CTV BUILDING COLLAPSE INVESTIGATION FOR CEBPTI PANEL

FOR DEPARTMENT OF BUILDING AND HOUSING

11TH AUGUST

Hyland
FATIGUE+EARTHQUAKE ENGINEERING

STRUCTURE SMITH
Consulting Engineers

REPORT PREPARED FOR:
DEPARTMENT OF BUILDING AND HOUSING

Report Title: CTV Building Collapse Investigation

Report Number: DBH 110628

REPORT PREPARED BY:

DR CLARK HYLAND

PhD, BE(Civil), BCom, CWE-NZIW, CPEng

Hyland Fatigue + Earthquake Engineering
PO Box 97282, Manukau, AUCKLAND 2241
Phone: (09) 262 0203
Mobile: 021 348 352
Email: clark@fatiqueandfracture.com
www.fatigueandfracture.com

ASHLEY SMITH

ME, MIPENZ, CPEng, IntPE(NZ)

STRUCTURESMITH Consulting Engineers
Level 1, 8 Railway Street, Newmarket
PO Box 275-2 Epsom, AUCKLAND 1344
Phone: (09) 377 9739
Mobile: 7 230 5153
Email: ashley@structuresmith.co.nz

Report Issue Date: 15th August 2011

Report Issue Status: Draft

Limitations and Applicability of Report:

This report has been prepared for the benefit of the Department of Building and Housing for the particular objectives described and within the stated scope. No liability is accepted by the company preparing this report or any employee or sub-consultant of these companies with respect to its use for any other purpose or by any other person or organisation.

This report has not been prepared for the purpose of assigning blame, fault or liability.

This disclaimer shall apply notwithstanding that the report may be made available to other persons to fulfil a legal requirement.

REPORT PREPARED FOR:
DEPARTMENT OF BUILDING AND HOUSING

1. INTRODUCTION

Hyland Fatigue and Earthquake Engineering, together with StructureSmith Ltd, are the consultant team reporting to the Department of Building and Housing (DBH) on the reasons why the CTV building collapsed during the earthquake of 22 February 2011.

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.

The terms of reference for our investigation are published on the DBH website at http://www.dbh.govt.nz/canterbury-earthquake-tor-technical-investigation

The investigation has made use of records of building design and construction, and 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 earthquake.

16 key witnesses have provided feedback on the collapse of the building – from the interior of the building and from without. The comprehensive investigation has provided insights that are summarised at the end of this document.

"…There's analytical studies with computer modelling. There's witness interviews to get multiple perspectives on what was seen. We've looked at the history of the building, how it was originally built and the alterations done to it. We've had material testing done to see the strength of the materials and the reinforcing and concrete. From this a convergence has arrived that allows us to understand more of what happened here – the physical explanations."

Clark Hyland

Note: In this report, the identity of the people who have helped us with this investigation has been protected. We appreciate their brave contribution, particularly as it meant them having to relive harrowing experiences. The full transcripts of these interviews are available if required.

2. EXECUTIVE SUMMARY

A. CONVERGENCE OF EVIDENCE

This investigation has drawn on all the varying forms of evidence available to build as accurate picture as possible of the cause of the collapse of the CTV Building in the after-shock of 22nd February, 2011.

Convergence of the evidence has helped guide the investigative process.

B. EYE WITNESS REPORTS

Over 16 people were interviewed as eye-witnesses of the collapse either from inside the building or from various locations around the building. The engineering analyses have drawn on these observations in exploring collapse mechanisms.

C. COLLAPSE SEQUENCE

Collapse appears to have initiated with the failure of a diaphragm drag bar at Level 4 on the Line E wall of the Lift Core. It then progressed to hinging of the perimeter columns that became restrained by the concrete precast panels that were installed between them, and following onto overload of the interior columns and collapse.

Further non-linear analyses are being undertaken to refine the assessment of the collapse initiation and development sequence.

D. BASIS OF DESIGN

The structure was analysed using the 3-D modal response spectrum analysis method, and designed for fully ductile response using the provisions of the New Zealand Loadings Standard NZS 4203:1984. This standard referenced the New Zealand Concrete Structures Standard NZSS3101:1982 and other relevant standards of the time for materials specific analysis and design criteria.

The primary seismic resisting system was designated as the lift and stair core walls at the north end of the building, in conjunction with the coupled shear wall on the south Line 1or Cashell Street face.

From review of the original design calculations and drawings, all the reinforced concrete beams and columns, and the masonry in-fill wall on the west side of the building on Line A appear to have been considered as Group 2 secondary elements under the provisions of NZS 3101:1982 clause 3.5.14.3 (a).:

> "Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $v\Delta$, specified in NZS 4203, and the assumptions of elastic behaviour."

However where the secondary elements are unable to maintain elastic behaviour under those imposed deformations the secondary elements were required under the provisions of NZS 3101:1982 clause 3.5.14.3 (b).:

> "Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $v\Delta$."

The $v\Delta$ deformation specified in NZS 4203:1984 were those derived from the response spectra analysis using the K/SM=2.75 spectra.

Analysis of the documented design undertaken as part of this investigation found that many of the columns were not able to maintain elastic behaviour under those specified deformations (Figure 91and Figure 92), and had not been designed to meet the additional seismic requirements of the concrete structures standard.

The additional seismic requirements included among other things:

- Increased confining reinforcement to the columns to prevent sudden compression failure.
- Continuity of beam reinforcement through and shear reinforcement in beam-column joints to prevent beams detaching from columns.

E. LEVEL OF EARTHQUAKE AND AFTER-SHOCK DEMANDS

The levels of loading demands placed on the structure in the September earthquake, and the December and February after-shocks, implied by the seismic records obtained at recording stations close to the site are not credible without some form of scaling or further calibration.

A comparison of the SM=0.8 design spectra relative to the records in each event (Figure 4 to Figure 6), indicates that major structural damage and even collapse should have occurred in all the events.

However what is considered in engineering terms to be minor or serviceability level damage was documented as occurring in the initial 4th September, 2010 earthquake.

Records are less clear on the damage after the 26th December 2010 after-shock however it appears that no significant additional structural damage occurred.

In both events there was significant disruption to office use due to furniture and partitioning damage.

The relative intensity of the events is also difficult to determine as the spectra for each event vary by up to a factor of 8 at the various recording sites, at the important 1second period relevant to the natural response of the CTV building.

An expert review of the earthquake records is required in conjunction with deep bore micro-tremor calibration of the recording sites, and the CTV building site before reliable use of the records can be made in terms of what demand was imposed by each event on the building. The approach recently taken by Japanese and Indonesian researchers following the earthquake in Padang in 2009 may be worth considering (Pradano, Goto et al. 2011).

As a consequence the performance of the building has been derived at this stage relative to the NZS 4203:1984 SM=0.8 and K/SM=2.75 design spectra.

F. EFFECTS OF SEPTEMBER AND DECEMBER EVENTS ON STRUCTURE

The September and December events were found to have only caused superficial structural damage. Though cracking damage to the perimeter columns foreshadowed the problem of their limited ability to deform without engaging in a damaging way with the p[recast spandrel panels installed between them.

Cracking to the perimeter columns would also have had an effect on the lateral stiffness of the building and the feeling that it was more flexible.

G. EFFECT OF DEMOLITION WORK IN ADJACENT SITE

The demolition that started immediately after the 4th September, 2010 earthquake caused significant distress to the occupants of the CTV Building. The building was found to be susceptible to heel drop vibration in open plan areas. The irregular thumping and vibrations caused by demolition are likely to have enhanced the feeling of liveliness. However the demolition work is not considered to have had any damaging effect on the building's structure.

H. EFFECT OF ALTERATIONS SINCE ORIGINAL CONSTRUCTION

Alterations to the building since construction are not considered to be significant in terms of its performance in eth earthquake. However it is noted that coring of the floor occurred at the locations where the slab pulled away from the lift core.

The attachment of drag bars between the lift core and the slab at Level 4, 5 and 6 are significant and appear to be material to the collapse.

I. CONNECTION OF FLOOR INTO LIFT CORE

The level 2 and 3 floors were not connected into wing walls D and E of the lift and stair core. This changed the response of the structure putting more demands on the Line 1 and 5 shear walls, however 3D dynamic response spectrum analysis of the structure with the floor diaphragm disconnected at these locations found that the structure could accommodate this. However this left the diaphragm connection at level 4 into the Line E shear wall with the greatest demand to capacity ratio. This was similar to that for the Line 1 wall for attainment of its nominal yield capacity.

It therefore appears that that the diaphragm connection in the slab itself from the Line E wall became overloaded shortly after the Line 1 wall reached its nominal yield capacity.

This loss of connection to the Line wall was found to then have increased the shear demand in the Level 4 to 5 portion of the Line 1 wall.

The demand vs capacity ratios calculated for the connections of the slab diaphragm into the stair and lift elevator core walls from Level 2 to Level 6 were consistent with the condition of the slab and drag bar remnants discussed in the Site Examination and Materials Tests report (Hyland 2011) and the collapse debris discussed in this report.

The profiled steel decking was found to have provided significant additional diaphragm capacity at Line 4 between walls C and CD. This was confirmed through observation on site post-collapse that it had been sufficiently anchored into the support at Line 4 that it was able to develop its tensile capacity at the location of maximum diaphragm demand. This was away from the zone of peak flexural tensile demand that occurred near the midspan of the slab.

Together this confirms that failure of the diaphragm as it connected to the lift and stair core walls did not initiate the collapse of the CTV building. The diaphragm however did pull away from the core walls as the floors were pulled down with collapse of the columns on Lines 2 and 3. The diaphragm fractured along the ends of the saddle bars that were placed over the Line 4 beams. This was the location of the weakest portion of the slab closest to Line 4.

The diaphragm connections to the lift core walls were governed by 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 Line 1 and the lift core walls were designed and detailed as fully ductile, the Line 1 wall was able to yield and displace inelastically well before the lift core walls.

J. LINE A IN-FILL MASONRY WALL

The Line A wall significantly increased the susceptibility of the Line 1 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, including the effect of the Line A masonry in-fill wall on the primary seismic resisting structure of the building. This was because the it appears care had been taken to detail the masonry infill with a level of inelastic resilience so that it could deform in accordance with the provisions for Group 2 secondary members.

In doing so effect the masonry in-fill block work on the overall behaviour of the structure did not need to be specifically considered..

This provision of the standard is not in accordance with sound earthquake engineering principles.

K. DUCTILE DESIGN OF TORSIONALLY IRREGULAR STRUCTURES

It can be seen that wall on Line 1 would be expected to yield and then deform inelastically well before the level 5 wall. Given the large difference in the capacity ratios its difficult to conceive that the Line 5 wall would ever have been able to becoming a yielding element in the structure.

It therefore raises questions about the adequacy of the provisions in the design standards for the ductile design of torsionally irregular structures. In this case the core shear walls were detailed for ductile performance but in fact responded as elastic elements working in conjunction with a fully ductile perimeter wall on Line 1.

There are no specific provisions in the design standards preventing this occurring. For buildings designed with primary structural systems acting in different directions the rule is that here should only be 1 change in the level of ductility between the two systems so that there is a level of inelastic compatibility between them in the event of an design earthquake occurring. In this case the comparative ratio of ductile response between Line 5 and Line 1 wall is equivalent to the ratio of their respective demand ratios or 5.9.

A requirement to ensure that the ratio between the relative flexural demand/ capacity ratios of any elements of a primary seismic resisting system in a torsionally irregular structure is limited to a value of 1.20 would prevent this level of irregularity in the ductile response occurring.

L. PRE-CAST CONCRETE SPANDREL PANELS

The development of a column collapse mechanism in the level 4 perimeter columns initiated the collapse of the building. Structural hinging damage at the top, bottom and mid-height of perimeter columns formed a mechanism in them that meant those columns could not sustain any vertical actions (Figure 18).

An explanation for the formation of mid-height perimeter column hinges is the restraining effect of the pre-cast concrete spandrel panels that were installed either side of the columns with a nominal 10 mm gap to the column faces. This would apply to the columns on Grid F and some on Line 1 and 4 where the pre-cast panels occurred. Columns at F/3 and F/4 were restrained on both sides by spandrel panels.

The actual as-built gap either side of the columns is expected to have ranged between 0 and 22 mm based on the guidelines for assessing combined construction tolerances BS 5606:1990 (Figure 14). 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.

M. CONCRETE STRENGTH

Testing of the concrete in the structure found it to be weaker than expected for its age. This is considered to be material to the collapse.

3. BUILDING DESCRIPTION

AS AT 22 FEBRUARY PRIOR TO AFTERSHOCK

Figure 1 - Canterbury Television Building in 2004 (Photo credits: Phillip Pearson, derivative work: Schwede66)

A. 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.

The gross floor dimensions were approximately 31m x 22.5m. The building had a lightweight roof supported on steel framing 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 columns grid was typically 7.5 x 7.0m.

The foundations comprised shallow strip and pad footings and foundation beams.

The primary earthquake resisting structure as defined by the standards of the time was provided mainly by 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 lightweight steel escape stair. The lower doorway opening had been partially in-filled with reinforced masonry to window sill height.

The secondary structure not considered by the design standards to contribute directly to the earthquake resistance of the building consisted of moment resisting frames of precast log beams supported on 400 diameter and 400 x 300 rectangular reinforced concrete columns. These appear to have been detailed as "non-seismic".

The key features of the structure that were relevant for 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.
- Extensive voids in the floor adjacent to the north side shear wall
- No or inadequate connection of the floor diaphragm to walls D and E at Levels 2 and 3.
- The presence of a column directly under the core wall at the north-east corner, attracting axial loads 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, the widely spaced ties in the 400 x 300mm rectangular columns and the lack of any special ties 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.

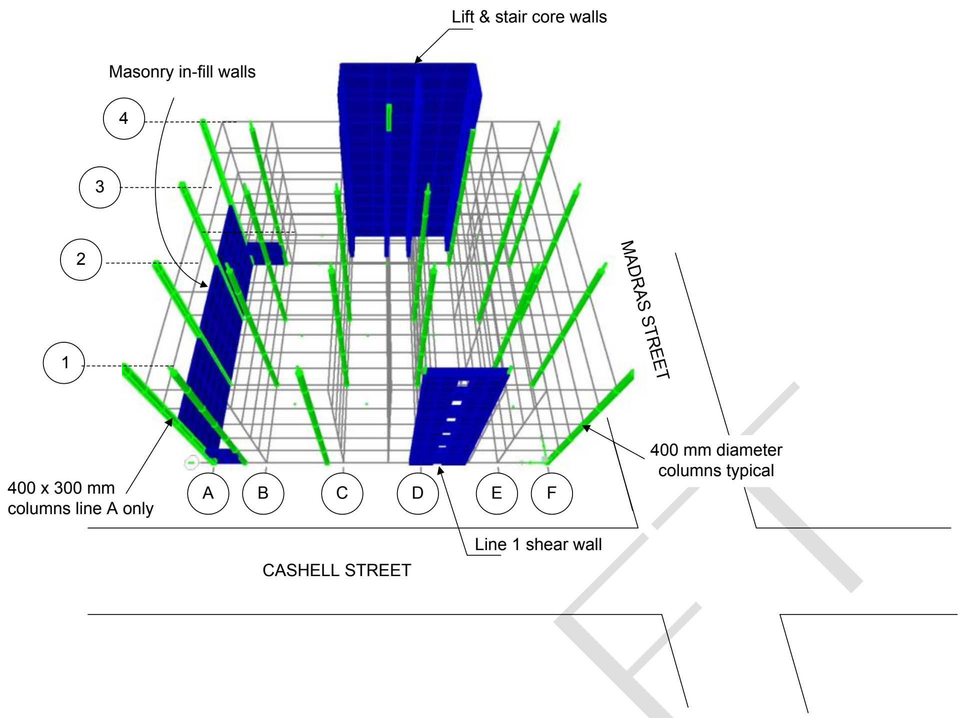


Figure 2 Building orientation and grid lines used in the report

B. PROCUREMENT PROCESS

The developer gained building permit approval in September 1986.

C. SITE INVESTIGATIONS (SOILS, SEISMOLOGY)

The original site investigation report dated 18 June 1986 and was reviewed by geotechnical engineers Tonkin and Taylor Ltd and found to be consistent with normal practice in Christchurch at the time.

They were able to recommend lower bound, most likely and upper bound soil spring stiffness values for input into the computer models for seismic analysis.

Liquefaction is not considered to have been be a significant factor for this building. There was a report of some water or liquefaction on the west side of empty site adjacent at the west side after the earthquake. No liquefaction was observed adjacent to the building itself (Figure 39) or in the streets around the site at the south and east sides.

Pits dug at the north face of the lift and stair core walls found no evidence of soft soli, settlement, uplift or liquefaction (Hyland 2011).

D. DESIGN, DRAWINGS AND SPECIFICATIONS

The building consent drawings and the Christchurch City Council property file were made available and reviewed.

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

Police, USAR and witness photos and TVNZ news video files were received showing the collapsed structure and the deconstruction process.

E. VARIATIONS DURING CONSTRUCTION

One drawing in the set of drawings provided by the structural engineer, drawing S26 had been amended to show a block wall in place of the consented precast panel wall at the ground floor entry off Madras St. This was not a significant structural change and would not have affected the seismic response if it had been built as documented.

F. POST-OCCUPANCY ALTERATIONS

A consulting engineer 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.

Post-earthquake site examination found some structural steel angle drag members bolted into the wall fins and the floor slab at the three upper levels but not the two lower floors Level 2 and Level 3.

Lundia Storage was added to an area during an office fit out.

Another consulting engineer designed a penetration into the Level 2 floor slab (ground is Level 1) for a new internal stairway at the south-east entry for CTV.

The building changed use from its original commercial office use to an education facility and studios resulting in increased design live loads according to the design Code.

Interviews with tenants have confirmed that the following tenancies within the building at the time of the February 22 aftershock.

- Level 6 – Office (Relationship Services) in west half. The east side was empty.
- Level 5 – Medical Clinic
- Level 4 - Kings Education
- Level 3 – Vacant office. Had been a Travel School but they had moved out so was vacant. Some fit out work was being done at the time.
- Levels 1 and 2 – Television and Radio studios (Canterbury Television)

G. SETTLEMENT AND CORE VERTICALITY

A survey of the remaining slab and exposed foundation beams found no obvious settlement had occurred. The slab in parts however had been damaged particularly on the west side of the building.

The elevator core had northward out-of vertical set of about 90 mm at its east end and 70 mm at its west end. This out-of vertical set is concluded to have been due to construction tolerances as no obvious damage to the foundation beams was found when inspected.

Geotechnical engineers Tonkin and Taylor advised that there was a layer at depth below the site that could have been subject to liquefaction that could have caused some slight settlement.

No liquefaction can be observed in photos in the immediate vicinity of the building after the collapse (Figure 39). However liquefaction was reported to have occurred near the neighbouring building near the Cashell Street roadway, on the adjacent site.

No evidence of liquefaction was found when a pit was excavated adjacent to the north side of the lift core footings.

The levels survey could not identify any obvious changes in level that could be attributed to ground movement that could not otherwise be attributed to construction practice and accepted tolerances.

4. EARTHQUAKE AND AFTERSHOCK RECORDS

A. 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 four 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)

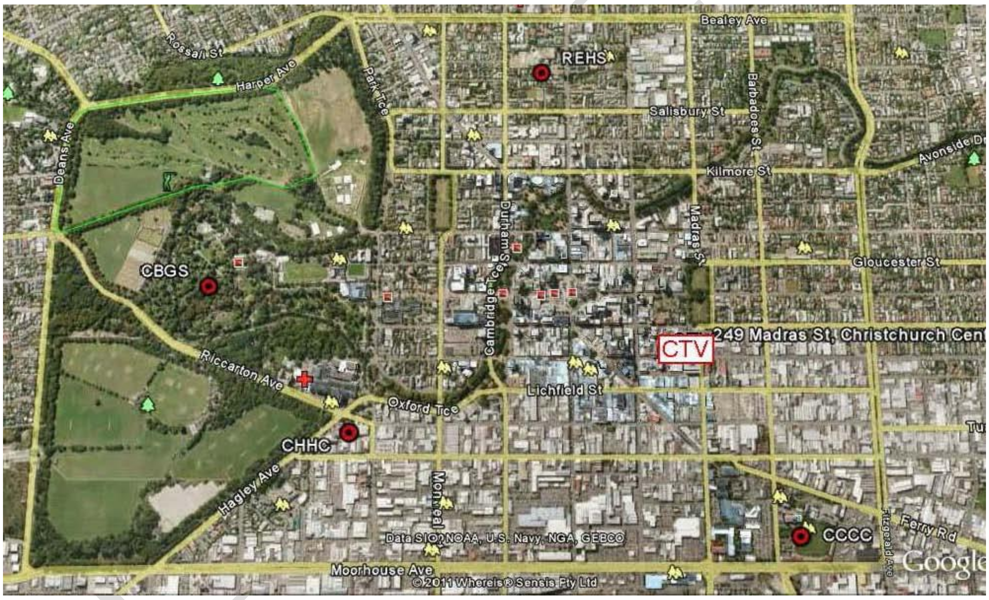


Figure 3 - Locations of Geonet Strong motion Recorders relative to CTV Site

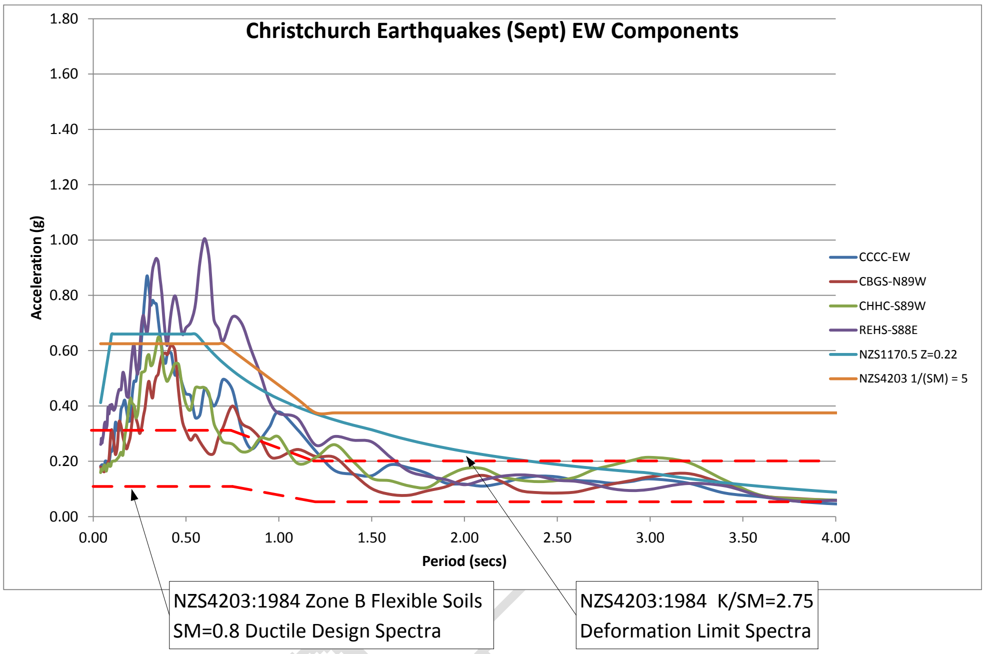
For each earthquake, or aftershock the four strong motion records have each been converted into a 5% damped response spectrum using the SAP computer program and are shown plotted in the following sections alongside the Code level elastic response (SM=5, μ=1) spectra according to NZS4203:1984 and NZS1170.5:2003 for comparison.

Except for the CCCC site, the axes of the instruments are very close to N-S and E-W, as are the axes of the CTV building. The record from the CCCC site has therefore been realigned into N-S and E-W components to enable direct comparison.

At this stage we do not have sufficient detailed information about the ground conditions at the four recording stations, or detailed information about the ground conditions between the recorders and the CTV site, to enable an accurate conversion of the earthquake records into an equivalent record for the CTV site. However, since the four stations effectively surround the CTV site on three sides at fairly close proximity then the records are helpful in demonstrating the general level of shaking that would have occurred at the site.

B. RESPONSE SPECTRA FROM 4TH SEPTEMBER 2010 EARTHQUAKE

Figure 4 shows response spectra plots of East-West and North-South components of the 4 September 2010 earthquake, from the four nearby recording stations. The NZS4203:1984 SM=0.8 ductile response spectra for Zone B and flexible soils, used to design the building is shown. In addition the fully elastic response spectra are shown from NZS4203:1984 SM=5, and the current NZS1170.5:2003 μ=1.



