CTV BUILDING

CTV BUILDING COLLAPSE INVESTIGATION

FOR DEPARTMENT OF BUILDING AND HOUSING
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Part 2 of 3

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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 February Aftershock must be interpreted in the light of observed condition of the CTV building after the September Earthquake and December 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 September 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 and identify likely collapse scenarios taking account of other information available.

Structural analyses and evaluation included the following:

- Elastic response spectrum analyses ("the ERSA) of the whole building
- Non-linear time history analyses ("the NTHA") of the whole building
- Non-linear pushover analysis ("the NPA") of the whole building
- Displacement compatibility analyses of frames on Line 1 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 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 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 was subject to 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 calculated 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. Due to the range of factors noted above, which are subject to variability and uncertainty, there remained some issues where interpretations by the authors varied.

OVERVIEW

Figure 16 presents a diagrammatic summary of the key considerations involved in evaluating the possible collapse scenarios involving columns. The diagram highlights that at the heart of the evaluation involved the comparison of "demand" with "capacity". "Demand" may be thought of as the loads and displacements imposed on the columns by the combined effects of gravity and earthquake loadings. "Capacity" may then be considered as the strengths of critical columns 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 a 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 divided by the inter-storey height is termed inter-storey drift or simply "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.

It was found that the drifts determined from the analyses were sufficient to exceed the capacities of columns if there was no diaphragm disconnection. 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 designed to that standard may be limited by the diaphragm connection capacity, which shouldn't be the case.

The Drag Bar connections were shown by analysis and assessment in Appendix G to be the most likely location for detachment of the slab due to in-plane actions of the floors. Review of the physical collapse evidence indicated that failure may not have 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 95. 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 and 6 had most likely occurred due to the floors losing their support along Lines 2 and 3 as those columns collapsed. In considering the possibility of diaphragm disconnection, 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 so failure would have occurred sooner than in cases where the diaphragm had remained connected.

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 and 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. 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 the ground motion record chosen.

Nonlinear pushover analysis (NPA) uses a model with the same features as that used in the NTHA. A static load distribution is applied, and increased incrementally to allow 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 self-weight of the building can be estimated within reasonable limits though estimation of the load due to contents can be more difficult and uncertain.

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 potential 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.

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 effective column stiffness section properties derived from moment-curvature analyses and non-linear rigid plastic hinges at their heads and bases.

The other used fixed end moment drifts derived using moment-curvature software Cumbia and adjusted for frame effects as described in Appendix 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.

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 for the more heavily loaded columns, and the limited 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 and arrangement of reinforcing steel.
- Vertical ground accelerations may have increased or reduced the loads on columns and thus increase or reduce drift capacity.

On top of these considerations was the potential influence of the Spandrel Panels on column capacity. Observations after the September Earthquake and inspection of structure remnants after February Aftershock indicated that there may have been interaction between the columns on the north, east and south faces with the adjacent Spandrel Panels. Such interaction was found by the NPA and displacement compatibility analyses to have possibly reduced the drift capacity of those columns.

In assessing Spandrel Panel interaction it was noted that the actions generated may have been limited by the capacity of the Spandrel itself or the bolts that connected the spandrels to the floor slab. Assessment of the maximum bracing capacity of a typical Spandrel Panel indicated that the capacity could be limited by the out of plane flexural strength of the Spandrel end walls.

The Spandrel Panels may have accelerated column head failure, but it was difficult to see how the development of the mid-height hinging observed on a number of columns had occurred. One possible explanation for the hinging in those locations may be due to the development of localised bursting stresses at the ends of the column vertical bar lap splices which may not have had sufficient spiral confinement reinforcing,.

It was also difficult to accurately assess the column shear capacity due to the different guidance available and the fact that the CTV Building columns had an unusually low core area compared to gross section area. There was less transverse reinforcement than it appears had been in the tests that formed the basis of the NZSEE 2006 assessment guidelines.

The NTHA and ERSA models did not allow for interaction with the Spandrel Panels but were used to estimate the likely drift demands at these positions. This was accounted for in the NPA analyses. 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.

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 detailed welded in place which were likely to provide engagement of all bolts simultaneously. The capacity of the fixings specified was 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.
- Maximum possible Spandrel Panel interaction (minimal gap between column and spandrel) would have 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.
- However the displacement demands of the February Aftershock were sufficient, based on application of the full ground motions, to fail a critical columns without any Spandrel Panel interaction.

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 slab 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 assessed as weaker than the reinforced slab at the location where the slab was found to have failed. This indicated that the slab may not have failed due to excessive in-plane diaphragm actions but may have failed due to loss of vertical support as the columns on Line 3 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 short embedment lengths of beam bars and the lack of specific 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 axial load in the columns. 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.

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 might have 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 used 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 September Earthquake, the analysis indicated in plane shear forces in excess of the shear capacity of the masonry if the infill was assumed to be fully restrained by the concrete header beams and columns; or in-plane drifts up to 30 mm if the masonry was assumed to be separated by the frame. Photographs of the walls and statements by Eyewitness 16 found no damage or spalling.

This suggests that the masonry walls, at least for the September Earthquake level of shaking, may have been stiffer than assumed in the NTHA analysis and that the response of the structure to the ground motion may have been 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 may 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.

COLLAPSE INITIATORS EXAMINED

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

1. Column failure on Line F or Line 1. This involved collapse initiation as a result of column failure on one of these lines, probably in a mid to upper level, with or without the influence of spandrel interaction. A Line F initiation was noted as being consistent with the arrangement of collapse debris and eye witness reports of an initial tilt to the east.
2. Column failure on Line 2 or Line 3. Collapse in this case would be initiated by failure of a column at mid to low level, under the combined effects of axial load (gravity and vertical earthquake) and inter-storey displacement. Low concrete strength could have made this scenario more likely.
3. Column failure due to diaphragm disconnection from the North Core at Level 2 or Level 3. This scenario requires that the diaphragm separated from the North Core causing a significant increase in the inter-storey

displacements in the floors above and below. The nature of the separation and resulting movement of the slab would have an influence on which of these highly loaded columns was the most critical. It was noted that no Drag Bars were installed at these levels.

