive strain exceeding 0.004. This was considered to be a potentially critical condition given that they were essentially unconfined by the widely spaced spiral reinforcement.

9. If full contact between the columns and the precast spandrels was assumed then column failure would have been initiated earlier, either by the 0.004 compressive strain criteria or by shear failure brought about by the resulting short column behaviour. The compressive strain being a combination of axial and flexural compressive strains.

10. Vertical accelerations alone were considered not to have caused columns to fail, unless concrete strength in critical columns was extremely low. However when combined with lateral drifts, vertical accelerations certainly could have contributed to column failure.

11. Concrete strengths were found from testing to vary widely and in many cases appeared to be below the minimum specified strengths at the time of the collapse. Low concrete strengths in critical columns, particularly if in combination with vertical accelerations would have led to premature failure by any of the above criteria.

APPENDIX E – ELASTIC RESPONSE SPECTRA ANALYSIS

EARTHQUAKE AND AFTERSHOCK RECORDS

Strong Motion Recordings

The nearest strong motion recordings of the three Canterbury earthquakes of 4 September 2010, 26 December 2010 and 22 February 2011 were downloaded from the GeoNet ftp site. (GeoNet is a collaboration between the Earthquake Commission and GNS Science that provides public access to hazards information including earthquake records at www.geonet.org.nz/earthquake).

The instruments are located at the following sites, and as shown on the map below in relation to the CTV site:

- Botanical Gardens (CBGS)
- Cathedral College (CCCC)
- Christchurch Hospital (CHHC)
- Rest Home Colombo Street North (REHS)
- Westpac Building (503A)
- Police Station (501A)

Figure 110 - Locations of Geonet Strong motion Recorders relative to CTV Site

For each earthquake, or aftershock, the response spectra records have been converted into a 5% damped response spectrum by GNS.

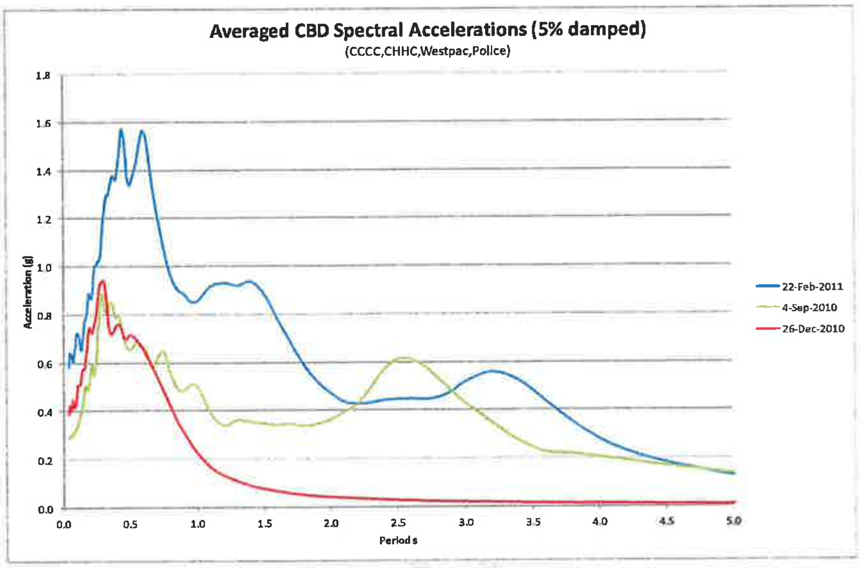
The axes of the instruments are very close to N-S and E-W, as are the axes of the CTV Building.

There is not sufficient detailed information about the ground conditions at the recording stations, or the ground conditions between the recorders and the CTV site, to be totally accurate about an equivalent record for the CTV site. However, since the stations effectively surround the CTV site on three sides at fairly close proximity then the records are very helpful and the best available to demonstrate the average level of ground motion.

It is important to recognise the difference between spectral records and the spectra used for design. It is appears that some level of calibration is required to determine building response reliably from earthquake ground motion and response spectra. The level of calibration required is difficult to determine and beyond the scope of this investigation. It is likely to be dependent on the specific nature and configuration of each structure.

Averaged Resultant Response Spectra

Averaged resultant spectra were derived from records at Westpac Building, CCCC, CHHC and the Police Station as these were closest to CTV. The resultant of the north-south and east-west accelerations at each recorded period was used as it is not possible to determine the sign of each component. Discretised linear curves were then fitted to the spectral plots to allow input of the spectra into ERSA software (Figure 111).



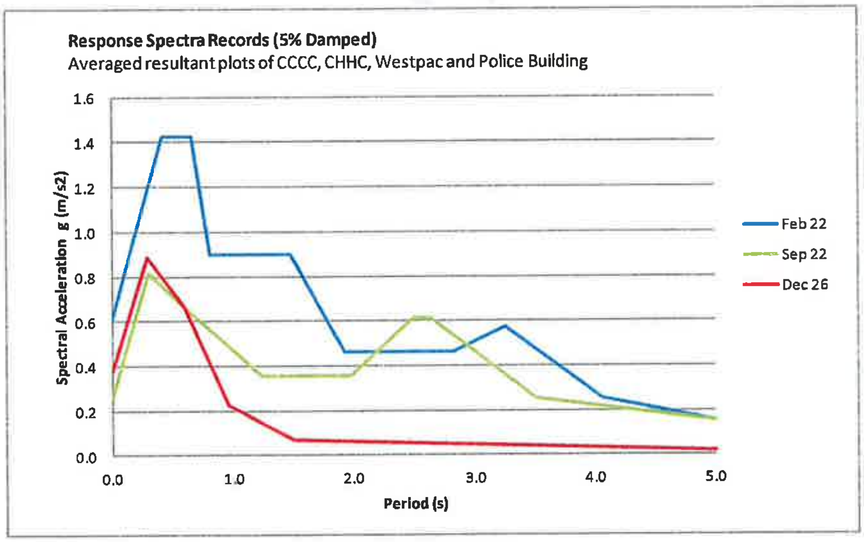


Figure 111 - Averaged resultant response spectral records (5% damped) from CCCC, CHHC, Westpac and Police building GNS records. The lower plot has been discretised into linear steps to facilitate use in ERSA.

The NZS 4203:1984 design spectra have been superimposed on these in Figure 112. These have been scaled to ensure the base shear form ERSA corresponds to 90% of that derived from a static analysis in accordance with NZS 4203:1984.

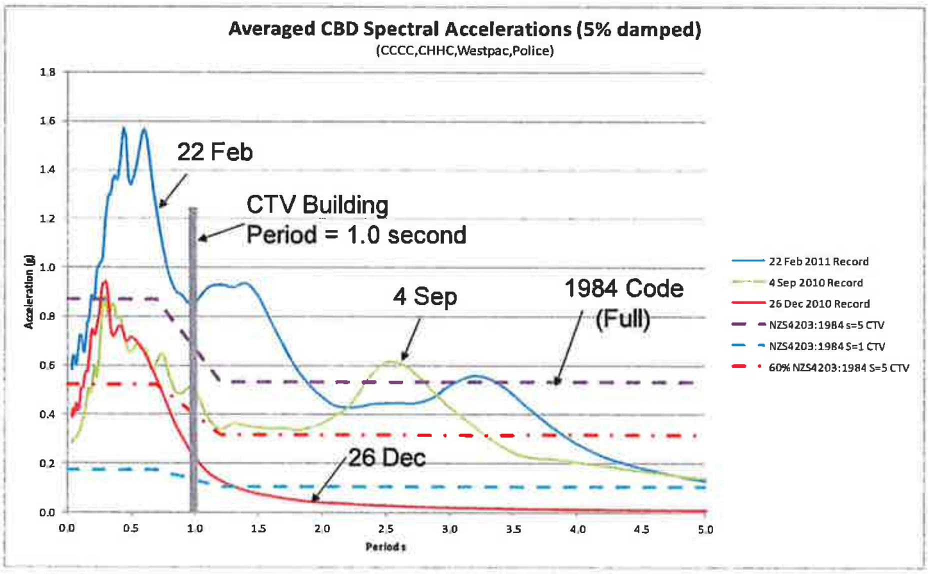


Figure 112 - Averaged CBD response spectra superimposed with design spectra for CTV Building according to NZS 4203:1984. The NZS 4203 spectra have been scaled by around 1.5 to achieve 90% of the first mode base shear derived from a static analysis in accordance with the requirements of NZS 4203:1984

RESPONSE SPECTRA AND DRIFTS

The February Aftershock spectra were plotted in Figure 113. The scaled spectra necessary to achieve 0.35% and 0.75% drifts along Line F were developed and also plotted. The 0.35% drift corresponded to the Line F north-south drift in response to north–south loading necessary to develop the nominal bending capacity at the base of the South Wall. the 0.75% drift corresponded to a lower bound marker at which column collapse initiation in conjunction with Spandrel Panel interference was found likely to occur in the displacement compatibility analyses in Appendix F.

The effect of damping on the spectra can be readily seen from the comparative plots. For the displacement compatibility analyses the 5% damped response spectra were used in line with common practice.

If collapse occurred at drifts along Line F of around 0.75% this indicates that the CTV Building collapsed prior to it achieving its intended fully ductile performance. The intention of NZS 4203:1984 was that a structure designed to be fully ductile (s=1) should be able to survive an earthquake response equivalent to that shown by the s=5 curve, with a low probability of collapse.

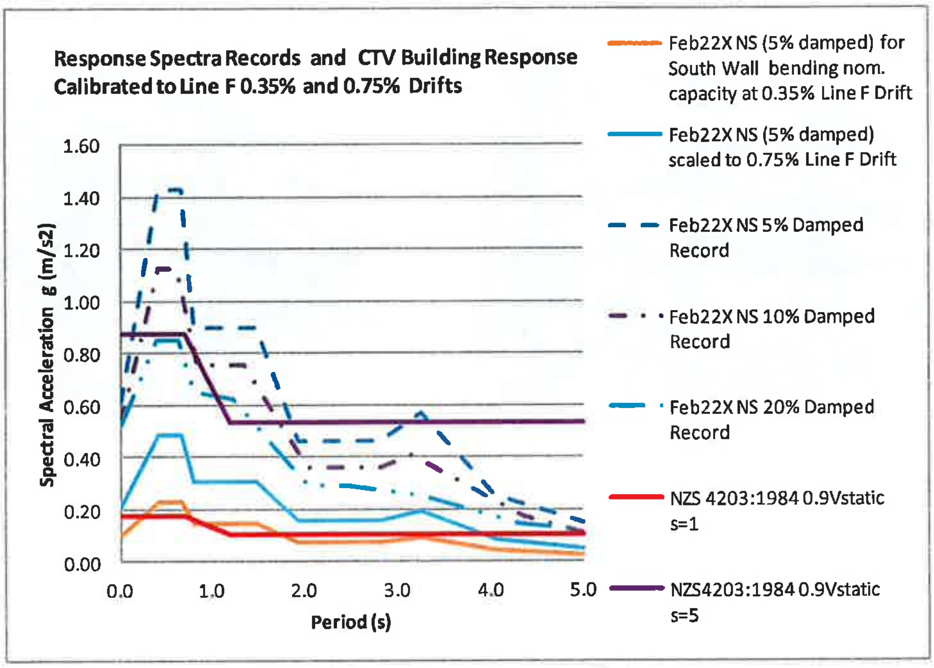


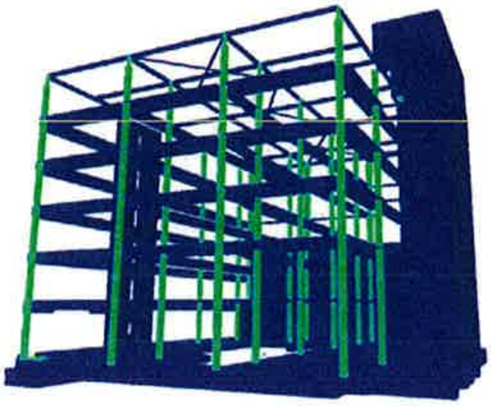
Figure 113 - Response spectra records for various levels of damping are shown alongside those calibrated for building response at development of 0.75% drift on Line F. Also shown is response at development of nominal bending capacity in the base of the South Wall at 0.34% drift on Line F. It appears likely that collapse occurred between 0.75% and 1.3% drifts on Line F, The expectation of NZS 4203:1984 was that drifts of 1.23% should have been attainable with a low probability of collapse.

ERSA MODELLING

Introduction

Linear 3D structural elastic response spectra analysis ("ERSA") using ETABS software was carried out. This was to enable checks against Standard loadings and to investigate structural behaviour in various configurations. These configurations included:

- Primary walls alone.
- Primary walls and the masonry infill wall on grid A.



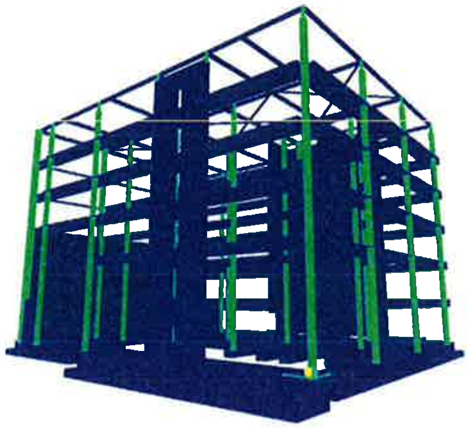


Figure 114 - ETABS computer model - views from north-east and south-east.

ERSA Computer Modelling Assumptions

This method of analysis was required to be used for this structure by NZS 4203:1984 and was reportedly used by the Design Engineer of the CTV Building in 1986. It is a method still commonly used today for the structural design of multi-storey buildings.

The main assumptions in modelling were as follows:

- Upper bound soil stiffness, as recommended by Tonkin & Taylor.
- Concrete walls only as seismic bracing, with secondary frames considered separately.
- Line A block walls as seismic bracing (because of lack of separation).
- Fully ductile response.
- Concentric, +0.1b and -0.1b accidental eccentricity.

Superimposed dead load was estimated as 0.55 kPa throughout.

Live load was taken as 2.5 kPa as applicable for "offices for general use" according to NZS4203.

Seismic live load was calculated to be 0.83kPa in accordance with NZS4203.

Material properties were calculated based 25 MPa for the North Core and South Wall.

Effective section properties of the walls, were calculated in accordance with the recommendations of NZS 4203:1984 and NZS3101:1982, - using the paper titled "The Analysis and Design of and the Evaluation of Design Actions for Reinforced Concrete Ductile Shear Wall Structure" by T. Paulay and R.L. Williams (NZSEE Bulletin Vol13 No.2 June 1980) as the basis.

Concrete walls and coupling beams in the South Wall were modelled, allowing flexibility in the beam/wall joint zones. Refer Figure 114 and Figure 115.

The subsoil was considered to be flexible as defined in NZS4203:1984.

In 1986 it was common practice by many engineers to assume that foundations were rigid, and this was allowed by NZS 4203:1984. However for this investigation, as the building was founded on flexible subsoil, with shear walls cantilevering off foundation beams, the assumption of flexible soil springs was considered to be appropriate to gain a better insight into the behaviour of the structure.

Sensitivity analyses were carried out using a range of values for foundation spring stiffness. The appropriate stiffness of soil springs for seismic analysis were calculated by geotechnical engineers Tonkin & Taylor Limited ("T&T") as outlined in (Sinclair 2011). T&T gave three sets of values for soil spring stiffness; one considered to be a lower bound stiffness, one considered to be the most likely stiffness and one considered to be an upper bound stiffness. For the purposes of this report the upper bound stiffness values (i.e. 1.36k) were used. This was to achieve a conservative estimate of the natural periods of the structure and of the design base shear.

These analyses did not incorporate the effects of the internal and perimeter frames columns along Line 1, 2, 3 4 and F in accordance with the primary and secondary frame analysis approach of NZS 4203:1984. The assessment of those frames and the effect of engagement of perimeter columns with the pre-cast concrete Spandrel Panels was accounted for separately in the displacement compatibility analyses described in Appendix F.

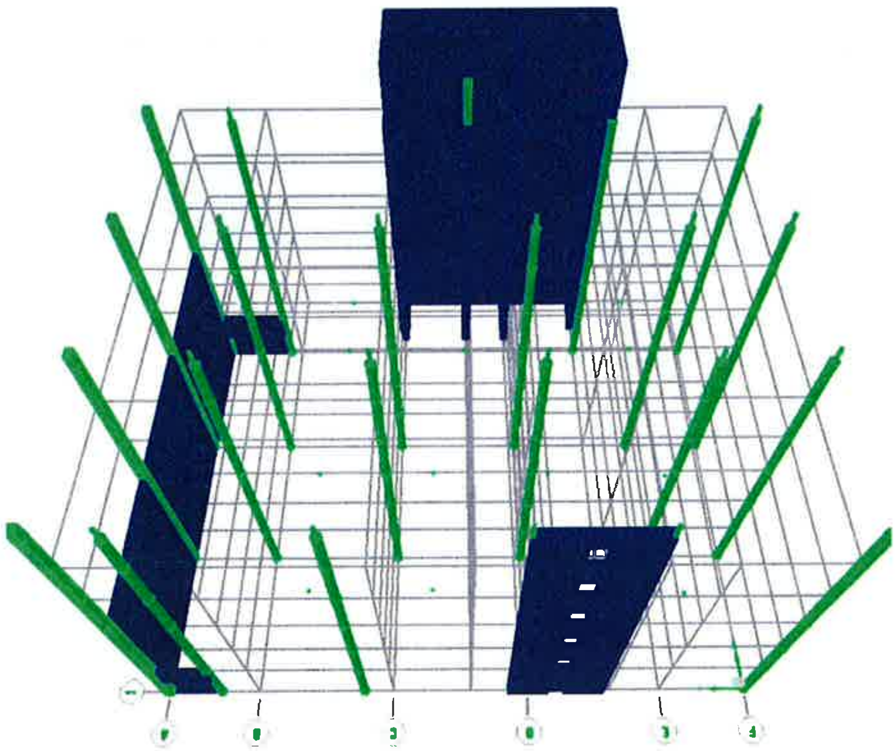


Figure 115 - 3-D view of ETABS model 1c showing layout of concrete shear walls, concrete masonry walls and columns (beams are not shown in this view for clarity).

Each of the models were analysed using ERSA taking into account the relevant natural periods. The dynamic base shear was scaled to 90% of the equivalent static value in accordance with the Code. The analyses were carried out for fully ductile response using a structural type factor S=1.0, and a structural material factor M=0.8, (SM=0.8) in accordance with NZS4203:1984. The resulting design actions were scaled upwards to determine design actions for an elastically responding structure with S=5.0, (SM=5.0).

One model included only the North Core and the South Wall as the primary seismic resisting system. This reflected the authors understanding of the original design intent based on the structural calculations provided by the Design Engineer. In their calculations the beam/column frames on Lines 1, 2, 3, 4 and F, and the concrete masonry wall on Line A were not included in the seismic analysis as part of the primary seismic resisting structure.

The recommendation of the Commentary to the Concrete Structures Standard NZS3101:Part2:1982 cl C3.5.14.1 on identifying whether the beam and column frames should be considered as part of the primary seismic resisting system was that "frames in parallel with slender shear walls should be designed as fully participating primary members".

There was no specific guidance as to what a slender shear wall was, however many engineers would consider the South Wall with a height to length ratio of 3.3, when measured to the underside of Level 6, to have been slender. The guidance did not extend to the perimeter frames transverse to the South Wall on Line F.

Commentary recommendations to Standards are generally recognised however as not requiring mandatory conformance. Displacement compatibility analyses of the secondary frames would have been expected to have ensured that the secondary frames were adequately designed for the anticipated inelastic displacements of the South Wall.

It is likely that other buildings designed to these standards may also have secondary frames that do not satisfy displacement compatibility demands of ductile shear walls or frames.

The column that was located at grid intersection 4-D/E was modelled as part of the North Core. This column was connected to the top of the core wall directly. It was considered to be an integral part of the North Core and therefore part of the primary seismic force resisting system.