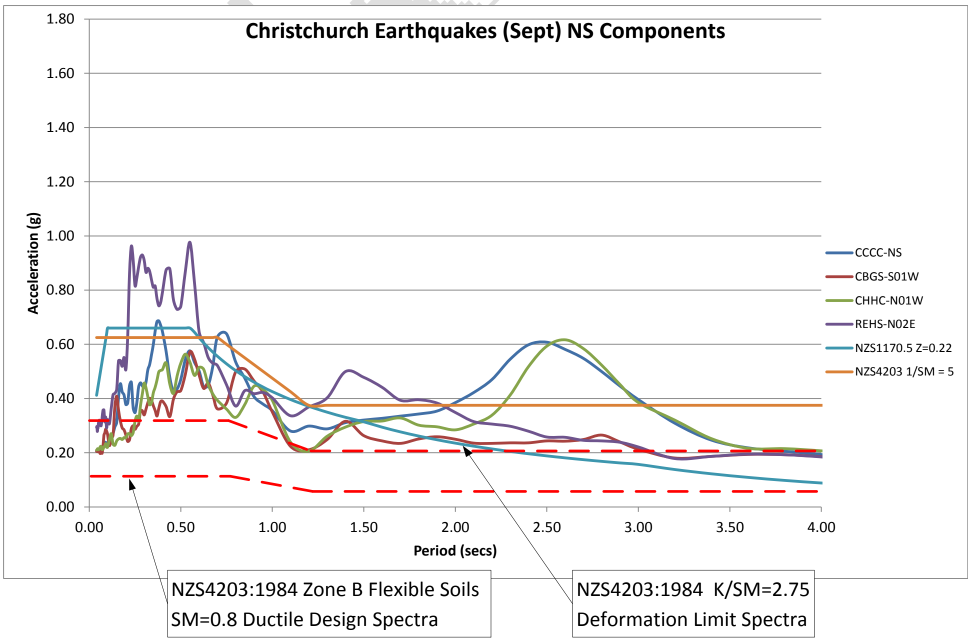
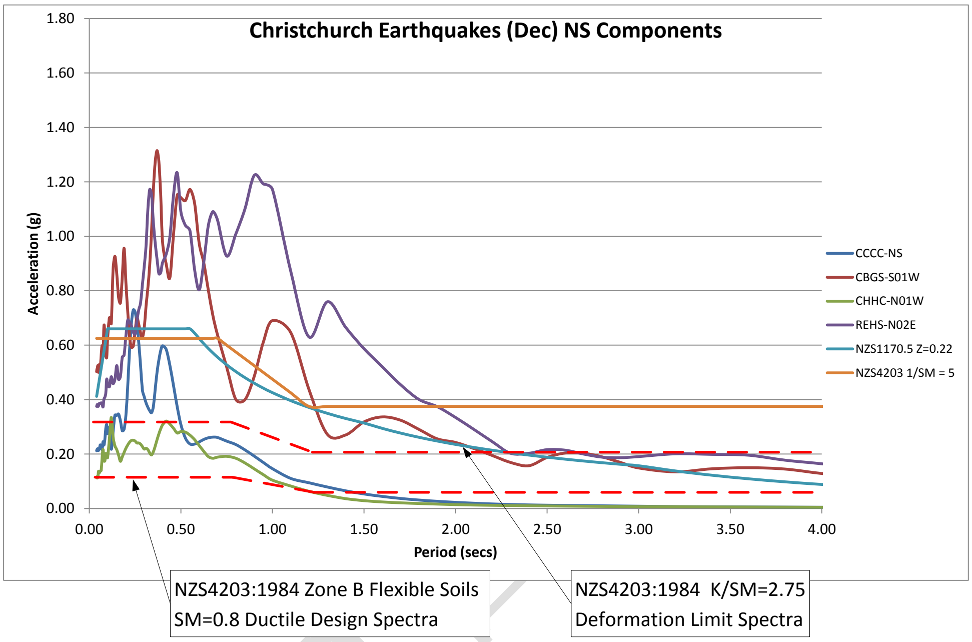


Figure 4 Response spectra for the earthquake of 4th September, 2010. The ductile and fully elastic response spectra form NZS 4203:1984 and NZS 1170.5:2003 are superimposed.

C. RESPONSE SPECTRA FROM 26TH DECEMBER 2010 AFTER-SHOCK

Figure 5 shows response spectra plots from the four nearby recording stations for the 26th December2010 after-shock and the relevant design spectra.



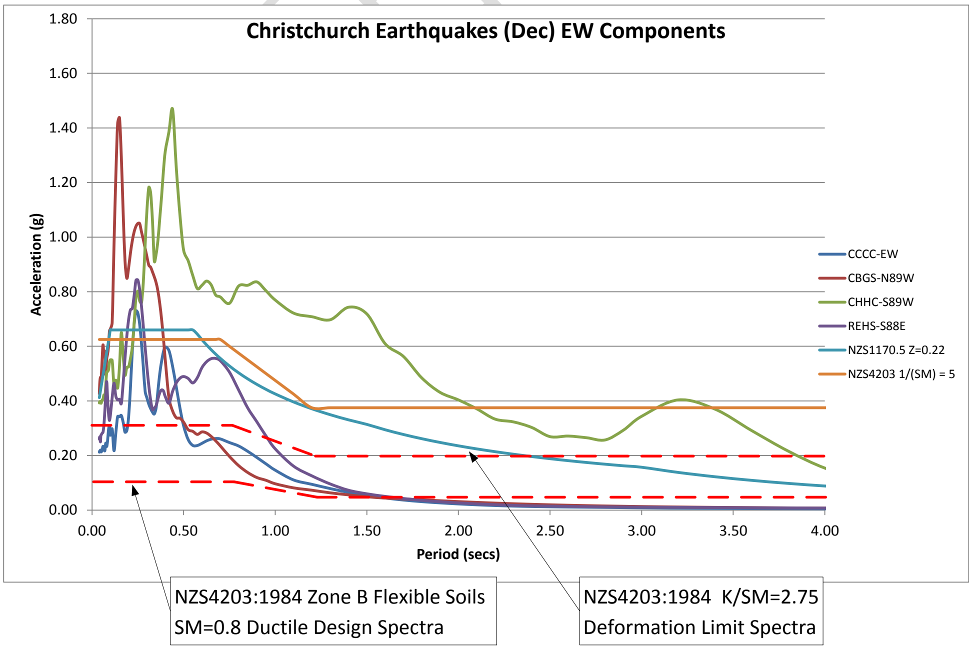
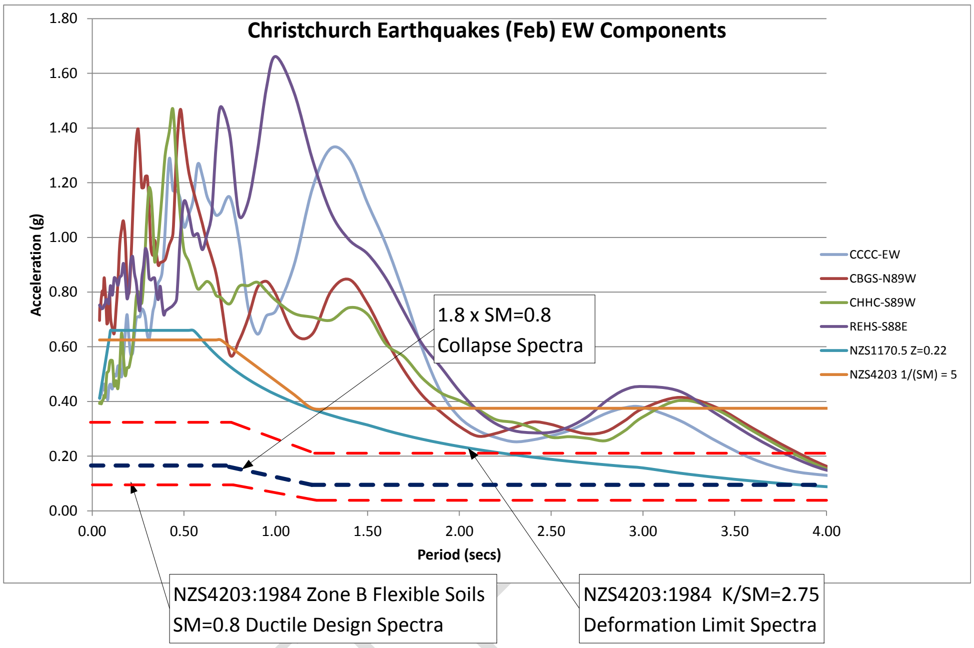


Figure 5 Response spectra for the earthquake of 26th December, 2010. The ductile and fully elastic response spectra form NZS 4203:1984 and NZS 1170.5:2003 are superimposed.

D. RESPONSE SPECTRA FROM 22ND FEBRUARY 2011 AFTERSHOCK

Figure 6 shows response spectra plots from the four nearby recording stations for the 22nd February 2011 after-shock and the relevant design spectra. The 1.8 x SM=0.8 scaled design spectra at which the collapse is calculated to have initiated is shown as the collapse spectra.



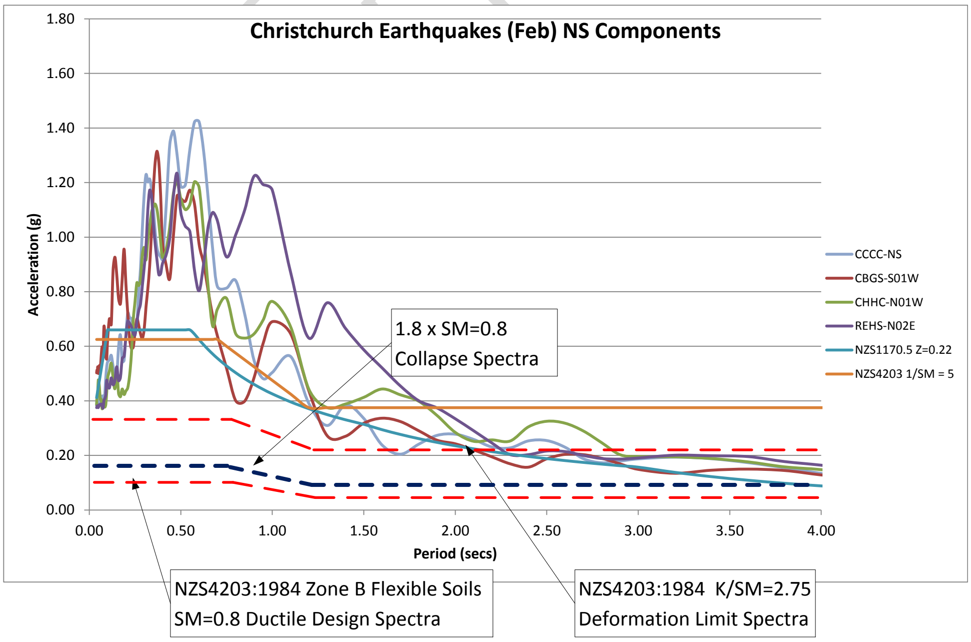


Figure 6 Response spectra for the after-shock of 22nd February, 2011. The ductile and fully elastic response spectra form NZS 4203:1984 and NZS 1170.5:2003 are superimposed.

5. EFFECTS OF 4TH SEPTEMBER 2010 EARTHQUAKE

The building gained a green placard in the rapid assessment after the earthquake. It was subsequently re-confirmed as a green placard.

An engineering report dated 6 October 2010 was prepared by a consulting engineer who had been engaged by the building owner to report on damage following the September earthquake.

The report identified minor damage in several areas, and included photographs. Work was in the process of being carried out to repair some of that damage, including epoxy grouting up of cracks in concrete columns and beams (Figure 7 to Figure 11).

Tenants interviewed described the building as feeling more flexible after this event. However this also coincided with demolition of the neighbouring building commencing immediately after this event and continued until the week before the February aftershock. Shudders were often felt through the building as the concrete structure was demolished with wrecking balls and concrete pokers.

After the 4th September earthquake no major structural damage was observed. However there was damage to partitions, glass and filing cabinets were toppled, and some cracking to the column adjacent to the elevator core. The damage was therefore consistent with serviceability limit state performance.

Figure 7 Level 6 400 mm diameter columns (Left to right) a) Column C18 outside lift; b) Column C3 on Line 1/A-B with hairline horizontal cracking

Figure 8 (Top to Bottom) a) hairline cracking in slab at Line 1 shear wall door at Level 4; b) Dmage to plaster on infill blockwall at Level 1Line 4

Figure 9 beam column joint on Line 2 or 3 at Level 1

Figure 10 damage to wall Linings

Figure 11 Contents damage on Level 2

6. EFFECTS OF DEMOLITION OF NEIGHBOURING BUILDING

Demolition of a reinforced concrete building and preparation of the site for a car p park, commenced on the adjacent site immediately after the 4th September, 2010 earthquake.

Work on the site continued until the after-shock and collapse of the CTV building on 22nd February, 2011 (Figure 77).

Heavy machinery with pneumatic pokers and pincers, and drop hammers, were used to break it up (Figure 12). This caused ongoing and disturbing vibrations to occupants in the CTV building.

It is considered however that no structural damage was caused by the demolition sufficient to affect the earthquake resistance of the CTV Building.

Figure 12 Heavy machinery demolishing a reinforced concrete building adjacent to the CTV Building after the 4th September, 2010 earthquake.

7. EFFECTS OF 26TH DECEMBER AFTER-SHOCK

A 'Christchurch EQ Rapid Assessment Form – Level 1', and a 'USAR Damaged Building Reconnaissance Report' dated 27 December 2010 were obtained from the CCC files.

The first page Rapid Assessment form identified a broken pane of glass that might fall onto a balcony. The second page 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 on Level 6. The damage 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 occurred in the offices further into the building (Figure 13).

No obvious damage 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 C18 by the lifts had visible wavy cracking which it had after the 4th September event.

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

A student interviewed from Level 4 also advised that a person thought to be an engineer inspected the building a week before the 22nd February aftershock. However the name or company that that person worked for is unknown. No damage was obvious to the student at the time.

It is concluded on the basis of the above that there was no significant structural damage to the building after the 26th December aftershock, though damage to glass, partitions and hairline cracking to some structural concrete.

Figure 13 Damage after 26th December , 2010 after-shock on Level 6 of CTV Building (clockwise from top left) a) Cabinet door had opened but hadn't fallen over though npot 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 Cashell 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).

8. DESCRIPTION OF COLLAPSE ON 22ND FEBRUARY, 2011

The six-story CTV Building located at 249 Madras Street, on the Cashell Street corner collapsed in the 22February 2011 Christchurch earthquake. CTV's main studios were destroyed and the building's lift cavity, the main part of the structure left upright, caught fire. Of the 166 confirmed dead by 12 March 2011, 94 were recovered from the CTV building. Many of the dead and missing were faculty and students at the located on Level 4 of the CTV building.

This section describes the collapse sequence using a combination of numerical analysis, materials testing, witness statements, review of photos of the debris layout prior to being removed from site, and examination of the failure condition of structural remnants at the site and at the designated secure area at the Burwood Landfill.

A summary of the witness statements is in Appendix B,

A description of the collapsed structure and structural remnants is made in Appendix D and in the Site Examination and Materials Tests Report.

Results of the numerical analyses are in Appendix F

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

Observations and comments are recorded about each item in the general text and in captions in the photos.

A. STRUCTURAL COLLAPSE SEQUENCE

i. Level 4 Slab Diaphragm Drag Bar Failure Initiation

Following failure of the attachment of a drag bar to the Line E wall of the lift core at Level 4 collapse initiated in the Line 1 and F perimeter columns at Level 4 and progressed into overload of the Line 2 and 3 columns at Level 1, at the Madras Street end of the building.

At actions equivalent to approximately 1.8 x the New Zealand Loadings Standard NZS 4203:1984 SM=0.8 design spectra loads the fixings attaching the drag bar connecting the slab to the Wall on Line E of the Lift core are expected to have failed.

The resulting increased differential inter-storey drifts between Level 4 and level 5 would then have led to 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.

The perimeter columns lost their load carrying capacity after breaking their backs on the adjacent precast concrete spandrel panels.

Increased inter-storey drifts and shears were found from the 3D structural analysis to occur at this level due to termination of the Line A masonry infill wall at the underside of Level 4. This caused a significant change in the torsional stiffness above that level.

High shear demands on the Line 1 shear wall at this level from the analysis may have also led to slippage on the construction joint of the west panel of the wall and to the diagonal cracking found in the east panel, increasing the inter-storey displacement (refer site and materials report item E4).

An alternative explanation for the E4 damage is that there may have been some degree of dynamic impact damage upon fracture of the Level 4 diaphragm drag bar connection that was resisted by the Line 1 shear wall at that level.

The shear demand on the Line 1 shear wall below Level 4 was found from the 3D structural analysis to be significantly less than that at Level 4. However the soft story effect between L4 and L5 and would have allowed displacements to perimeter columns sufficient for them to become restrained by the precast spandrel panels.

The columns at all levels have been calculated to have been able to maintain axial load capacity on their core concrete after formation of reinforcing yielding controlled flexural hinging if the concrete had properties fitting within the bell curve of 30 MPa aged by 25% at L1 to L3 and 17.5 MPa aged by 25% elsewhere.

The perimeter columns have been calculated to have been able to sustain axial capacity on their core alone if the concrete had properties consistent with 17.5 MPa concrete aged by 25% at all levels.

The internal columns could also sustain capacity as long as the concrete had strength greater than the lower 5% aged strength and displacement demand was less than approx 4.

Observations of column damage on site and at the Burwood landfill showed a number of 400 mm diameter columns with flexural hinging damage at their bases and additional hinging 1350 to 1600 mm above floor level. This is unusual as the columns would have been 2690 mm high between the top of the floor and the underside of the 550 mm deep beams they supported. However the vertical column bars were specified with laps extending 1200 mm above floor level. The end of the laps correspond roughly with the location of the hinging zone.

Hinging top and bottom would have been expected, but not mid-height hinging.

The bottom hinging damage indicates flexural or tension yielding behaviour, as the concrete remaining has horizontal cracking through it and remains uncrushed and is still held in place by the vertical reinforcing steel. The R6 spiral ties surrounding the vertical bars were found to have fractured in Item E33. However it is thought that this was not due to the tie bursting because of hoop stresses induced by the need to confine crushed concrete, but rather by the need to restrain vertical reinforcing steel after it had elongated in tension and then was forced to compress as the flexural hinging developed.

At the mid-height failure zone the concrete had in most cases completely disintegrated, leaving a spear headed form consistent with shear/compression./ flexural hinging. In others a flexural hinge like that found at the base occurred.

ii. Engagement with Precast Spandrel Panels

An explanation for the formation of mid-height column hinges is the effect of the pre-cast concrete spandrel panels that were installed either side of the columns with a nominal 10 mm gap to the column faces (Figure 17). This would apply to the columns on Grid F and some on Line 1 and 4 where the pre-cast panels occurred. Columns at F/3 and F/4 were restrained on both sides by spandrel panels.

The actual as-built gap either side of the columns is expected to have ranged between 0 and 22 mm based on the guidelines for assessing combined construction tolerances BS 5606:1990 (Figure 14). 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: $10\,mm \pm \sqrt{10^2 + 5^2 + 3^2} = 10 + 12\,mm \text{ and } 10 - 10mm$

Some of the columns are expected to therefore have been restrained by the pre-cast panels prior to the earthquake occurring and would not have developed hinging at their bases prior to engagement with the spandrel panels and formation of a mid height hinge. This was seen in the beam-column remnant from Line 4 (Figure 69).

Other columns would have had a greater gap and been able to develop a hinge at the top and bottom of the column before engaging with the spandrel panels and developing a third hinge forming a localised collapse mechanism. This is seen in the perimeter column remnant E33 (Figure 70).

Due to the highly torsional response of the structure the displacements along Grid F are similar to the displacements along Grid 1 in the orthogonal direction (Figure 15).

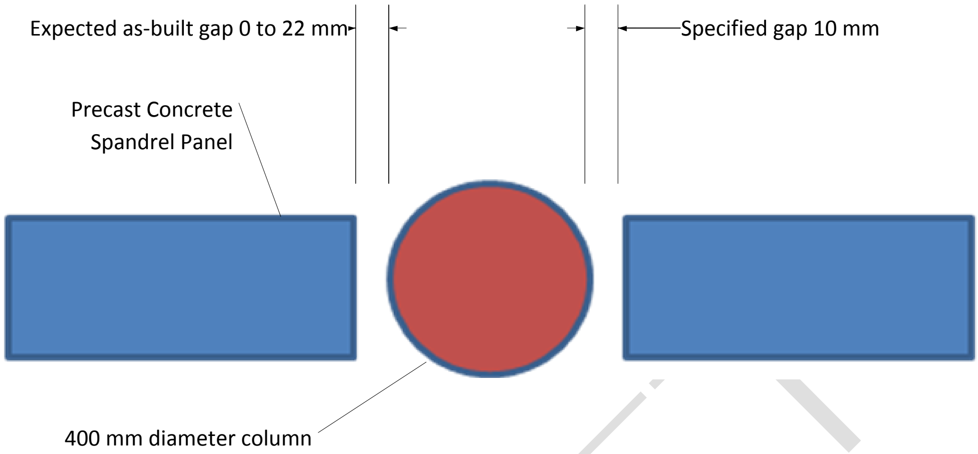


Figure 14 expected as-built gaps between spandrel panels and columns based on BS 5605:1990 combinations of tolerances

iii. Line 1 Shear Wall Inelastic Deformation

The Line 1 wall was found to have developed some level of inelastic flexural response, as the end bars had strain hardened to 3% at the approximately mid-height of the wall between Level 1 and 2. However this level of flexural deformation of the Line 1 shear wall would have been insufficient to achieve the level of inter-storey displacement necessary to bring the Grid F columns into contact with the spandrel panels and then form a mid-height hinge.

The formation of a third hinge in the columns on Grid F is significant, as without the development of a third hinge the columns of the secondary frames while damaged would have been able to maintain axial load carrying ability, and avoid collapse, as nominally pinned columns restrained by the shear walls at Line 1 and 4 and the masonry boundary wall on Grid A up to the underside of Level 4. Once a third hinge developed in the perimeter columns along Grid F a collapse mechanism was developed that would have led to rapid progressive collapse (Figure 18).

No cracking was evident in the west panel which should have borne half of the shear demand on the wall at that level. Below Level 4 the cracking damage became much lighter and strongly uni-directional. Horizontal construction joints above and below the slab levels in the Line 1 shear wall were found to be typically un-roughened and smooth. The 3D analysis showed a significant reduction in the shear in the Line 1 wall below Level 4 due to the masonry wall on Grid A adding significant torsional restraint to the structure below that level.

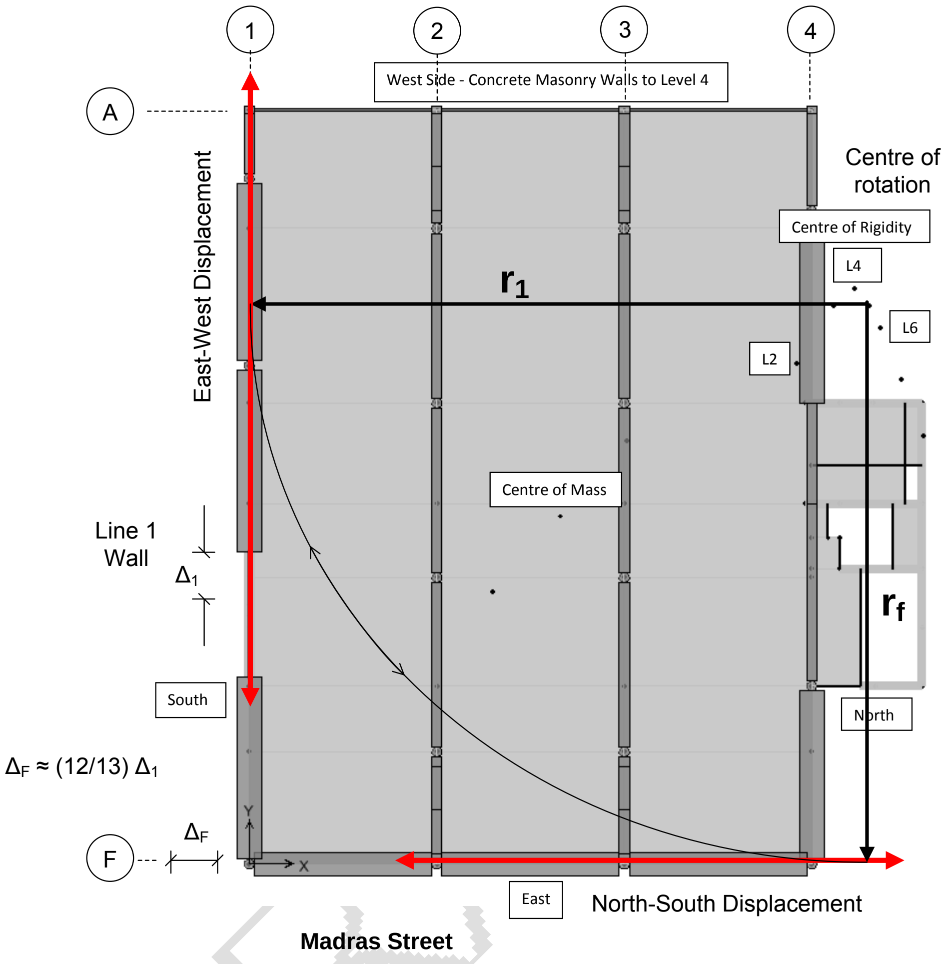


Figure 15 Torsional behaviour of the building increased above Level 4

It is therefore possible that the west panel of the Line 1 shear wall developed slippage on the construction joint above of below the slab shedding the inter-story shearing demand onto the east panel which had a shear capacity of 1159 kN using assessed material properties from testing of fc'=33.5 MPa and fy= 448 MPa for the reinforcing steel with strength reduction factor =1.0. This corresponds to 1.1 time the SM=0.8 NZS4203:1984 design load and less than the 1.8 x SM=0.8 actions at the time of failure of the Level 4 diaphragm tie to Wall E .

The damage to the east panel of the Line 1 shear wall may have further increased the differential inter-story floor displacements between Level 4 and 5 and the resulting damage to the perimeter columns (Figure 16).