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

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

Figure 16 outlines the key considerations involved in evaluating these scenarios. Further explanation and background is given in the section on the Four Scenarios.

CRITICAL COLUMN IDENTIFICATION

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

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

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

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

KEY DATA AND RESULTS

Elastic Response Spectra Analysis

Figure 11 shows the response spectra used in the ERSA. In this graph, the vertical axis represents the expected response of a building to the ground shaking. The horizontal scale shows the natural period of vibration of a building (low buildings generally having low periods and high buildings having high periods). The natural vibration period of the CTV Building was around 1.0 to 1.3 seconds.

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

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

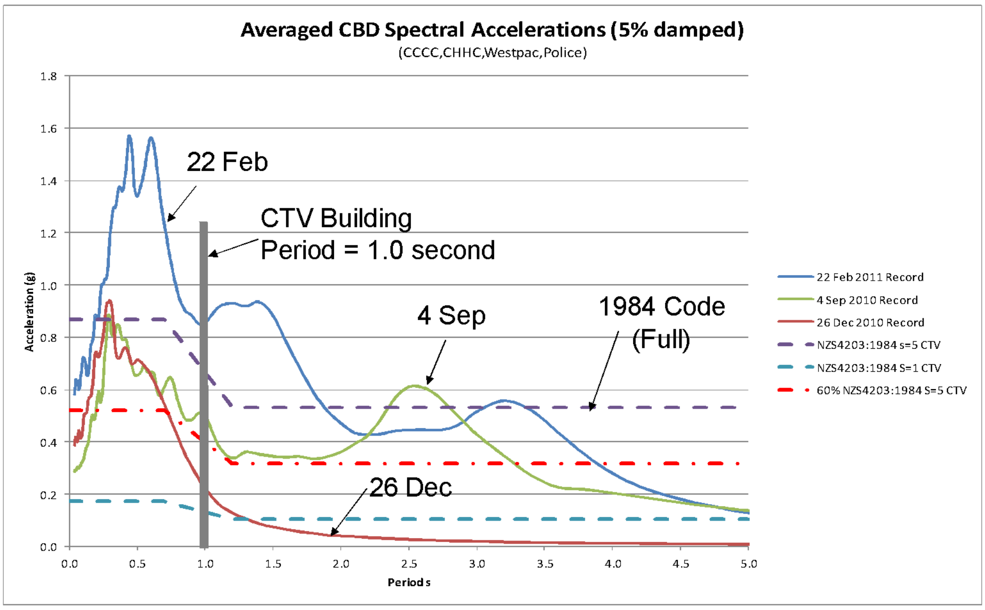

Figure 49 – Response spectra records for the September Earthquake, December Aftershock and the February Aftershock. Also shown (dashed lines) are the design spectra for the CTV building according to NZS 4203:1984. The lower dashed line is the spectra for ductile design that the North Core and South Wall were required to have axial / flexural dependable strength in excess of. The upper most dashed line is the elastic response spectra that the structure was expected to be able to match in terms of ultimate displacement without collapsing.

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

Although direct comparison of such spectra can be misleading, it can be seen that at a period of 1.0 second, the acceleration shown for the February Aftershock significantly exceeded the full 1984 value required for the design of elastically responding structures. The CTV Building had been designed for ductile response using forces derived from the lowest design spectra shown in Figure 11.

The ERSA indicated that the demand of the February record was approximately 2.2 times the demand of the September record, which was in turn almost 1.8 times the demand of the December Aftershock.

Seismic Detailing Requirements Check

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

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

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

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

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

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

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

Column F2 Level 3 – Demand versus Capacity

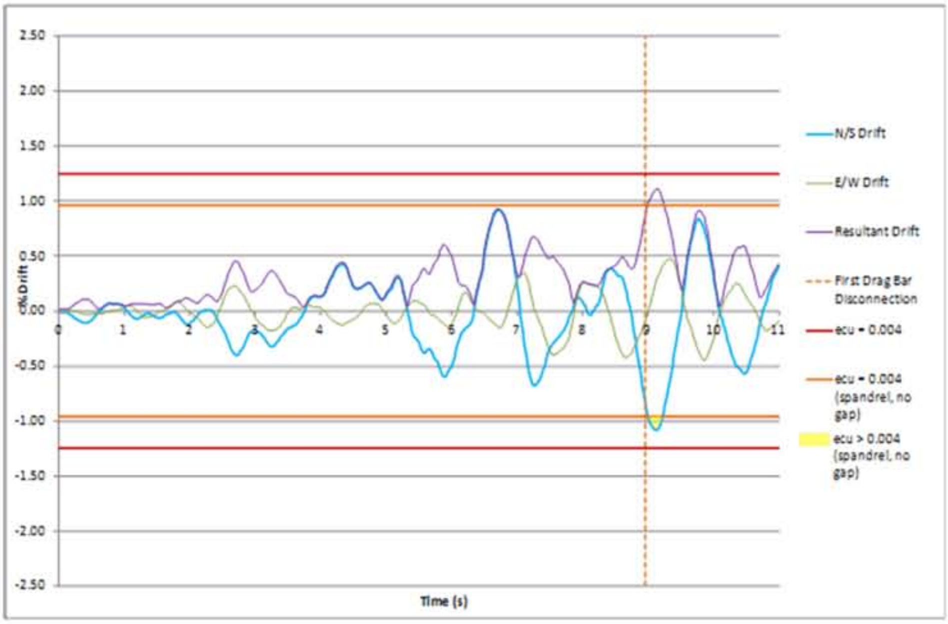

Figure 50 - Comparison of drift demand and capacity – column F2 Level 3 - CBGS, 4 September Darfield Earthquake, no masonry.