Modelling of Line A Masonry Infill Wall

The masonry infill wall on Line A was included in the ERSA model as part of the primary seismic resisting system at the time of the September Earthquake. This was to reflect the lack of effective separation to the outer face of the masonry infill prior to the 22 February Aftershock as reported by Eyewitness 16.

The design drawings show that the Line A wall should have been fully grouted of the horizontal pre-cast concrete boundary members. Greased starter bars at 600 mm centres were fixed into the under-side of the precast beams along Grid A (Figure 116 DENG Dwg S9 section 6). There was a D12 horizontal bar shown in the shaded top course of the infill masonry in Section 6 of S9, and a note on Dwg S17 required "Grade B masonry all cells filled" indicating that it was to have been filled. As it was Grade B masonry, it required observation by an engineer during construction.

The Design Engineer calculations indicate that the intent may have been to leave the top courses only partially filled to reduce interference with the structure; however this did not get shown on the Drawings.

However based on the statement of Eyewitness 16 complete grout filling of the top courses may not have been achieved. The adjacent building had been demolished after the 4 September Earthquake leaving the wall exposed to the weather. Eyewitness 16 and another worker were required to remove mortar trimmings off the face of the wall in preparation for strapping and cladding the wall a day or so before the 22 February 2011 Aftershock. They found that the top courses of the masonry infill at each level were apparently hollow, and no vertical gaps were apparent to them between the masonry and the columns, but some horizontal gaps were found in places between the top courses and the beams. They were able to knock out the face of one top course block on the Level 1 portion of the wall with hammer blows which showed it was hollow (Figure 117).

They also found that the wall wasn't fully grouted when they later drilled holes into it for timber strapping fixings, and that the movement joints in the masonry between the panels and the columns were filled with mortar. The outside face joint appeared to have had a nominal amount of mortar filling the outside edge. This may have been an attempt to ensure the boundary wall fire rating was achieved. This would have limited the ability of the masonry panels to move as three separate panels and increased their collective stiffness.

The rectangular columns sat out proud of the wall face by 20 mm or so. Photos showed vertical separation gaps between the corner column (Grid 1/A) and the short Line 1 return wall on the south face of the building. A horizontal separation gap appeared to be evident between the Line 1 wall and the beam above it (Figure 116). However no vertical or horizontal gaps were evident from the photo on the West wall along Grid A.

An engineering inspection by the OIE after the 4 September 2010 Earthquake found sealant on the inside face adjacent to a car park column (Figure 119). This indicated that the specified vertical gap appeared to be as specified when viewed from the inside of the building. At the time of the inspection the boundary wall of the adjacent building would have still been in place. The wall showed no signs of any cracking. The top course of block work can be seen to have been fitted snugly under the precast beam above it as specified.

The inside of the west wall, after the 4 September 2010 earthquake showed some damage to the linings at Level 2 (Figure 119).

During the collapse (Figure 120) the masonry wall along Grid A broke apart, in some cases as distinct panels, consistent with the design drawings (DENG Dwg S17). Two way diagonal shear fracturing, indicative of severe cyclic demands on the Level 2 masonry was also evident.

This indicates that infill masonry above Level 2 on Line A fully developed its shear capacity prior to the collapse and therefore affected the response of the structure to the Aftershock (Figure 121).

Gravity actions and confinement of the masonry by the surrounding beams and columns due to the compromised gaps, means it is likely that the masonry infill wall acted as confined masonry making the wall very stiff and strong.

The extent of that interaction with each other and the columns either side through the full earthquake response was difficult to quantify accurately. Therefore, for the ERSA the masonry wall panels were modelled two ways. One as cantilevers pinned to the floor diaphragm above, ignoring the effect of interaction between the sides of the panels and the columns. . The second as a fixed edge panel.

The level of stiffness introduced into the structure even without contact between panels and columns with this approach was sufficient to move the centre of stiffness significantly towards the western wall, compared to that found using the model ignoring the effect of the masonry infill wall. The additional effect of fully locking up the walls as an integral unit would further moved the centre of stiffness westward but by a smaller amount and similarly reduced north-south displacements on Line F.

The modelling of the Line A masonry wall was therefore difficult to define accurately, but assumptions were made as follows:

- Connection to the floor diaphragm was assumed to occur at the top of the masonry wall, although no vertical load carrying load paths were included.
- The masonry walls were input assuming the 10 mm gap between panels and the 25 mm gap between the masonry and concrete framing was present.
- The masonry material properties were E = 15 GPa.

The masonry walls at level 1 on Line 1 and 4 were not been included in the computer modelling as they were specified as separated structurally from the columns each side with reasonable gaps - but had reinforcing steel connecting them to the floor beam above (DENG Dwg S9 Section 2 and 3) .

For normal design purposes, to allow for various torsional effects, the loadings Standard requires the seismic force to be applied at points +0.1b and -0.1b eccentric from the centre of mass - where b was the breadth of the building perpendicular to the horizontal loading direction under consideration. The object of this study was to identify the cause of collapse rather than to design the building, and for this reason the analyses focussed on the concentric mass analysis runs.

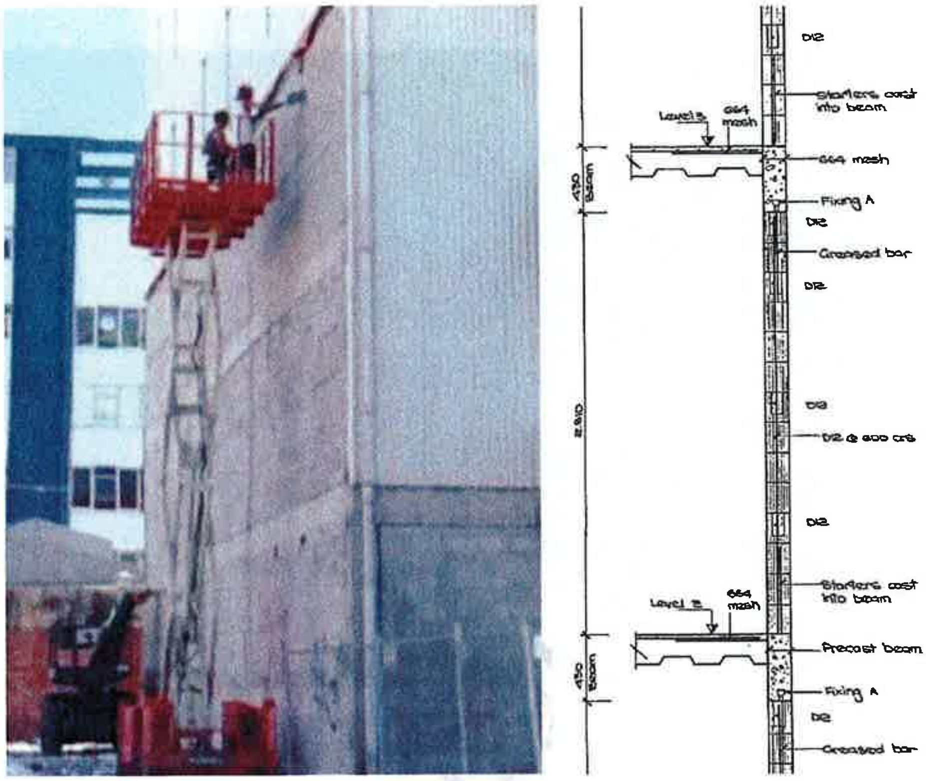


Figure 116 - West wall on Line A (left to right) : Being prepared for strapping and cladding a day or so before collapse on 22nd February; b) Connection of west wall block work into floor beams top and bottom (portion of DENG Dwg S9 Section 6), showing the fixing of the top of the wall into the structure.

Figure 117 – Workers (including Eyewitness 16) hammering face of top course block away on west wall near Line A / 1 corner column. This indicates hollow blocks occurred in the top course and no separation joints on the outer face of the masonry.

Figure 118 - Line A infill masonry wall adjacent to column with no obvious cracking after the 4 September earthquake. Flexible sealant is visible between masonry and column.

Figure 119 - Inside of the west wall at Level 2 after the 4 September 2010 Earthquake shows some damage to the linings.

Figure 120 - West wall on Line A at southwest corner shortly after the collapse.

Figure 121 - West wall shortly after collapse. The corner Grid 1/A column is still standing and the wall panels have broken free in panel sections in places. The edges of the panel section are square consistent with the design drawings. Diagonal fracture of the masonry infill that has fallen outwards from level 2 is highlighted. This indicates that the infill masonry above Level 2 fully developed its shear capacity prior to the collapse and therefore affected the response of the structure to the Aftershock.

ERSA RESULTS

Irregularity and Torsional Response

One of the features of the CTV Building seismic force resisting structure was the asymmetrical plan layout of the concrete bracing walls. The North Core being substantially stiffer and stronger than the southern coupled shear wall in the east-west direction, meaning that the structure had a severe plan irregularity. This can be seen in the following Figure 122 showing the plan location of the centre of mass and the plan location of the centre of rigidity for each of the main floor diaphragms at levels 2 to 6.

Note - The centre of rigidity is defined as follows:

> *When translational lateral loads are applied at the centre of rigidity of a particular floor diaphragm, with no loads applied to any of the other floor diaphragms, the displacements of that diaphragm will have only translational components with no rotations. It should be noted that the resulting displacements of the diaphragms at other levels in general will contain translational as well as rotational components.*

With the concrete masonry walls also participating as part of the seismic force resisting system, the structure was highly irregular in plan in both directions and also a vertical irregularity was introduced at level 4 due to the participation of the masonry walls below that level. The level 4 floor acted as a transfer diaphragm transfering seismic loads from the north core to the west side masonry walls and vice-versa.

With the secondary concrete frames added, the irregularities were moderated slightly by the action of the frame which was located more centrally than the walls.

The authors consider that the seismic resisting system in this building was irregular and there are warnings in the loadings Standard NZS4203 that the seismic performance of such irregular structures is less predictable than for equivalent symmetrical structures.

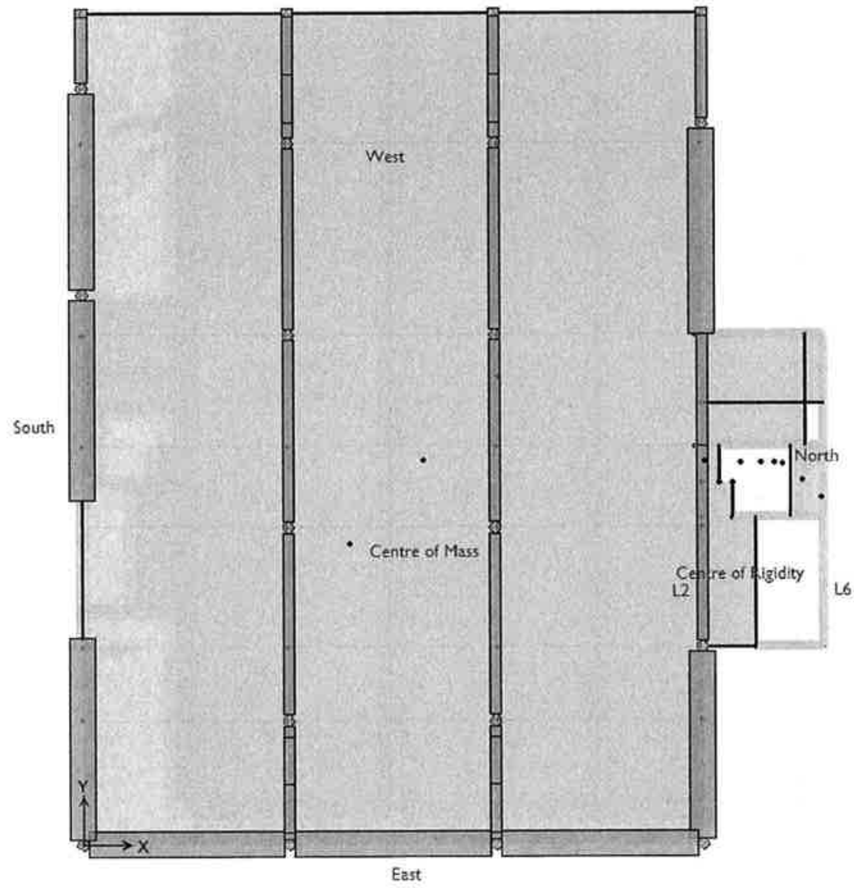


Figure 122 - Centre of Mass and Centres of Rigidity for each Floor (North Core and South Wall only as primary seismic resisting system) The centre of rigidity is close to alignment with the centre of mass for North-South excitation, but highly eccentric from the centre of mass for east-west excitation. This means that the building would have more torsional or twisting response to east-west components of earthquake ground accelerations than to north-south ground accelerations if the Line A masonry infill wall was adequately separated from the structure.

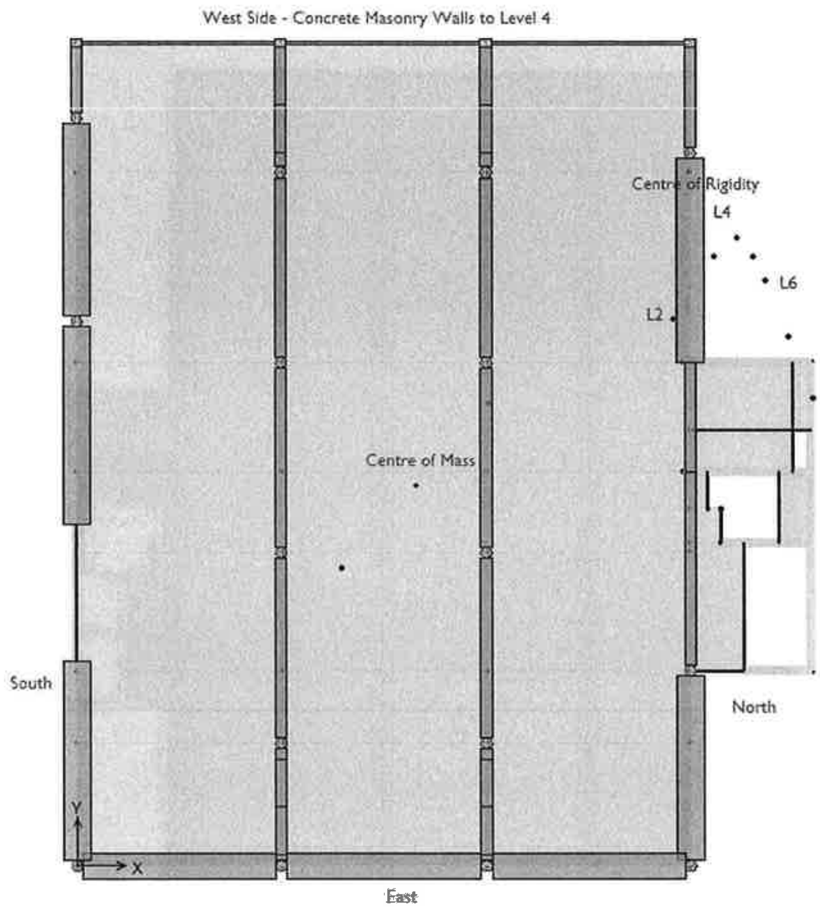


Figure 123 - Centre of Mass and Centres of Rigidity for each Floor (North Core, South Wall and Line A masonry infill wall in contact with structure). The centre of rigidity is highly eccentric from the centre of mass in both directions due to the participation of the west side Line A masonry infill wall below Level 4. The earthquake loads act through the building's centre of mass at each floor level, and the building tries to resist the earthquake actions through its centre of rigidity at each level. The offset between the centre of mass and the centre of rigidity is the eccentricity that determines the level of twist or torsion that results. With Line A masonry wall in full contact with the structure the building will have increased torsional response to north-south earthquake ground motions.

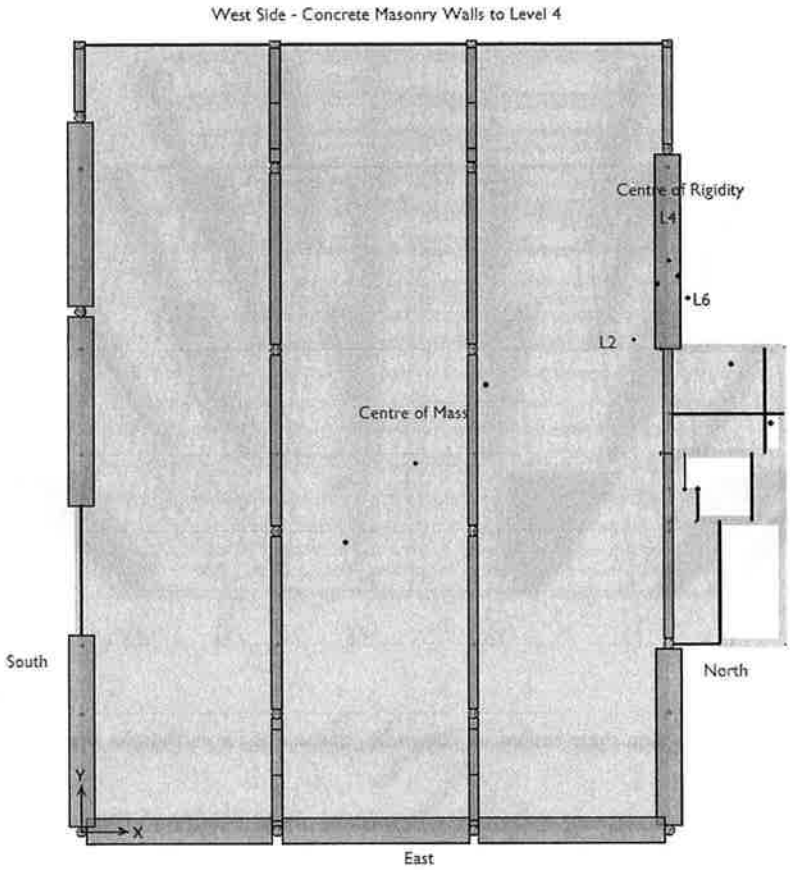


Figure 124 - The changes in the locations of the Centre of Mass and Centres of Rigidity for the building each Floor (North Core, South Wall and Line A masonry infill walls and secondary frames on Lines 1, 2 3, 4 and F). The centre of stiffness moved south and west, reducing the torsional response of the building, due to the effect of the secondary frames.

The effect of the eccentricity between the centre of mass and centre of rigidity on the torsional behaviour on the columns can be seen in the following plots of column shear actions for earthquake shaking in the east-west direction.

Figure 125 is a plot of shear actions (which are related directly to the level of column drift) on the east-west axis of the columns. The columns nearer to the south side and nearer to the top of the building were subject to higher drifts and shear actions (and corresponding bending moments). This is because they were furthest from the centre of rigidity and so experienced more seismic drift, and because the frame attracted a bigger proportion of the total storey shear compared with the walls nearer the top of the building.

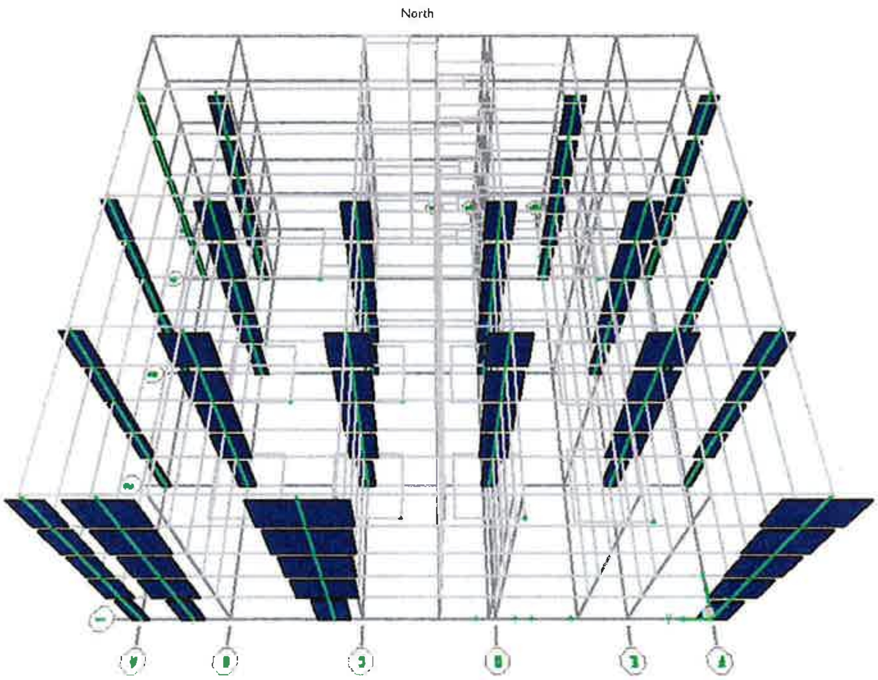


Figure 125 - Plot of column shear actions on east-west column axis for earthquake shaking in east-west direction.