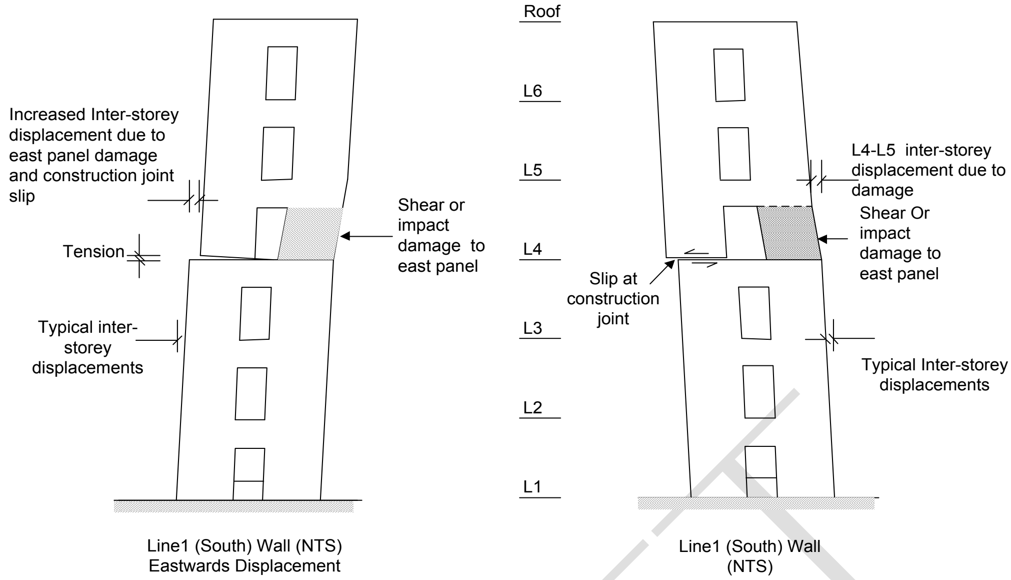


Figure 16 Shear or impact damage on L:ine 1 shear wall at Level 4 to 5

Once columns on Grid F at level 4 began to collapse the collapse would quickly spread west through the structure as the next line of columns between level 4 and 5 gained additional tributary areas to support and collapsed in compression (Figure 19).

As the interior columns on Line 2 and 3 collapsed the slab and beams they supported would have pulled downwards on the Line 1 shear 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 as was observed by eye witnesses.

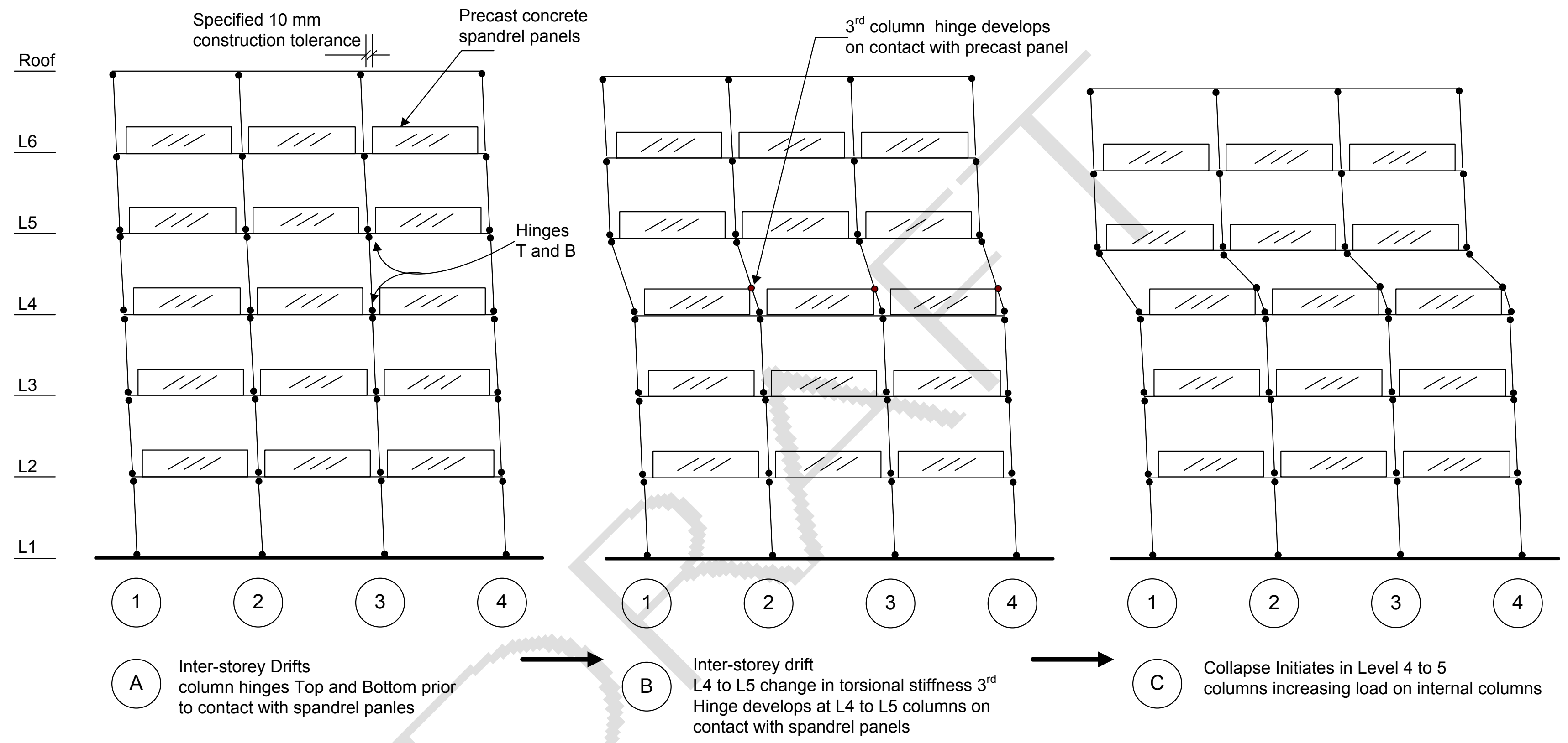


Figure 17 Development of perimeter column collapse mechanism on Madras Street Line F at Level 4

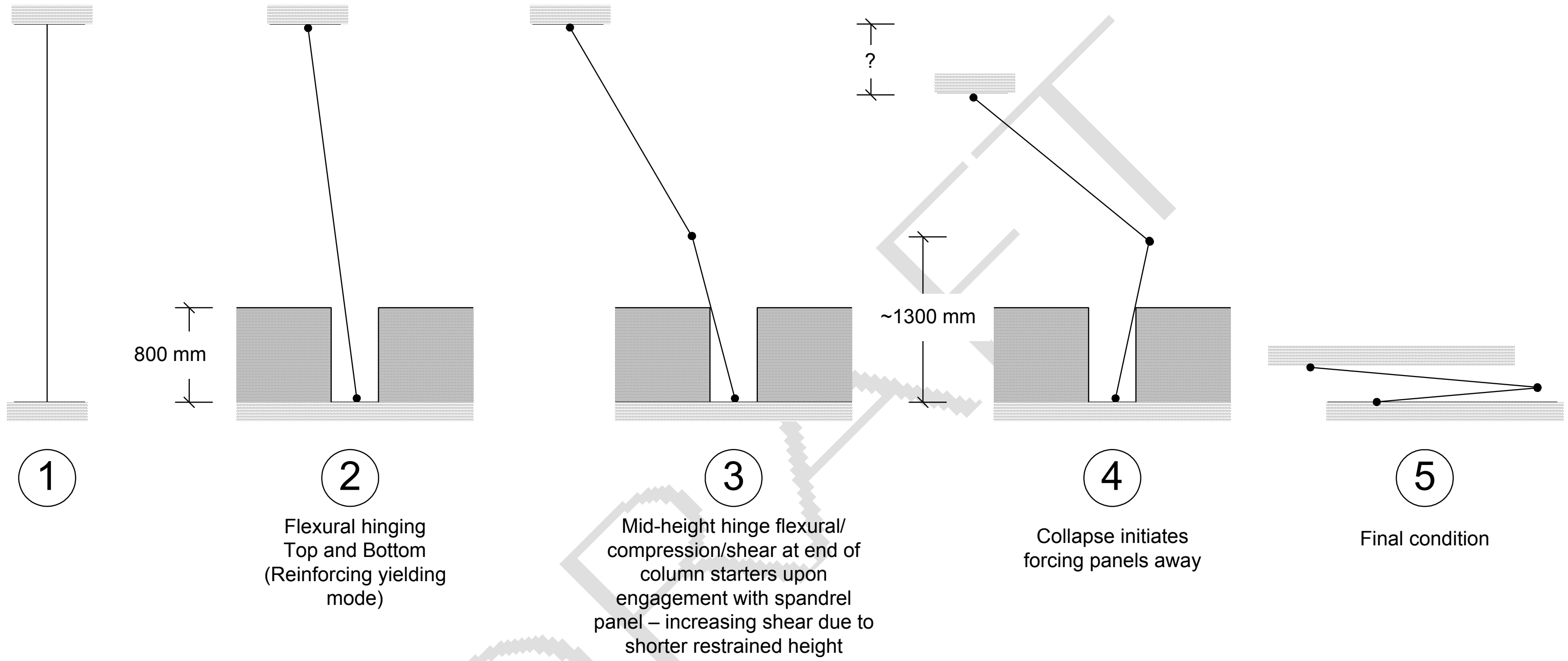


Figure 18 Perimeter column collapse mechanism development from interaction with spandrel panels

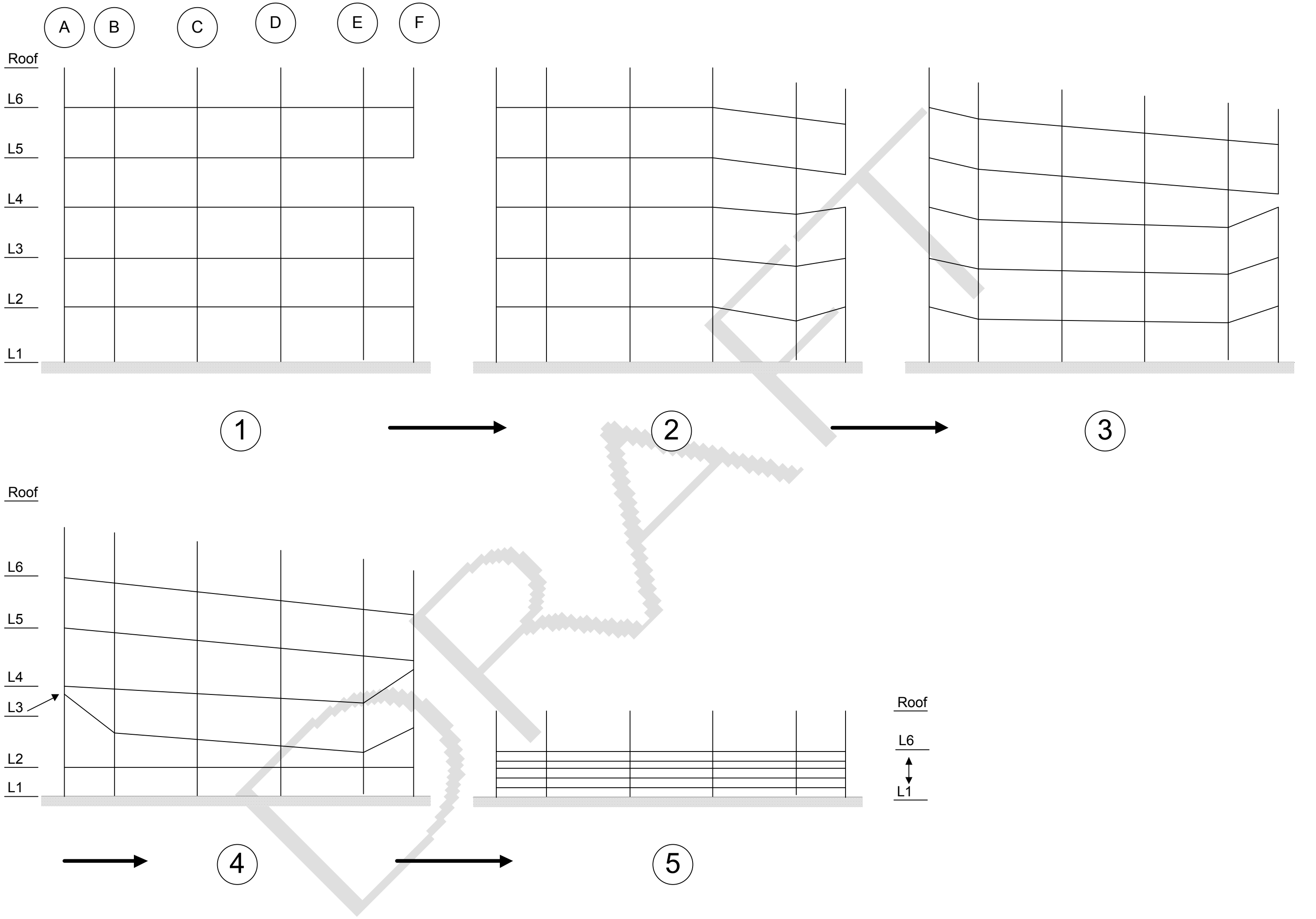


Figure 19 Line 2 collapse development sequence following column collapse initiation on Line F

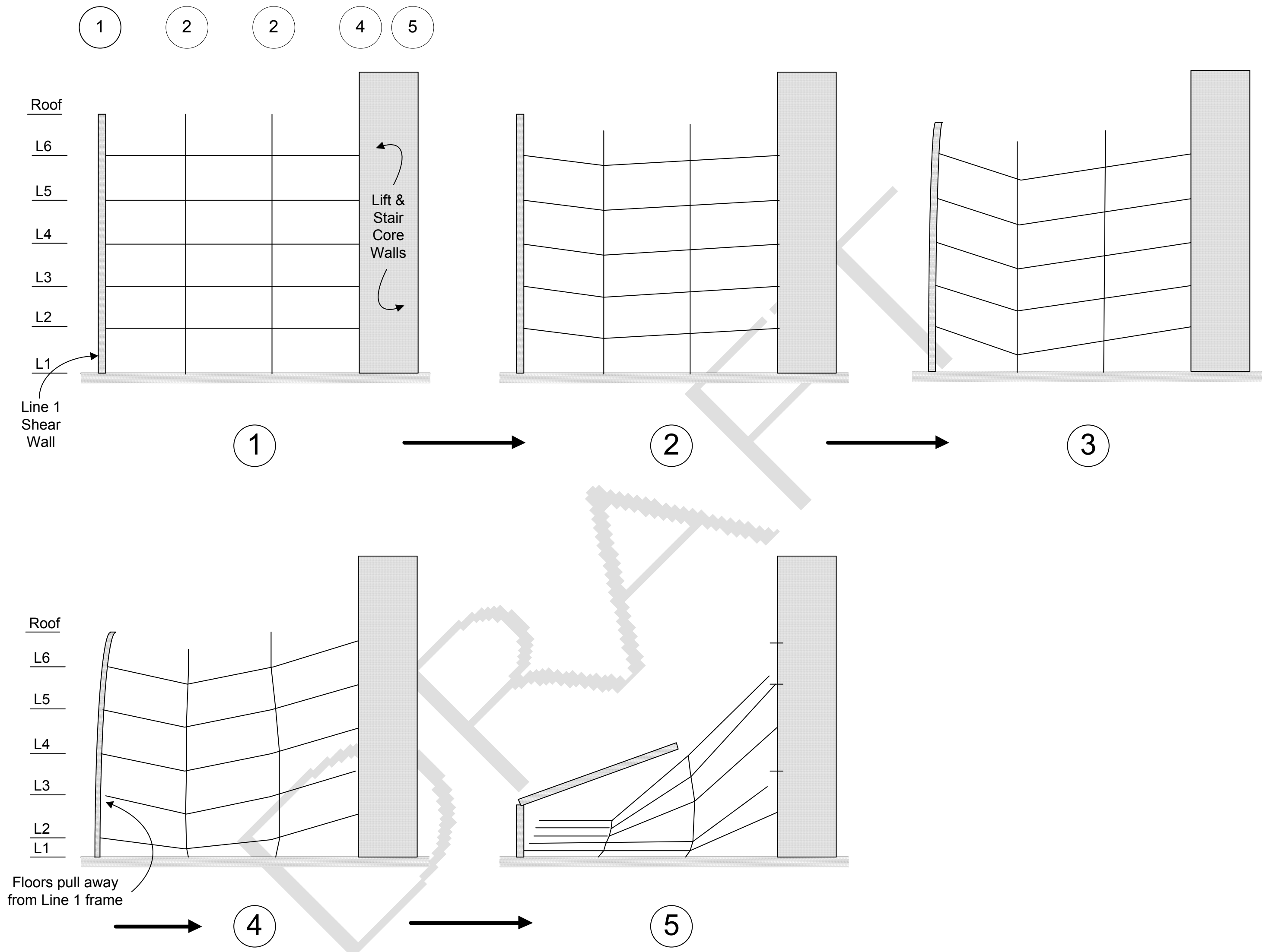


Figure 20 Line D collapse sequence simultaneous with Line 2 collapse sequence

B. ALTERNATIVE STRUCTURAL COLLAPSE MECHANISM

i. L1 or L2 Column Compressive Flexural Failure on Line 2 or 3

Another potential mechanism of collapse, but not preferred as the primary mechanism , is initiation in compressive –flexural failure of the most highly loaded secondary frame columns C7, C8, C13 and C14 on Grids 2 and 3. In this scenario this would occur after them being subjected to lateral displacements required to maintain compatibility with the primary ductile reinforced concrete shear walls.

As the floors sunk over these columns 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 wall on Line 1 was found to have cracking patterns, localised compression spalling of concrete at one end and indications of flexural reinforcement yielding indicating that it had developed a certain amount of inelastic behaviour before the collapse.

The strength of concrete in the columns at the lower levels is a key factor for this collapse mechanism. The detailing of tie and spiral reinforcement in the columns and in the beam-column joint regions did not comply with the Codes of the time for secondary frames as inelastic demands were found to occur at the limiting compatibility displacements set in the loadings Standard NZS 4203:1984. However this would only be critical in terms of initiating collapse if column shears were high or compression failure of the concrete rather than tensile yielding of the reinforcing steel column occurred. Where steel yielding initiates the column failure the shears drop away quickly as the steel elongates plastically and the column connection stiffness reduces and the lateral displacement required to develop the same action on the next cycle of motion increases.

A check of the columns was made under various levels of representative axial actions at the expected range of concrete strengths in the columns from testing. This found that the failure mode was always reinforcing tension limited rather than concrete compression crushing limited except below Level 3 for concrete strengths below the mean of 27.5 MPa for 17.5 MPa strength concrete at 28 days aged by 25%. Similarly below Level 2 for concrete below the upper 95% strength of 33.1 MPa for 17.5 MPa strength concrete at 28 days aged by 25%.

The others were also found to be able to sustain the axial actions from the tributary areas above on the unconfined core of the concrete column, once the cover concrete spalled. Whereas these ones could only sustain those axial actions below their interaction design capacity limit. This would have required the displacements imposed on the columns at the lower levels to have been less than what was needed to exceed the design capacity limit.

The concrete specified for the columns below Level 3 was 30 MPa at 28 days, and 35 MPa at 28 days for columns below Level 2. However at least three columns at Level 1 were found to have concrete properties more consistent with 17.5 MPa strength at 28 days aged by 25 % rather than with the 35 MPa concrete specified.

This is the reason for consideration of the lower strength concrete in the column collapse scenario.

Tests on the 400 mm square stub adjacent to the lift core C18 found it to have concrete that would not have conformed to that for 17.5 MPa at 28 days aged by 25%. The concrete also has traces of silt in it indicating the aggregates and sand had not been appropriately washed before missing. However the column stub was also charred by the fire that occurred after the collapse. While the temperatures that developed are not thought to have been large there can be a reduction of concrete compressive strength that may have occurred. To avoid this effect, the concrete cores were taken so that the tested area was thought to be in concrete unaffected by fire. However this has not been conclusively shown at this stage.

Similarly two single level columns at the Grid F/4 area C23, and C21 or C22 were found to have concrete only consistent with 25 % aged 17.5 MPa 28 day strength concrete rather than the35 MPa strength specified.

The concrete from columns known to have been from Level 5 and above had concrete properties consistent with 17.5 MPa strength at 28 days aged by 25%, whereas concrete with 25 MPa 28 day strength had been specified.

Three columns were found during testing amongst the other samples that had concrete strengths consistent with 30 MPa 28 day strength aged by 25%. One of these was still connected by vertical reinforcing steel to a column above it which had much lower properties consistent with 17.5 MPa at 28 days aged by 25%. The distinct change in concrete properties is consistent with the change specified at Level 3, though 25 MPa concrete at 28 days was specified at Level 3 and above.

However in conjunction with the other two columns of unknown location it can't be said that no columns below Level 3 had concrete conforming with the specified 28 day strengths of 30 or 35 MPa.

Similarly it can't be said with confidence that all the most highly loaded columns on Grid 2 and 3 had concrete better than 17.5 MPa at 28 days.

A check of the gravity actions on the columns in accordance with 1986 Codes, and assuming the lower bound concrete strengths were in fact what occurred , shows that level 1 columns would have been working at the upper Code limit for axial load (

Figure 90).

The analysis shows that if the lower strength concrete was used in the structure the most heavily loaded columns C7, C8 , C13 and C14 were reliant on lower than specified in-service live loads and the safety margins within the design assumptions to maintain their integrity under gravity loads. They would have been operating near the upper limits of their theoretical axial capacity and could sustain only small amounts of lateral displacement from earthquake motion before collapsing (Figure 21).

The sway of the building under seismic loading, and the resulting actions on columns, have been calculated from the 3D ETABS model. The frames while designated as not part of the primary earthquake resisting system by the designer were found to be subject to deformations sufficient to lead to collapse in conjunction with the low concrete strengths found from tests undertaken to date. The asymmetrical layout of the bracing walls meant that the building was subject to significant torsional deformation under an earthquake striking in the east to west or north to south directions. The direction of the aftershock that occurred on 22nd February appears to have been southeast-northwest as both north and east spectra are similar.

The 'non-seismic" detailing of reinforcement in the columns (small diameter ties and spiral at wide spacings) 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 collapse mechanism is a credible option that can't be totally discounted, but depends on the Level 1 and Level 2 columns having concrete strengths consistent with 17.5 MPa at 28 days and aged by 25% rather than the specified 35 MPa and 30 MPa at 28 days respectively.

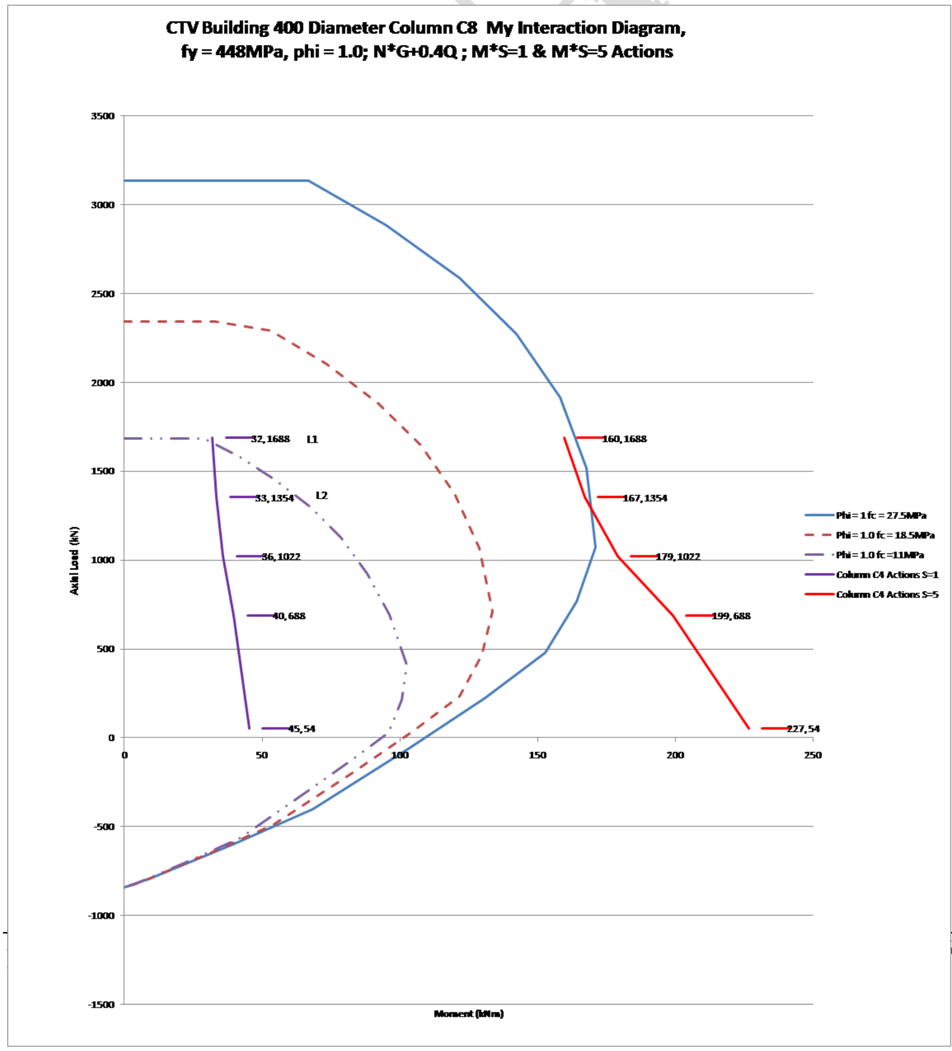


Figure 21 C8 (Grids C/2) column interaction curve used tested material properties and displacement cvompatibi8li9ty actions for S=1 and S-5. Collapse is ost likely where the load demand li8ne breaches the 18.5 MPa average concrete strength interaction curve at around s-3 to 3.5 displacements.