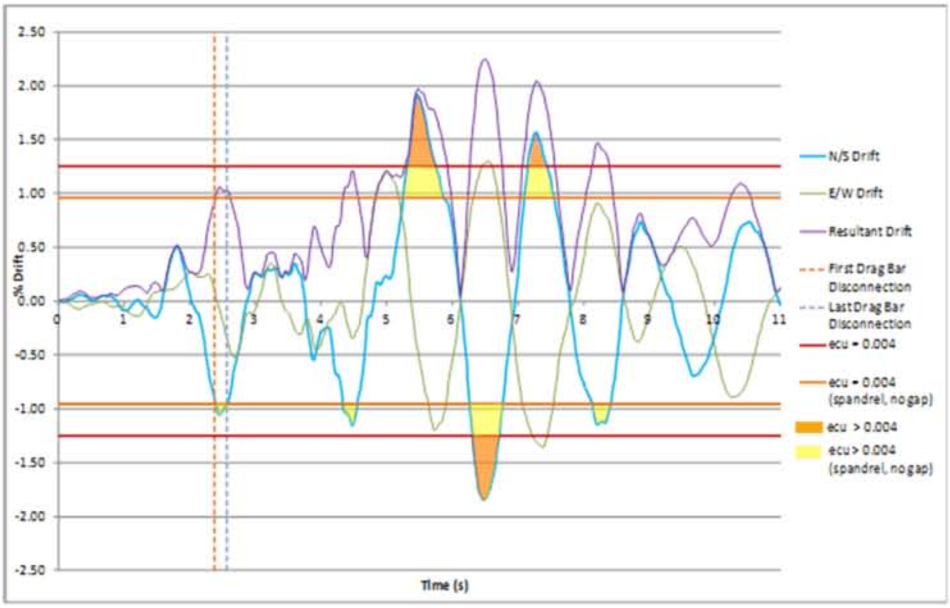

Figure 51 - Comparison of drift demand and capacity – column F2 Level 3 - CBGS, February Aftershock, no masonry.

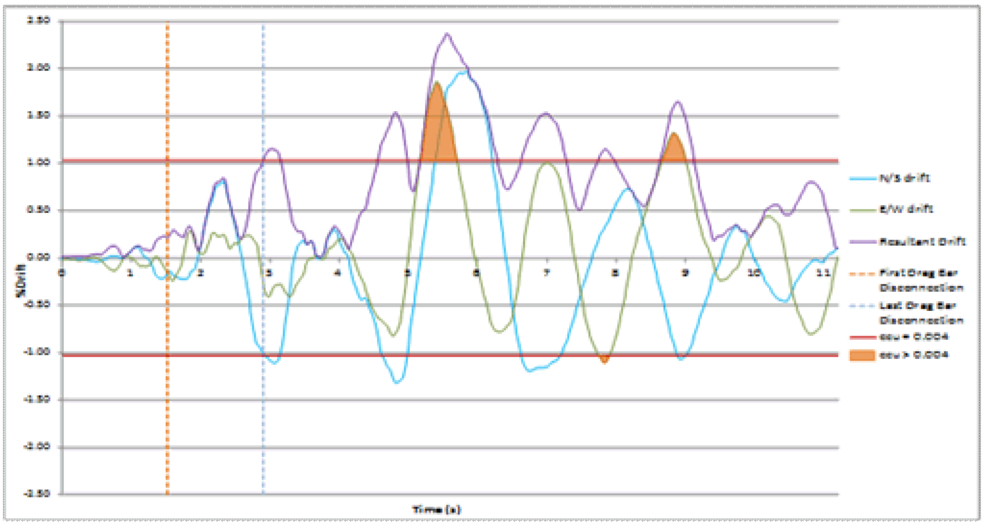

Figure 52 - Comparison of drift demand and capacity – column D2 Level 3 - CHHC, 22 February Lyttelton Aftershock, no masonry.

Figure 12 and Figure 13 show output from the non-linear time history analyses for column F2. Figure 14shows output from the analyses for column D2. The vertical axis shows the amount of inter-storey displacement (drift) at this column location. The horizontal axis is the time from start of shaking (as input into the analysis). The wavy lines plot the drift level over time and are based on application of the full ground shaking record in the analyses. This drift is a key measure of demand on the column. The blue line shows the north-south drift which is critical for the grid F columns, taking into account the stiff façade beams and the potential interaction with Spandrel Panels. The thin brown line indicates the resultant drift of the north-south and east-west drifts.

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

The horizontal lines represent the estimated capacity of this column to sustain the drift without failing according to various criteria (assuming expected concrete strength and without vertical earthquake effects). The band between the horizontal lines in Figure 12 and Figure 13 reflects the difference between "no interaction with the spandrels" (higher value) and "full interaction with the spandrels". The areas where the drift had exceeded the estimated capacity are shown shaded. The band showing the range of capacities would be wider if allowance was made for the effect of variable concrete strength and vertical earthquake forces in the column.

Estimates were made of the influence of axial force and concrete strength on the drift capacities of columns in different locations. Three key capacity points were identified for each case: the displacement to cause initial yield in the reinforcing steel, initiation of concrete yield at compressive strain of 0.002, and the displacement to cause the 0.004 ultimate compression strain in the concrete (at which failure was taken to occur).

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

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

Similar plots were made for column D2 at Level 3, shown in Figure 14, with similar conclusions being reached regarding the likely performance of this column in the 22 February 2011 event.

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

These comparisons give some indication of the challenges of determining which column or mechanism initiated failure. However, the plots indicate clearly that there is a strong likelihood that the demands of the February Aftershock were enough to cause column failure, whereas the demands of the September Earthquake were much less.

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

Effect of Vertical Acceleration

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

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

The NPA found that the drift capacity of the F2 Level 3 column may have been reduced by approximately 12% for a 60% increase in axial load from vertical earthquake effects. The alternative method indicated a reduction of drift of between 0.25% to 0.50% /1000 kN increase in axial demand. Indications were that at greater compression demands the reduction in drift capacity may be higher, however that has not been established accurately.