The internal columns did not experience significant shear actions in the north-south direction because the Line 2 and 3 floor beams ran east-west. Similarly the columns at the west side above level 4 did not experience significant shear actions in the north-south direction because there were no beams at the west side above level 4.

The columns at the east side and nearer to the top of the building were subject to the highest drifts (and corresponding shear and bending moments). This was because they were furthest from the centre of rigidity and so experienced more seismic drift. The magnitude of the column drifts in this north-south direction along Line F were of a similar order to column drifts in the east-west direction along Line 1, the direction of earthquake shaking modelled. The effect of torsion was therefore significant. The columns at the east side of the building formed part of a two-way moment frame and so they also experienced concurrent actions in each direction.

Shear Distribution in South Wall

The distribution of shear actions in the Line 1 wall and the susceptibility to damage from north-south seismic events, were found to have been significantly affected by the Line A masonry infill wall - and the lack of connection of the floor diaphragm to walls D and E at levels 2 and 3.

ERSA undertaken neglecting the Line A infill masonry wall, found that inter-storey shear design actions that adequately enveloped the design actions for the worst case East/West and North/South design condition (Figure 126).

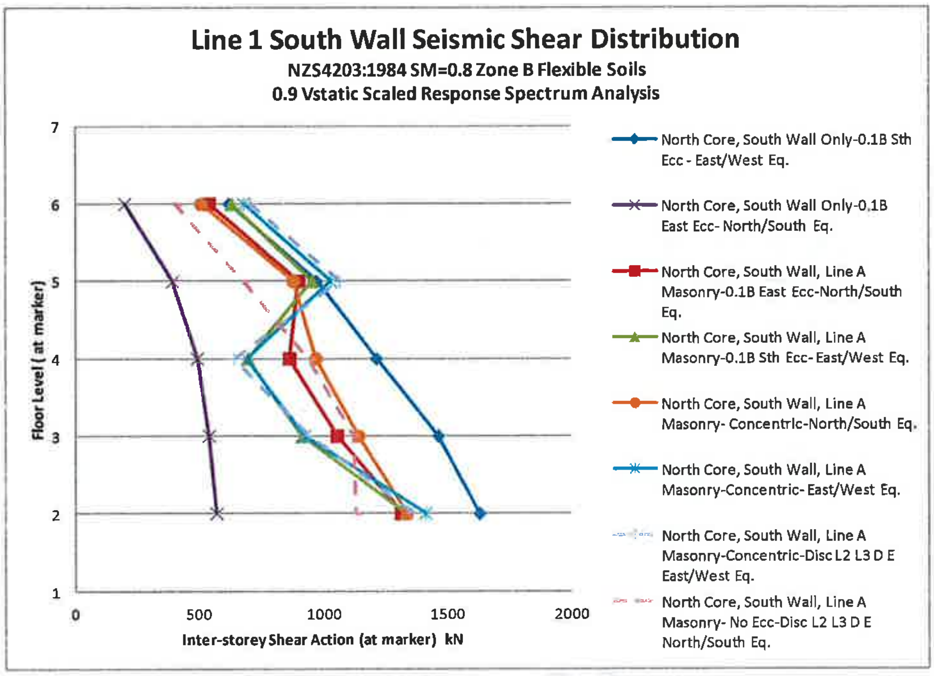


Figure 126 – Seismic shear distribution in accordance with NZS 4203:1984 on South Wall with varying conditions of mass eccentricity and with or without the Line A masonry infill wall. This shows that the Line A wall did not affect the design of the South Wall but made it more susceptible to damage under North-South earthquake loading.

When buildings are designed for earthquakes, lateral loads are applied to simulate the earthquake effects in each direction of the building. In this case the east/west event dominated the design when the masonry infill wall on Grid A was excluded. For north/south events with no Grid A masonry the actions on the Line 1 wall are 35% of those for the east/west event.

The Line A masonry wall elevated the actions on the Line 1 South Wall, in response to North-South seismic events. As a consequence the South Wall attracted similar inter-storey drifts and shear actions for both north-south and east-west events, with a slightly worse condition in a North-South event (Table 11)

It can also be seen that the inter-storey shear actions reduce significantly between level 4 to 5 (13.4 m marker) and Level 3 to 4 (10.2 m marker) for east-west events when the Line A masonry in-fill wall is included in the analyses.

As a consequence greater damage would be expected in the Line 1 South Wall panels between level 4 and 5 than between Level 3 and 4. This was initially thought to be consistent with the comparative damage observed in the shear wall remnants E3 and E4 discussed in the Site Examination and Materials Tests report (Hyland 2011). However if collapse had occurred at Line F drifts of 0.75% 1.3%, the level of shear demand in the South Wall would not appear to have been sufficient to have caused that damage. So it was concluded that the damage likely occurred as a consequence of the wall falling onto the debris.

The Line A wall therefore significantly increased the susceptibility of the Line 1 South Wall to damage in a north-south seismic event, and by implication also to an event with resultant northwest-southeast direction.

It was permissible according to the New Zealand Concrete Structures Standard NZS 3101:1984 cl. 3.15.14.3 (a) to ignore the seismic requirements of the standard, if the Line A masonry in-fill complied with the provisions for Group 2 Secondary members.

The calculations by the Design Engineer indicate that the intention may have been to more fully isolate it from the structure than what was shown on the Drawings.

However the Group 2 provisions effectively required the masonry in-fill block work to be protected, but not for its effect on the overall structural response to be considered.

This provision of the standard should be reviewed and could mean that other buildings of the era may have unanticipated responses to earthquakes.

Better guidance is required as to what is acceptable interaction of Group 2 secondary elements with structures.

Line A Masonry In-Fill Wall Shear

The distribution of shear actions on the Line A wall, higher at level 4 and least at Level 1.

The wall had a nominal capacity of 2822 kN, including partial contribution from the columns entrapped by the infill.

ERSA indicated tha the wall would have been expected to maintain shear capacity until the South Wall minimal bending capacity developed but may have become severely damaged in shear between Levels 2 and underside of Level 4 prior to drifts of 0.75% to 1.3 % developing on Line F.

Severe diagonal fracture of Level 2 wall panels was observed in photos of the debris immediately after the collapse (Figure 61).

South Wall and North Core Flexural Action to Capacity Ratios

The flexural demand to capacity ratios of the walls on Line 1 and 5 in East-West direction were calculated relative to the SM=0.8 response spectrum analysis actions and subject to axial gravity actions of G+Qu in accordance with the loading standard NZS 4203:1984 (Table 9).

As an isolated element the South Wall was adequately strong. However its large differential in its strength and that of the North Core meant that the building was torsionally unsymmetrical. This links back to the issue of the lack of geometrical symmetry in the structure.

The South Wall is likely to have yielded or sustain permanent damage and then deform inelastically but in a reliable way well before the Line 5 wall developed its nominal capacity. Given the large difference in the capacity ratios it is likely that the Line 5 wall remained largely elastic at drifts between 0.75% and 1.3% on Line F.

This study raises questions about the adequacy of the provisions for the ductile design of torsionally irregular structures. In this case though the North Core walls were detailed for ductile performance they were so strong they in fact responded as elastic elements, working in conjunction with a fully ductile perimeter South Wall.

Flexural Demands vs Capacity of South Wall

The flexural demand on the Line 1 wall was greatest at its base where the introduction of the partial masonry in-fill to the Level door constrained the wall to act as a cantilever wall between level 1 and 2. This behaviour was indicated by the cracking patterns in the wall after the collapse (Figure 82).

The nominal bending capacity of the wall, without strength reduction factors, was calculated to be 21103 kNm based on the average tested concrete strength from cores in the wall of f'c= 32.0 MPa and average tested yield stress of the reinforcing steel of Re =448 MPa. The flexural demand on the wall for the SM=0.8 spectra was M*=12605 kNm. An actual S value at which yield is calculated to have initiated in the Line 1 South Wall is

$$S_{act} = \frac{M_n}{M^*} = \frac{21202}{12605} = 1.7$$

The displacement of the structure as whole was therefore calculated to have remained constrained by the elastic displacement of the Line 1 South Wall up to demands 1.7 times the SM=0.8 spectra.

Actions and displacements on the secondary structural members should have been able to be sustained up to at least this level of structural demand. This was found by the displacement compatibility analysis to have been the case with The Line 1 South Wall bending yield at its base occurring at a calculated drift along Line F of 0.45%. This was less than the lower bound drift of 0.75% at which collapse could have initiated in columns on Line F.

Flexural Demands vs Capacity of Line 5 Wall

The flexural demand on the Line 5 wall was greatest at its base.

The nominal bending capacity of the wall without strength reduction factors was calculated to be 167904 kNm based on the average tested concrete strength from cores in the wall of f'c= 32.0 MPa and average tested yield stress of the reinforcing steel of Re =448 MPa.

The flexural demand on the wall for NZS4203:1984 SM=0.8 spectra was approximately M*=20500 kNm.

The ratio of nominal capacity over demand for the SM=0.8 spectra.

$$\frac{M_n}{M^*} = \frac{167900}{20500} = 8.2$$

It is therefore not surprising that no obvious damage was sustained by the North Core walls prior to the South Wall developing its nominal bending capacity.

The North Core was very strong relative to the South wall. While the South Wall suffered damage prior to collapse, that damage is not considered to have caused the collapse.

Wall	Flexural Capacity Mn kNm	SM=0.8 Action M* kNm	Demand / Capacity M*/Mn	Comment
1				
Level 1	21103	15056	0.71	E/W eq
		13440	0.64	N/S
5				
Level 1	167900	20400	0.12	E/W

Table 9 - Flexural demand / capacity ratios for walls on Line 1 and 5 relative to NZS 4203:1984 design actions.

APPENDIX F - DISPLACEMENT COMPATIBILITY ANALYSIS TO STANDARDS

INTRODUCTION

NZS 4203:1984 required that the secondary frames in earthquake resisting structures were able to satisfy certain prescribed displacement compatibility criteria. These were intended to ensure that the secondary frames would remain reliable under the specified earthquake loadings.

In this section the structure as it appeared to be prior to the 4 September Earthquake was assessed against those criteria.

METHOD

The displacement compatibility performance of the Secondary Frames on Line 2 and F was analysed using displacement profiles from ERSA of the Primary Frame analysed using NZS 4203:1984 design spectra and also actual records. The Primary Frames being the South Wall, North Core and the Line A masonry infill wall. The Line A wall was included in this assessment as it appeared to have been engaged with the structure at the time based on the report of Eyewitness 16.

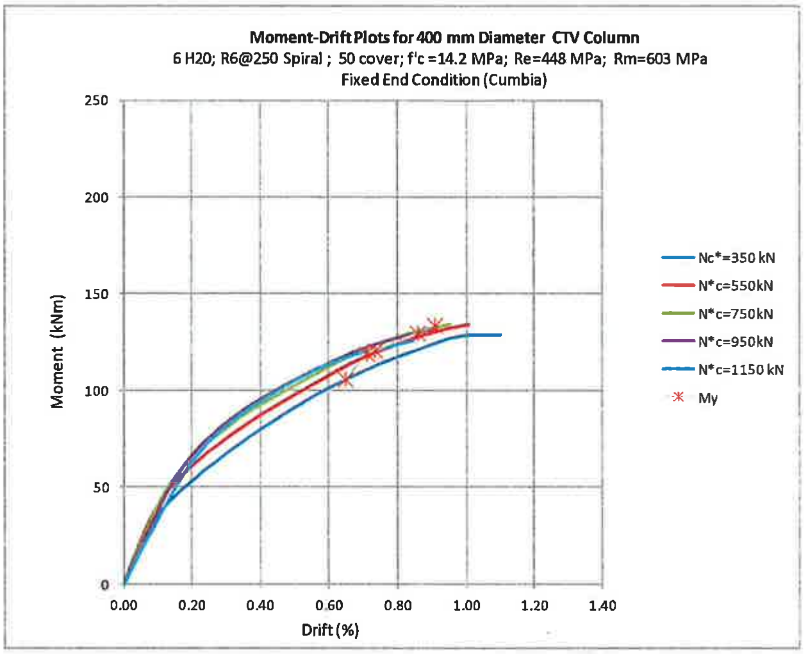
The displacements determined by ERSA at secondary frame column lines were then applied at floor level of plane frame analyses of Line F. The plane frame analyses used inelastic column moment curvature relationships to better approximate the effective inelastic stiffness EI_{eff} of the columns and the bending moment and shears in the frame.

PLANE FRAME MODELLING OF LINE F

The moment displacement and moment curvature relationships at varying axial actions were developed using Cumbia software (Montejo and Kowalsky 2007). The plot of the moment-drift curves for f'c =14.2 and f'c =27.5 MPa concrete are shown in Figure 127. These show how the moment–drift relationship varies with concrete strength and axial compression.

The effective stiffness of the T-beams, with slab over-hang modelled as 2.5 time slab thickness, was set at 0.6 EIg in accordance with the recommendation of NZS 3101:2006.

Concrete strength was set at 29.6 MPa in the columns on Line F as this was the average tested concrete column strength.



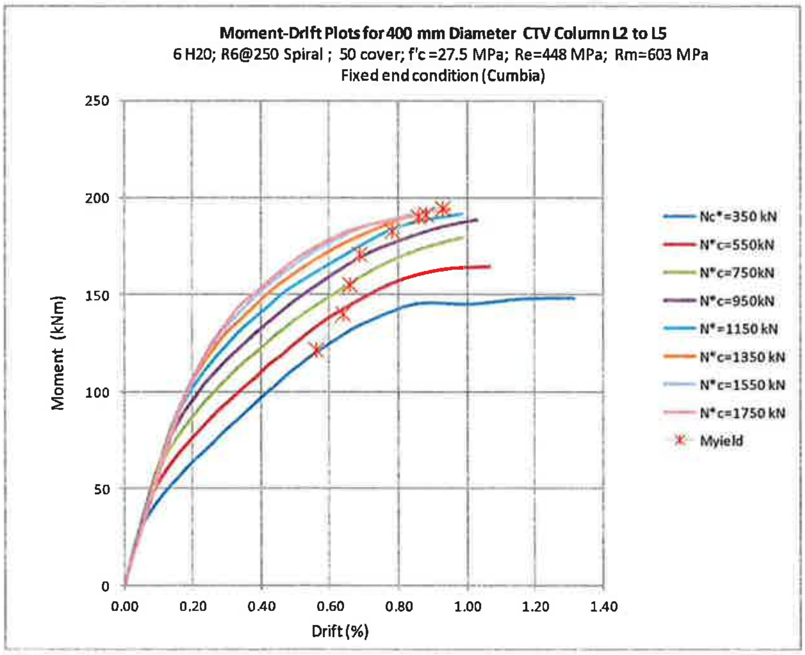


Figure 127 - Moment-Drift plots for 400 mm diameter CTV columns for f'c=14.2 and 27.5 MPa concrete, using Cumbia software for fixed end conditions. Concrete limiting strain was set at 0.004. This shows that yielding of the reinforcing steel starts at higher drifts as the axial compression action increases. Similarly the ability of the columns to bend more after starting to yield reduces as the axial compression action increases. Columns in the upper levels had lower axial compression actions compared to the lower level columns, and so were able to sustain more inelastic demand than those at lower levels. The crosses indicate the point at which yield of the extreme reinforcing steel bar occurs designated as the yield moment of the column.

DISPLACEMENT COMPATIBILITY ANALYSES

The ERSA were run for North-South and East-West applications of the response spectra of the earthquake records and the NZS 4203:1984 design spectra applied concentric with the centre of mass to aid comparison of the relative demands on the structure with the limiting assumption of elastic response.

Column F/2 ERSA Displacements and Inter-storey Drifts
Using unscaled earthquake spectral records from 4 Sep 2010, 26 Dec 2010 and 22 Feb 2011

Level	Load	Displacement UX (N-S) m	Inter-floor Drift (N-S) m	Inter-floor Drift (N-S) %	Displacement UY (E-W) m	Inter-floor Drift (E-W) m	Inter-floor Drift (E-W) %
North -South Earthquake							
L6	SEP4X	0.1569	0.0350	1.08	0.1004	0.0243	0.75
L5	SEP4X	0.1219	0.0349	1.08	0.0762	0.0238	0.74
L4	SEP4X	0.0870	0.0328	1.01	0.0523	0.0217	0.67
L3	SEP4X	0.0541	0.0292	0.90	0.0306	0.0180	0.55
L2	SEP4X	0.0249	0.0249	0.67	0.0127	0.0127	0.34
L1	SEP4X	0.0000			0.0000		
L6	DEC26X	0.0682	0.0151	0.47	0.0595	0.0149	0.46
L5	DEC26X	0.0531	0.0150	0.46	0.0447	0.0144	0.44
L4	DEC26X	0.0381	0.0141	0.43	0.0303	0.0125	0.39
L3	DEC26X	0.0240	0.0127	0.39	0.0178	0.0103	0.32
L2	DEC26X	0.0113	0.0113	0.31	0.0075	0.0075	0.20
L1	DEC26X	0.0000			0.0000		
L6	FEB22X	0.3113	0.0694	2.14	0.2010	0.0486	1.50
L5	FEB22X	0.2419	0.0693	2.14	0.1524	0.0478	1.47
L4	FEB22X	0.1726	0.0652	2.01	0.1046	0.0433	1.34
L3	FEB22X	0.1074	0.0580	1.79	0.0613	0.0359	1.11
L2	FEB22X	0.0494	0.0494	1.34	0.0254	0.0254	0.69
L1	FEB22X	0.0000			0.0000		
East-West Earthquake							
L6	SEP4Y	0.0921	0.0205	0.63	0.0842	0.0209	0.65
L5	SEP4Y	0.0715	0.0205	0.63	0.0632	0.0203	0.63
L4	SEP4Y	0.0511	0.0191	0.59	0.0430	0.0177	0.55
L3	SEP4Y	0.0319	0.0170	0.53	0.0253	0.0146	0.45
L2	SEP4Y	0.0149	0.0149	0.40	0.0107	0.0107	0.29
L1	SEP4Y	0.0000			0.0000		
L6	DEC26Y	0.0418	0.0091	0.28	0.0602	0.0153	0.47
L5	DEC26Y	0.0327	0.0091	0.28	0.0449	0.0146	0.45
L4	DEC26Y	0.0236	0.0084	0.26	0.0303	0.0124	0.38
L3	DEC26Y	0.0153	0.0076	0.23	0.0179	0.0102	0.31
L2	DEC26Y	0.0077	0.0077	0.21	0.0077	0.0077	0.21
L1	DEC26Y	0.0000			0.0000		
L6	FEB22Y	0.1821	0.0406	1.25	0.1700	0.0424	1.31
L5	FEB22Y	0.1415	0.0405	1.25	0.1277	0.0410	1.27
L4	FEB22Y	0.1010	0.0379	1.17	0.0867	0.0358	1.10
L3	FEB22Y	0.0631	0.0337	1.04	0.0509	0.0294	0.91
L2	FEB22Y	0.0294	0.0294	0.80	0.0215	0.0215	0.58
L1	FEB22Y	0.0000			0.0000		

Table 10 - Displacements and inter-storey drifts along Line F at Line 2 from ERSA using full spectral records and assuming fully elastic response. These show that the inter-storey drifts in the North-South direction imposed on Line F in the 4 September 2010 Earthquake (Sep4X) were approximately twice those in the 26 December aftershock (Dec26X). The drifts imposed by the 22 February Aftershock (Feb22X) were around twice those of the 4 September 2010 Earthquake (Sep4X).