9. STRUCTURAL ANALYSIS

A. DESCRIPTION OF COMPUTER MODELLING

Linear elastic 3D Structural modelling using the computer analysis programs ETABS (Figure 22 and Figure 23) was carried out to enable checks against Standards and to facilitate consideration of structural behaviour with various configurations including the secondary frames with the primary structural walls on Line 1 and 4, the walls alone, and the walls and the masonry infill wall on grid A which a witness had observed to had been built without seismic separation against the sides of the columns.

The structure was analysed using response spectra analysis commonly used for such buildings. Nominal specified material properties appropriate for the time are being used to assess design capacity ratios.

Refer to Appendix F (page 111) for a detailed description of the computer modelling assumptions and key analysis results.

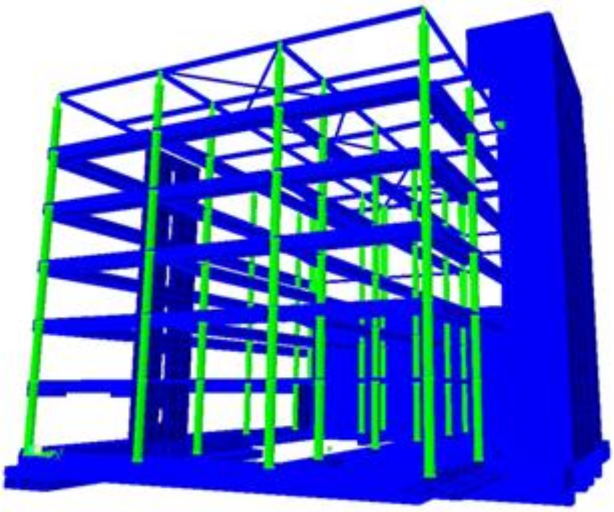


Figure 22 - ETABS computer model - view from north-east

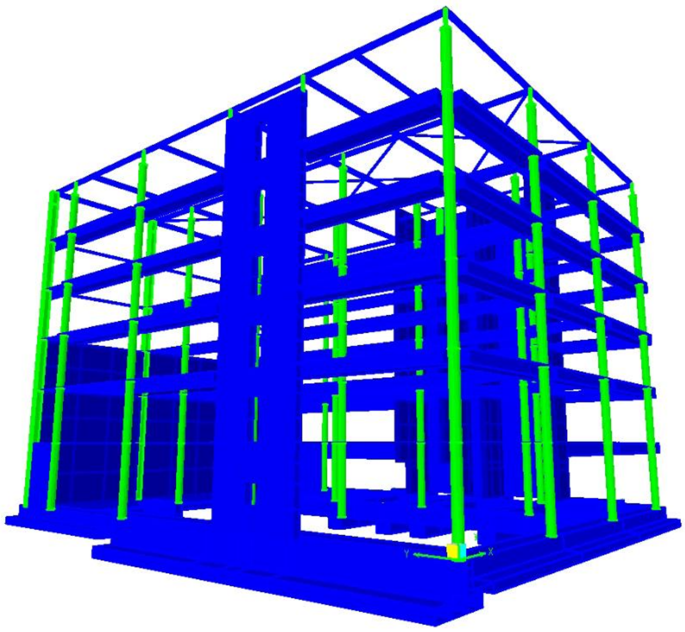


Figure 23 - ETABS computer model - view from south-east

Cracked section properties appropriate for 1986 are based on Paulay and Williams NZSEE Journal paper referenced by the NZ Standards of the time.

The attached Table (page **Error! Bookmark not defined.**) is an outline of the various assumptions that have been made and the sensitivity analyses that have been carried out in arriving at the model that was felt to best represent the structure that existed prior to the February collapse.

The three analysis models 1, 2 and 3, as shown in the three main columns of the table, have been set up however our work to date has focussed on ETABS Model 1. This model is based on Codes applicable in 1986, namely NZS4203:1984 with amendment 3 and NZS3101:1982. Type of analysis is 3D linear elastic, gravity & equivalent static EQ or response spectrum EQ uses specified material properties

Output shaded yellow to compare first mode period and base shear only This results of the runs shaded orange in the table are the ones being used to check Code compliance in 1986. The main assumptions in these runs are as follows:

- upper bound soil stiffness, as recommended by Tonkin & Taylor
- concrete walls only as seismic bracing, with secondary frames considered separately
- with grid A block walls as seismic bracing (because of lack of separation)
- fully ductile response assumed initially

- concentric, +0.1b and -0.1b accidental eccentricity
- output shaded orange will be used for 1986 Code compliance checks

10. BEHAVIOUR OF CONCRETE SHEAR WALLS

A. SUMMARY

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."

In the absence of the documentation of the connection of the drag bar ties into the slab and

The level of damage in the Line 1 shear wall and the calculated assessment of onset of failure indicate that the felt earthquake response of this building was in the order of s=1.7 of NZS4203:1984 for flexible soils in Zone B (Figure 21).

The displacement of the structure as whole is therefore calculated to have remain constrained by the elastic displacement of the Line 1 shear wall up to demands 1.7 times the SM=0.8 spectra.

Actions and displacements on the secondary structural members can be considered to have been able to be sustained up to this level of structural demand.

B. LINE 1 SHEAR WALL

i. Shear Distribution in Line 1 Wall

The distribution of shear actions in the Line 1 wall and the susceptibility to damage from north-south seismic events, are found to have been significantly affected by the Line A masonry in fill wall and the lack of connection of the floor diaphragm to walls D and E at levels 2 and 3.

Analysis undertaken neglecting the Line A wall, leads to inter-storey shear design actions that adequately envelopes the design actions assuming the worst case East/West and North/South seismic events (Figure 24).

This was because 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.

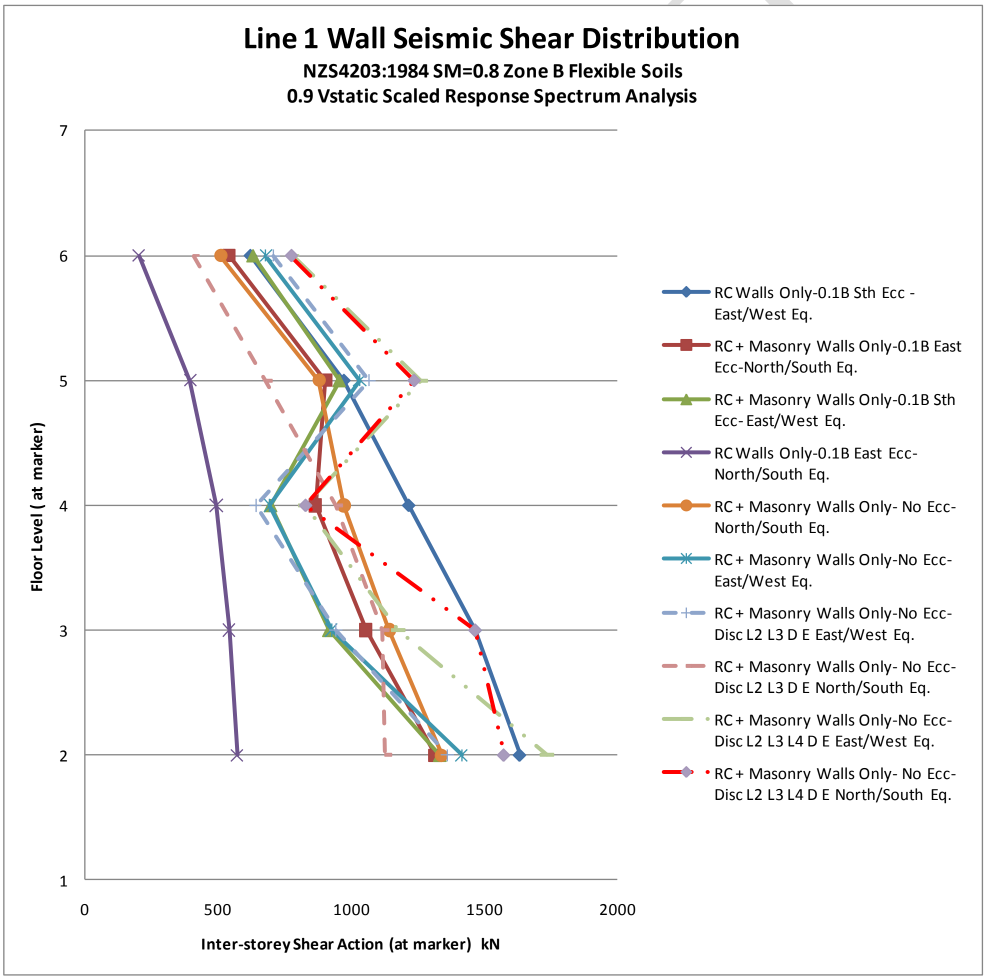


Figure 24 Line 1 Shear Wall shear action distribution

However the introduction of the Line A masonry wall into the analysis elevates the actions on the Line 1 wall, in response to north-south seismic events. As a consequence it then attracts similar inter-storey shear actions for both north-south and east-west events. It in fact the worst condition for the Line 1 wall becomes that where a north-south event occurs and the accidental eccentricity of mass is located east of the nominal centre of mass.

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) when the Line A masonry in-fill wall is included in the analyses. This is particularly so for events with East/West directionality or for North/South events where the accidental eccentricity of mass is shifted eastwards.

As a consequence greater damage would be expected in the Line 1 shear wall panels between level 4 and 5 than between Level 3 and 4. This is 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)

The Line A wall therefore significantly increased the susceptibility of the Line 1 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, including the effect of the Line A masonry in-fill wall on the response of the designated primary seismic resisting structure of the building, if the masonry was detailed in accordance with the provisions of Group 2 Secondary members.

In effect the masonry in-fill block work needed to be protected but not its effect on the overall structure.

This provision of the standard is not in accordance with sound earthquake engineering principles.

ii. Flexural Demands vs Capacity of Line 1 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 confirmed by the cracking patterns in the wall after the collapse (Figure 74).

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 eth wall of fc'= 32.0 MPa and average tested yield stress of the reinforcing steel of Re =448 MPa. When the maximum tested yield stress found in a bar taken from eth east end of the wall of 464 MPa is used the bending demand in eth wall at the point of collapse is calculated to be 21690 kNm.

The flexural demand on the wall at SM=0.8 was M*=12605 kNm. An actual S value at which yield is calculated to have initiated in the Line 1 shear wall is

$$S_{act} = \frac{M_n}{M^*} = \frac{21202}{12605} = 1.7$$

Therefore SM_{act}=1.35.

The displacement of the structure as whole is therefore calculated to have remain constrained by the elastic displacement of the Line 1 shear wall up to demands 1.7 times the SM=0.8 spectra.

Actions and displacements on the secondary structural members can be considered to have been able to be sustained up to this level of structural demand.

C. LINE 5 WALL

i. Shear Distribution and Diaphragm Connection Actions in Line 5 Wall

The Line 5 shear wall forms the north face of the lift and stair core walls. It was detailed and proportioned using capacity design principles.

The distribution of shear actions in the Line 5 wall are found to have been significantly affected by the Line A masonry in fill wall.

Disconnection of the floor diaphragm to walls D and E at levels 2 and 3 increased the shear actions on the Line 5 wall, as the couple between it and the Line 1 wall picked up the additional torsional actions.(Figure 25) .

Floor diaphragm connection actions are greatest at Levels 4 to 6 (Figure 26).

It only suffered hairline diagonal shear cracking near the base in the earthquakes and after-shocks (Hyland 2011).

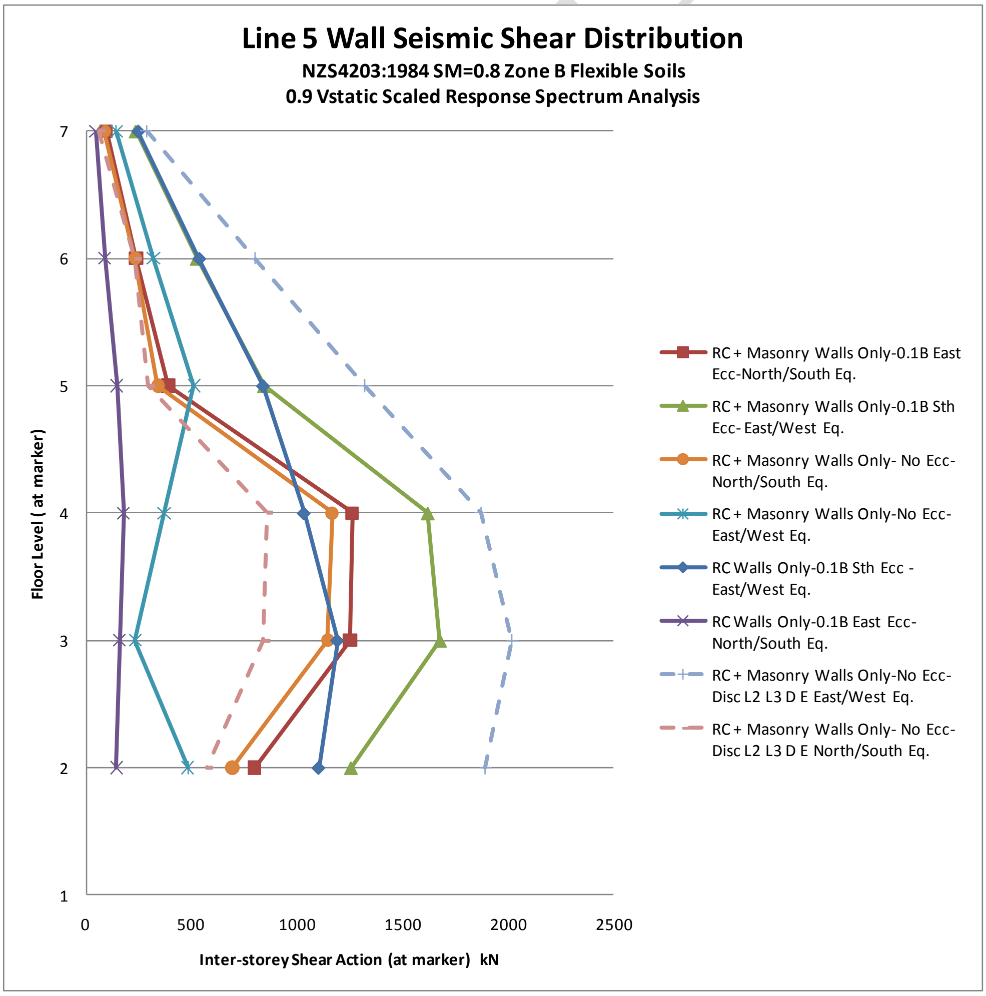


Figure 25 Line 5 Shear Wall shear action distribution

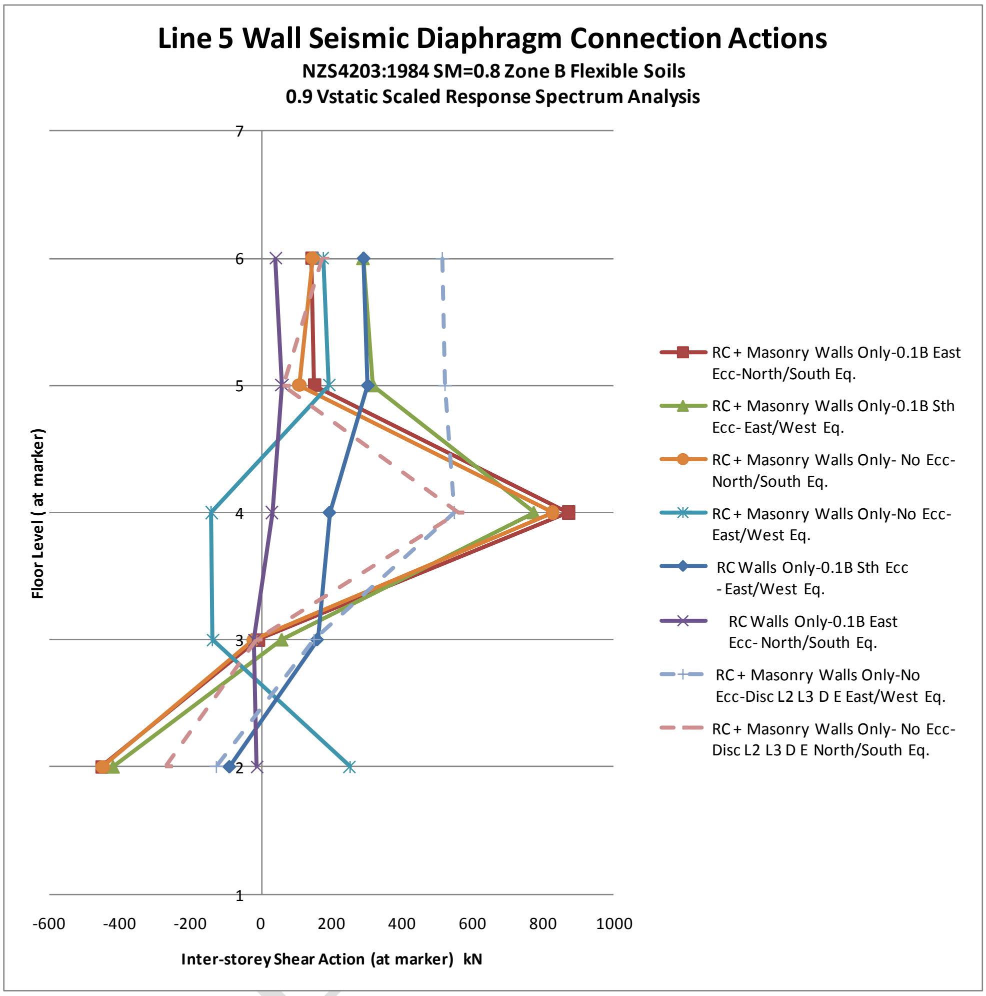


Figure 26 Line 5 Wall seismic diaphragm connection actions

ii. 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 eth wall of fc'= 32.0 MPa and average tested yield stress of the reinforcing steel of Re =448 MPa.

The flexural demand on the wall at SM=0.8 was approximately M*=20500 kNm.

The ratio of nominal capacity over demand at SM=0.8.

$$\frac{M_n}{M^*} = \frac{167900}{20500} = 8.2$$

It is therefore not surprising that no obvious damage was sustained by the Core walls prior to the building collapse.

D. LINE A MASONRY IN-FILL WALL

The distribution of shear actions on the Line A wall indicates that it acted as a pivot to the north-south translation and torsional rotation of the building above Level 4.

The greatest demand occurs on the Line A wall during east-west seismic events (Figure 27).

This shows that the wall strongly influences the torsional response of the structure.

The effect of the disconnection of the floor diaphragms at level 2 and 3 on walls D and E can be seen to have reduced the demand on the Line A wall.

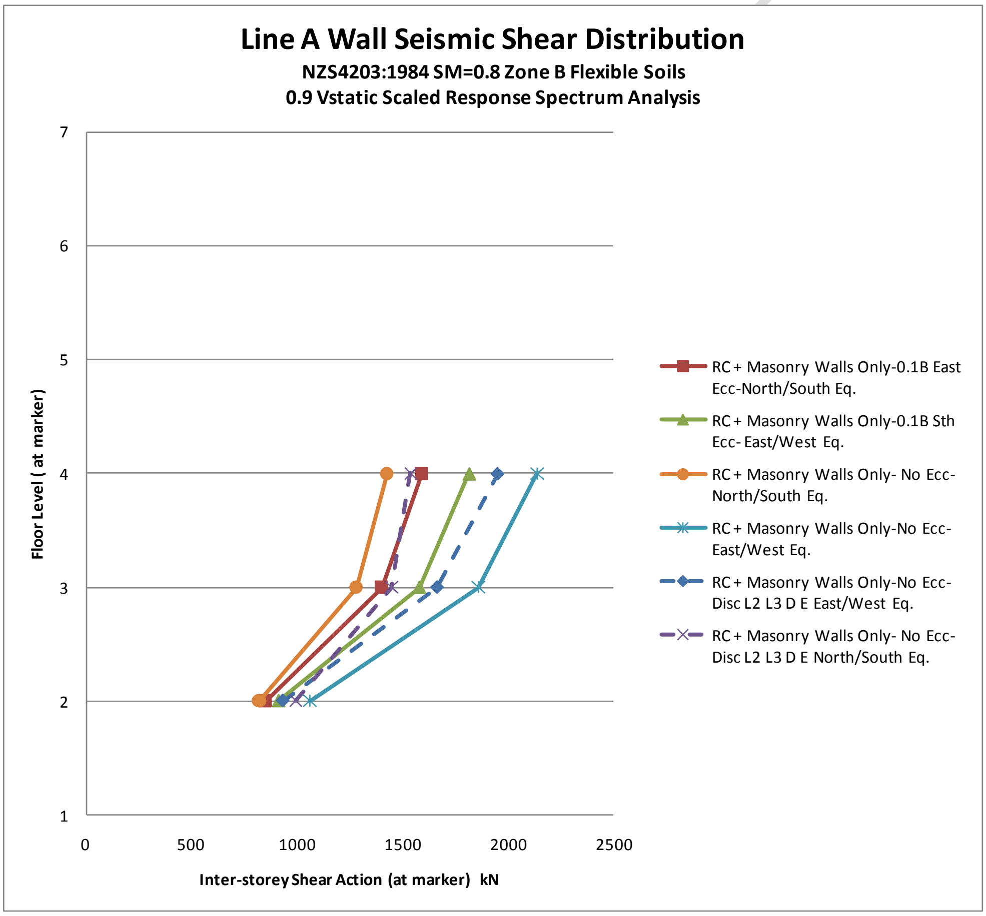


Figure 27 Line A Wall seismic shear distribution

E. LINE C WALL

i. Shear Distribution and Diaphragm Connection Actions in Line C Wall

The Line C shear wall forms the west face of the lift and stair core walls. It was detailed and proportioned using capacity design principles.

The distribution of shear actions in the Line C wall are found to have been significantly affected by the Line A masonry in fill wall.

Disconnection of the floor diaphragm to walls D and E at levels 2 and 3 increased the shear actions on the Line 5 wall, (Figure 28).

Diaphragm connection actions are greatest at Levels 4 and 5 (Figure 29).

It only suffered hairline diagonal shear cracking near the base in the earthquakes and after-shocks (Hyland 2011).