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

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

Drift Demand Capacity Comparison

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

Each table shows the average maximum drift demand for the September Earthquake, and the December and February Aftershocks for the full records as noted. For the February Aftershock, Also shown are two 1986 standard design limits for the CTV Building:

The "1986 Non-ductile detailing" figure is the drift demand computed in accordance with 1986 standards to determine the need or otherwise for ductile detailing of the columns. Non-ductile detailing would have been allowed provided that the actions induced in the column at this point did not exceed the prescribed limit.

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

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

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

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

Table 4 - Indicative drift demand and capacity values on column at Grid F2 at Level 3.

B. Column on grid D2 at Level 3		
Demand or Capacity	Event / Condition	East-West Column drifts (% of floor height)
	Event / Condition	Full Record
Demand	22 February 2011 (NTHA - CHHC)	1.9
	26 December 2010 (estimate)	0.40
	4 September 2010 (estimate)	No analysis
	1986 Non-ductile Detailing	0.5
	1986 Ultimate	1.0
	2010 Ultimate	1.8
Capacity	Failure (No spandrel effect)	1.1 - 1.2 (range)
	Failure (Full spandrel effect)	No spandrel

Table 5 - Indicative drift demand and capacity values on column at Grid D2 at Level 3.

POSSIBLE COLLAPSE SCENARIO

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

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

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

Once the interior columns began to collapse, the beams and slabs above fell down and broke away from the North Core, and the South Wall and the beams and columns attached to it then fell northwards onto the collapsed floors and roof.

Figure 17 shows the situation for this scenario with no spandrel interaction (A) and with spandrel interaction (B and C) and Figure 18. Figure 20 illustrates the case of failure of ground floor columns on Line D for this scenario and the subsequent collapse of the floor slabs and frames for this inferred collapse sequence. Figure 19 shows the case along Line 2 for the scenario involving initiation on Line F.

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

THE FOUR SCENARIOS

Preferred Collapse Scenario

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 had been moved in the rescue showed a slight throw eastward. These observations and the structural analyses seem to support Line F column failure scenario, possibly including Spandrel Panel interference effects, being the likely point of initiation of the collapse.

Scenario 1: Line F or 1 Column Collapse Initiation

In this scenario collapse may have initiated in the Line F perimeter columns with bases at Level 2 to 5 at drifts between 0.9% and 1.3%. This may then have then led to overload of the Line 2 and 3 columns at mid to low level, at the Madras Street end of the building.

The Line 1 and F column lines were found in to experience the highest inter-storey drifts in the structure.

ERSA indicated that If the infill masonry wall on Line A restrained the frame the centre of rigidity shifted westward (Figure 53).

The level at which the Line F columns may have initiated collapse may have been dependent on the size of the gaps between the columns and the precast Spandrel Panels. Interaction with the panels would likely have reduced the drifts necessary to exceed the column concrete compression strain limit at the column heads.

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 and northwards on the Line 1 South Wall and frame. The slab and beams connected into the columns at Grid A would have pulled down and inwards on those columns and this may explain the beam-column joints pulling out in places. The upper levels and roof above the column failure initiation on Line F could have then dropped as a unit, and with a slight lean towards Madras Street, collapsing the structure below to the ground.

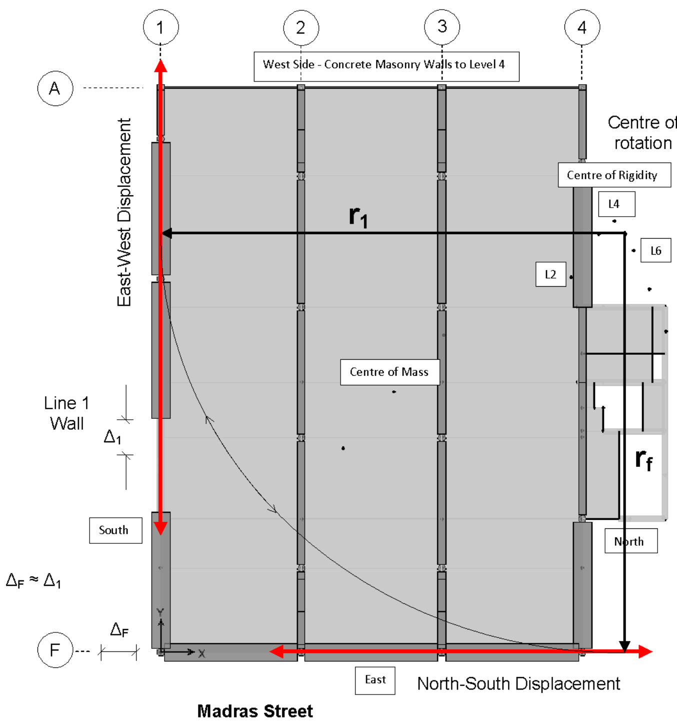

Figure 53 – ERSA indicated that torsional behaviour of the building increased if the masonry infill wall on Line A was not adequately separated from the frame. This shows that in that case the building would have had a tendency to twist about the centre of rigidity that was moved towards the Line A wall because of its stiffness. The centre of rigidity was furthest west at Level 4. This may have 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 September Earthquake and February Aftershock. This could have made the columns on these lines more susceptible to being damaged and initiating collapse during the February 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 internal columns. This was considered in the NPA and as part of the displacement compatibility analysis. This scenario 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 February Aftershock.

In this scenario it was envisaged that following a line 2 or 3 internal column failure the floors would have sunk and the slabs would have been forced into catenary type behaviour. The structure then would have progressively collapsed inwards onto itself.