The results in Table 10 show that the inter-storey drifts in the North-South direction imposed on Line F in the 4 September 2010 Earthquake (Sep4X) were twice those in the 26 December aftershock (Dec26X). The drifts imposed by the 22 February Aftershock (Feb22X) were twice those of the 4 September 2010 Earthquake (Sep4X).

The results in Table 10 also show that Line F north-south drifts where greater in response to loading spectra applied in the north-south direction than in the east-west direction.

CHECK ON ADEQUACY OF NON-SEISMIC DETAILING IN COLUMNS

A check was made to determine the performance of the columns under the displacement demands of NZS 4203:1984 cl 3.8.1.1 and NZS 3101:1982 cl 3.5.14.3(a) for non-seismic detailing. These required the vertical and spiral reinforcing steel in the columns to remain elastic or undamaged when the ductile design spectra (S=1.0) displacements were scaled by a factor of K/SM=2.75. If the columns could not achieve that then they would have been required to have met the more severe ductile detailing provisions of NZS 3101:1982.

This occurred on Line F with application of North-South loading at a drift of 0.67% between Level 5 and 6 as shown in Table 11.

The bending moments and shears determined from the plane frame displacement compatibility analysis at this drift level were found to exceed the elastic limits for bending at Levels 3, 4 and 5 and at Levels 4 and 5 for shear. The dependable strength of the column should exceed the demand at that displacement to satisfy the requirements.

The columns therefore did not appear to satisfy the conditions of NZS 4203:1984 to allow them to be detailed with non-seismic detailing.

ADEQUACY OF PRIMARY FRAME DRIFT CAPACITY TO NZS 4203:1984

The primary frame, (the North Core and South Wall) was required to limit drifts to 0.83%. at computed displacements at K/SM=2.75 (Table 11 2.75SPECXDUCTILE). These were calculated to be a maximum of 0.67% for design loading in the North-South direction, including the effect of the masonry in-fill wall on Line A. This indicates that the structure satisfied this requirement to limit the drift, in its apparent condition at the time of the September Earthquake.

The structure by implication should also have been able to sustain greater drifts equivalent to those implied by development of fully ductile behaviour in the structure. This was not a specific requirement of NZS 4203:1984 that had to be checked however.

This is seen to be a short-coming of NZS 4203:1984.

The ultimate drift calculated to occur under the development of fully ductile behaviour of the structure was 1.23% at Level 5 to 6 for the North-South design condition (Table 11 5.0SPECXDUCTILE). However collapse was shown previously to have been able to have occurred at drifts between 0.75% and 1.3%.

This indicates that the structure could only sustain between 60% and 100% of the drift expected to be achieved for fully ductile behaviour implied by NZS 4203:1984.

Column F/2 ERSA Displacements and Inter-storey Drifts

North-south earthquake spectra scaled to development of South Wall L1 nominal bending capacity at 0.34% drift and at 0.75% drift. Also NZS 4203:1984 displacements at K/SM=2.75 elastic displacement limit and s=5 ultimate capacity displacement limit.

Level	Load	Displacement UX (N-S) m	Inter-floor Drift (N-S) m	Inter-floor Drift (N-S) %	Displacement UY (E-W) m	Inter-floor Drift (E-W) m	Inter-floor Drift (E-W) %
North -South Earthquake							
L6	FEB22X SW 0.34%	0.0498	0.0111	0.34	0.0322	0.0078	0.24
L5	FEB22X SW 0.34%	0.0387	0.0111	0.34	0.0244	0.0076	0.24
L4	FEB22X SW 0.34%	0.0276	0.0104	0.32	0.0167	0.0069	0.21
L3	FEB22X SW 0.34%	0.0172	0.0093	0.29	0.0098	0.0057	0.18
L2	FEB22X SW 0.34%	0.0079	0.0079	0.21	0.0041	0.0041	0.11
L1	FEB22X SW 0.34%	0.0000			0.0000		
L6	FEB22X F 0.75%	0.1090	0.0243	0.75	0.0704	0.0170	0.53
L5	FEB22X F 0.75%	0.0847	0.0243	0.75	0.0533	0.0167	0.52
L4	FEB22X F 0.75%	0.0604	0.0228	0.70	0.0366	0.0152	0.47
L3	FEB22X F 0.75%	0.0376	0.0203	0.63	0.0215	0.0126	0.39
L2	FEB22X F 0.75%	0.0173	0.0173	0.47	0.0089	0.0089	0.24
L1	FEB22X F 0.75%	0.0000			0.0000		
North -South NZS4203:1984							
L6	2.75 SPECXDUCTILE	0.0980	0.0219	0.67	0.0630	0.0152	0.47
L5	2.75 SPECXDUCTILE	0.0761	0.0218	0.67	0.0478	0.0150	0.46
L4	2.75 SPECXDUCTILE	0.0543	0.0205	0.63	0.0328	0.0136	0.42
L3	2.75 SPECXDUCTILE	0.0338	0.0183	0.56	0.0192	0.0113	0.35
L2	2.75 SPECXDUCTILE	0.0156	0.0156	0.42	0.0080	0.0080	0.22
L1	2.75 SPECXDUCTILE	0.0000			0.0000		
L6	5.0 SPECXDUCTILE	0.1781	0.0397	1.23	0.1145	0.0277	0.85
L5	5.0 SPECXDUCTILE	0.1384	0.0396	1.22	0.0869	0.0272	0.84
L4	5.0 SPECXDUCTILE	0.0987	0.0373	1.15	0.0597	0.0247	0.76
L3	5.0 SPECXDUCTILE	0.0615	0.0332	1.02	0.0349	0.0205	0.63
L2	5.0 SPECXDUCTILE	0.0283	0.0283	0.76	0.0145	0.0145	0.39
L1	5.0 SPECXDUCTILE	0.0000			0.0000		

Table 11 - Displacements and Inter-storey drifts from north-south ERSA loadings at development of bending yield at the base of the South Wall (Feb22X SW 0.34%), and at development of 0.75% drift (Feb22X F 0.75%). Also shown are the displacements and drifts at the K/SM=2.75 elastic detailing limit of NZS 4203:1984 (2.75 SPECXDUCTILE). The ultimate drift expected by NZS 4203:1984 (5.0 SPECXDUCTILE (1.23%) were at the upper end of the drift capacities by the columns of between 0.75%. and 1.3%.

ADEQUACY OF DRIFT CAPACITY FOR 2010 STANDARDS

Ultimate limit state drifts were also calculated based on the demand from the ERSA model, neglecting p-delta effects, and multiplied by the drift modification factor of 1.24 from NZS1170.5 Table 7.1. This indicated a drift demand of 2.3%.

This well in excess of the requirements of the standards in 1986 and indicates that the CTV Building may have had an average comparative drift capacity in the order of 40% to 50% of 2010 requirements.

According to the 2010 standards, the calculated 2.61% Ultimate Drifts along gridline 1 at levels 4 and 5, exceed the inter-storey deflection limit of 2.5% specified in

NZS1170.5 (refer clause 7.5.1). This means the line 1 seismic resisting structure would need to be stiffened to comply with current standards.

EFFECT OF SPANDREL PANELS ON LINE F COLUMNS

To assess the effect of Spandrel Panel interaction on column head bending and shear demands was made using a plane frame analysis with prescribed displacements with gaps of 3, 5 and 10 mm at a nominal drift on Line F of 0.75%. The drifts were applied as prescribed displacements at the head and base of the columns. Where the displacements were sufficient to cause contact with the Spandrel Panels the displacement was restricted at the top of Spandrel Panel level to the set gap. For example with 3 mm gaps a 3 mm prescribed displacement was set at that location if contact occurred. If not then no restraint was applied at hat location.

At drifts along Line F of 0.75% (Refer Table 11 Feb22X 0.75% and Table 12), severe distress of the column heads at F/2 and F/3 at the underside of Levels 3, 4, 5 and 6 could have occurred if they were restrained by the Spandrel Panels with gaps of 3 mm or less.

Where the Spandrel Panels had gaps between them and the columns of 10 mm, no contact was indicated to have occurred at 0.75% inter-storey drifts. Even so the analysis showed that the columns at Levels 4 and 5 would suffer bending yield at around 0.6% drift. These columns had some reserve of displacement ductility to cope with cyclic demands past yield.

The Level 4 columns had the least reserve of remaining displacement ductility to cope with cyclic demands post yield. Below Level 4 the columns were calculated to have remained elastic in that analysis and so would have remained largely (Table 12).

Where the Spandrel Panels gap was 5 mm then bending demands were increased at Levels 4 and 5. Below Level 4 the columns were calculated to have remained elastic (Table 12).

With gaps of 3 mm or less then bending and shear capacity at Levels 2 to 5 was exceeded for columns. At Level 1 the columns were calculated to have remained elastic (Table 12)..

Collapse initiating on the upper levels of Line F is consistent with eyewitness observations.

DEVELOPMENT OF SOUTH WALL NOMINAL BENDING CAPACITY

Development of the South Wall nominal bending capacity was indicated when north-south drifts shown in Table 11 of 0.35% (Level 6 Feb22X South Wall) on Line F occurred. At this level of drift the infill masonry on Line A remained at shear levels less than the nominal capacity limit allowed for in the reinforced concrete masonry standard NZS 4230:2004 (**Error! Reference source not found.**).

It was therefore concluded that both the Drag Bars and the Line A masonry infill wall would have remained effective in resisting seismic response at the levels of Line F drift of 0.35% at which the South Wall developed nominal bending capacity at its base.

It was also found that at the same 0.35% drift levels on Line F, the columns on Line 2 and F remained elastic and were not overloaded in shear, compression or bending for mean concrete strength of 29.6 MPa.

CTV Displacement Compatability Analysis Column : Frame Line F Checks

Line 2 F/2 10 mm Spandrel gaps **f'c** 29.6 MPa

					Frame Analysis Results			Limits		
BOC Level	Floor Height mm	Beam Depth mm	TOC Displacement mm	Drift %	Bending kNm	Shear kN	Axial kN	My kNm	Mu (ec=0.004) kNm	Shear Vcsp kN
L5-6	3240	550	24.3	0.75	142	105	300	122	143	114
L4-5	3240	550	24.3	0.75	166	123	601	140	168	140
L3-4	3240	550	22.8	0.70	162	121	895	171	187	159
L2-3	3240	550	20.3	0.63	182	136	1183	183	191	175
L1-2	3700	550	17.3	0.47	141	92	1470	193	193	187

Line 2 F/2 5 mm Spandrel gaps

					Frame Analysis Results			Limits		
BOC Level	Floor Height mm	Beam Depth mm	TOC Displacement mm	Drift %	Bending kNm	Shear kN	Axial kN	My kNm	Mu (ec=0.004) kNm	Shear Vcsp kN
L5-6	3240	550	24.3	0.75	149	114	300	122	143	114
L4-5	3240	550	24.3	0.75	176	137	601	140	168	140
L3-4	3240	550	22.8	0.70	163	121	895	171	187	159
L2-3	3240	550	20.3	0.63	182	136	1183	183	191	175
L1-2	3700	550	17.3	0.47	149	92	1470	193	193	187

Line 2 F/2 3 mm Spandrel gaps

					Frame Analysis Results			Limits		
BOC Level	Floor Height mm	Beam Depth mm	TOC Displacement mm	Drift %	Bending kNm	Shear kN	Axial kN	My kNm	Mu (ec=0.004) kNm	Shear Vcsp kN
L5-6	3240	550	24.3	0.75	190	167	300	122	143	114
L4-5	3240	550	24.3	0.75	228	202	601	140	168	140
L3-4	3240	550	22.8	0.70	217	189	895	171	187	159
L2-3	3240	550	20.3	0.63	236	203	1183	183	191	175
L1-2	3700	550	17.3	0.47	152	95	1470	193	193	187

Table 12– Line F frame displacement compatibility results for 0.75% drifts on Line F Showing effect of Spandrel Panels with varying gaps to the columns on bending demands on column heads

LIMITATION OF ERSA DRIFT CALCULATIONS

The ERSA drift calculations are recognised to be low approximations of the actual drifts, as they have been calculated in the table as the difference between the displacements at each level. The reported displacements are the combination of the magnitude of modal displacements without the effect of signs of higher mode displacements being accounted for. NZS1170.5 :2004 indicates that an allowance for this effect of up to 30% may be justified.

However a review of drifts calculated by ETABS specifically accounting for modal displacement effects found that the use of drifts calculated using the difference between inter-storey displacements was suitable for this structure.

SUMMARY

The columns did not satisfy the criteria of NZS 4203:1984 that would have allowed them to be detailed using the "non-seismic" provisions of the concrete structures standard NZS 3101:1982.

The primary frames provided adequate stiffness to limit the drifts of the structure to those required by NZS 4203:1984.

It appears that a number of columns did not possess sufficient ductility to sustain drifts imposed by development of fully ductile behaviour in the structure with a low probability of collapse. This was particularly so if Spandrel Panel interference occurred.

The expectation of NZS 4203:1984 was that drifts of 1.23% should have been attainable with a low probability of collapse.

The effect of Spandrel Panel interference with the Line F columns was to increase the bending and shear demands on the Line F column heads. This may have been sufficient to initiate collapse on Line F if the condition had occurred.

Development of South Wall nominal bending capacity at the base was indicated concurrent with drifts of approximately 0.35% occurring along Line F.

APPENDIX G - DIAPHRAGM FAILURE ANALYSIS AT NORTH CORE

In order to investigate whether the floor diaphragm detached from the North Core before or after column collapse occurred, the in-plane bending capacity along two critical diaphragm failure sections was analysed.

Review of the collapse photos such as Figure 128and analysis indicated the following order of collapse:

- The slab on the south side of the Line 4 beam broke away due to loss of vertical support following column collapse on Line 3.
- The slab attached to the lift and stair well walls (D and D/E) then detached from the Drag Bars as column 4-D/E collapsed.

Analysis of the in-plane bending capacity of the diaphragm compared in-plane bending capacity along two critical failure sections shown in Figure 131 The first along the perimeter ABCD running from the edge of the Line C wall out 1200 mm to the ends of the slab saddle bars and along 11.35 m then back 1200mm the Line D/E wall. The second failure section EFGA ran from the attachment point of the Drag Bars at walls D/E and D and along the slab at Line 4 between Walls C and C/D.

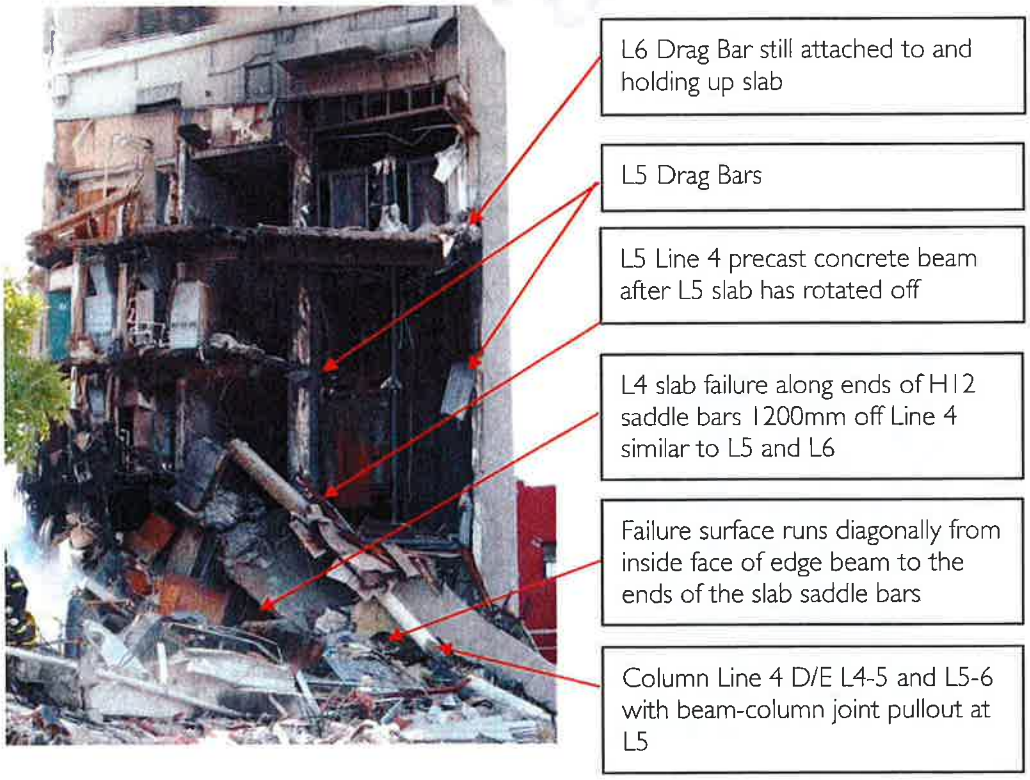


Figure 128 - Failure of slab adjacent to North Core.

The Drag Bars had been installed during remedial work to the building in 199. The copies of the sketches issued for their installation are shown in Figure 132 and Figure 133.

The in-plane bending capacity along ABCD was found to be greater than that along EFGA as shown in Figure 131. The in-plane bending capacity along ABCD included shear contributions of the AB and CD portions and the shear capacity of the with edge beam. The bending capacity along EFGA was limited by the tension capacity of the Drag Bar connections. The contribution of the profiled metal deck in tension was conservatively ignored but would have further increased the differential of strength between the ABCD and EFGA failure sections.

The diaphragm in-plane bending capacity at EFGA at the Drag Bars was the weakest link in terms of diaphragm attachment to the North Core. Failure of the slab was evident along line ABCD (but not through the perimeter beam) at L4 to L6 from a careful look at the photos (Figure 128and Figure 129).

For this to have occurred, column collapse along Line 2 and/or 3 would have occurred pulling the slab at the core downwards so that it failed in flexure and tension at the end of the saddle bars. This was consistent with the way the Level 3 and 4 slabs were found to have fallen, lying diagonally against the North Core after the collapse as shown in Figure 130.

The Line 4-D/E column adjacent to the North Core may then have been pulled down as the collapse progressed and the portions of slab immediately outside the lifts between walls D and D/E rotated downwards and pulled away from the Drag Bars.

The epoxied concrete anchors, that attached the Drag Bars to the slab, appeared to have held adequately in shear (as evidenced by the Level 6 slab which was still being held up by the Drag Bars). However as the slab portions rotated downwards the slab pried away from the anchors in tension, leaving the anchors at the north end of the Drag Bars vertical and those on the bent down outstand bent over by around 30 degrees..

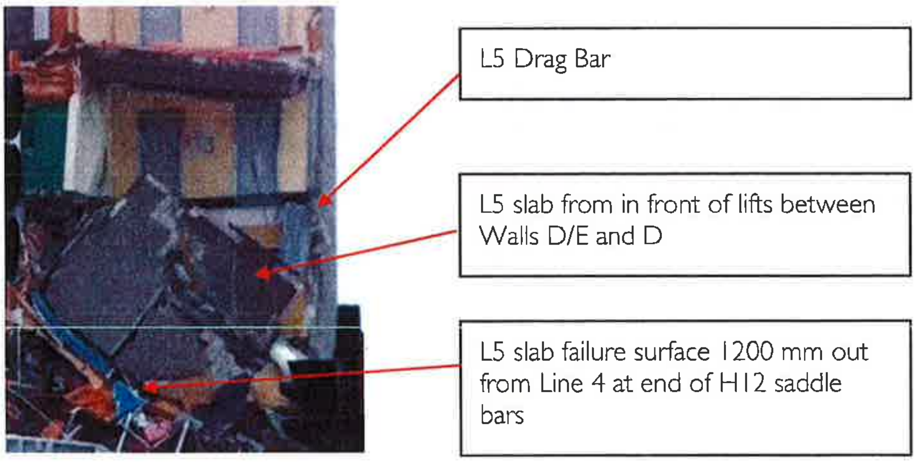


Figure 129 - Level 5 slab from in front of lifts shortly after the collapse.