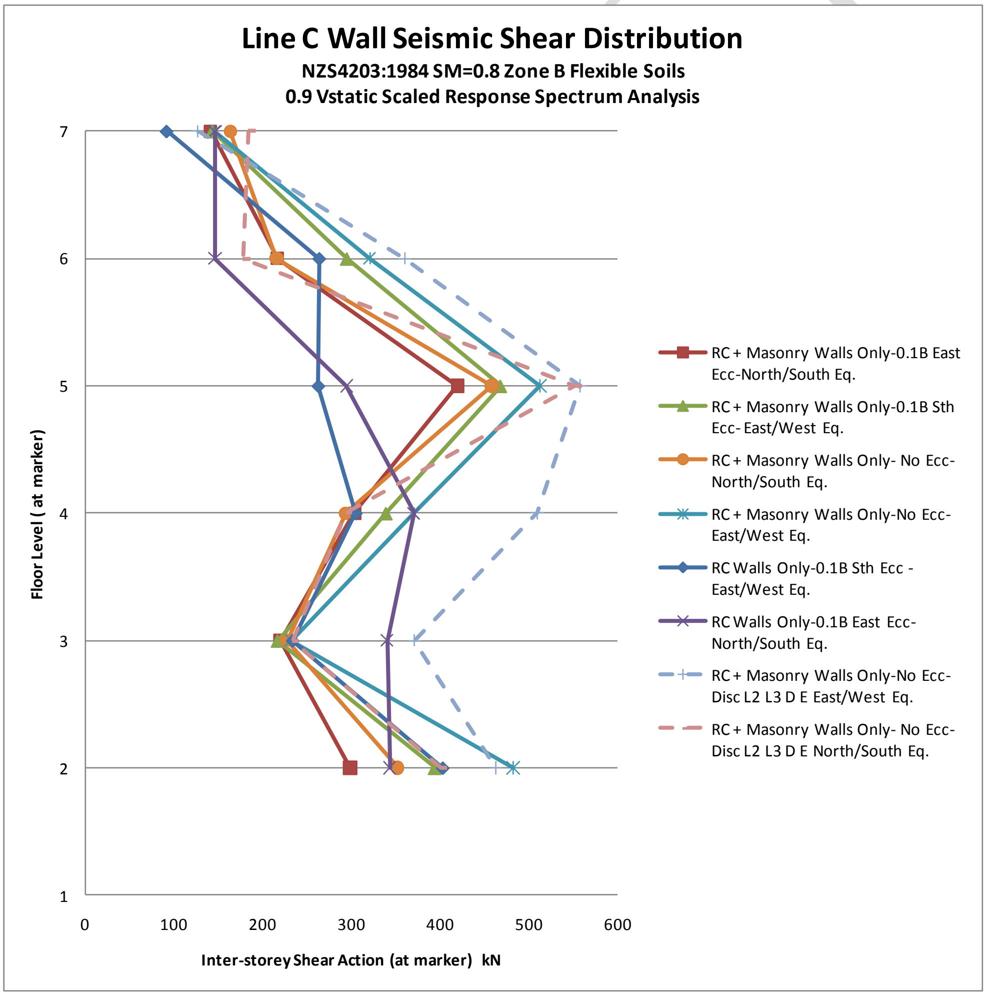


Figure 28 Line C Wall seismic shear distribution

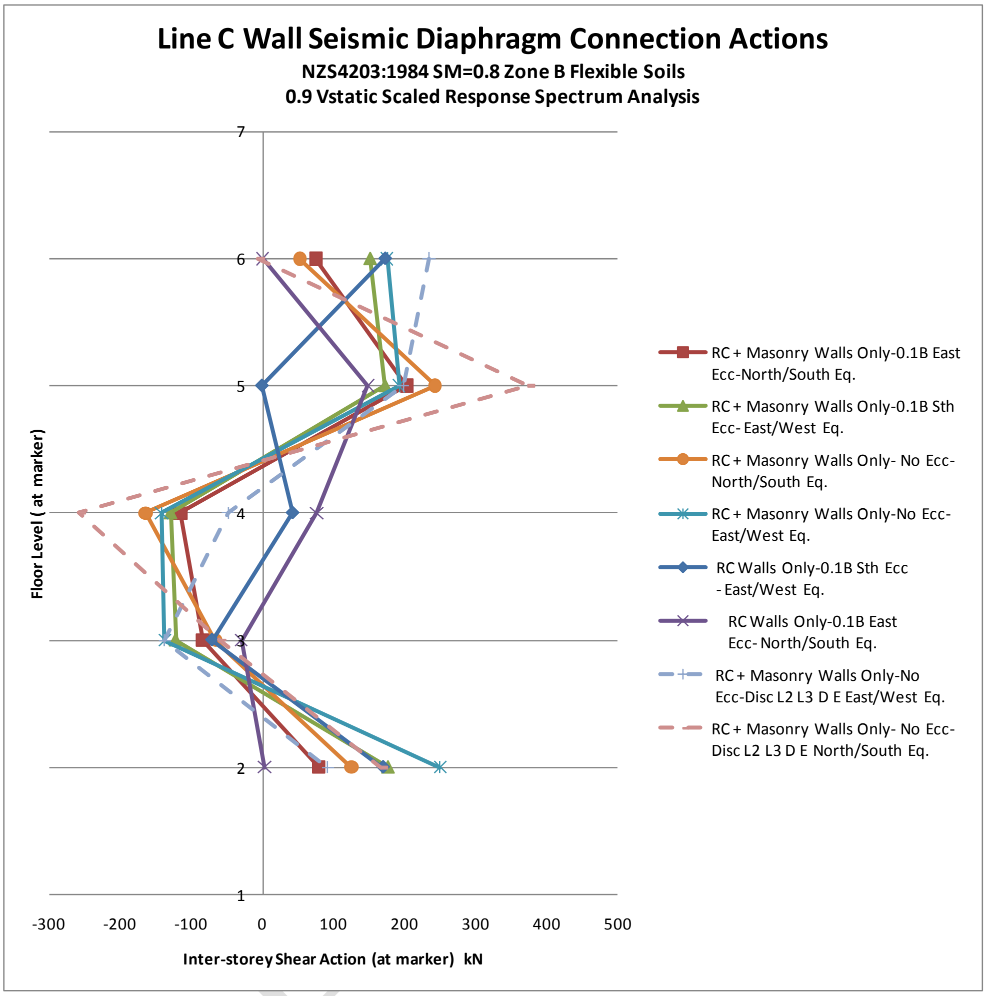


Figure 29 Line C Wall seismic diaphragm connection actions

F. LINE CD WALL

i. Shear Distribution and Diaphragm Connection Actions in Line CD Wall

The Line CD shear wall forms the west face of the lift and stair core walls. It was detailed and proportioned using capacity design principles.

The distribution of shear actions in the Line C wall are found to have been significantly affected by the Line A masonry in fill wall.

Disconnection of the floor diaphragm to walls D and E at levels 2 and 3 increased the shear actions on the Line CD wall, (Figure 30).

Floor diaphragm connection actions are greatest at Level 4 (Figure 31).

It only suffered hairline diagonal shear cracking near the base in the earthquakes and after-shocks (Hyland 2011).

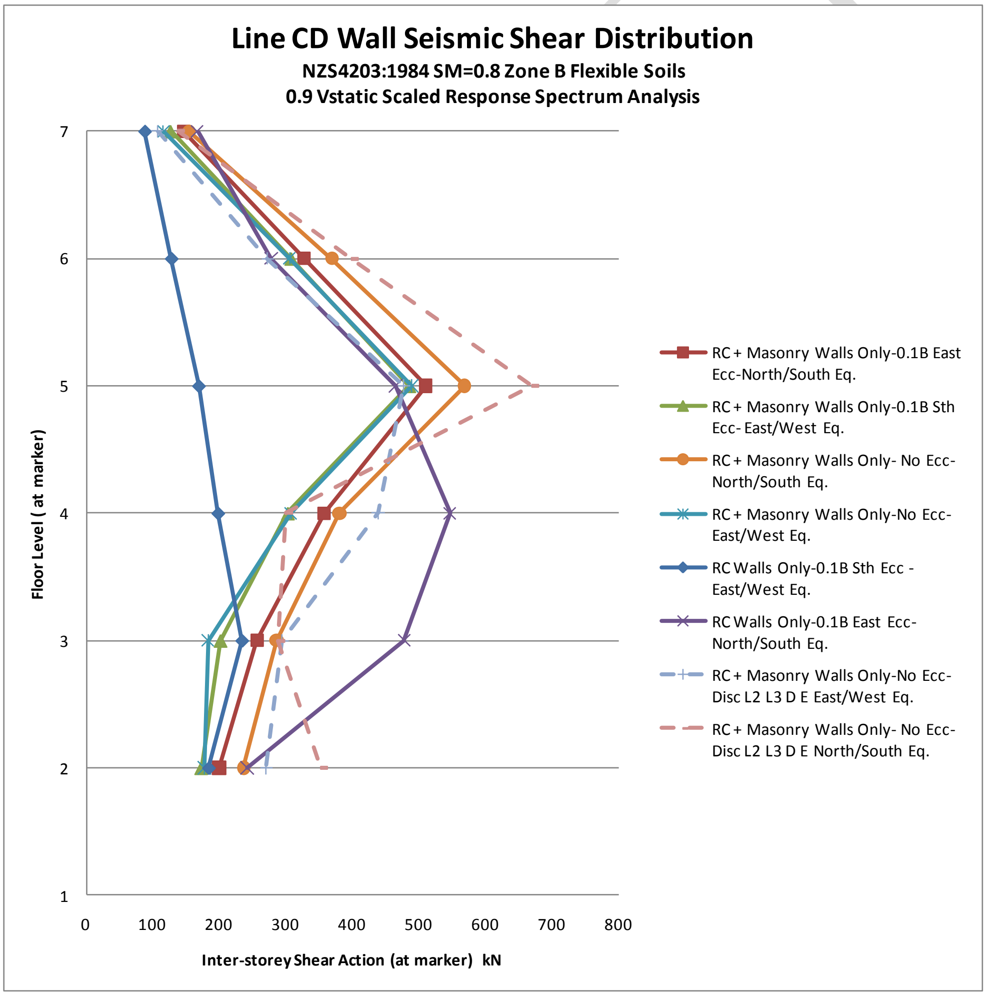


Figure 30 Line CD Wall seismic shear distribution

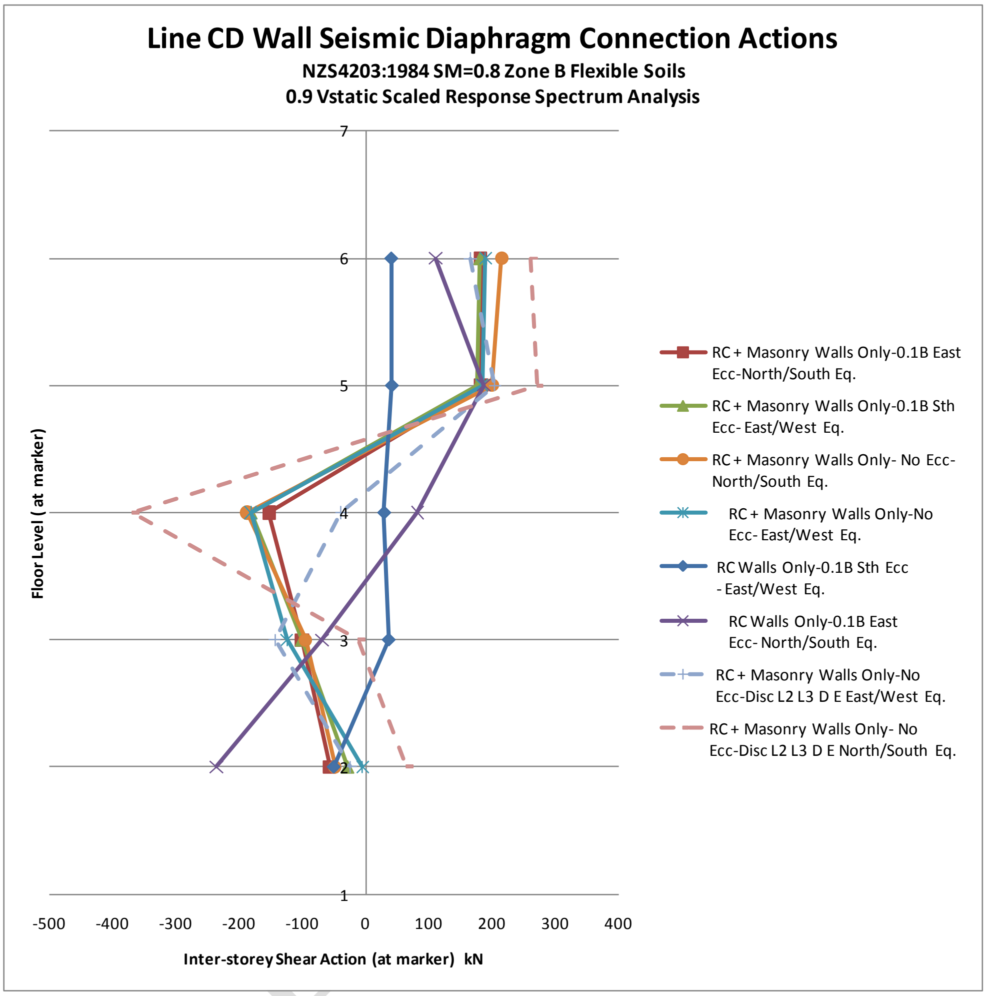


Figure 31 Line CD Wall seismic diaphragm connection actions

G. LINE D WALL

i. Shear Distribution and Diaphragm Connection Actions in Line D Wall

The Line D shear wall forms the west face of the lift and stair core walls. It was detailed and proportioned using capacity design principles.

The distribution of shear actions in the Line D wall are found to have been significantly affected by the Line A masonry in fill wall .

Disconnection of the floor diaphragm to walls D and E at levels 2 and 3 changed the shear actions on the upper levels of the Line D wall, (Figure 32).

Floor diaphragm connection actions are greatest at Level 6 (Figure 33).

It only suffered hairline diagonal shear cracking near the base in the earthquakes and after-shocks (Hyland 2011).

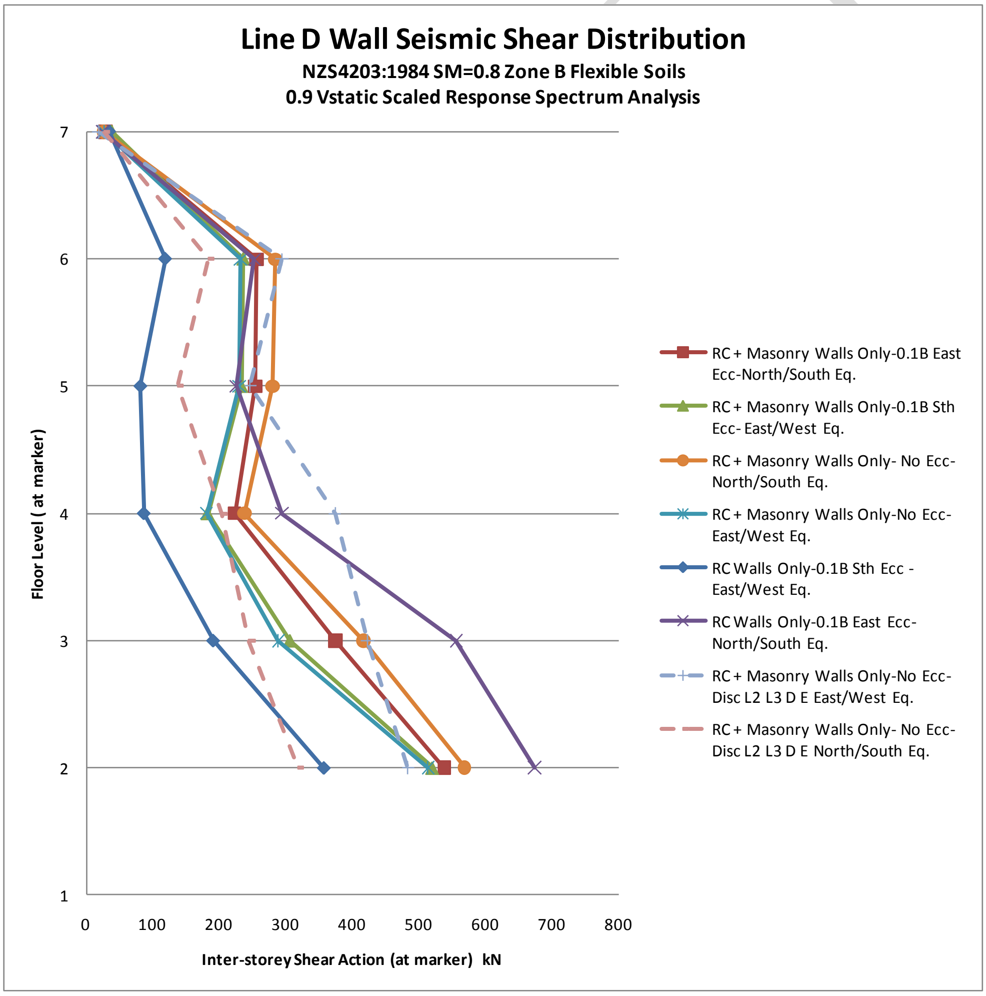


Figure 32 Line D Wall seismic shear distribution

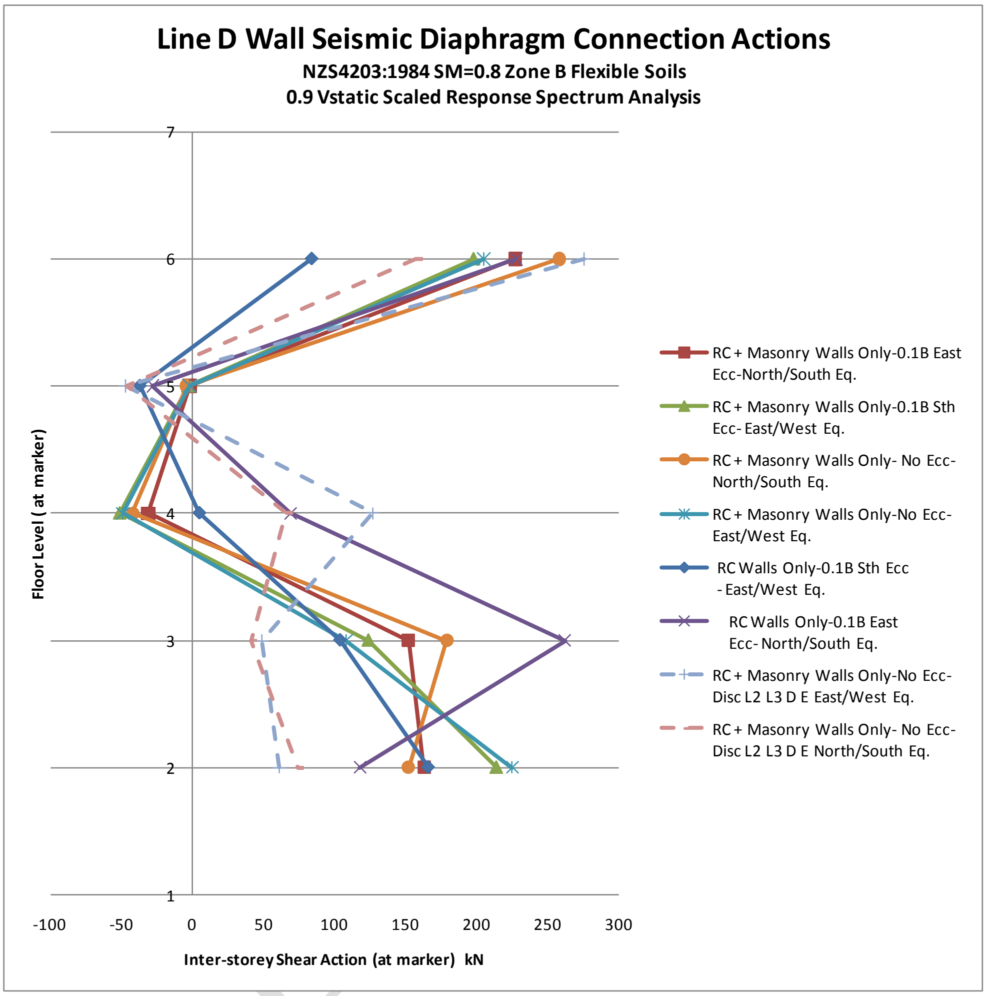


Figure 33 Line D Wall seismic diaphragm connection actions

H. LINE E WALL

i. Shear Distribution and Diaphragm Connection Actions in Line E Wall

The Line E shear wall forms the west face of the lift and stair core walls. It was detailed and proportioned using capacity design principles.

The distribution of shear actions in the Line E wall are found to have been significantly affected by the Line A masonry in fill wall .

Disconnection of the floor diaphragm to walls D and E at levels 2 and 3 increased the shear actions at level 4 of the Line E wall, Subsequent disconnection of the diaphragm at Level 4 increased the shear action between Level 5 and 4 (Figure 34).

Diaphragm connection actions are greatest and Level 4 when there are no diaphragm connections at level 2 and 3. When the connection at Level 4 is also removed the diaphragm connection action at Level 5 reduces (Figure 35).

It only suffered hairline diagonal shear cracking near the base in the earthquakes and after-shocks (Hyland 2011).

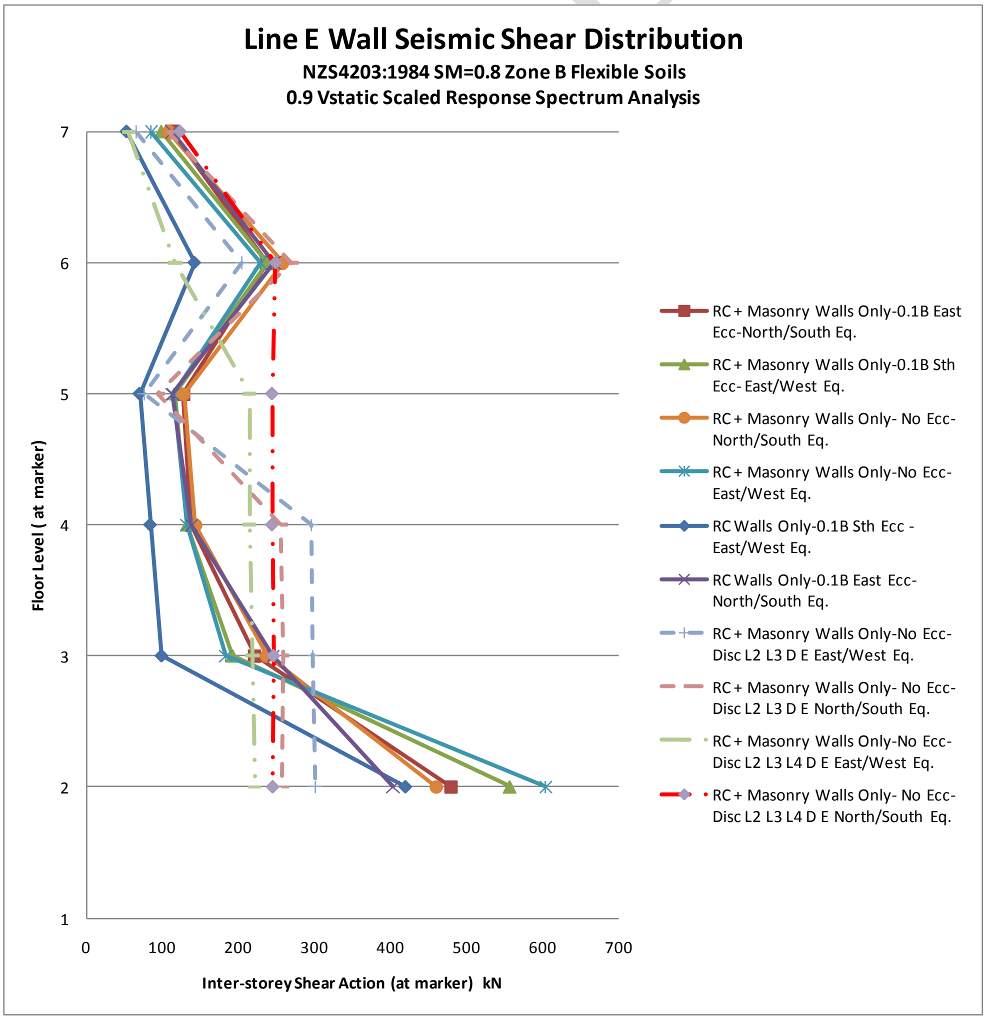


Figure 34 Line E Wall seismic shear distribution

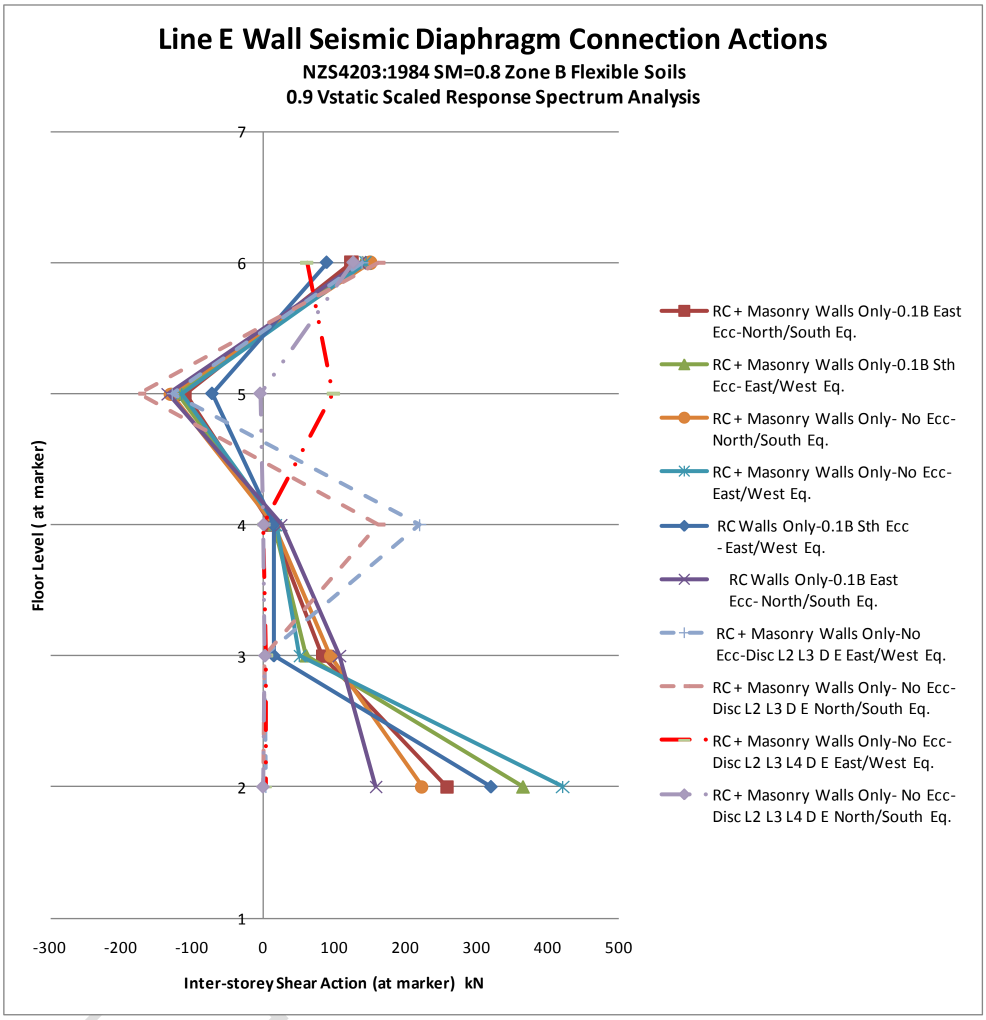


Figure 35 Line E Wall seismic diaphragm connection actions

I. FLOOR DIAPHRAGM CONNECTIONS TO THE LIFT CORE WALLS

The diaphragm connections to the lift core walls were governed by 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 Line 1 and the lift core walls were designed and detailed as fully ductile, the Line 1 wall was able to yield and displace inelastically well before the lift core walls.

Initial analysis with the floor diaphragm connected at Level 2 and 3 at Lines D and E showed that the floor would disconnect at low levels of seismic demand.

The frame was then analysed with those diaphragm connections removed and the demand versus capacity ratios calculated. This showed that the Line 1 and 5 shear walls picked up additional shear to compensate for the loss of diaphragm connections at level 2 and 3 (Figure 24and Figure 25).