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

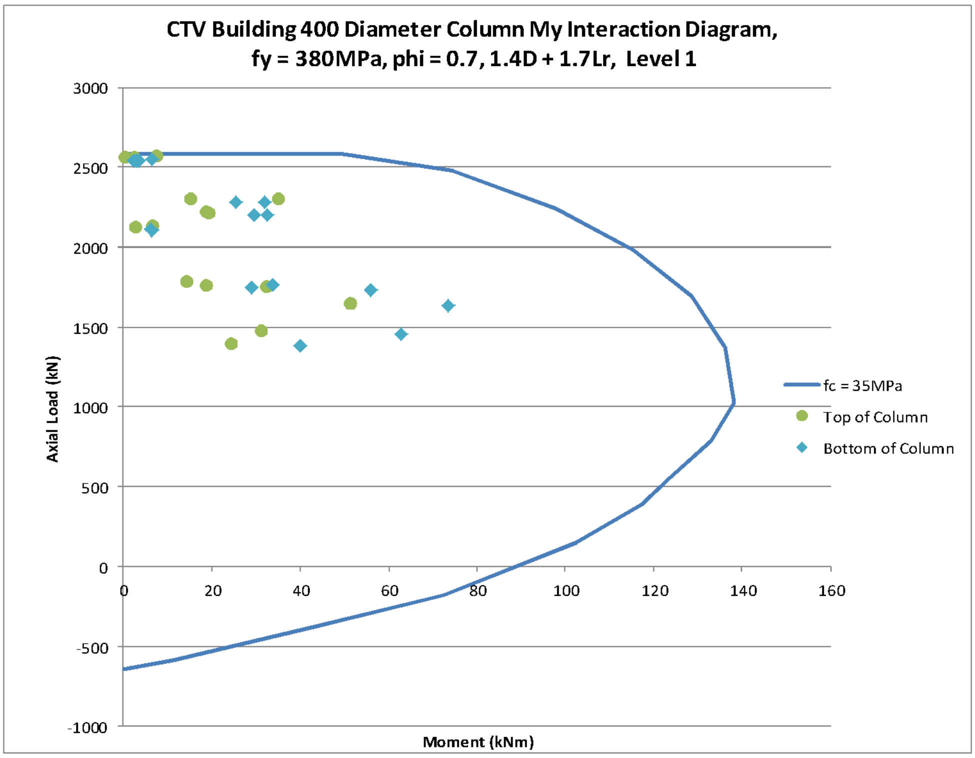

Figure 54 - Column Chart for factored design Gravity Load 1.4D + 1.7 Lr on Level 1 columns for concrete with specified 28-day strength of 35 MPa. This indicates that a number of the Level 1 columns were getting close to the blue line and nearing the standard axial load design limit. The phi factor of 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 as required in the Standards. For the condition used in the collapse scenarios these safety factors have been reduced to 1.0 to better reflect actual loading conditions and expected strength at the time of the collapse.

However the testing of column remnants found the concrete to have lower 5 percentile strength of 17.3 MPa (when adjusted for possible testing orientation effects). This was significantly less than the lowest 28-day strength concrete specified for columns in the CTV Building. For this reason lower than specified concrete strength was considered in the column collapse scenarios.

The most highly loaded secondary frame columns were on Lines C and D, 2 and 3. A check of the gravity actions on these columns at Level 1in 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 54.)

The 'non-seismic' detailing of reinforcement in the columns (small diameter spirals 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 initiated in the lower floor columns.

In summary this isolated Line 2 or 3 internal column collapse mechanism was a credible option that cannot be discounted.

However it may 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 perhaps have been more likely to have resulted in an even more concentric debris pile on the site than what was observed.

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

The demands imposed on the connections of the diaphragms to the North Core were difficult to accurately assess. The lack of Drag Bars at these levels at the lift shaft wall\ increased their vulnerability.

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 potentially high in-plane flexural demands that could occur at loadings recently proposed for diaphragm design (Uma, Zhao et al. 2009).

The effect of diaphragm detachment at Level 2 or 3 would have been to overload Level 1, 2 or 3 columns by imposing greater lateral displacement on them due the loss of restraint from the North Core.

It appeared based on ERSA and assumptions of equivalent static analysis that the effect of the Line A masonry infill may have been to reduce the demand on the diaphragm connection to the North Core between walls C and C/D. The connection however remained vulnerable.

The Level 3 slab was found from the collapse photos to have remained lying against the North Core after the collapse indicating that it had lost support at the Line 3 end of the slab rather than at the North Core required by this scenario.

In conclusion it appeared that diaphragm disconnection at Level 2 and 3 was not entirely consistent with the collapse evidence and less likely than Scenario 1 and 2.

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

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

The NPA indicated that significant uplift could occur at the southern face of the North Core as the wall displaced to the North.

It was considered a possibility that rocking or tensile extension of the south face of the North Core related to primary North/South response may have initiated failure and detachment of the floor slabs due to a combination of in-plane and out-of plane diaphragm actions.

The physical collapse evidence showed floor slabs sloping down from the North Core. If collapse had initiated by disconnection of the slab on Line 4 from the North Core then it would have been expected that the slabs would have been expected to have rotated downwards about Line 3 rather than about Line 4.

The NTHA indicated in Figure 140 that at drifts as low as 1.0% along Line F, the calculated capacity of the Level 4 Drag Bar may have been reached. This is within the range of the expected failure drifts of the columns of between 0.9 and 1.3%. The Drag Bar disconnection drift value from the NTHA should be treated with caution however due to the complexities recognised in the engineering profession with respect to accurate analysis of diaphragm to shear wall interaction effects. It may have been larger or smaller than this amount.

It is possible that Drag Bar disconnections did not take place prior to the collapse because it appears from the photos of the North Core immediately after the collapse that the Level 4, 5 and 6 slabs may not have failed initially at the Drag Bars. This is discussed in more detail in Appendix G.

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

In conclusion it appeared that this scenario was not entirely consistent with the collapse evidence and so was considered less likely than Scenarios 1, 2 or 3.