Level	Diaphragm Bending Capacity at Core Walls (kNm)	
	Failure Section ABCD	Failure Section EFGA
Level 4	18737	9543
Level 5	18737	11365
Level 6	18737	12901

Table 13 - Diaphragm in-plane bending capacity at critical sections adjacent to North Core (Refer Figure 131 for identification of failure sections ABCD and EFGA)

FLOOR DIAPHRAGM CONNECTIONS TO THE NORTH CORE WALLS

The diaphragm connections to the North Core walls were required to be designed using the "Parts and Portions" provisions of the New Zealand Loadings Standard NZS 4203:1984.

These provisions did not make sufficient allowance for buildings such as this where significant inelastic displacement was expected in the primary seismic resisting frame.

In this case while both South Wall and the North Core walls were designed and detailed as fully ductile, the South Wall was able to yield and displace inelastically well before the North Core walls.

Initial ERSA using NZS 4203:1984 design loads with the floor diaphragm connected at Level 2 and 3 at Lines D and D/E indicated that these would be overstressed at low levels of seismic demand. However it was analysed further and found that the Line 1 and 5 shear walls could pick up additional shear to compensate, should those diaphragm connections to walls D and D/E at level 2 and 3 be lost (Figure 126).

This counters the view that lack of diaphragm Drag Bars to walls D and D/E at Levels 2 and 3 necessarily initiated the collapse.

The need for ties or Drag Bars to the shear walls on Line D and E were identified during a pre-purchase review for a potential purchaser in early 1990. Correspondence from the design engineering company and the reviewer states:

> "The agreed maximum tie load is 300 kN per tie. We understand that this load would be reduced on lower floors, in accordance with the "parts and Portions" section of NZS 4203:1984."

The documentation of the connection of the Drag Bar ties into the slab and walls obtained from the Design Engineer (Figure 132 and Figure 133), showed that the Drag Bar actions were calculated following the provisions of NZS 4203:1984. Bars were not designed or installed in Levels 2 and 3. This seems to have been deliberate and appeared to be based on the assumption that adequate shear capacity was provided at Walls C and C/D into the North Core at those levels to cope with diaphragm demands.

The authors assessment of the Drag Bar nominal capacities (ϕ=1.0) at level 4, 5, and 6 - are shown in Table 14.

Limit state capacities were calculated as the minimum of Drag Bar tensile yield; wall anchor shear, concrete crushing and pull-out; Drag Bar anchor shear, concrete crushing and pull-out.

Anchor capacities were calculated in accordance with the July 2011 edition of the FIB Design of Anchorages in Concrete guide (FIB 2011).

Wall	Level	Drag Bar kN	Wall Anchors kN	Slab Anchors kN	**Limit Capacity kN**
D	L4	698	302	420	**302**
D	L5	698	503	420	420
D	L6	698	603	630	**603**
D/E	L4	540	403	420	**403**
D/E	L5	540	503	558	**503**
D/E	L6	540	703	698	**540**

Table 14 - Diaphragm Drag Bar nominal capacities.

The slab diaphragm capacity itself was found to be less critical than the Wall D and D/E connections even ignoring the contribution of the profiled metal decking. The profiled metal decking would have been able to develop its tensile capacity in proportion to the level of development of the decking from the slab support to the critical location. Its contribution to shear capacity would also have been significant.

Along Line 1 and 2 the profile metal decking had pulled free of the beam lines during the collapse. This is consistent with the columns on that line settling and the slab being temporally held up along Line 1 and 4.

Figure 130 - North Core slabs leaning against the North Core showing that their collapse occurred after collapse of the Line 3 frame

continued

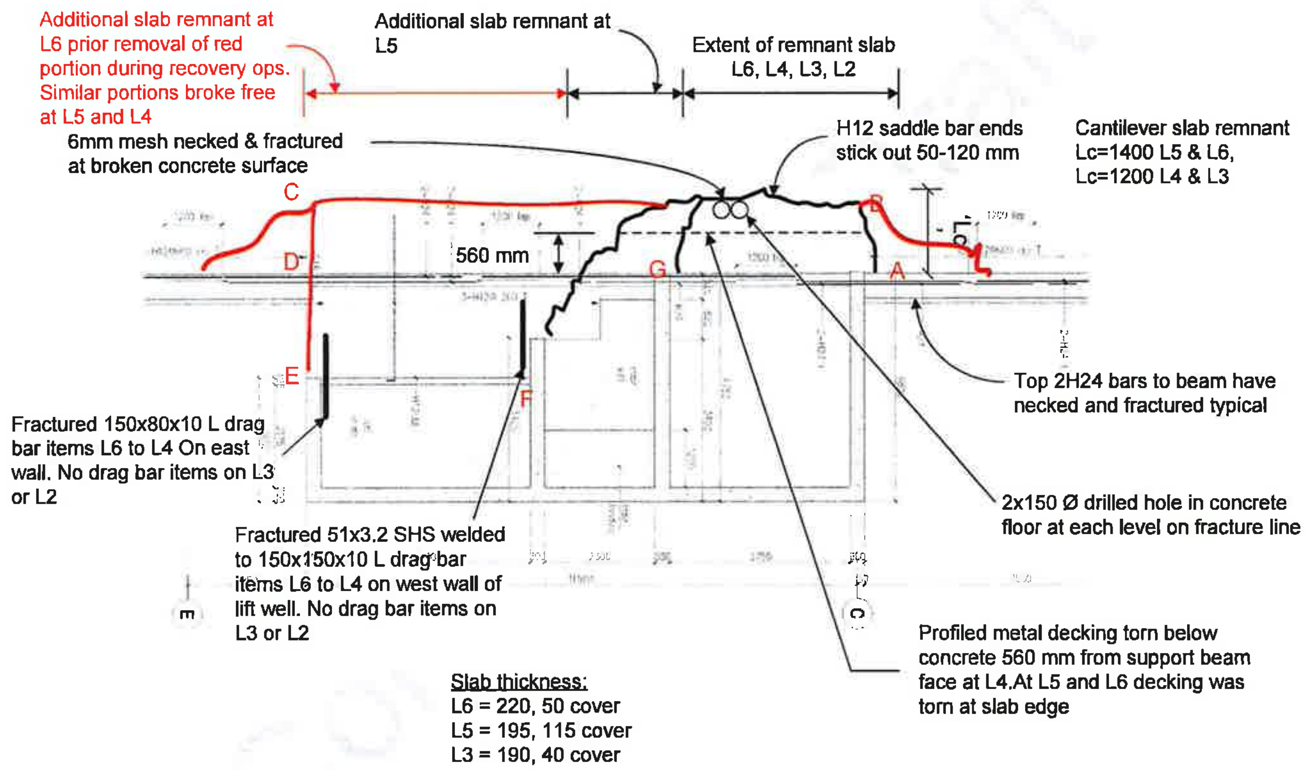


Figure 131 - North Core slab remnants after collapse based on site measurements in black and inferred by collapse photos in red..

continued

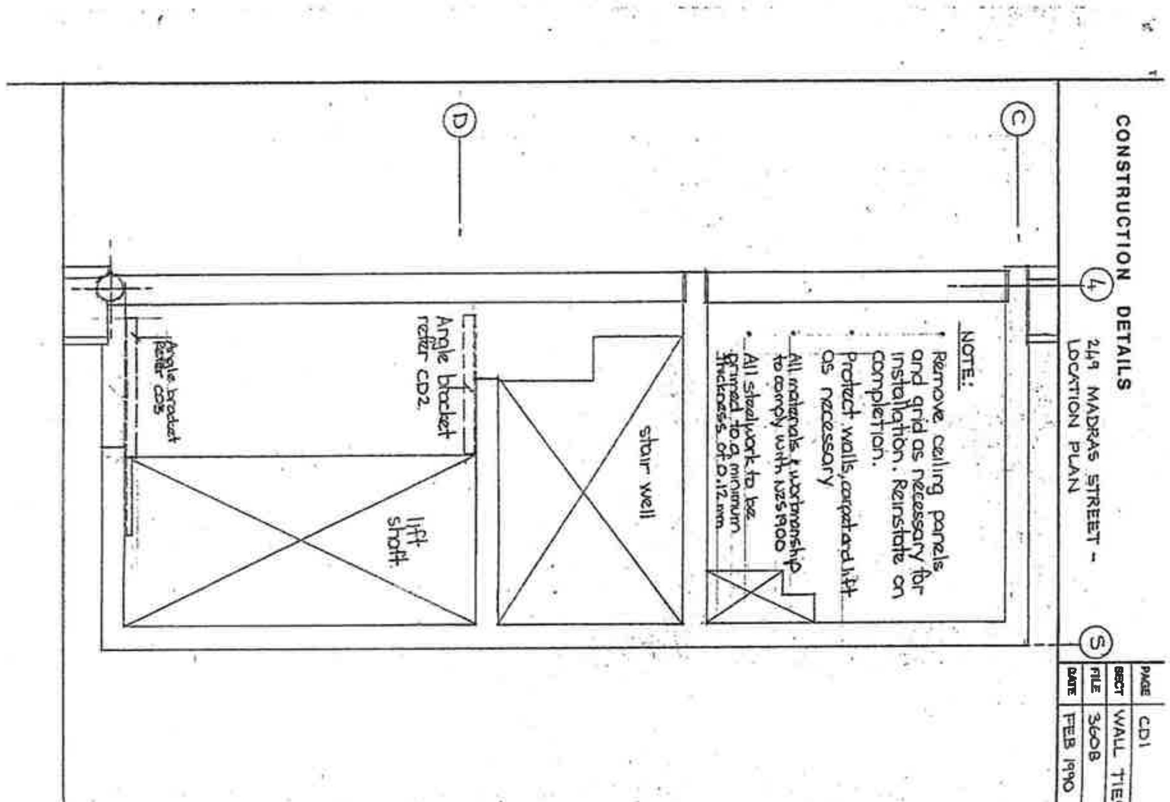


Figure 132 - Drag Bar locations on North Core walls

continued

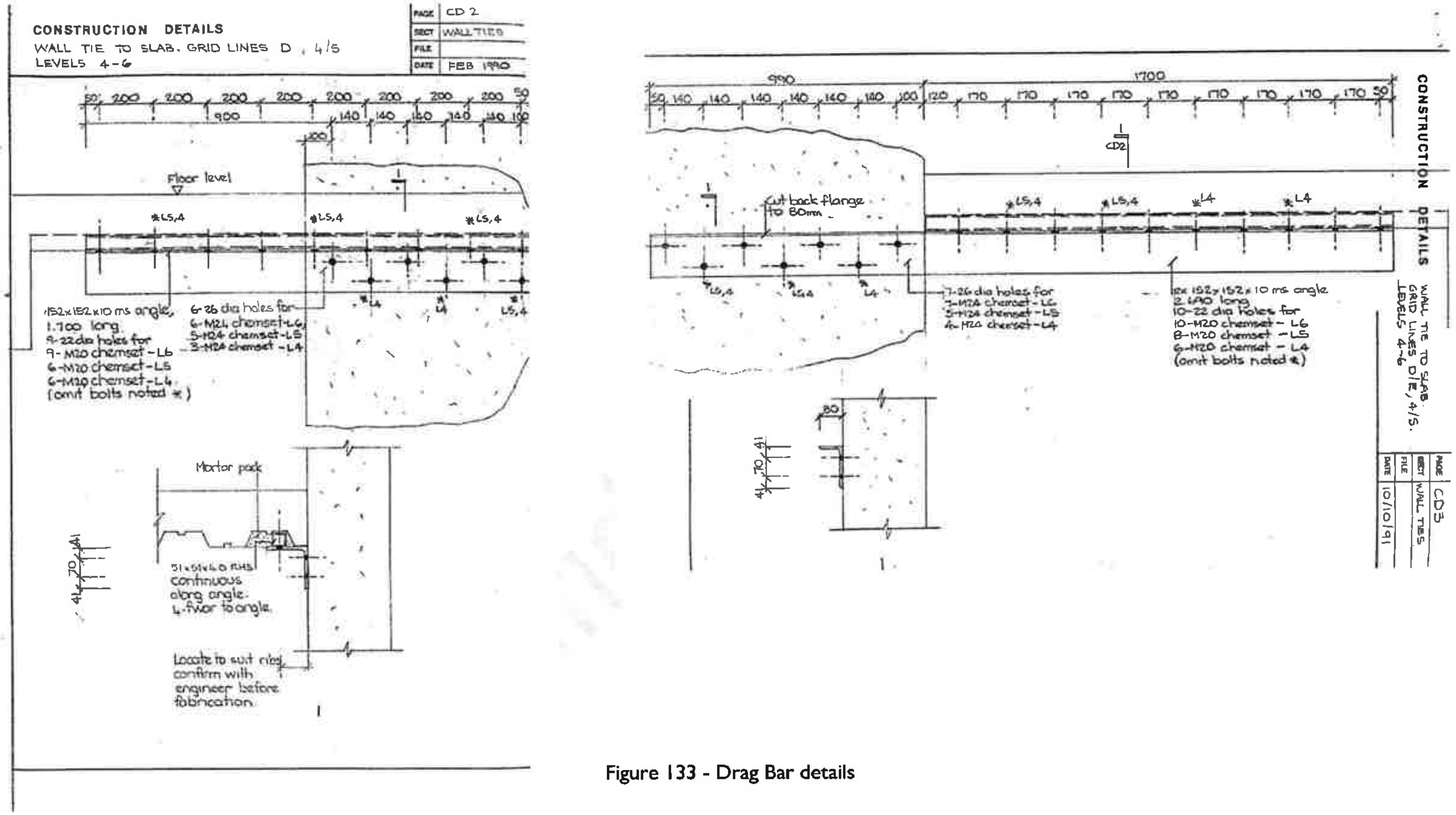


Figure 133 - Drag Bar details

APPENDIX H- GEOTECHNICAL REPORT SUMMARY

The general ground conditions at the site are described in Tonkin and Taylor's report as follows:

"The top four metres of the soil profile appear very consistent over the whole site, with silt (moist, firm) generally down to 1.5 m depth, overlying silty fine to medium sand. The water level is within this sand.

The geotechnical report of 1986 interpreted site conditions to differ below this level as follows:

- Over the major portion of the site, a thick dense gravel layer of 5 to 6 m thickness is present, overlying a deep layer of dense sand.
- For the remainder of the site, over the NE quadrant, the gravel is not present and is replaced by more sand and silt.

The 1986 report pointed out that:

> "... the transition between the gravel and soft sediments overlying the sand .... is quite abrupt and crosses the north-east corner of the site."

The appended Geotechnical Advice by Tonkin and Taylor concluded:

> "The geotechnical investigation carried out (by others) in 1986 was typical of the time and appropriate for the expected development. The report contained recommendations for further investigation. A modern investigation would now likely involve more deeper boreholes with more sampling and SPT's. Cone Penetration Tests would offer the opportunity of mapping the "transition" between gravel/no-gravel areas and also quantitative data for liquefaction analysis. Shear wave measurements would enable assessment of dynamic response parameters for dynamic analyses."

Liquefaction was not mentioned in the 1986 geotechnical report though the potential for liquefaction in Christchurch was well known at the time. Some of the soils at depth could have been subject to liquefaction or strength loss.

The type of foundations employed for the CTV building were typical for the size of the building and the Christchurch CBD. Provided liquefaction was not an issue, the shallow spread footings would seem appropriate and design recommendations were conservative for static conditions...."

One area of localised surface water or liquefaction was reported on the west side of the adjacent empty site to the west of CTV adjacent but this may have been due to the fire fighting that occurred. Otherwise there have been no reports of obvious liquefaction in the immediate vicinity of the CTV building.

On the subject of liquefaction, from Tonkin and Taylor's geotechnical review; "In summary, a thin layer, between water level at 2.5 – 3 m depth and gravel at 3.5 to 4 m depth, may have liquefied during and following the February earthquake. At the NE quadrant, this may have extended deeper. The limited thickness of the layer and the confining effect of the larger footings would mean complete bearing future would be unlikely, but "yield" with resulting settlement and differential settlement could have occurred.

In order to carry out a dynamic analysis of the CTV building for earthquake loading, the structural analysis required representation of the soil-foundation interaction as "subgrade reaction" stiffnesses. Tonkin and Taylor carried out computations using the Barkan formulae to give probable lower bound soil stiffness parameters, most likely parameters and probable upper bound parameters for use in the structural analyses that were carried outfor this investigation.

Seismic ground motions at the CTV site were deduced from strong-motion recordings surrounding the CBD. The five stations of interest were:

Botanical Gardens: CGBS

Cathedral College: CCCC

Christchurch Hospital: CHHC

Rest Home Colombo Street: REHS

Page Road Pumping Station: PRPC

The last two of these (REHS and PRPC) showed significantly higher amplification than the others, both with respect to Peak Ground Accelerations (PGA) and spectral accelerations.

A borehole (BH 103) drilled for the Department of Building and Housing (DBH) at the REHS site logged significant thickness of "very soft organic silt" and "very soft peat". The PRPC station is located in a known liquefaction zone, with a nearby borehole (ECAN – M35/5124) logging sand to 27m depth, overlying sands and gravels.

The other three stations (CGBS, CCCC, CHHC) were all expected to have generally similar profiles of variable inter-bedded silts, silty and gravelly sands, overlying dense sands.

For this reason Tonkin and Taylor considered the REHS and PPPC records should be disregarded and the CTV site response should be assumed as similar to the average of the other three stations.

APPENDIX I - DESIGN AND CONSTRUCTION STANDARDS AND SPECIFICATION CLAUSES

A selection, but not exhaustive listing of relevant design and construction clauses, from Standards, Specifications and the Building Permit, referred to in the text are listed for the readers convenience as follows:

PLAN AND VERTICAL IRREGULARITY

Plan and vertical irregularity criteria in the General Structural Design and Design Loadings Standard NZS 4203:1984 are as follows:

Cl. 1.4.2 "...the deflections of the structure as a whole, and any of its parts, shall not be such as to impair strength or serviceability of the structure."

Cl. 3.1 "The main elements of a building that resist seismic forces shall, as nearly as is practicable, be located symmetrically around the centre of mass of the building."

C3.1.1 "....Geometrically dissimilar resisting elements are unlikely to develop plastic hinges simultaneously, and ductility demands may also be increased by torsional effects."

Cl. 3.4.7.1(c) "For irregular structures more than 4 storeys high, horizontal torsional effects shall be taken into account by 3-D modal analysis of cl 3.5.2.2.2." ie ERSA

C3.4.7.1 "It should also be remembered that in torsional situations energy dissipation cannot usually be distributed evenly among resisting elements.... Structures of moderate eccentricity are those for which the torsional component of shear load in an element most unfavourably affected does not exceed three quarters of the lateral translational component of shear load".

INTER-STOREY DRIFT LIMITS

Drift limit criteria in NZS 4203:1984 were as follows:

Cl. 3.8.1.1 "Computed inter-storey deflections shall be those resulting from the application of the horizontal actions specified in section 3.4 or 3.5 and multiplied by the factor K/SM appropriate to the structural type and material, ... and K=2.2 for the method of section 3.5 (ERSA)".

Cl. 3.8.1.2 "Computed deformations shall neglect foundation rotations."

Cl. 3.8.3.1" Inter-storey deflections computed in accordance with 3.8.1 between two successive floors shall not exceed .... 0.010 times the zone factor ... where the zone is: 5/6 for seismic zone B ..."

SEPARATION OF SECONDARY STRUCTURAL ELEMENTS

Separation of elements criteria in NZS 4203:1984 were as follows:

Cl. 3.8.4.1(a) "...infillings... (cl 3.8.4.1(b)) shall be so separated from the structure that there is no impact when the structure deforms to twice the extent computed by clause 3.8.1."