Assessment of the capacity versus design action on the diaphragm connections into the lift core walls shows the Level 4 connection into the Line E wall then became the element with the highest relative demand to capacity ratio (Table 1). This was similar to the ratio of the SM=0.8 level bending action compared to the yield capacity of the Line 1 shear wall.

The capacity to demand ratio of approximately 1.7 on the Level 4 Wall E diaphragm connection is consistent with it having been designed using the Parts and Portions provisions of NZS 4203:1984.

The specific level of capacity is not known as the connection of the drag bars into the slab at Level 4 is not known. The capacity has been based on the connection capacity of the remnant drag bar in the wall.

The slab diaphragm capacity itself was found to be less critical than the Wall D and E connections due to the presence of the profiled metal decking. This was found to have been able to develop its tensile capacity during the collapse without losing it anchorage to the supporting beams along Line 4 between Walls C and CD.

Elsewhere the 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.

Wall	Connections	SM=0.8 Action/Capacity	Comment
C-CD			
Level 2	Slab tension/shear	0.15	N/S eq
	Slab shear	0.39	N/S
Level 3	Slab tension/shear	0.08	N/S
	Slab shear	0.39	
Level 4	Slab tension/shear	0.23	
D			
Level 4	3 M24 wall bolts	0.52	N/S
Level 5	4 M24 wall bolts	0.15	N/S
Level 6	6 M24 wall bolts	0.57	N/S
E			
Level 4	4 M24 wall bolts	0.50	N/S
		0.68	E/W
Level; 5	5 M24 wall bolts	0.43	N/S
Level 6	7 M24 wall bolts	0.27	N/S

Table 1 Floor diaphragm connection ratios relative to analysed demand actions for SM=0.8 dynamic response spectrum analysis. The floor diaphragm is considered disconnected at Level 2 and 3 on walls D and E in this analysis.

J. WALL LINE 1 AND 5 FLEXURAL ACTION TO CAPACITY RATIOS

The flexural demand to capacity ratios of the walls on Line 1 and 5 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 2).

It can be seen that wall on Line 1 would be expected to yield and then deform inelastically well before the level 5 wall. Given the large difference in the capacity ratios its difficult to conceive that the Line 5 wall would ever have been able to becoming a yielding element in the structure.

It therefore raises questions about the adequacy of the provisions for the ductile design of torsionally irregular structures. In this case though the core shear walls were detailed for ductile performance they in fact responded as elastic elements working in conjunction with a fully ductile perimeter wall on Line 1. There are no specific provisions in the design standards preventing this occurring. For buildings designed with primary structural systems acting in different directions the rule is that here should only be 1 change in the level of ductility between the two systems so that there is a level of inelastic compatibility between them in the event of an design earthquake occurring. In this case the comparative ratio of ductile response between Line 5 and Line 1 wall is equivalent to the ratio of their respective demand ratios or 5.9.

A requirement to ensure that the ratio between the relative flexural demand/ capacity ratios of any elements of a primary seismic resisting system in a torsionally irregular structure is limited to a value of 1.20 would prevent this level of irregularity in the ductile response occurring.

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 2 Flexural demand / capacity ratios for walls on Line 1 and 5

APPENDIX A – BUILDING DATA

A. DRAWINGS

Note to panel - refer to CTV workspace, under Building Data folder for drawings, as follows:

- 'CTV Structural Drawings Part 1' pdf also includes architectural drawing A6.
- 'CTV Structural Drawings Part 2' pdf also includes architectural drawings A1 to A5, A7, A8 and A14. These pdf files were obtained from the Christchurch City property file.
- Amended drawing S26, with detail for Blockwall 4a added was provided subsequently by ARCL.

B. SPECIFICATION

Note to panel - refer to CTV workspace, under Building Data folder for Structural Specification, as provided by ARCL.

C. GEOTECH REPORT

Note to panel - refer to CTV workspace, under Building Data folder for Site Investigation Report dated 18 June 1986 by Soils and Foundations (1973) Limited.

APPENDIX B – EYE WITNESS SUMMARIES

A. THE INTERVIEWS WITH THE EYE WITNESSES

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 witness location map (Figure 36).

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.

i. Witnesses Inside the CTV Building

1. Level 6: East side of the south west 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.

ii. Witnesses Outside the CTV Building

5. Les Mills building

6. IRD building

7. IRD building

8. In front of CTV driveway on Cashell 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 Cashell Street

16. Working on the recladding on the CTV at south west corner of CTV building

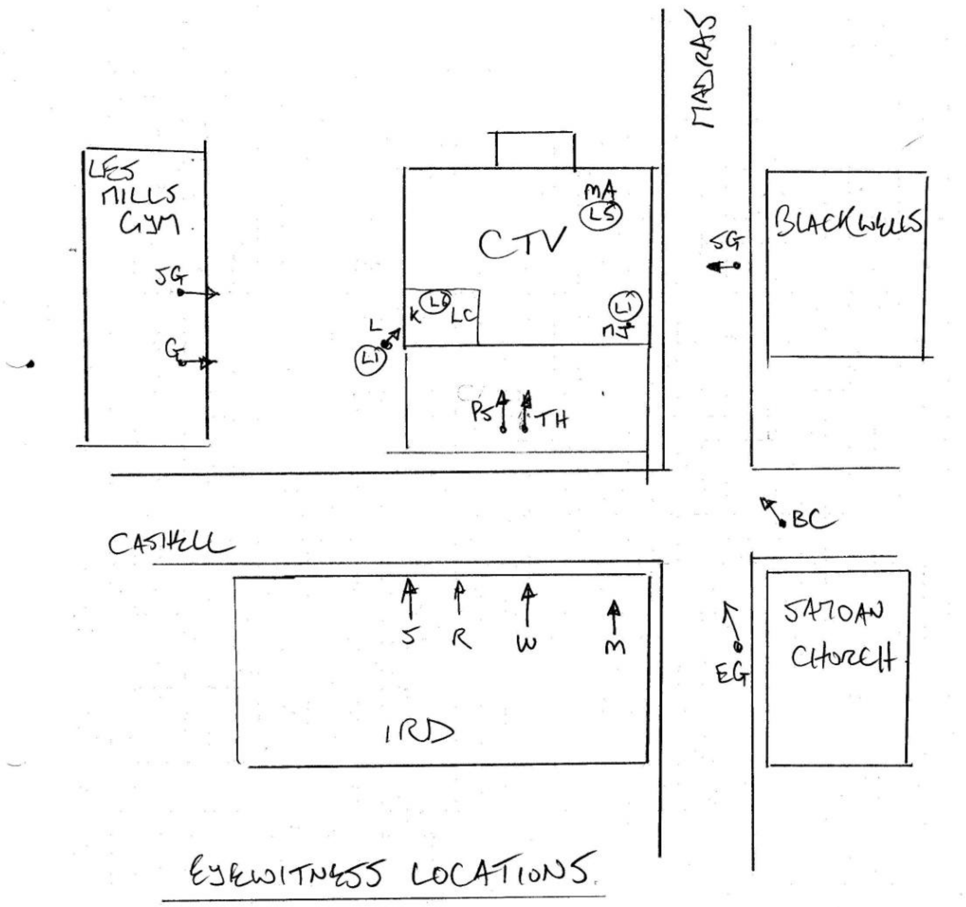


Figure 36 Eye witness location map (draft only)

B. INTERVIEW SUMMARIES

i. Eye Witness I.

Eyewitness I was in the eastern side of a room on the southwest corner of Level 6 at the time of the earthquake. (See Witness Map.)

- *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 Cashell 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.

- *Timeframe.*

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 witness 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.

ii. Eye Witness 2.

Eyewitness 2 was on Level 1, 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 on 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 fallWhen 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 was 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."

• Timeframe.

"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 witness 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 witness 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.

iii. iEye Witness 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 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 on the west side and back 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."

• After the building had collapsed.

A colleague that I had made contact with managed to crawl to my side of the desk. There was still movement and lots of noise, and screaming and shouting – and I smelled smoke for the first time. We possibly could have smelled smoke earlier, it is hard to tell because of the dust and dryness. This smoke spurred us on to try and get out and move towards the daylight my colleague had seen on the other side of my desk. I could see that the way that the furniture had moved that I was tucked in this sort of corner. We were passing rubble, insulation, whatever back so we could move forward. I managed to push myself out to chest level. Then my colleague shouted that there were two men out on some kind of concrete ledge. One called to me - I was picked up, and pulled out and dropped to to the guy below. " I am pretty sure I got out into the alley way right next to my desk, through what would have been the window.

• 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. She described seeing how the two buildings had an about 2 inch or more gap between them, with metal rods tieing them together brick to brick. (Square star shaped black thin metal gauge.) She could see them as she drove into the car park, and noticed those rods had gone.

iv. Eye Witness 4.

• Eyewitness 4 was against a side wall on Level 6 that comes out to Cashel Street just in front of the IRD building. The side next to where the demolition work was.

" Usually our meeting would have been in the middle of our premises – but on this particular day we were sitting furtherest 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 – even 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, 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 where 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 Cashell 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 Cashell 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.

Timeframe

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, then the fire broke out in the lift or lift shafts.

• Pre-earthquake observations about the building.

This witnessed 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 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…its 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, before the earthquake collapse, that they were just superficial.

v. Eye Witness 5.

Eye Witness 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 witness 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 they way it all went down.

Timeframe.

"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 witness was definite that the CTV was down during that first earthquake, 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.

Observations at the site.

He was standing at the front, Cashell 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 smokey 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.

iv. Eyewitness 6.

Eyewitness 6 was in the IRD building on the third floor as the earthquake hit.

" 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 just collapsed straight down."

Direction of the fall.

"It was just a side 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 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."

Timeframe.

The time that this witness 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.

Observations of the site.

His observations of the site where 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 they had been demolishing 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."

.

C. SUMMARY OF DESCRIPTIONS

The experiences of those who survived the collapse of the building, combined with those viewing it from different angles from outside - give us helpful clues as to what actually happened to the structure of the building.

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

- The subjectivity of time.

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

- The subjectivity if sensation.

For example, if someone has no sensation of falling, it suggests a "slow" fall. In real terms there is a "rush" that is experienced with a fast fall, for example, like 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.

- The subjectivity of words.

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.

(to be written)

D. CONCLUSIONS FROM THE INTERVIEWS

We can conclude from the interviews that the building fell in the following way and there were consistent themes of concern and observation that needed to be addressed by the technical analysis.:

(to be written)

APPENDIX C - POST-COLLAPSE CONDITION

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

Observations and comments are recorded about each item in the general text and in captions to the photos.

A. OVERHEAD VIEWS

Figure 37 Aerial view from southeast with debris being removed by heavy machinery (Dominion Post)

Figure 38 Aerial view from northwest with heavy machinery removing debris (NZ Herald)

B. WEST WALL (LINE A)

Figure 39 West side of building with lift core partially obscured by smoke, prior to heavy machinery removing debris

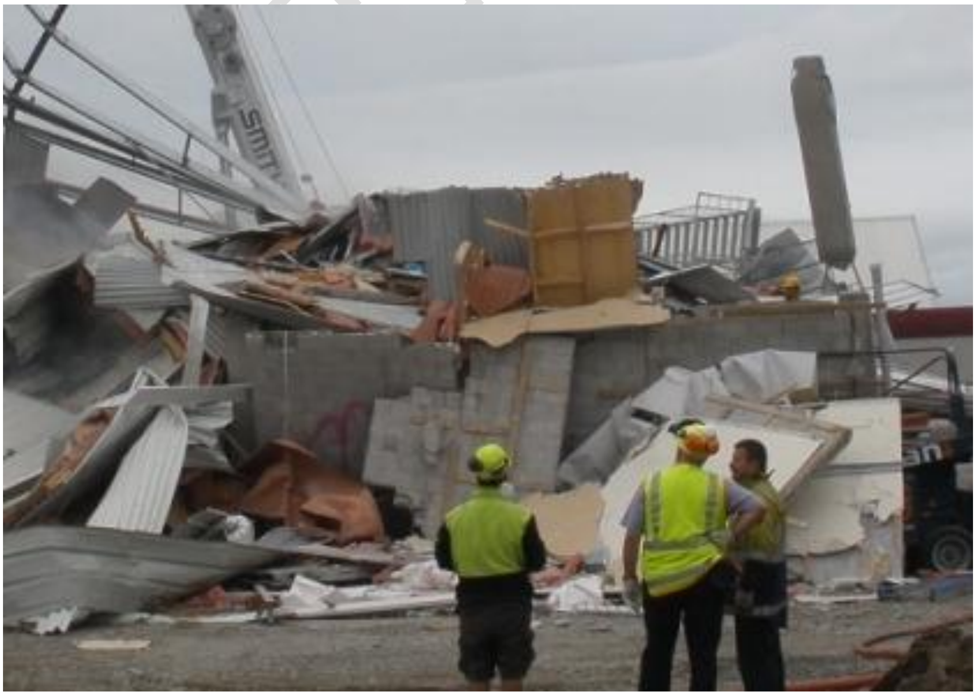


Figure 40 South west corner (Grid A/1) with corner column still standing. Collapsed work platform under wall panels.

C. CASHELL ST (SOUTH, LINE 1)

Figure 42 Cashell St face with lift core tower in background prior to fire starting


Figure 43 Western end of south face (Line 1). Collapsed Line shear wall with escape stair to the right.

Figure 44 Corner of Cashell and Madras Street faces (Line 1/F). Fractured columns in foreground

D. MADRAS STREET (EAST, LINE F)

Figure 46 Corner of Cashell and Madras Streets with columns and spandrel panels (msn photo)

Figure 47 Madras St with precast spandrel panels fallen onto cars

Figure 48 View from across Madras Street with Line 2 column at left and Line 3 column at right

APPENDIX D- REMOVAL OF DEBRIS

The debris from the collapse was removed from site and taken to a secure designated area at the Burwoood Landfill.

The photos (show stages in the sequence of debris removal.

Eventually the lift core tower was left on site with a selection of structural remnants that are described in the Site Examination and Materials Tests report.

Figure 49 Spandrel panels and beams at Cashell Street Line 1 and on Line 4 in background standing vertical. Roof debris visible.

Figure 50 Debris being cleared from Madras Street face

Figure 51 View form Cashell Street with debris being cleared away from west wall

Figure 52 View from southwest corner face Line 1 with pre-cast edge beam being removed and emergency stair on Line 1 shear wall visible.

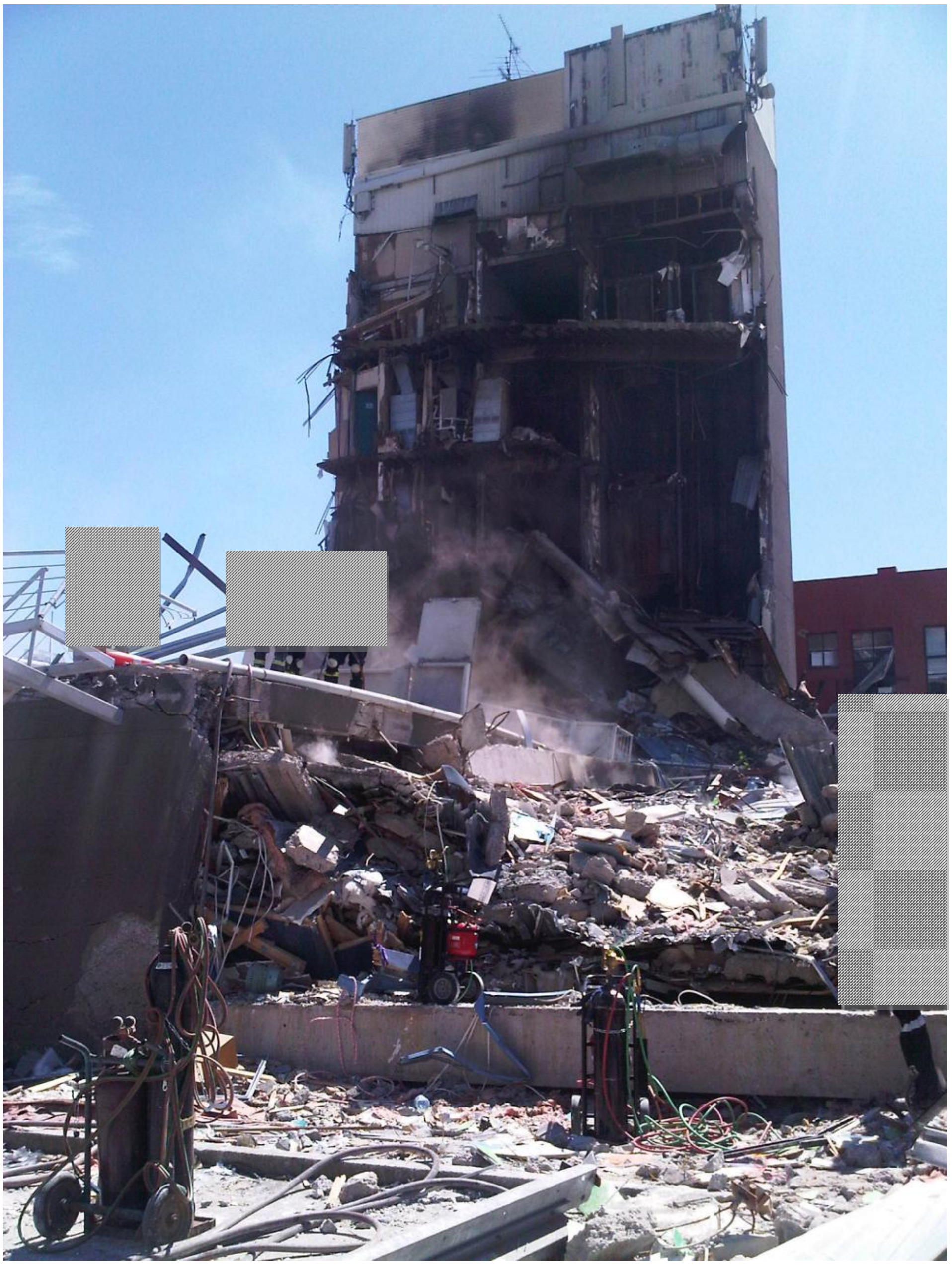


Figure 53 View from Cashell Street east side of Line 1 with Line 1 shear wall lying on debris at left; trapezoidal end profile of floor slabs laying on top of each other in foreground; Remnant of lift core slabs and column C18 at rear. Level 6 slab in front of lift core still in place though column C18 has collapsed below.

Figure 54 Portion of Line 1 shear being lifted out by crane.. The Level 6 slab in front of lift core has been removed for safety reasons.

Figure 55 Level 3 portion of Line 1 shear wall being prepared for removal

Figure 56 Line 1 shear wall has been removed.

Figure 57 Slabs sloping diagonally from lift core. Line 2 debris still visible

Figure 58 Line 3 and Lift core slab debris remaining

Figure 59 Lift core slabs remaining to be removed.

Figure 61 Lift core slabs removed.

Figure 62 All debris removed leaving the Level 1 slab on grade and remnants of the lift core

APPENDIX E- REMNANTS OF STRUCTURE

A. ARCHITECTURAL CLADDING SPANDREL PANELS

Figure 63 Concrete spandrel panels, perimeter beams and columns on Cashell Street face (Line 1 / B-D)

Figure 64 Building from Madras Street during construction showing spandrel panels on Line F and 4 fitted in between columns above floor levels

Figure 65 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

B. SLAB AND INTERIOR BEAMS

Figure 66 Line 2 beams laying rotated northwards

Figure 67 Line 3 beams lying rotated southwards

Figure 68 Lift core slab lying diagonally against the core

C. 400 MM DIAMETER PERIMETER CONCRETE COLUMNS

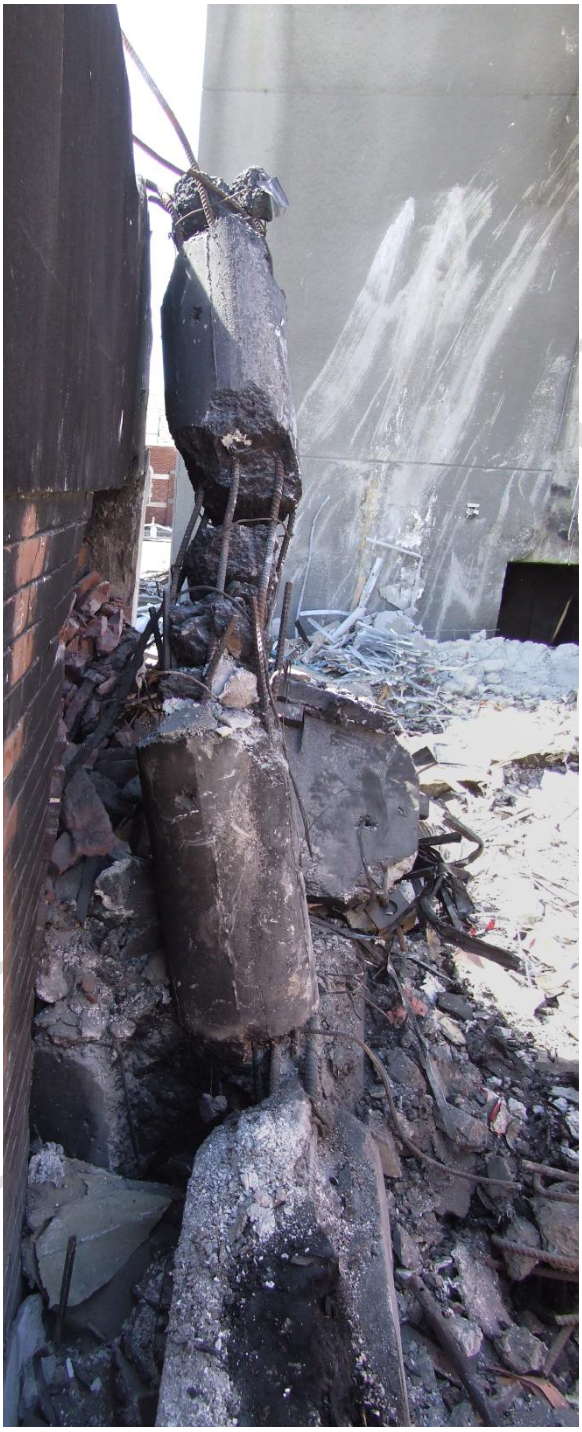


Figure 69 Line 4 / B spandrel column with B22 precast log beam in foreground and B23 shell beam at rear. No hinging is apparent at the base of the column compared to the perimeter column Item E33 (refer Site Examination and Materials Test Report)

Figure 70 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 (refer Site Examination and Materials Test Report)

Figure 71 Perimeter columns at beam -column joint with shell beam on right side

D. LIFT CORE COLUMN C18

Figure 72 Lift core column C18 remnants among debris

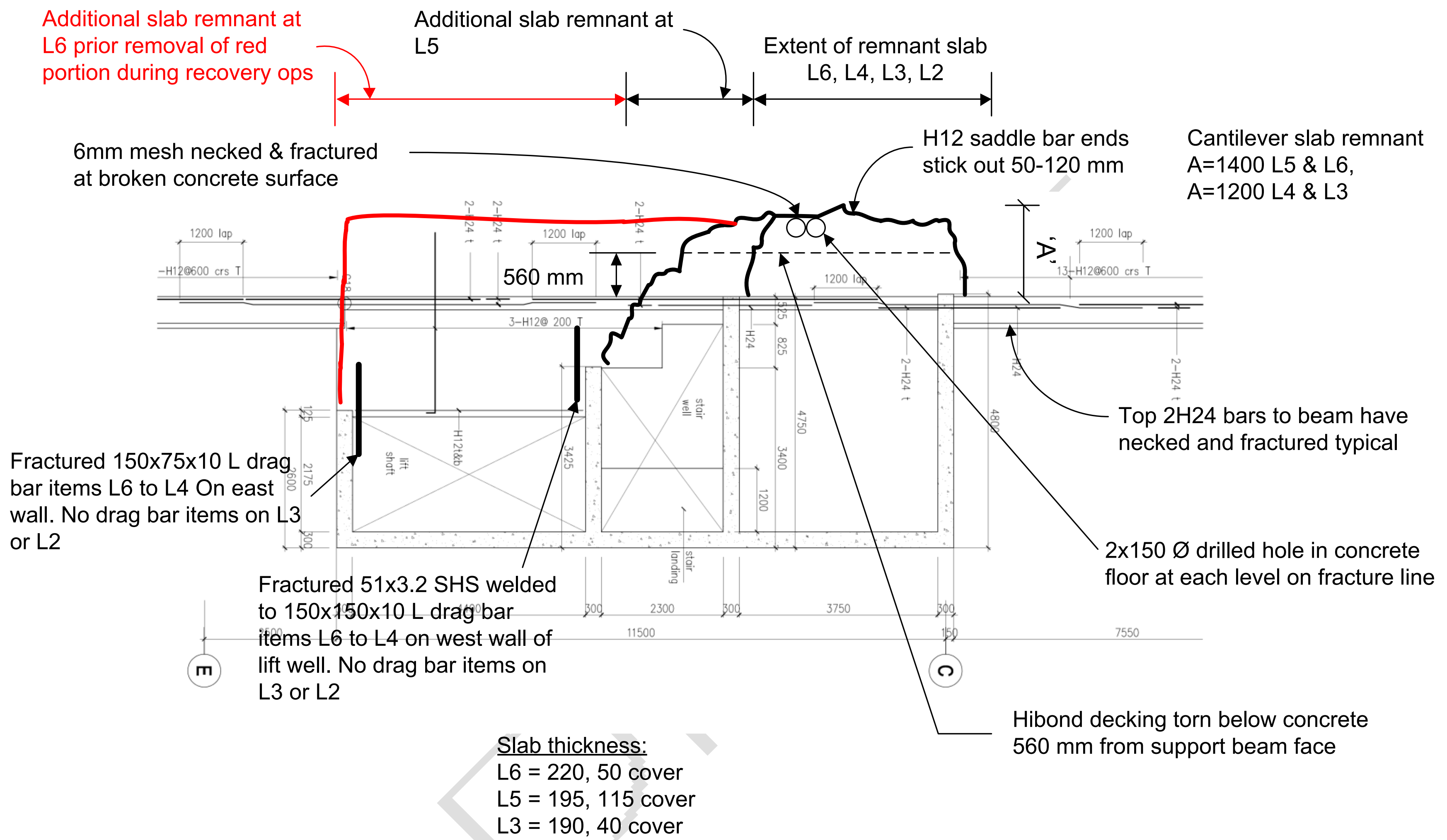


Figure 73 Lift core slab remnants after collapse

E. LINE I SHEAR WALL

Figure 74 Line 1 shear wall at Level 1 showing masonry in-fill at door opening, in-plane flexural cracking and spalling of concrete at right (east) end

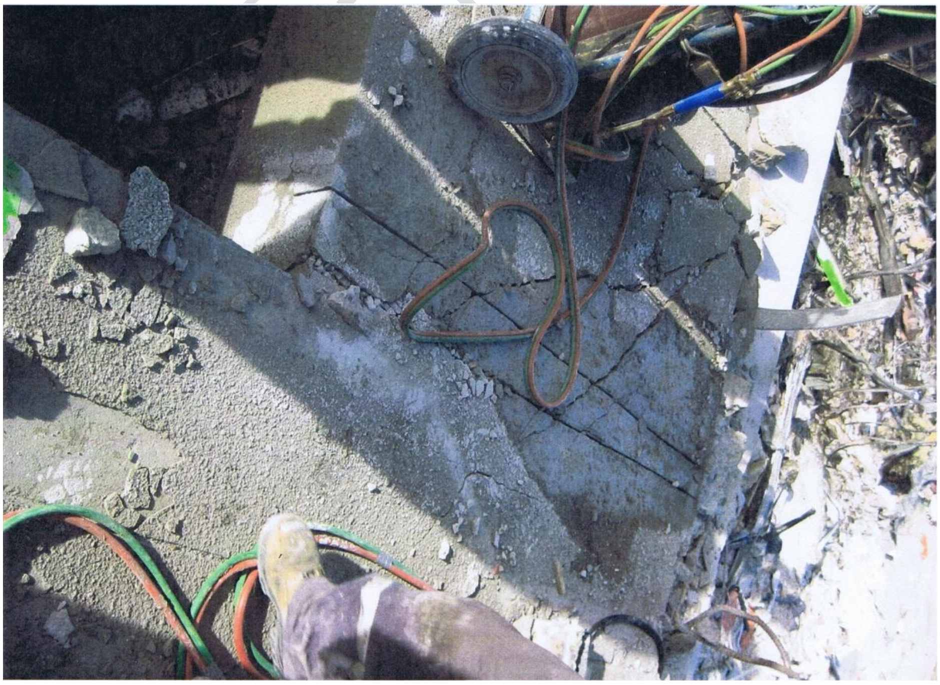


Figure 75 Line 1 shear wall at Level 4 being prepared for cutting and removal. Shear cracking in panel is visible under cutting equipment and hoses.