9 DESIGN, CONSTRUCTION AND STANDARDS ISSUES

INTRODUCTION

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

DESIGN ISSUES

Building Inter-storey Drift Limits

The building as a whole was required to have sufficient lateral stiffness to not exceed the inter-storey K/SM factored drift displacement and drift limits for Zone B (Christchurch). This drift limit 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 used to calculate this drift allowed for some level of foundation rotation and for effective stiffness of shear walls according to NZSEE journal Vol.13 No.2 June 1980. Full fixity of foundations was allowed at the time of the design (NZS 4203:1984 cl3.8.1.2).

For Levels 2 to 6 with inter-storey heights of 3.24 m the drift limit was calculated to be 27 mm. For Level 1 with inter-storey height of 3.66 m the limit was 30 mm. This set the minimum stiffness requirements for the primary seismic resisting structure.

In Appendix F the primary frame was found to have satisfied the building inter-storey drift requirements if no account was made of the effect of inelastic deformation initiating in the South Wall at the K/SM deformation levels. It is therefore debatable whether the drift limit was satisfied.

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 3101:1982 cl. 3.5.14.1(b) and cl. 3.5.14.3) if they were not considered part of the primary system. A displacement compatibility analysis was required to determine whether the requirements for seismic design were should be applied. The columns were checked to determine whether they remained elastic at drifts imposed on them by the movement of the primary structure when its design displacements were factored by K/SM= 2.75.

It was found in Appendix F that a number of columns would have exceeded their elastic deformation limit strength under the applied K/SM factored drifts. 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.

Minimum Shear Reinforcing of Columns

The columns did not satisfy the minimum requirements for shear reinforcement in columns of NZS 3101:1982. These requirements applied irrespective of whether seismic detailing was required or not.

These requirements were for minimum spacing of the spiral reinforcing of approximately 150 mm (cl 7.3.5.4) and a minimum cross sectional area (cl.7.3.4.3).

Spiral reinforcing of R6 @ 90 mm centres approximately or R10 @ 150 mm centres, with the same steel properties as those specified, would have been required. The spiral reinforcement detailed was R6 @ 250 mm centres.

Spandrel Panel Separation

The displacement compatibility and the pushover analyses found that column collapse could have initiated without Spandrel Panel interference. However spandrel interaction may have contributed to column collapse at lower levels of drift.

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 column 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 potential 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). This was published after the project was completed and is not considered a mandatory compliance document in New Zealand. However it provides useful guidance on realistically achievable tolerances based on research undertaken in Great Britain. 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 for K/SM drifts; Spandrel Panels 820 mm high above floor level would have required a minimum 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 55). 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 NZS3109:1980:

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 the tolerances a little.

Construction personnel interviewed indicated it was likely that the panels were lined up to give an even visual line up the building rather than to give a minimum clearance.

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 potential effect of the Spandrel Panels interfering with the movement of the columns was to accelerate critical column head flexural/compressive damage.

The mid-height column hinging may have occurred during the collapse rather than contributing to the collapse.

On the other hand the column hinging above the Spandrel Panels adjacent to the ends of the 1200 mm long lap splices may indicate a localised initiation of the hinge at the ends of the column reinforcing steel splices. In such a case it is possible that the lack of adequate confining steel around the ends of compression splices may have allowed localised bursting of the cover concrete to initiate the hinge. Column hinging near the location of the ends of the bars splices and base damage appeared to be evident in the beam-column remnant seen in Figure 106.

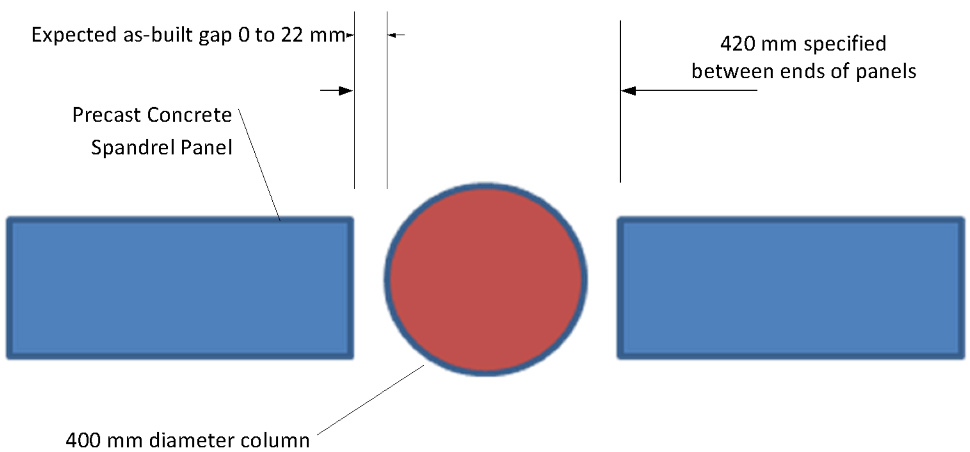

Figure 55 - Expected gaps achieved between Spandrel Panels and columns to achieve a specified gap of 420 mm between ends of panels. This based on BS 5606:1990 guidelines on construction tolerances.

No minimum seismic gap was specified.

A nominal gap of 420 mm was specified between the ends of adjacent precast concrete Spandrel Panels on Line 1, 4 and F. However the Drawings didn't specify a minimum clearance gap to the columns, 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 had no specific spiral or hoop reinforcing detailed to provide confinement or 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 R6 @ 250 mm centres column spiral reinforcement would have been difficult to achieve in practice. As an integral part of the columns, the joints would also have been required to be designed using the additional seismic design requirements of NZS 3101:1982.

It is conceivable that the lack of continuity steel through the beam column joints meant that the beams were unable to cope with much loss of vertical support as isolated columns were damaged and failed. Instead of being able to redistribute some of the load along the frames, 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 as recommended in NZS 4203:1984. The centre of stiffness of the designated primary seismic resisting elements was significantly eccentric to the centre of mass

The North Core and the South Wall, which was a coupled shear wall, had 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 4203:1984 cl. 3.8.4.1(a) and 3.8.4.2(a)).