Cl. 3.8.4.1 (b) "Pre-cast concrete claddings"... (cl3.8.4.2 (b)) "shall be separated so that there is no impact when the structure deforms to the computed deformations in cl 3.8.1"

DESIGN OF REINFORCED CONCRETE SECONDARY ELEMENTS

The requirements for the design of secondary structural elements by the Code of Practice for the Design of Concrete Structures NZS 3101:1982 were as follows:

Designation of Group 1 and 2 Secondary Elements

Cl. 3.5.14.1 "Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads transmitted to them, or due to deformations of the structure as a whole. These are classified as follows:

(a) Elements of Group 1 by virtue of their detailed separations are not subjected to loading induced by the deformation of the supporting primary elements or secondary elements of Group 2.

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to ... loadings induced by deformation of the primary elements."

Group 1 Separated Elements

Cl. 3.5.14.2 "Group 1 elements shall be detailed for separation to accommodate deformations $\mathbf{v\Delta}$ .... Such separation shall allow adequate tolerances in the construction of the element and adjacent elements, ... For elements of Group 1:

...(c) ....Fixings for precast units shall be designed and detailed in accordance with 3.5.15."

Cl. 3.5.15.1"When seismic deflection of the structure results in relative movement between a precast element and the points on the structure to which it is fixed, the fixings shall be designed to give clearance for the relative movements at these fixing points, corresponding to the seismic deflection computed in NZS 4203."

Cl. 3.5.15.2 "In buildings where the relative movements at the fixing points, computed in accordance with 3.5.15.1, are provided for by the capacity of the steel fittings for ductile deformation, and the relative movements do not require deflections in the fixings in excess of twice their yield deflection, the clearances required by 3.5.15.1 need not be provided."

Cl. 3.5.15.3 "For exterior elements and elements adjacent to any means of egress, the fixings, together with their anchorages shall be designed to deform in a ductile manner under movements exceeding the clearances required by 3.5.15.1.

Group 2 Non-separated Secondary Elements

Cl. 3.5.14.3 "Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. For elements of group 2:

(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $\mathbf{v\Delta}$, specified in NZS 4203, and the assumptions of elastic behaviour.

(b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $\mathbf{v\Delta}$,...

(d) Loadings induced by the deformation of the primary elements shall be those arising from the level of deformation, $\mathbf{v\Delta}$ specified in NZS 4203 having due regard to the pattern and likely simultaneity of deformation.

(e) Analysis may be by any rational method, in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one quarter of the amplified deformation, $\mathbf{v\Delta}$, of the primary elements, as specified in NZS 4203.

(f) Where elastic theory is applied in accordance with (e) for deformation corresponding to 0.5 $\mathbf{v\Delta}$ or larger, the design and detailing requirements of Section 14 may be applied, but otherwise the additional seismic requirements of other sections shall apply."

CONSTRUCTION MONITORING AND INSPECTION REQUIREMENTS

It is likely that the Council by-laws required construction monitoring and inspection relevant to the CTV Building construction to be as follows:

Building Permit Conditions (Application No. 1747)

"Item 2 The Engineer responsible for the structural design (including the foundation system) confirming in writing that the intent of his design has been complied with before the building is occupied."

Code of Practice for the Design of Concrete Structures NZS3101:1982

Section 1.1 states that "It is only applicable to structures and parts of structures complying with the materials and workmanship requirements of NZS 3109".

Specification for Concrete Construction NZS 3109: 1987

Cl. 1.3.1 "All structural concrete shall be inspected by the person responsible for the design or by a competent representative nominated or approved by him. Such inspection shall establish that the design is being interpreted correctly and that the works are being carried out generally in accordance with the standards specified."

Cl. 5.6.3 "Types of joint. Construction joints shall be one of the following basic types:

....Type B construction joints shall be made at locations indicated on the drawings where it is necessary to develop shear friction across the joint. The surface of cast concrete shall be prepared by one of the methods specified in clause 5.6.2 the extent of treatment shall be such as to produce a roughened or broken surface to a depth of approximately 3 mm above and below the average level."

Cl.6.2.1 "... Ready mixed concrete and concrete used in the production of precast products off the site shall comply with NZS 3104."

Cl. 6.10.1"General. Prior to commencement of the supplying of concrete, the constructor shall produce evidence to the satisfaction of the engineer supervisor that the concrete mixes proposed for the project are adequately designed and that the production standards nominated can be achieved consistently."

Cl. 6.10.3.2 "Mix design. Evidence shall be provided to the satisfaction of the engineer supervisor that each concrete mix proposed has a target mean strength in compliance with eth requirements of table 7 for the appropriate plant grading and specified strength."

Cl.6.11.1 " When the constructor wishes to change, in a manner likely to reduce its mean strength, a mix design which the engineer supervisor has approved as specified in 6.9.2 or altered as provided in 9.5.6.3, the engineer supervisor's approval shall first be obtained..."

Cl. 9.1 Tests shall be carried out during construction to check the compliance of the concrete with this specification... Proposals for location of sampling and frequency of testing shall be submitted to and subject to the approval of the engineer supervisor."

Specification for Concrete Production- High Grade and Special Grade NZS 3104: 1983Cl. 102 Definitions

"Engineer Supervisor means the professional engineer (or architect), his deputy, or authorized representative, nominated on behalf of the owner to supervise the works to which concrete is being supplied."

"Engineer to the Plant means the engineer experienced in quality control of concrete production, and in mix design, nominated by the concrete producer to assume responsibility for mix designs and for the standard of production..."

Cl. 211.3 Availability of (Mixing) Records

"The records shall be available for inspection on request by the engineer supervisor."

s being supplied."

"Engineer to the Plant means the engineer experienced in quality control of concrete production, and in mix design, nominated by the concrete producer to assume responsibility for mix designs and for the standard of production..."

Cl. 211.3 Availability of (Mixing) Records

"The records shall be available for inspection on request by the engineer supervisor."

APPENDIX J - DRAWINGS AND SPECIFICATION

Portions of structural and architectural drawings prepared by DENG and ARCH are shown to aid with interpretation of the report. (Portions are included with permission of DENG and ARCH).

A3 versions of some of the drawings are presented in an attachment to this Appendix.

DRAWINGS

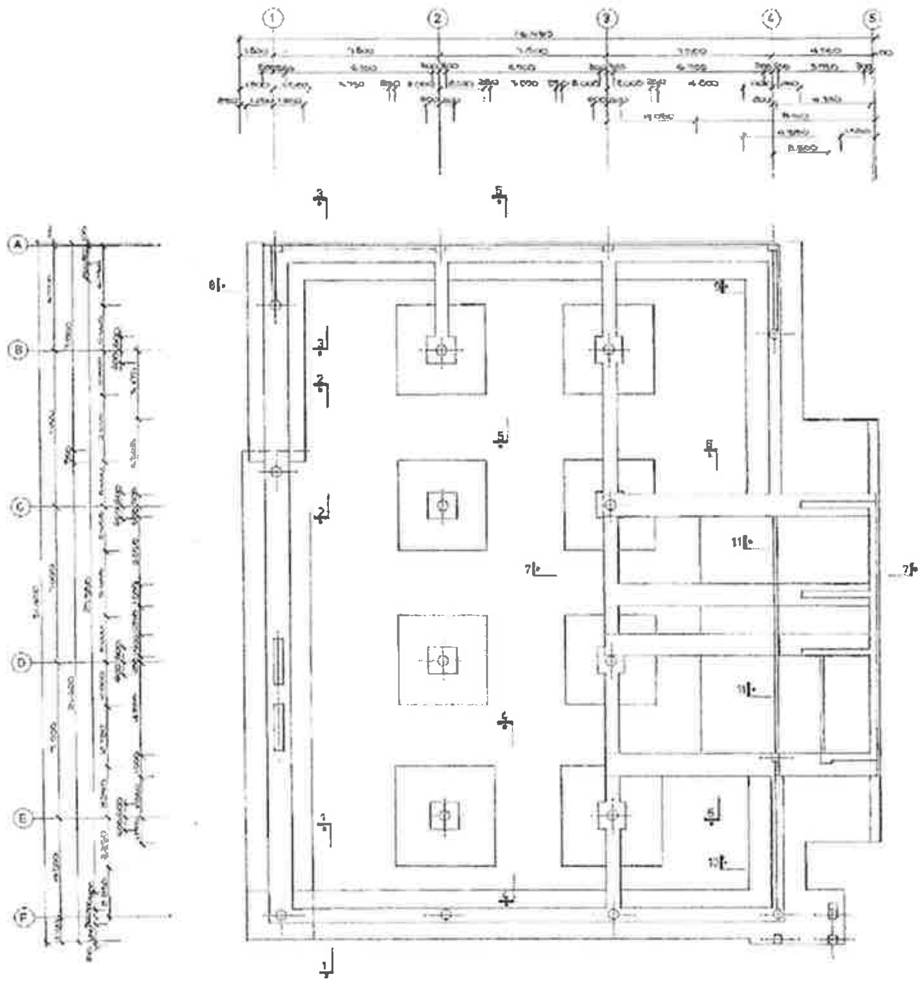


Figure 134 -Foundation Layout (Extract from DENG Dwg S2)

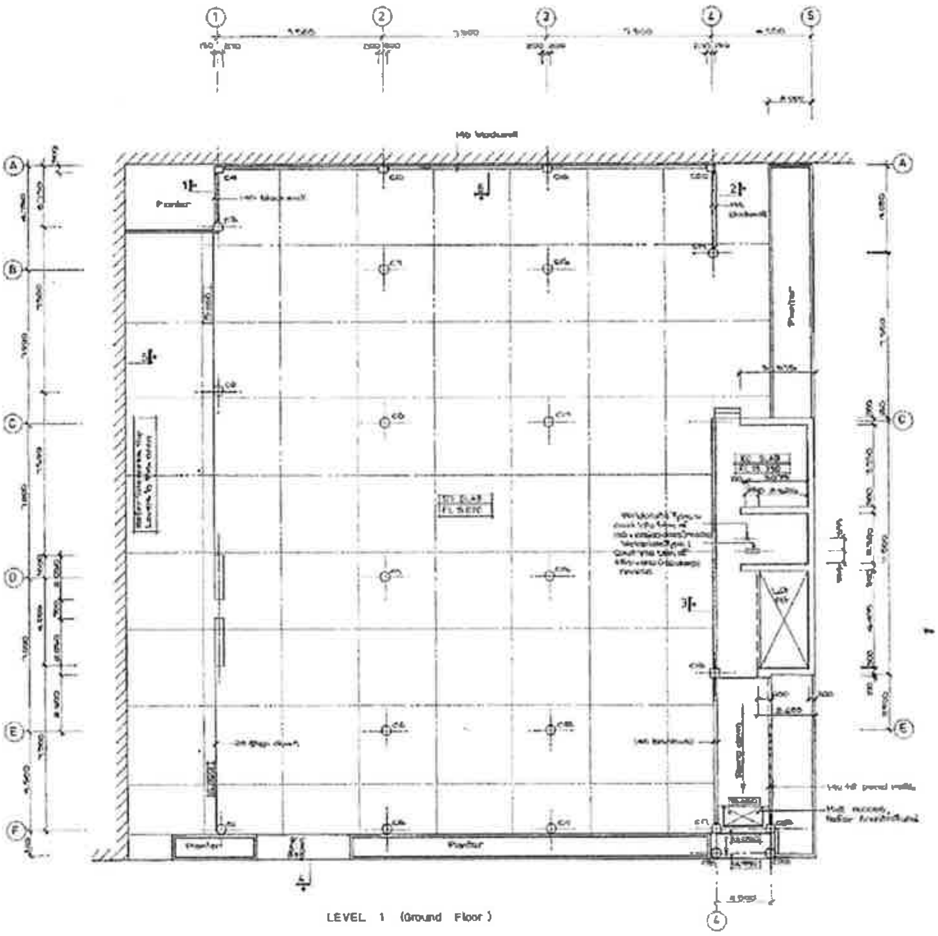


Figure 135 -Level 1 ground floor slab layout (extract DENG Dwg S9)

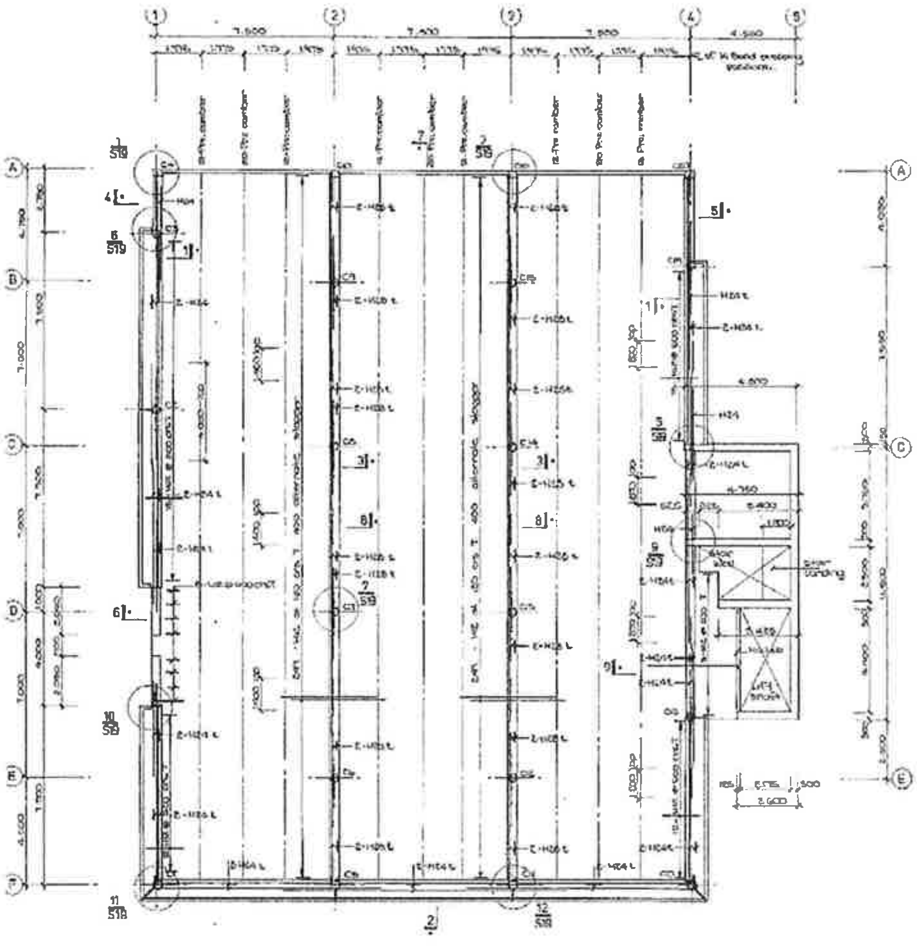


Figure 136 -Level 2 to 6 Floor Layout (Extract from DENG Dwg S15)

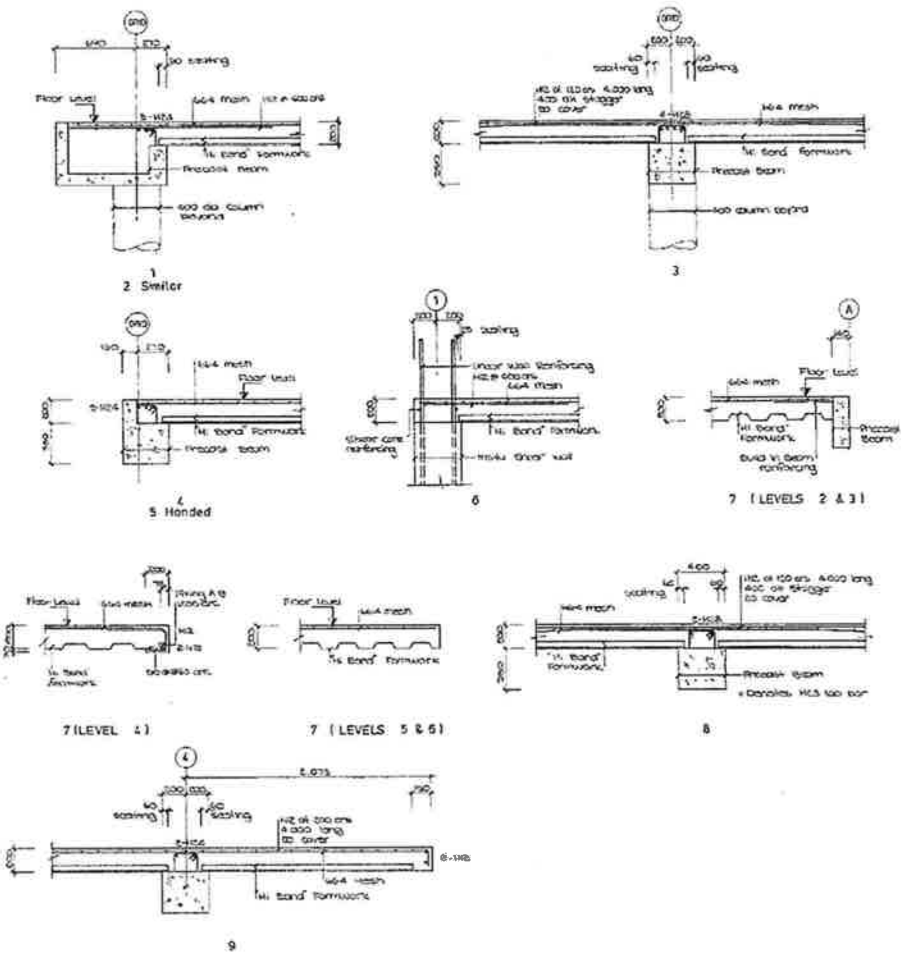


Figure 137 -Level 2 to 6 floor slab details (Extract from DENG Dwg S15)