APPENDIX F – EVALUATION/ANALYSIS DETAILS

A. COMPUTER MODELLING ASSUMPTIONS

Computer modelling assumptions and comparison of first mode period and seismic base shear for the various models analysed are summarised in Table # below and discussed in the notes following.

CTV Building - ETABS Computer Analyses									
Modelling Assumptions and First Mode Period, Base Shear Comparisons									
	Units	**Analysis Model 1**							
References		**1986 Codes**							
		NZS4203:1984							
		NZS3101:1982							
Objective		1986 Design Code compliance check							
Software program used		ETABS							
Analysis Type		Elastic, 3D, Dynamic Spectral Modal Analysis							
Seismic Load Input		Response Spectra							
Superimposed Dead Load	kPa	0.55							
Live Load	kPa	2.50							
Seismic Live Load	kPa	0.83							
T & L Beams - slab overhang each side	mm	300							
Material Properties	Various Units	Specified Material Properties (f'c 25MPa typical - up to 35MPa for level 1 columns)							
Effective Section Properties, I_e, Av_e									
- I_e, T & L beams	Fraction of I_g	0.50							
- I_e, Columns	Fraction of I_g	1.00							
- I_e, Walls	Fraction of I_g	0.60							
- I_e, Diagonally reinforced coupling beams, Grid 1	Fraction of I_g	0.40							
- Av_e, Diagonally reinforced coupling beams, Grid 1	Fraction of A_g	0.83							
Code Subsoil Flexibility / Site Subsoil Class (for seismic load input)		Flexible subsoil							
Modelled foundation spring stiffness - where k = expected stiffness, 0.77k = lower bound stiffness and 1.36k = upper bound stiffness (refer Tonkin & Taylor report)		Rigid Foundation		1.36k					
Accidental Eccentricity		Concentric		**Concentric**		+0.1B		-0.1B	
	EQ Direction	N-S (X)	E-W (Y)	**N-S (X)**	**E-W (Y)**	N-S (X)	E-W (Y)	N-S (X)	E-W (Y)
Model 1a. - Concrete Walls only (As-Drawn)									
First mode period of vibration, T1	seconds	0.82	0.79	**1.20**	**0.94**	1.22	0.81	1.21	1.02
Base Shear - (ductile S=1, M=0.8)	kN	2718	2776	**1797**	**2488**	1796	2728	1796	2220
				*	*				
Model 1b. - Concrete Walls + Masonry Walls (As-Built)									
First mode period of vibration, T1	seconds	N/A	N/A	**1.03**	**0.70**	N/A	N/A	N/A	N/A
Base Shear - (ductile S=1, M=0.8)	kN	N/A	N/A	**2342**	**2660**	N/A	N/A	N/A	N/A
				*	*				
Model 1c. - Concrete Walls + Masonry Walls + Frame (As-Built)									
First mode period of vibration, T1	seconds	N/A	N/A	**0.88**	**0.60**	N/A	N/A	N/A	N/A
Base Shear - (ductile S=1, M=0.8)	kN	N/A	N/A	**2590**	**2996**	N/A	N/A	N/A	N/A
				*	*				
LEGEND		= Analyses to compare first mode period and / or base shear only							
	*	= Analyses referred to in this report							

Figure 76 - Computer modelling assumptions - first mode periods and base shears

Notes to table:

1. The intention has been to model the building structure as it understood to have been prior to the February 22 2011 aftershock.

2. Seismic analyses have been carried out using the industry standard ETABS software package. This package was commonly available at the time the CTV building was designed in 1986 and is still commonly used today for design of multi-storey buildings.

3. The method of analysis used is 3-dimensional dynamic 'spectral modal analysis' as described in NZS4203 and NZS1170.

4. Analyses have been carried out to determine conformance with applicable Standards and to facilitate consideration of potential failure modes.

5. Analysis model 1 is based on the Standards applicable at the time the building was initially constructed in 1986. These include NZS4203:1984 with Amendment 3 dated December 1984 and NZS3101:1982 (check relevant Amendment #).

6. Superimposed dead load has been estimated as 0.55kPa throughout.

7. Live load has been taken to be 2.5kPa withNZS4203, or 3.0kPa with NZS1170.

8. Seismic live load has been calculated in accordance with the relevant Standard, 0.83kPa with NZS4203

9. An effective width of 300mm of floor slab overhanging the beams at each side has been assumed in the calculation of effective T-beam properties for use in the analysis, as recommended in the Standards.

10. Material properties have been calculated based on the specified 28 day concrete strengths as follows:

    a. Level 1 columns – 35MPa

    b. Level 2 columns – 30MPa

    c. Remainder – 25MPa

11. Effective section properties including effective moment of inertia, Ie and effective shear area, Ave have been calculated as specified in the relevant edition of NZS3101. For Analysis Model 1 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, which is referenced from NZS4203 was the basis for the effective section properties for shear walls.

12. The subsoil is considered to be flexible as defined in NZS4203.

13. Sensitivity analyses have been carried out using various values of foundation spring stiffness. In 1986 it was common practice by many engineers to assume that foundations were rigid. However for this building, which is founded on flexible subsoil and with shear walls cantilevering off foundation beams which are only activated when some flexibility is assumed, the assumption of flexible soil springs is considered to be most appropriate.

The appropriate stiffness of soil springs for seismic analysis has been calculated by geotechnical engineers Tonkin & Taylor (T&T) Limited as outlined in Appendix #. T&T give 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 we have used the upper bound stiffness values (i.e. 1.36k) since that will give a conservative estimate of the natural periods of the structure and of the design base shear.

14. Analysis model 1 has been subdivided into models 1a, 1b and 1c incorporating the key features of the structure that are believed to have most influenced the response of the building to earthquake shaking and to give an indication of the upper bound and lower bound seismic force on each component under the Code level shaking. Each of these three models has been analysed using the response spectrum analysis method taking into account the relevant natural periods and scaling of the dynamic base shear to 90% of the equivalent static value in accordance with the Code. The analyses have been carried out for a structural type factor 1.0 with NZS4203, or for a structural ductility factor 5 with NZS1170.5. The results can then be scaled up as required for limited ductile or elastic response.

15. Model 1a includes only the concrete shear walls, being the core walls at the north side of the building and the coupled shear wall at the south side of the building as the primary seismic force resisting system. This reflects our understanding of the original design intent based on the structural calculations provided by the design engineer. In their calculations the concrete masonry wall on Line A was not included in the seismic analysis as part of the primary seismic resisting structure.

    This appears to have been permissible under the provisions of the Concrete Structures Standard NZS 3101:1982 Group 2 secondary structural elements provisions cl 3.5.14

    Model 1a also incorporates the column that is positioned at grid intersection D/E4. This column is connected to the top of the core wall directly and so is considered to be an integral part of the core structure and therefore part of the primary seismic force resisting system.

16. Model 1b incorporates the concrete shear walls in model 1a as well as the three-storey high concrete masonry walls at the west side along grid A (Figure 77). The secondary beam and column frames are not included as these are post-processed separately for displacement compatibility with the primary system

    The design drawings show that it was intended that the Line A wall be fully grouted. Greased starter bars at 600 mm centres were fixed into the under-side of the precast beams along Grid A (Figure 77 DENG Dwg S9 section 6). There is a D12 horizontal bar shown in the top course of the infill masonry in Section 6 of S9, and a note on Dwg S17 requires Grade B masonry all cells filled so intention was to have it filled. Grade B masonry required observation by an engineer during construction, so it should have been grout filled as intended.

Shear fixity at the top of each approximately 2 metre wide panel, and shear and flexural fixity at the panel bases appears to have been the design intent. Gravity actions and confinement by the beams and walls above would mean it would be difficult to achieve much flexural tension slippage in the greased bars at the lower two levels.

Workers removed mortar trimmings off the face of the wall in preparation for strapping and cladding the wall a day or so before the 22nd February after-shock. The adjacent building had been demolished leaving the wall exposed to the weather.

Vertical separation gaps between the corner column (Grid 1/A) and the short Line 1 return wall can be seen. A horizontal separation gap appears also to be evident between the Line 1 wall and the beam above it.

However no obvious vertical or horizontal gaps are evident on the West wall along Grid A.

The workers 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 (

). They found that the wall wasn't fully grouted when they later drilled holes into it for timber strapping fixings. The rectangular columns sit out proud of the wall face by 20 mm or so.

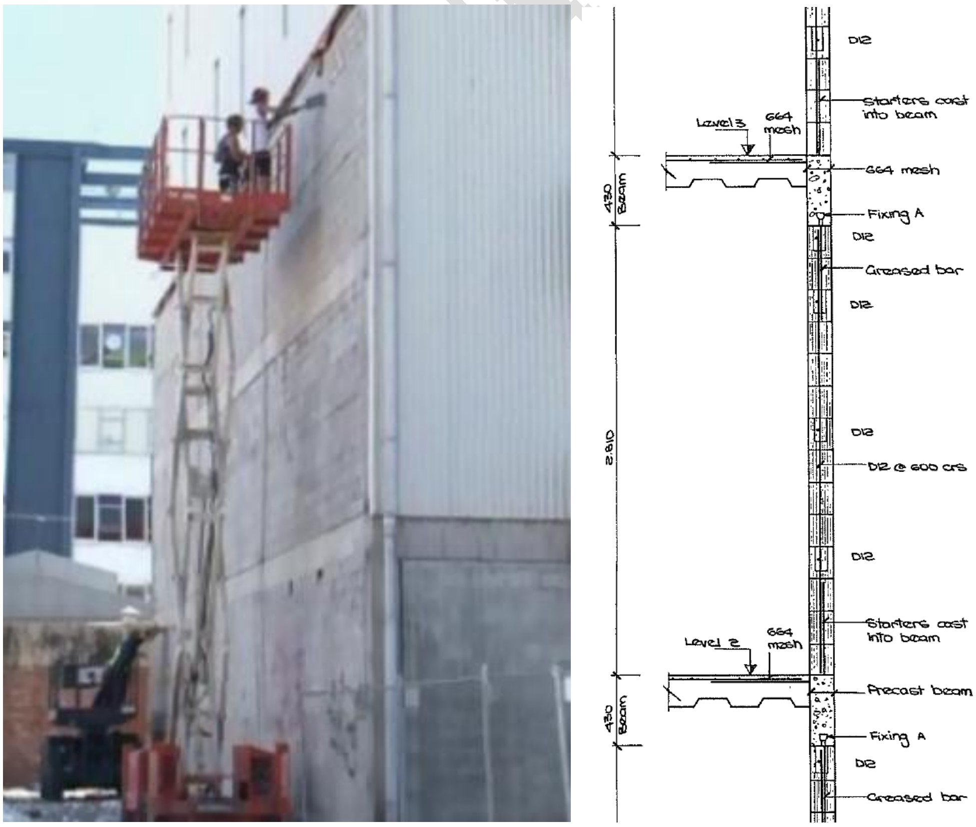


Figure 77 West wall on Line A (left to right) : Being prepared for strapping and cladding a day or so before collapse on 22nd February; b) Connection of west wall block work into floor beams top and bottom (portion of DENG Dwg S9 Section 6).

An engineering inspection after the 4th September earthquake found sealant on the inside face adjacent to a car park column (Figure 80) indicating 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 adjacent building wall abutting the CTV building was likely to have been still 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.

Figure 78Workers hammering face of top course block away on west wall near Line A / 1 corner column

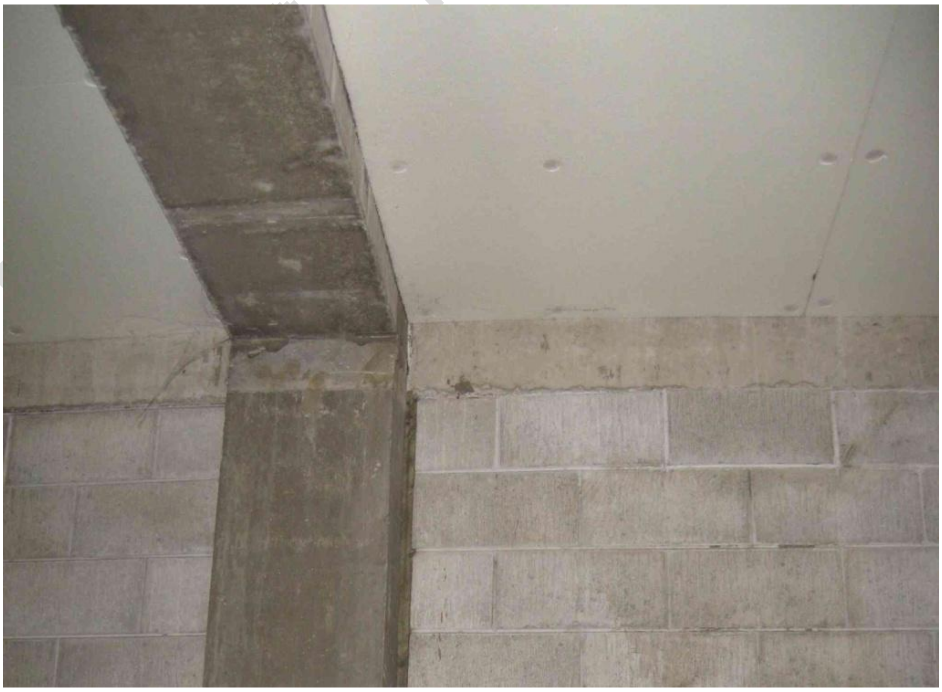


Figure 80 West Wall inside Level 1 car park after 4th September earthquake showing no apparent damage with sealant on vertical joint against column apparent on near side and no gap to underside of Line 1 floor beam above.

The inside of the west wall on Level 2 after the 4th September earthquake shows some damage to the linings (Figure 81).

Figure 81 Inside of the west wall on Level 2 after the 4th September earthquake shows some damage to the linings

Figure 82 West (Line A) wall at south (Line 1) corner showing collapsed masonry in-fill panels

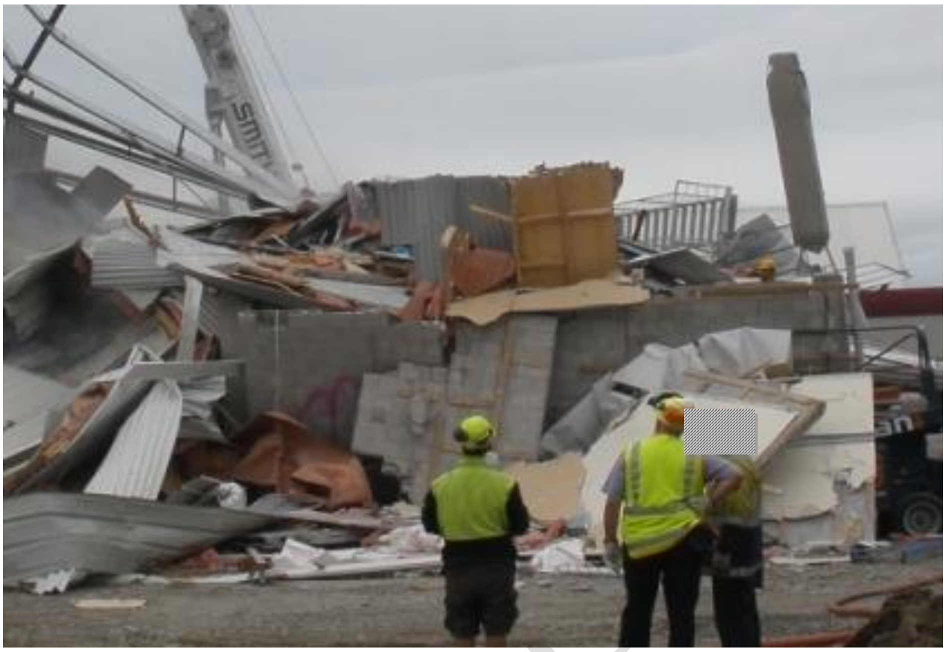


Figure 83 West wall shortly after collapse. The corner Grid 1/A column is still standing and the wall panels have broken free in panel sections in places. The edges of the panel section are square consistent with the design drawings

During the collapse (Figure 82) the masonry wall along Grid A broke apart, in some cases as distinct panels, consistent with the design drawings (DENG Dwg S17). The outside face joint appears to have had a nominal amount of mortar filling the outside edge. However this would have reduced the panels to move as three separate panels and increased their collective stiffness further. However the extent of that interaction with each other and the columns either side is difficult to quantify accurately.

Therefore, for the response spectrum analyses the masonry wall panels were modelled ignoring the effect of interaction between the sides of the panels and the columns. The level of stiffness introduced into the structure with this approach was sufficient to move the centre of stiffness significantly towards the western wall compared to that found using Model 1a. The additional effect of fully locking up the walls as an integral unit would further move the centre of stiffness westward but by a smaller amount.

The modelling of the Line A masonry wall in Models 1b and 1c are therefore 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 10mm gap between panels and the 25mm gap between the masonry and concrete framing was present.
- The masonry material properties were E = 15 GPa.

The masonry walls at level 1 on grid 1 and 4 have not been included in the computer modelling as they were specified as separated structurally from the columns each side with reasonable gaps and had no reinforcing steel connecting them to the floor beam above (DENG Dwg S9 Section 2 and 3) .

17. Model 1c incorporates the concrete shear walls and the concrete masonry walls in model 1b as well as the concrete beams and the remainder of the concrete columns (the concrete frame).

As shown later in this report the elastic limits of the concrete frame are exceeded for the specified Code level of shaking and therefore the frame will not be fully effective. This means that the most likely response will be somewhere between models 1b and 1c, and probably closer to the model 1b response.

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 is that "frames in parallel with slender shear walls should be designed as fully participating primary members".

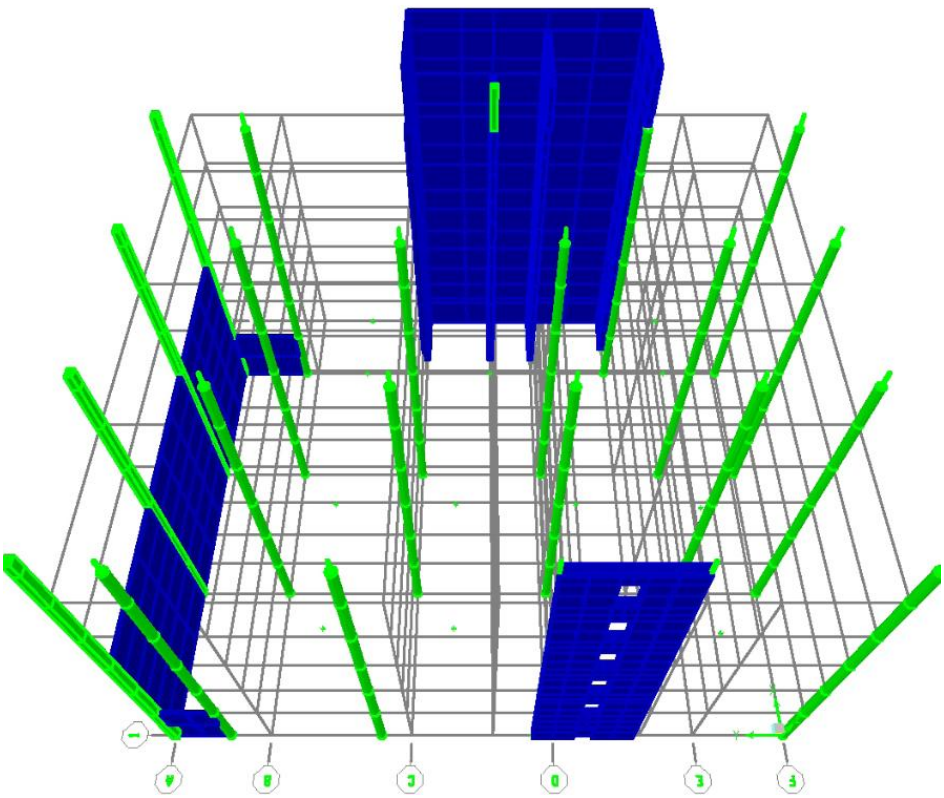


Figure 84 - 3-D view of ETABS model 1c showing layout of concrete shear walls, concrete masonry walls and columns (beams have been switched off in this view for clarity.

18. 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 is the length of the building

perpendicular to the horizontal loading direction under consideration. However the object of this study is to identify the cause of collapse rather than design and for this reason we have focussed on the concentric mass analysis runs shaded orange in the table. The other analysis runs, shaded yellow in the table have been carried out to enable comparison of the first mode periods and base shear for other configurations.

19. From the orange shaded results in the table we see that, as expected the stiffness and the base shear increase progressively as we move from Model 1a to 1b to 1c. What are not obvious from the table are the severe structural irregularities and the wide variation of torsional behaviour between the three models. Irregularity and torsional response are discussed in the following section F2.

20. For this project the effective section properties for shear walls from NZS4203 are believed to be close to reality for the shaking that occurred on 22 February 2011, taking into account the level of cracking that was observed in the walls after the collapse.

B. IRREGULARITY AND TORSIONAL RESPONSE

One of the key features of the CTV building seismic force resisting structure, as drawn and as simulated in ETABS computer model 1a, is the asymmetrical plan layout of the concrete bracing walls. The northern core walls being substantially stiffer than the southern coupled shear wall in the east-west direction, meaning that the structure has a severe plan irregularity. This can be seen in the following Figure 85showing 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, as in ETABS model 1b, the structure is highly irregular in plan in both directions and also a major vertical irregularity is introduced at level 4 due to the participation of the masonry walls below that level. The level 4 floor acts as a major transfer diaphragm here in transfering seismic loads from the north core to the west side masonry walls and vice-versa.

With the concrete frame also added, as in ETABS model 1c the situation is similar to that in 1b but the irregularities are moderated slightly by the action of the frame which is located more centrally than the walls.

Whichever way you look at it the seismic resisting system in this building is highly 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.