A nominal 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 showed the top course to be grout filled and 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 OIE reported after the September earthquake that on Level 1 the wall was visible and undamaged with flexible sealant in vertical joints intact. On Level 2 the wall was concealed by plaster board linings. However there was a crack in between the internal framing/ lining and the northwest corner column where the OIE could see daylight through indicating that the sealant may have fallen out. Eyewitness 15 also said he could feel air and see little holes of light in the corners of the archive room in the northwest corner of Level 2. The archive storage room was located between Line 3 and 4.

Eyewitness 16 reported that the top courses may have been partially grouted and some horizontal gaps between the top course and the underside of the beams were observed in places. He also reported that the vertical separation joints between the masonry infill panels and the columns were filled with mortar on the outer face.

In conclusion the authors consider that the vertical seismic separation joints in the masonry infill may have been compromised to some extent by mortar on the outer face. This may have caused the Line A frame and infill masonry to act as confined masonry bounded by the precast beams and columns. As such it may have increased the stiffness and strength of the Line A considerably above what was intended by the Design Engineer.

Diaphragm Connection

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.

No specific 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 review by an Independent Consulting Engineer, and resulted in steel angle Drag Bars being designed by the Design Engineer and installed on Levels 4, 5 and 6.

The Drag bars that were added appear to have been 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 this 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

Robustness means the ability of the structure to sustain damage without causing progressive collapse of the building as a whole.

The secondary beam and column frames lacked the level of robustness expected of frames designed to cope with the cyclic drift demands of earthquakes.

The seismic design provisions of the 1982 concrete structures standard would have improved robustness if they had been applied to the beams, and beam-column joints.

Documentation

The preparation requirements of construction joints requiring shear friction to develop across them in the North Core and South Wall were not shown on the drawings in accordance with NZS 3109:1980.

The top course of the masonry infill on Line A was indicated on DENG Dwg S9 section 6, as fully grouted. However the design calculations indicate that it was intended to be 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 gap between the Spandrel Panels was not identified as a minimum gap between the Spandrel Panels and the columns 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 February 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 September 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 starter bars and separations from columns had been specified in order to reduce the moment capacity and stiffness of the block work, it had not been detailed as isolated from the header beams at each level. 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 detailed column drift 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 55%. 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 distribution found in the column remnants 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 vulnerable.

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 and cement paste is less likely to leak out. A curing membrane was reportedly sprayed on to the column surfaces after the shutters were removed, so curing may have been adequate. No areas of "bony" concrete, where the aggregate lacked adequate cement, were found in the columns examined.

Figure 56 - Concrete from column on Line 4-D/E (C18) showing possible discolouration from silt.

Cores taken from the Line 4-D/E column were however found to have some discolouration that may or may not have been due to silt, had low density and did not exhibit the degree of aggregate breakage expected for concrete of the specified strength (Figure 56).

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 such as those in the CTV Building shear walls it was important to ensure that the surfaces were roughened using methods prescribed in the concrete construction code of practice NZS 3109: 1980. There were a number of construction joints in the South Wall that did not show signs of having been roughened as expected.

Bent –up Bars

Where precast components, such as the beams on Line 4, are required to be tied into in-situ 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 be taken in such circumstances to contact the design engineer 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, such as between the masonry infill walls and the surrounding frame, 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 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.

STANDARDS AND CODE ISSUES

Introduction

In the process of the investigation a number of shortcomings were found in the Standards and Codes of Practice applicable at the time of the design and construction of the CTV Building. Some of these shortcomings appear to remain up to the current time and need to be addressed.

The following extract puts in context the state of seismic engineering knowledge in New Zealand near the time of the publication of the concrete structures design code of practice NZS 3101:1982:

> The emphasis of current New Zealand loadings and concrete design codes is on good structural concepts and detailing. It is recognised that uncertainty exists regarding the selection of the mathematical model representing the behaviour of the structure and the form of the imposed ground shaking. Major damage observed in overseas earthquakes has been shown to be due mainly to poor structural concepts (for example: column side sway mechanisms and/or considerable twisting, due to soft storey, or lack of symmetry and uniformity), and poor ductile detailing (for example: brittle connections, inadequate anchorage of reinforcement or insufficient transverse reinforcement to prevent shear failure, buckling of compressed bars and crushing of concrete). The aim in seismic design is to impart to the structure features which will result in the most desirable behaviour, which implies establishing a desirable hierarchy in the possible failure modes for the structure. This philosophy may be incorporated in a rational capacity design procedure. A proper assessment of the strength and ductility of a structure cannot be made using the working stress design method. Hence the new concrete design code does not permit the working stress design method to be used; instead, design is required by the strength method.

Extract Park, R., "Review of Code Developments for Earthquake Resistant Design of Concrete Structures in New Zealand:, Bulletin of NZNSEE, Vol. 14, No. 4, December 1981.

Design and Construction Documentation

There were no requirements to ensure collapse critical components requiring special attention were identified on site drawings easily identifiable by site personnel in NZS 3109:1980. These include:

- Seismic gaps
- Concrete strength and testing
- Construction joint preparation and locations
- Primary seismic resisting elements

- Record keeping of inspections and test results and construction sequence
- Product certificates for reinforcing steel, sealants, etc.

There did not seem to be adequate guidance on determining how to combine multiple component construction tolerances when determining appropriate allowances for seismic separation gaps in NZS 3101:1982 and NZS 3109:1980. BS 5606:1990 now appears to offer an improved approach.

Fire-proofing requirements may conflict with seismic separations requirements.

There appeared to be no requirement to inspect and maintain seismic separation joints.

Non-ductile Columns

Undue reliance seemed to be placed on the appropriate selection of cracked section properties of concrete members to trigger the important seismic detailing requirements for concrete structures in NZS 3101:1982.

Similarly the requirement that elastic behaviour was maintained in the CTV Building columns up to only 55% of calculated ultimate limit state drifts from an analysis using the assessed cracked section properties seems too low in NZS4203:1984 and NZS 3101:1982.