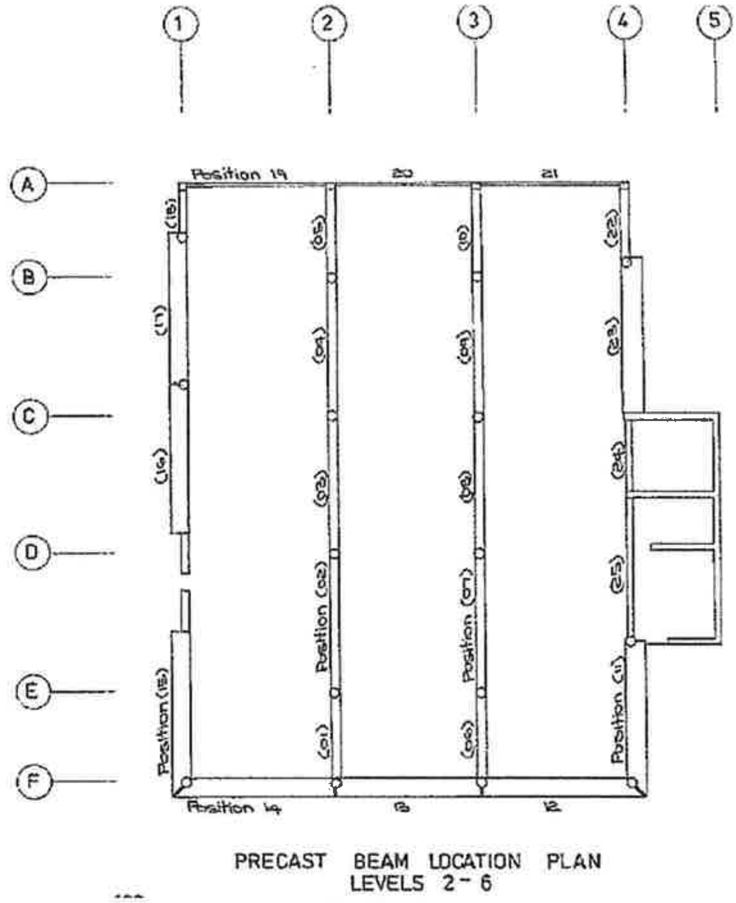


Figure 138 -Precast beam layout drawings (Extract DENG Dwg S18

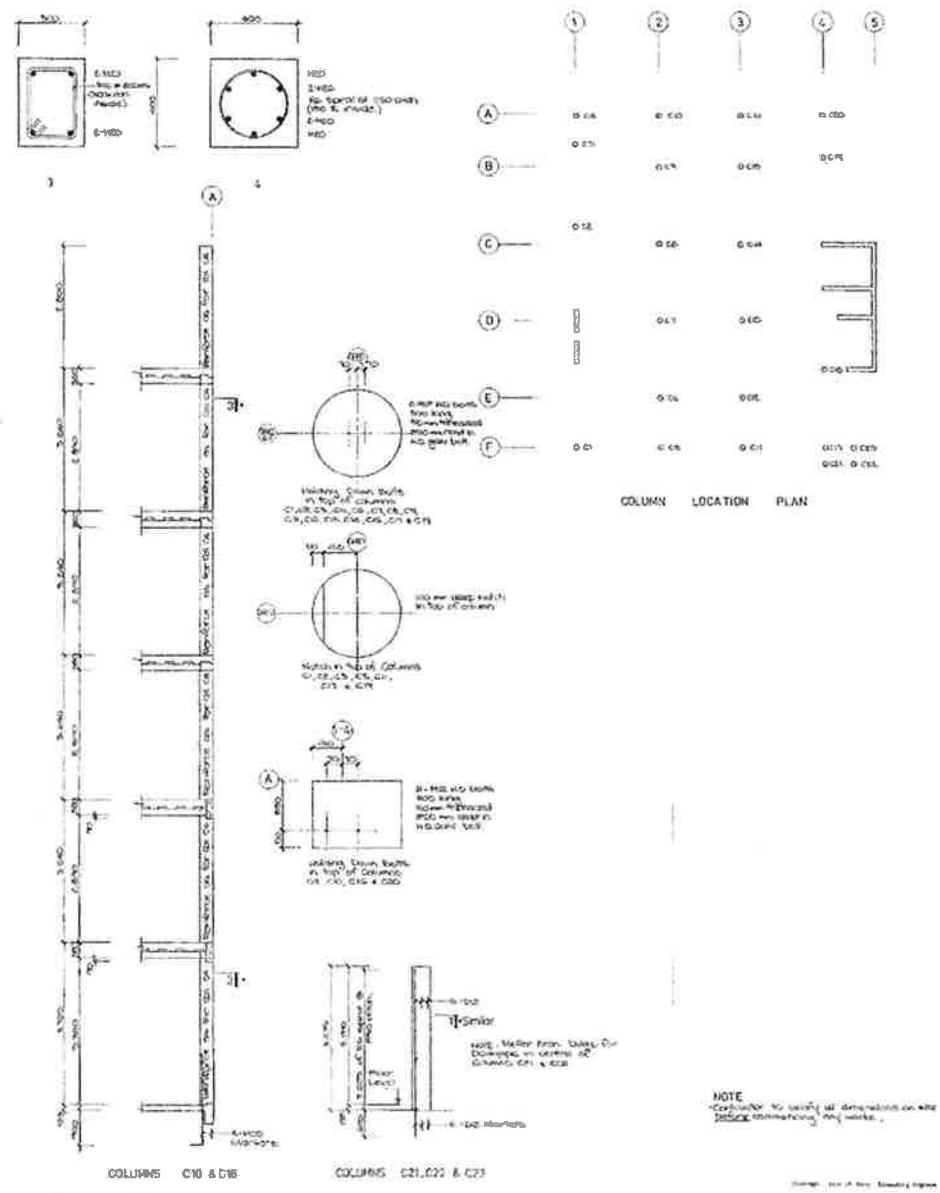


Figure 139 -Columns (Extract DENG Dwg S14)

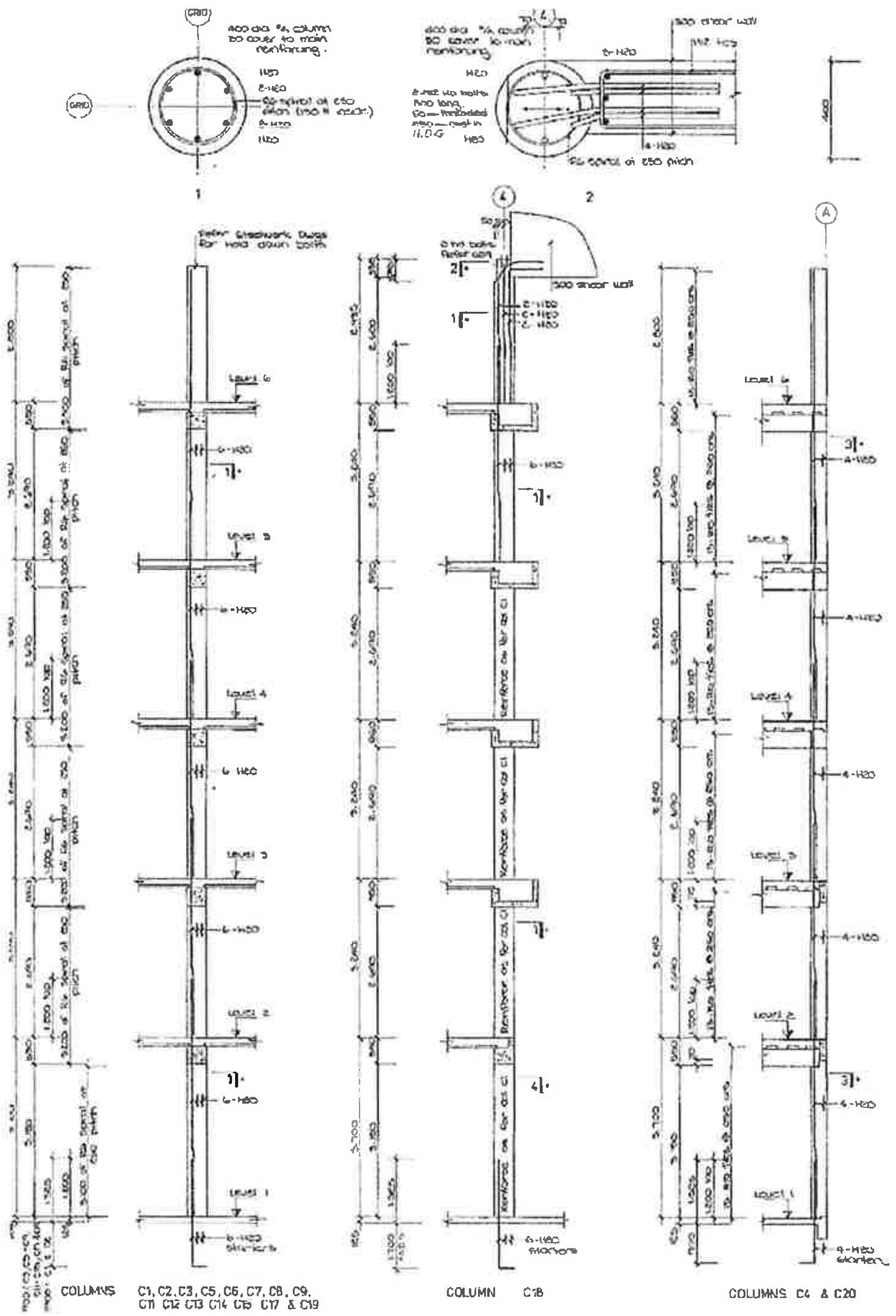


Figure 140 -Columns (Extract DENG Dwg S14)

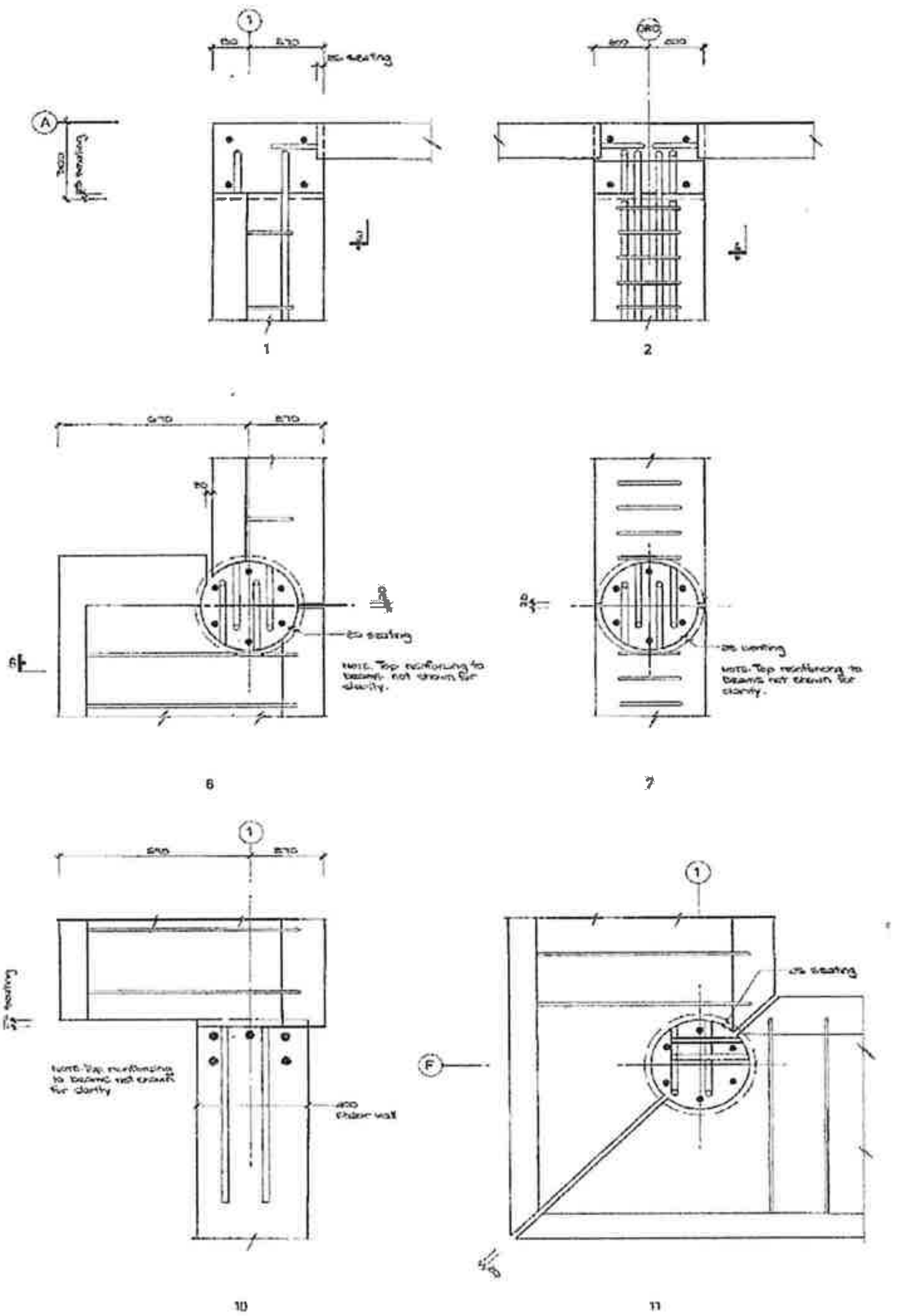


Figure 141 -Beam-Column Joints (Extract DENG Dwg S19)

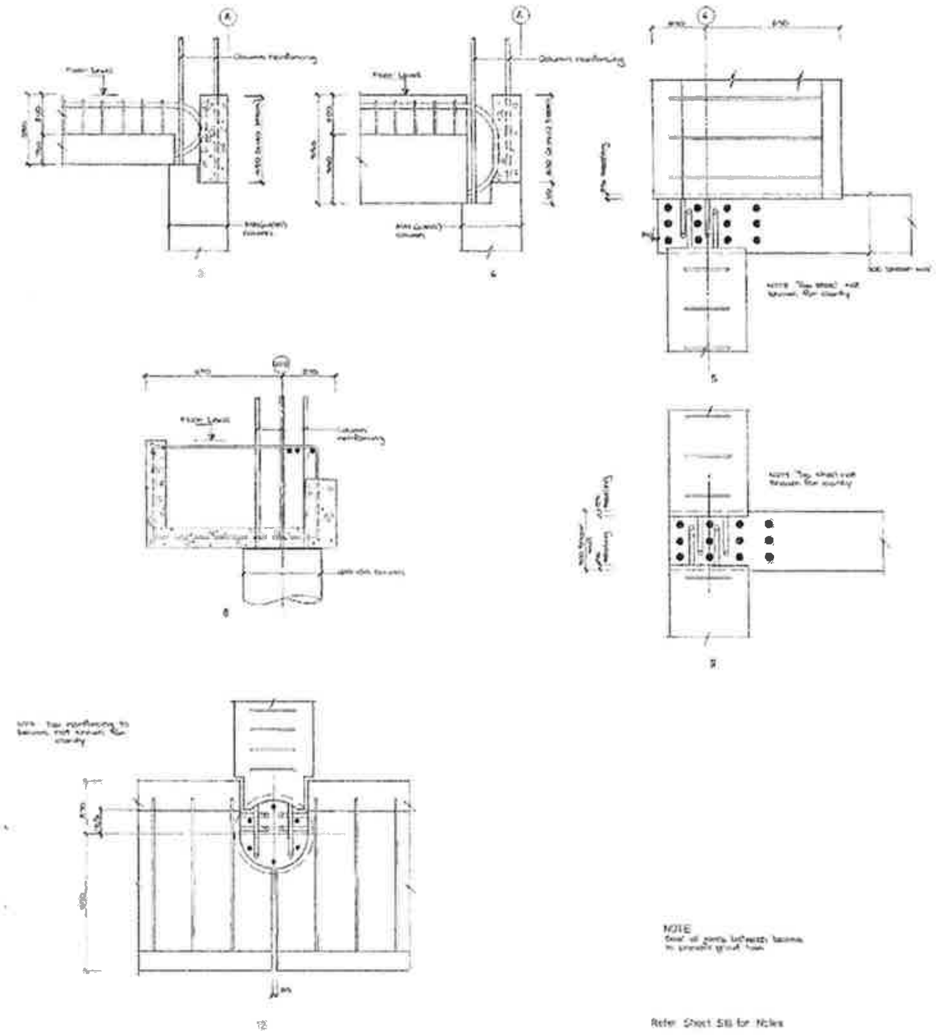


Figure 142 -Beam-Column Joints (Extract DENG Dwg S19)

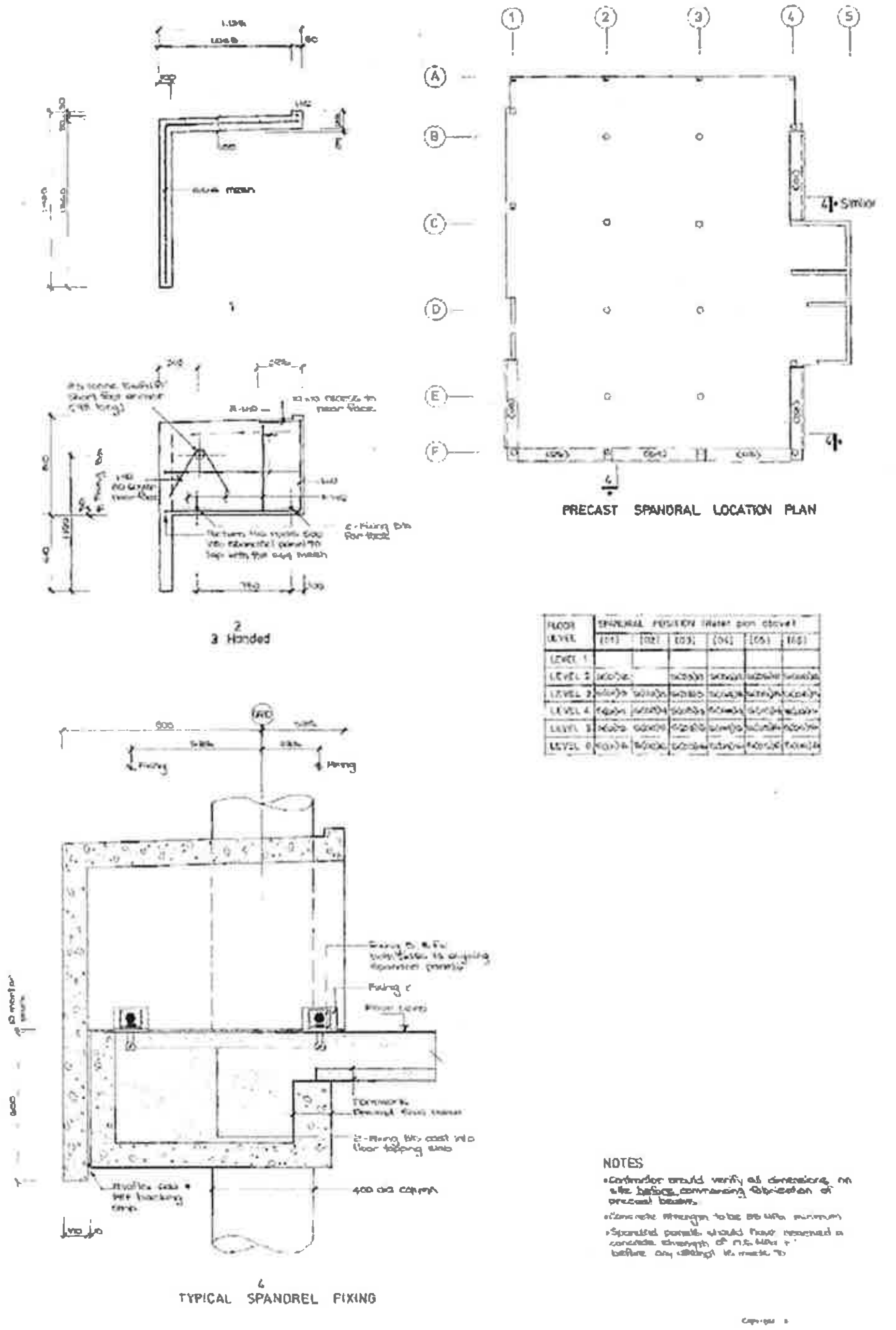
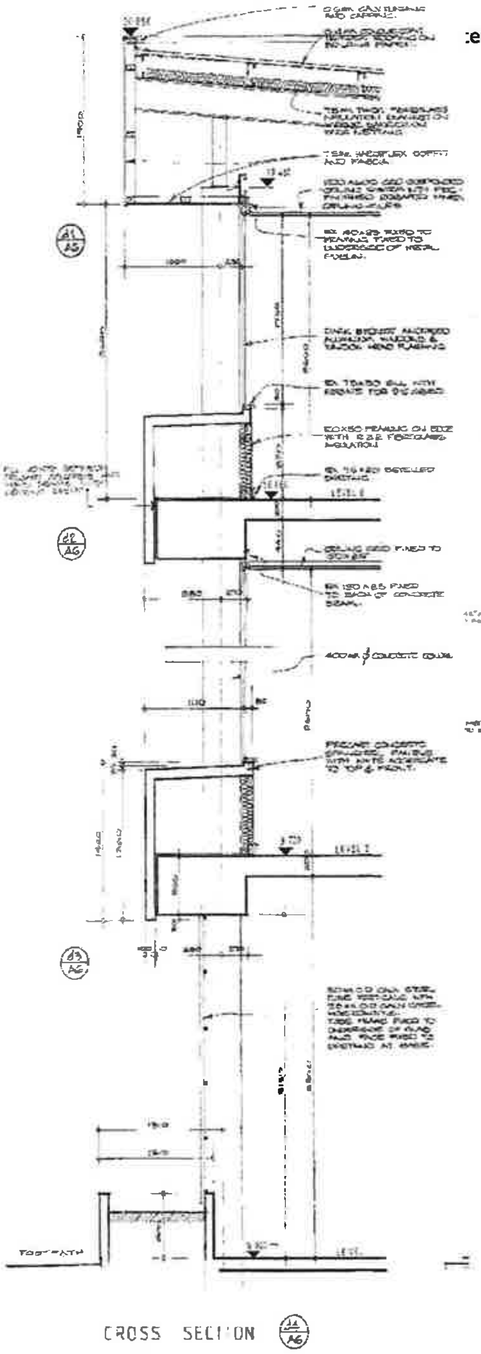
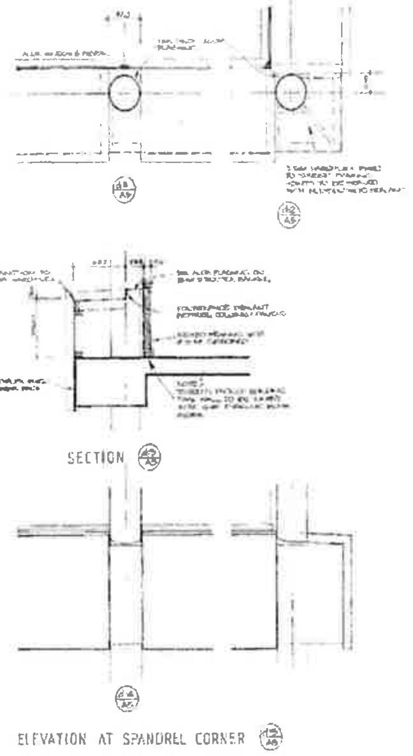
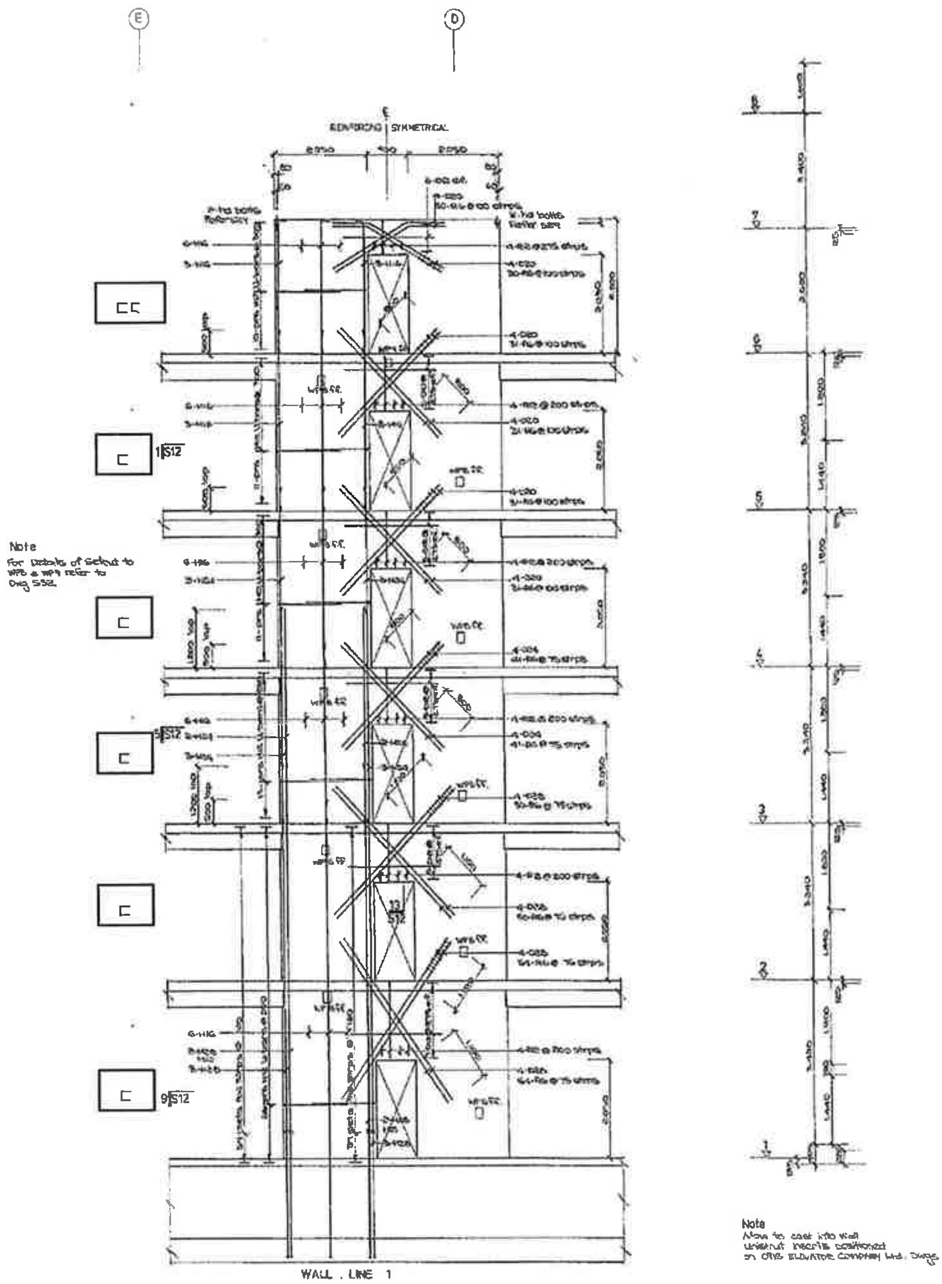