With respect to the displacement compatibility analysis requirements the requirement to satisfy elastic theory is not well defined in NZS 3101:1982. It was disturbing that only the Line 1 and Level 5 Line F columns triggered the requirement for seismic design and detailing.

Buildings designed before NZS 3101:1995, and especially those designed prior to NZS 4203:1992 (which increased the drift demand), with non-ductile gravity columns may be unacceptably vulnerable. They should be checked and a retrospective retrofit programme considered.

There was no comprehensive analytical model for assessing shear/flexure/compression interaction of concrete columns and particularly with allowance for the effects of cyclic earthquake demands in NZS 3101:1982. This remains an issue internationally (Mostafaei, Vecchio, Kabeyasawa, 2009).

NZSEE Guidelines for the assessment of existing structures for shear strength of Group 2 columns do not clearly identify validity range of shear strength provisions. It may not be appropriate to fully apply those provisions in cases such as those found in the CTV Building with low reinforcing ratios, wide spacing of transverse reinforcing and low core area to gross section area based on the testing range used (Kowalsky, Priestley, 2000).

There was no maximum gross to core section area ratio for columns for "non-seismic" secondary columns in NZS 3101:1982.

Vertical Acceleration Effects

No clear analytical models were identified for assessing effects of vertical acceleration on columns and other potentially critical components in NZS 4203:1984. The authors developed approximations of these effects for the purposes of this investigation by considering NPA results in Appendix D, and column drift capacity curves of varying strengths as shown in Appendix F.

More guidance in this area is required for practicing design engineers.

Concrete Strength Effects

No clear analytical models were identified for appropriately assessing the effects of concrete strength effects on columns. The authors developed approximations of these effects for the purposes of this investigation by considering column drift capacity curves of varying strengths as shown in Appendix F.

More guidance in this area is required for practicing design engineers.

Analysis and Design of Irregular Structures

There was a requirement to locate the main seismic resisting elements as nearly as practical symmetrically about the centre of mass of the structure. However most structures require some level of torsional irregularity to satisfy reasonable architectural requirements. There were no clear limits for torsionally irregular structures in terms of compliance requirements. Some guidance was given in the Commentary to the Loadings Standard NZS 4203:1984.

Requirements to consider Group 2 elements in the analysis of structures, when they could have significant effect on response in NZS 3101:1982, were not clear.

Ductile design and analysis compliance provisions did not seem sufficient to adequately envelope drift and ductility demands where differential yielding of components occurred in a structural system. There were warnings about the issue in NZS 4203:1984 and NZS 3101:1982.

Development and codification of better methods and limits for applying and designing torsionally irregular structures using static and ERSA analysis seems necessary. More guidance on appropriate modelling using NTHA methods also seems necessary to improve consistency.

Diaphragm Connections

There was a lack of detailed guidance for the design and detailing of diaphragms and connections to ensure robust performance in NZS 3101:1982. This remains a problem in current standards.

The “Parts and Portions” in the NZS 4203:1984 design provisions for connection of diaphragms to seismic lateral resisting walls seem inadequate. They did not ensure that diaphragm ties were not a weak link limiting the overall strength of the structure under severe seismic demands. The provisions did not appear to account for full displacement and strength demands, or higher mode response characteristics of the structural system.

Buildings with connections between floor slabs and shear walls (diaphragm connections) designed to the provisions of the Loadings Standards NZS 4203 prior to 1992 may be at risk. Further investigation into the design of connections between floor slabs and structural walls is needed.

10 CONCLUSIONS

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

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

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

Analyses using the full February Aftershock ground motion records indicate drift demands on critical column elements to have been in excess of their capacities even assuming no spandrel interaction and no vertical earthquake accelerations.

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

- The stronger than design-level ground shaking.
- The low displacement-drift capacity of the columns due to:
    - The low amounts of spiral reinforcing in the columns which resulted in sudden failure once concrete strain limits were reached.
    - The large proportion of cover concrete, which would have substantially reduced the capacity of columns after crushing and spalling.
    - Significantly lower than expected concrete strength in some of the critical columns.
    - The effects of vertical earthquake accelerations, probably increasing the axial load demand on the columns and reducing their capacity to sustain drift.
- The lack of sufficient separation between the perimeter columns and the Spandrel Panels which may have reduced the capacity of the columns to sustain the lateral building displacements.
- The plan irregularity of the earthquake-resisting elements which further increased the inter-storey drifts on the east and south faces.
- Increased displacement demands due to diaphragm (slab) separation from the North Core.

- The plan and vertical irregularity produced by the influence of the masonry walls on the west face up to Level 4 which further amplified the torsional response and displacement demand.
- The limited robustness (tying together of the building) and redundancy (alternative load path) which meant that the collapse was rapid and extensive.

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

11 RECOMMENDATIONS

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

Irregular Structures

Geometrically irregular structures may not perform as well as structural analyses indicate. There is a need to review the way in which structural irregularities are dealt with in design standards and methods.

Non-ductile Columns

Buildings designed before NZS 3101:1995, and especially those designed prior to NZS 4203:1992 (which increased the design drift demand), with non-ductile gravity columns may be unacceptably vulnerable. They should be checked and a retrospective retrofit programme considered.

Pre-cast Concrete Panels and Masonry Infill Walls

Existing buildings with part-height pre-cast concrete panels (or similar elements) between columns may be at risk if separation gaps are not sufficient and maintained. Such buildings should be identified and remedial action taken.

Diaphragm Connections

Buildings with connections between floor slabs and shear walls (diaphragm connections) designed to the provisions of Loadings Standard NZ 4203 prior to 1992 may be at risk. Further investigation into the design of connections between floor slabs and structural walls is needed.

Design and Construction Quality

There is a need for improved confidence in design and construction quality. Measures need to be implemented which achieve this. Design and Construction Features Reports should be introduced and made mandatory. Designers must have an appropriate level of involvement in construction monitoring. There should be a focus on concrete mix designs, in-situ concrete test strengths, construction joint preparation and seismic gap achievement.

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

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

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