Figure 143 -Pre-cast spandrel panels (Extract from DENG Drawing S25)

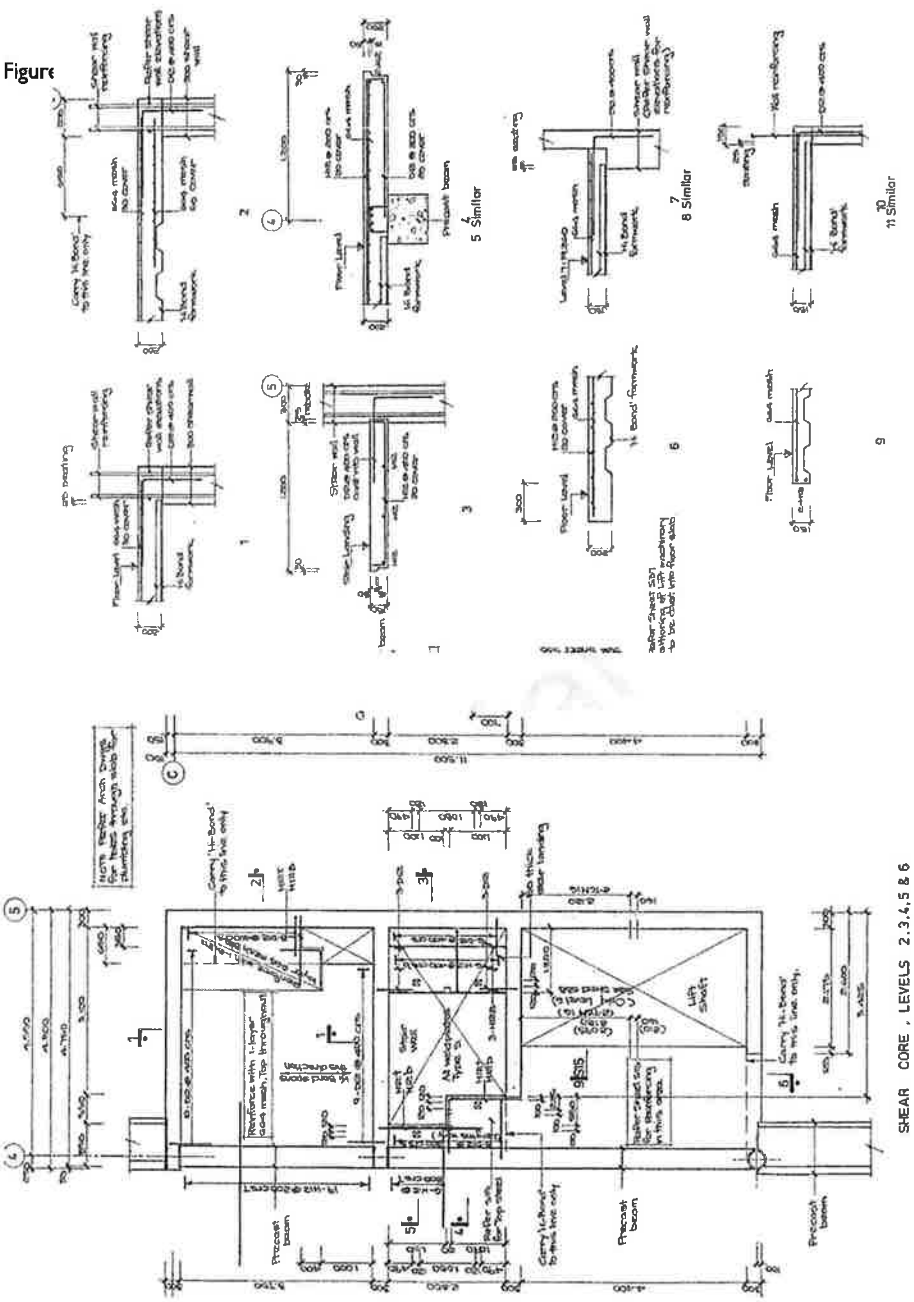


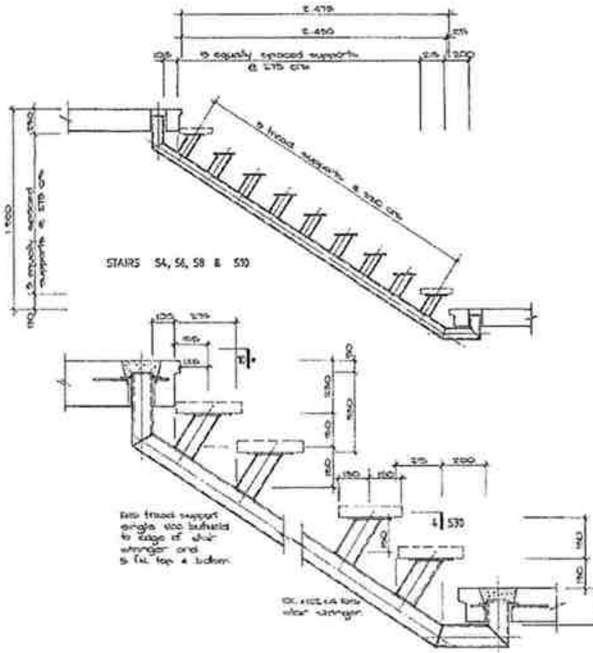
:er Columns (Extract ARCH Dwg A7)

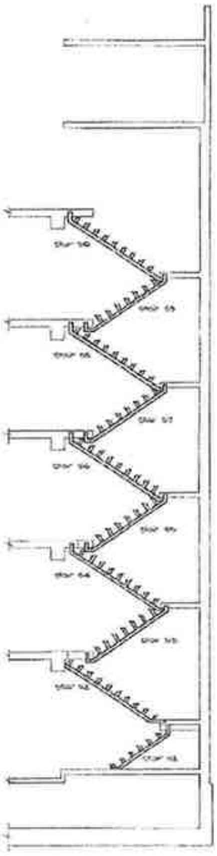



REINFORCING SYMMETRICAL
WALL . LINE 1
Note
For Details of Setout to WP8 & WP9 refer to Dwg S52.

Figure




STAIRS S4, S6, S8 & S10



CONCRETE AND REINFORCING STEEL SPECIFICATION

2. cont'd... 2503

2.6 REINFORCEMENT
All reinforcement shall comply with NZS 3402 (1973). Bars prefixed with a 'D' on the drawings shall be deformed Grade 275 steel. Bars prefixed with a 'R' on the drawings shall be plain Grade 275 steel. Bars prefixed with an 'H' on the drawings shall be deformed Grade 380 steel. Mesh shall be hard drawn steel wire fabric to NZS 3422 (1972). All reinforcement and workmanship shall conform to the requirements of NZS 3109:1980.

2.7 FAIRFACE FINISHES
All concrete surfaces that will be visible in the finished job, or covered with paint, Enduit plaster, or tiles, shall be finished fairface. All concrete required to have a fairface finish shall be cast to a high standard using accurately constructed form work and to a high standard of workmanship. In addition to surface tolerances specified below, the finished surface shall conform for blowholes with illustration 4 in the NZ Standard NZS 3114:1980 "Specification for Concrete Surface Finishes." Refer to the Architect's drawings for the finish required on concrete surfaces.

2.8 SLAB FINISH
Except as specified below, all slabs have a steel trowelled finish. Screed off and lightly wood float. Finish slabs with approved power floating and compacting machines to leave a dense, level surface which does not vary more than 6mm from a 3 metre straight edge, and not more than ± 15mm from true level.

2.9 SITE CONCRETE
Form and cast 50mm site concrete beneath main foundations and elsewhere as necessary to provide a clean, dry working platform. Ensure ground surface is clean and dry and there is no evidence of soft spots.

2.10 FOUNDATIONS
Form and cast main foundation beams as detailed. It is envisaged that the beams will be cast in stages with construction joints. Allow to scrabble or green cut the faces of these joints. The exact location and details of all construction joints are to be agreed with the Engineer before pouring concrete.

2.11 LIFT PIT
Form and cast lift pit walls and floor with sump as detailed. Build in PVC 140mm HYDROFOIL waterstop or similar to all construction joints in floor and walls. Waterproof the concrete with SIKA Plastocrete-N-Waterproofer or approved equivalent.

2 cont'd... 2503

2.12 GROUND FLOOR SLAB
Form and cast ground floor slab on damp proof course on compacted hardfill. Cast in strips and sawcut into panels where agreed by the Engineer on site. The maximum spacing of sawcuts or construction joints shall not exceed 3.75 metres.

2.13 PROPPING OF PRECAST BEAMS
Precast beams shall be propped to support the dead weight of the beam until the floor concrete has reached 20 MPa.

2.14 CHASES, HOLES AND NIBS
Form all chases, holes, upstands and nibs as shown on the drawings or required by other trades. Chases and holes shall be accurately positioned and formed at the time of casting the concrete.
Set concrete shall not be hacked unless specific approval is obtained from the Engineer.

2.15 BUILDING IN
As the work proceeds, build in all necessary bolts and other fixings. The Concretor shall ascertain from all other sub-contractors all particulars relating to their work with regard to order of its execution and details of all such provisions of fixings sleeves, chases, holes, etc., and of all necessary items to be built into concrete and shall ensure that all such items are provided for and/or positioned.

No claim will be recognized or allowed for at extra cost of cutting away or drilling concrete work already executed in consequence or any neglect of the Contractor to ascertain these particulars and make the necessary provision beforehand.

2.16 FLOOR SLABS
Concrete floors have been detailed to use the 'DIMOND HI-BOND H.S.' composite steel/concrete floor system. This has a profiled metal deck of 54mm overall depth, made from G500 steel, 0.75mm thick.

The floor shall be handled, laid, and fixed in accordance with the manufacturer's written "laying instructions".
Provide temporary propping to floors as shown on the drawings, with an upward camber to the propping lines as detailed. Floors shall be constructed of a uniform thickness, so that slab surfaces as constructed shall follow the cambered profile of the floor decking.
Propping shall extend over at least three levels at all times, to distribute the weight of the floor being poured into three lower floors, and to support mobile scaffolds being used to erect precast floor beams.

3. PRECAST CONCRETE

3.1 GENERAL
Refer to the General and Special Conditions of Contract clauses which shall apply to all work in this section of the Specification.

3.2 SCOPE
This section of the specification includes the manufacture and supply on site of the following pre-cast units:-

1. Precast beams
2. Precast wall panels

The work includes the fabrication and supply of all structural steel fittings to be built into the units as detailed on the drawings.

3.3 MATERIALS AND WORKMANSHIP
All formwork, concrete and concreting and finishing shall be in accordance with the relevant clauses of Concrete and Reinforcing Steelwork Specification except where noted otherwise in this section.

3.4 CONCRETE
All concrete shall be HIGH or SPECIAL GRADE complying with NZS 3109 Clause 6.2.Concrete for all precast work shall be 25 MPa at 28 days with 18mm maximum size aggregate.

3.5 TOLERANCES
All precast units shall be manufactured to the following tolerances unless stated otherwise on the drawings:

- Length	± 6 mm
- Cross Section	± 3 mm
- Squareness (of cross section and ends)	± 3 mm
- Twist (dimensions from plane containing the other three corners	± 3 mm
- Built in Items	± 5 mm

The above tolerances are given as a guide. Their application in any particular case shall be subject to interpretation by the Engineer.

3.6 FINISHES
All precast concrete exposed in the finished building shall be cast to a high standard using accurately constructed formwork and a high standard of workmanship. Precast items that do not meet the required standard to the satisfaction of the Engineer will be rejected. Formwork shall be such as to produce a high quality fair face finish on all exposed surfaces. Formwork shall be made from sheet steel or dressed plywood treated with a polyurethane finish to a high quality smooth surface, or similar.

3. cont'd... 2503

In general finished surfaces shall be smooth and formed with moulds or by careful trowelling. Surfaces shall be free from honeycombing, grout loss, excessive air holes or other imperfections. Arrises shall be straight clean and sharp and free from spalling or damage.
All exposed surfaces shall have a similar appearance and standard of finish. Surfaces finished by trowelling shall be finished to the same standard and uniformly match surfaces against formwork.
Formwork shall be sealed at all corners, joins and inserts to prevent all grout loss.
All surfaces against which concrete is later to be cast shall be left roughened by brooming the poured face while the concrete is still plastic. Clean surfaces thoroughly from all laitance and loose concrete.

3.7 HANDLING
A high standard of finish is required and handling shall be such as to prevent any damage to units.
Approved lifting devices or hooks shall be provided in all precast units and these shall be made available to the Contractor for erection purposes and removed cleanly after use. Units shall be handled only by the hooks or devices provided. They shall be loaded and transported so that no forces are applied in excess of those occurring during normal lifting. Twisting forces shall not be permitted to occur. Units shall be strapped and secured to prevent movement or damage during transportation.

Details of lifting hooks and devices, and their positions, shall be submitted to the Engineer for approval before manufacture commences. Care shall be exercised at all times, that hooks or devices suffer no bending or other damage. Lifting hooks or devices set permanently in the units shall have a safety factor of at least 4 and for repetitive use shall have a safety factor of at least 6.

3.8 STACKING
Units shall be stacked on timber dunnage and suitable soft packing placed under the lifting points. Stacking shall at all times be such as to minimise the effects of creep and to avoid undue distortion of units.
Stacking of units shall be carried out on an area capable of withstanding the bearing pressures involved and in such a way that damage to units, lifting hooks, and to other embedded fixtures and to other units shall not occur.

3.9 MARKING
Mark all units with a mark number, orientation in finished job, and date of casting. The marking shall not be permitted to affect the fairface finish.

3.10 INSPECTION
The Engineer or his representative will inspect the precast units at all stages of manufacture to ensure conformity with this specification. Units which do not conform to the required tolerances, which shown grout leakage, which have been damaged, or which are otherwise defective shall be liable to rejection and may be used in the structure only at the Engineer's discretion.

3. cont'd... 2503

No repair work shall be done without specific instruction from the Engineer.

3.11 BUILDING IN
Supply and fix all lifting bolts, cast in sockets, timber grounds and other fixings as shown on the drawings or as required for the proper erection of the units in the finished work.

3.12 PRECAST SHELL BEAMS
Form and cast the beams as detailed including all reinforcing starters, structural steel fixings, holes for services, rebates, etc, as detailed.
The beams have been detailed to minimise their weight and hence crane capacity. The surface of the beams inside the stirrups shall be roughened to ensure good bond to the infill concrete. Outside the stirrups the surface shall be straight and level to receive the proprietary floor system.

Sides and soffits shall be finished as clause 3.6 where exposed in the completed building, otherwise to a reasonable fairface finish.

Figure 148 Extract from DENG Pre-cast Concrete Specification