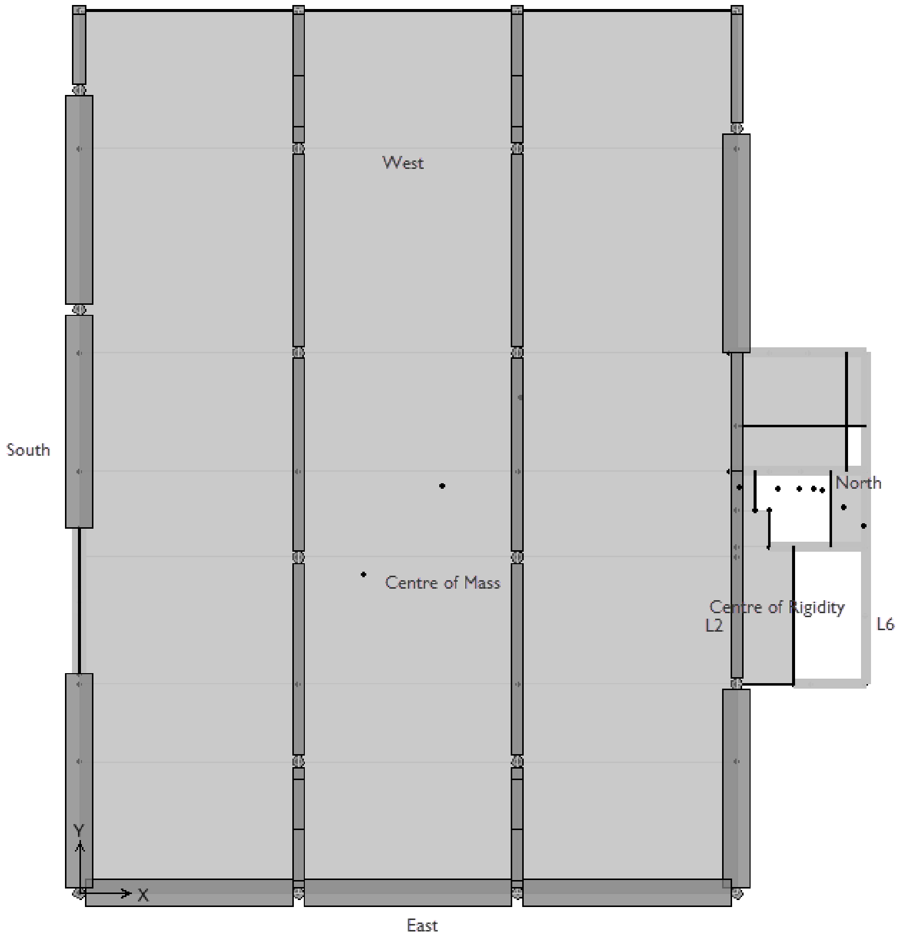


Figure 85 - Centre of Mass and Centres of Rigidity for each Floor (ETABS model 1a – Concrete Shear Walls only as seismic system)

Note – the centre of rigidity is close to the centre of mass in the east-west direction, but highly eccentric from the centre of mass in the north-south direction (towards the north).

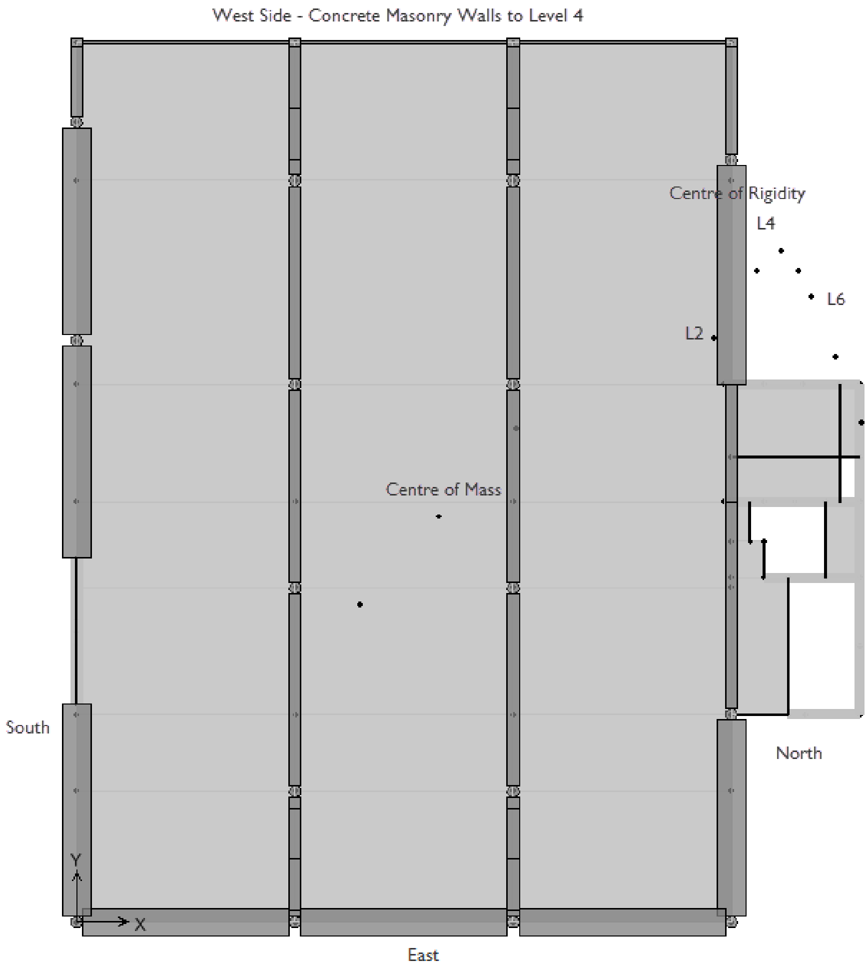


Figure 86 - Centre of Mass and Centres of Rigidity for each Floor (ETABS model 1b – Concrete Shear Walls and Concrete Masonry Walls as seismic system)

Note – the centre of rigidity is highly eccentric from the centre of mass in both directions, with a major vertical irregularity and transfer diaphragm being introduced at level 4 due to the participation of the west side masonry walls below that level.

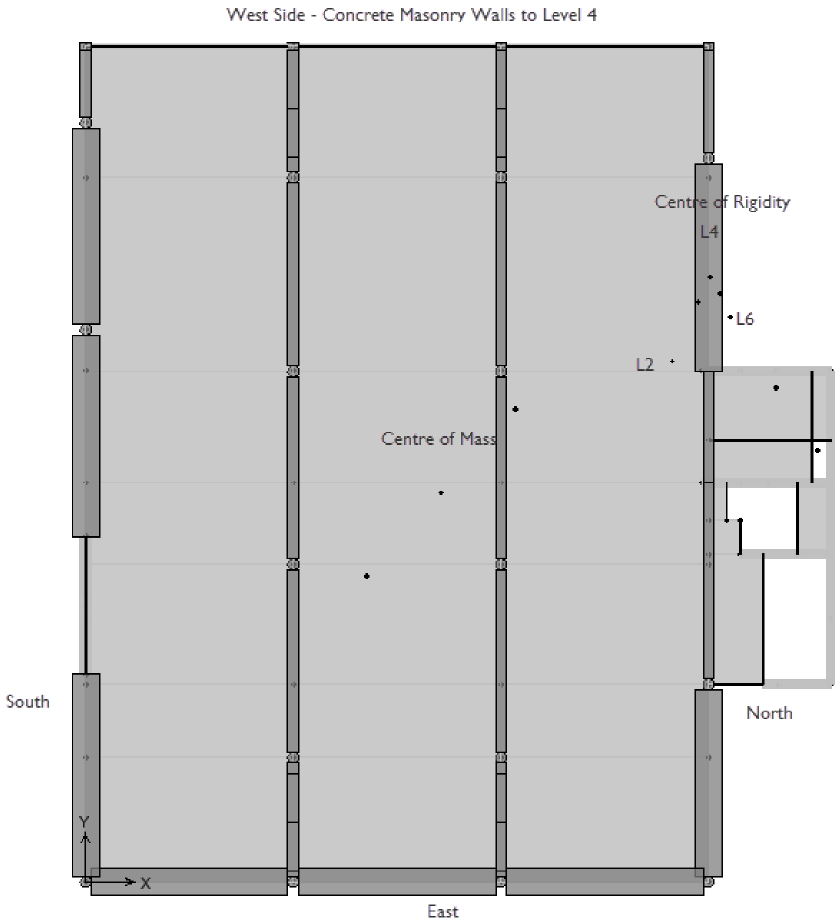


Figure 87 - Centre of Mass and Centres of Rigidity for each Floor (ETABS model 1c - Concrete Shear Walls and Concrete Masonry Walls and Concrete Frame)

The effect of this torsional behaviour on the seismic displacements and column actions can be seen in the following plots of column shear forces for earthquake shaking in the east-west direction. Figure 88Error! Reference source not found. is a plot of shear force on the east-west axis of the columns. It can be seen that columns nearer to the south side and nearer to the top of the building are subject to higher shear forces (and corresponding bending moments). This is because they are furthest from the centre of rigidity and so experience more seismic drift due to torsion, and because the frame takes a bigger proportion of the total storey shear compared with the walls nearer the top of the building.

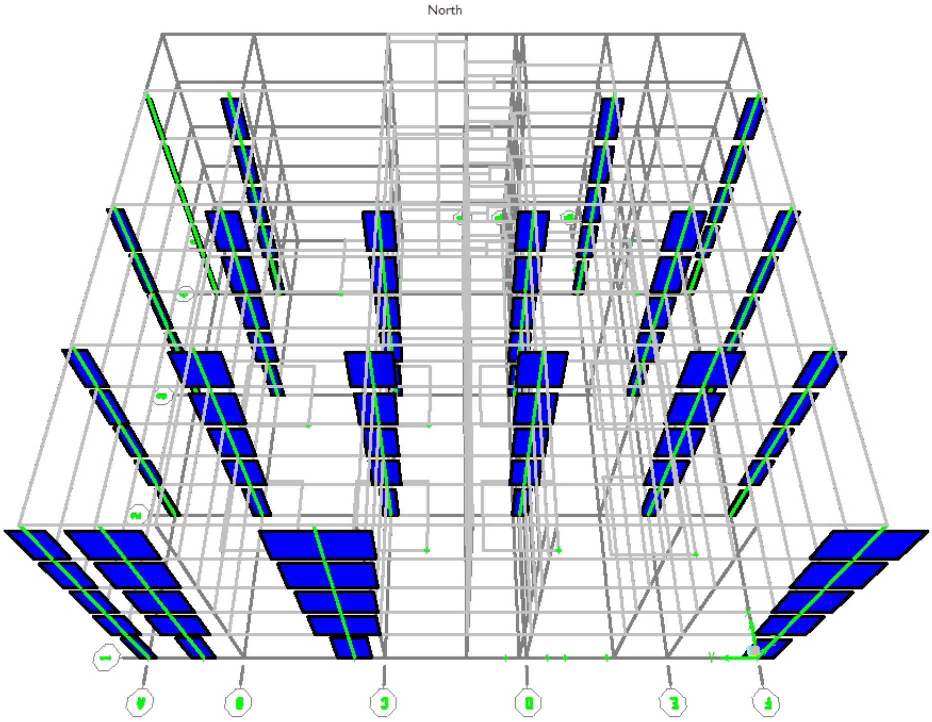


Figure 88 - Plot of column shear force on east-west column axis for earthquake shaking in east-west direction

Figure 90 is a plot of shear force on the north-south axis of the columns, for earthquake shaking in the east-west direction. The north-south shear forces shown here arise purely because of torsion.

The internal columns do not experience shear forces in the north-south direction because the floor beams run east-west. Similarly the columns at the west side above level 4 do not experience shear forces in the north-south direction because there are no beams at the west side above level 4.

The columns shear forces at the west side should be ignored because they are due to interaction of the columns with the concrete masonry walls, an effect of the way the structure has been modelled rather than a real effect.

It can be seen that columns at the east side and nearer to the top of the building are subject to the highest shear forces (and corresponding bending moments). This is because they are furthest from the centre of rigidity and so experience more seismic drift due to torsion, and because the frame takes a bigger proportion of the total storey shear compared with the walls nearer the top of the building. The magnitude of the shear forces in this north-south direction are of a similar order to shear forces in the east-west direction, the direction of earthquake shaking modelled. The effects of torsion are therefore very significant. The columns at the east side of the building form part of a two-way moment frame and so they experience concurrent actions in both orthogonal directions.

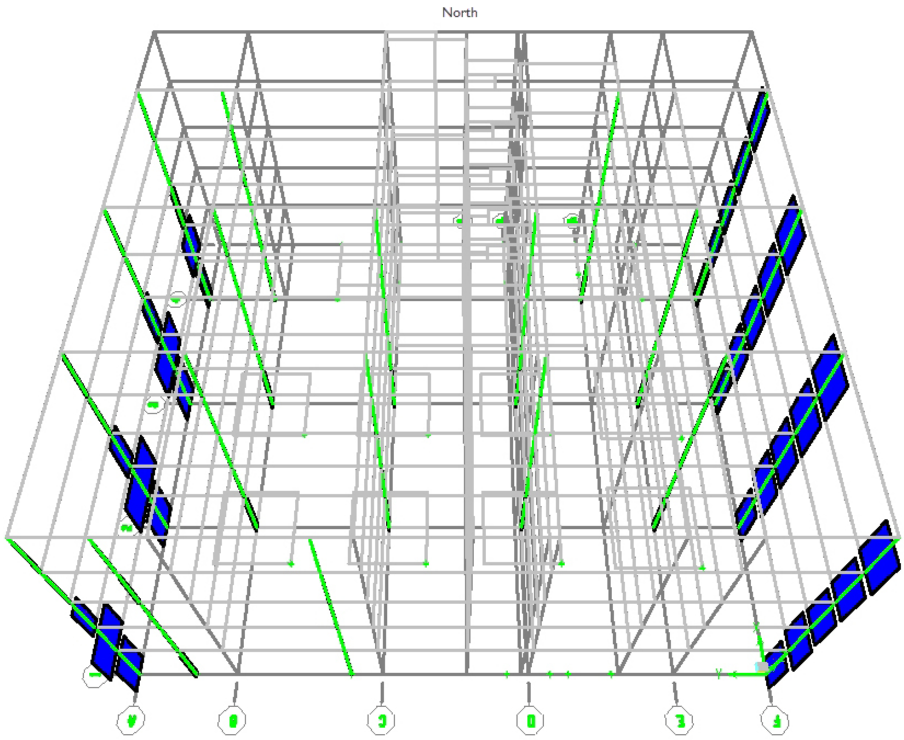


Figure 89 - Plot of column shear force on north-south column axis for earthquake shaking in east-west direction

C. CODE CHECKS AT THE TIME OF THE 22ND FEBRUARY 2011 AFTERSHOCK

i. General

Code checks have been carried out for the structure as-built at the time of the 22 February aftershock, against the design Standards that were applicable at the time the building was initially constructed in 1986. These Standards include NZS4203:1984 with Amendment 3 dated December 1984 and NZS3101:1982 (check relevant Amendment #).

The focus here is on the components that are believed to have contributed to the collapse in the 22 February aftershock, based on the physical evidence and on eye witness accounts.

The concrete shear walls exhibited some damage, but not sufficient to have initiated the collapse.

Eye witnesses reported that the building collapsed quite quickly and relatively uniformly across the site. This would not be consistent with the collapse being initiated by the failure of the floor or the floor beams, which would be more localised, at least initially.

The floor beams, especially the perimeter beams were found to be relatively intact after the collapse. They may have contributed to the collapse by placing demands on the beam column joint zone.

The columns and the connections of floor diaphragms to shear walls are therefore the main focus of the following Code checks.

ii. Column Flexure and Axial Load

The internal columns at grid intersections C2, D2, C3 and D3, are all highly loaded under gravity actions. This is shown in

Concrete tests on the available column remnants showed lower strengths than expected. The actual concrete strengths for the most highly loaded columns are uncertain and so this remains as a potential initiator of the collapse, taking into account the additional demands that would have been placed on the internal columns during the earthquake.

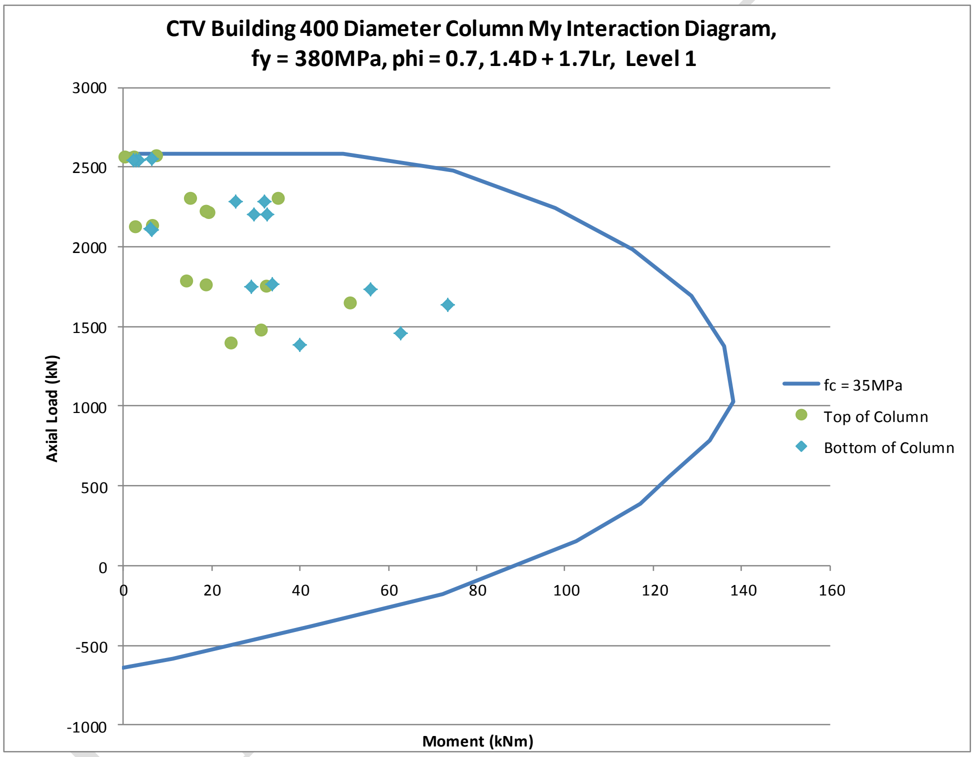


Figure 90 - Column Chart for Factored Gravity Load 1.4D + 1.7 Lr

However, as shown in

Figure 91 and Figure 92 the perimeter columns on the south and east sides of the building experience higher seismic forces and so these are another focus.

The light gauge and widely spaced horizontal ties in the columns do not comply with the additional seismic requirements of the Standard NZS3101 if inelastic demand is required in them. From the ARCL calculations the concrete frame was designed as a secondary structural frame, not forming part of the primary seismic force resisting system. The columns of the frame would be classed as Group 2 secondary elements as defined in NZS3101:1982 Clause 5.5.14.1(b).

Clause 3.5.14.3 is relevant and that clause states 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 v**Δ**, specified in NZS4203, and the assumptions of elastic behaviour".

And under (b) "Additional requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below v**Δ**".

The level of deformation v**Δ** to be considered here would be the "computed deformations" defined in Clause 3.8.1 of NZS4203:1984. These are the deformations resulting from the spectral modal analysis multiplied by a factor K/(SM) = 2.75.

The analysis intended here is an analysis with the primary structure only. For the CTV building the original design intent would be with the concrete walls only as the primary system. However, as-built the masonry walls at the west side, which were not separated, should also be included.

For convenience, the secondary frame actions based on the computed deformations taken directly from ETABS model 1c have been used, that is the model with the concrete shear walls, the masonry walls and the concrete frame all included. The computed deformations from this model are less than for models 1a or 1b and so the column actions will have been underestimated here as far as the Code is concerned. However, despite this underestimation of the required column actions, the columns have been found to have actions greater than the elastic capacity of the column and so would have needed to develop and be detailed for inelastic behaviour to cope with these demands. This is shown in the following interaction charts, where the data points outside the curves represent exceedance of elastic behavioural limits.

The flexure and axial load charts below are for columns at Levels 3, 4 and 5 in the building, which have been shown above to be critical as far as seismic actions are concerned.

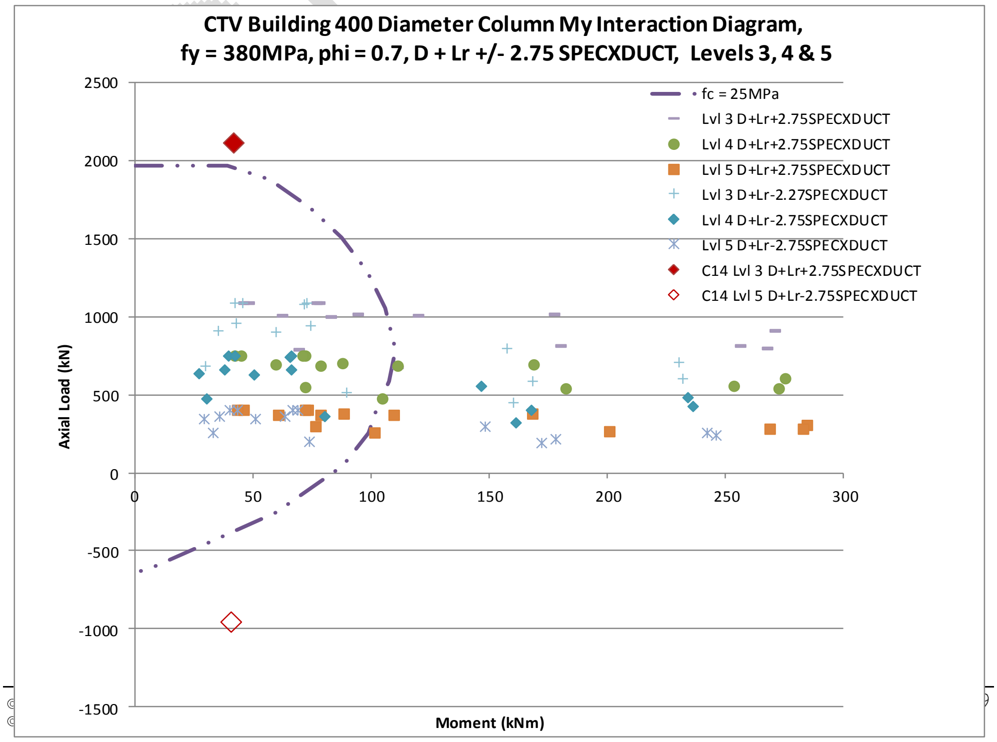


Figure 91 - Column Interaction Chart showing demands greater than elastic limits on columns with Standard NZ3101:192 Clauses 3.5.14(a) and (b) (for earthquake shaking in north-south direction)

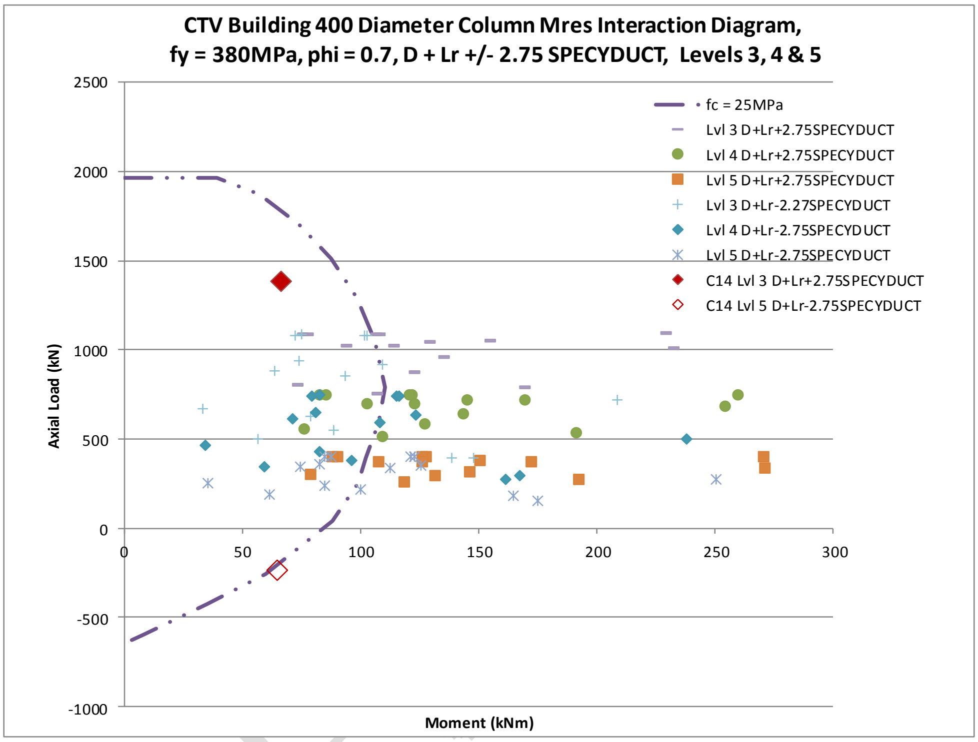


Figure 92 - Column Interaction Chart showing columns with demands greater than elastic limits under NZS3101:1982 Clauses 3.5.14(a) and (b) (for earthquake shaking in east-west direction)

Note –Data points within curve are not subject to inelastic demand, those outside would be.

iii. Column Shear

Shear demands on the columns reduces significantly as inelastic deformation occurs at the columns heads and bases and the columns tend to pin ended column behaviour.

With a measured hinging length of 400 mm only small amounts of flexural tensile yielding of reinforcing will result in significant rotations for small changes in bending resistance in column connections to beams above and the floor below.

APPENDIX G- REFERENCES

Hyland, C. W. K. (2011). CTV Building: Site Examination and Materials Tests. Auckland, Hyland Fatigue + Earthquake Engineering.

Pradano, M. H., Y. Goto, et al. (2011). Strong Ground Motion by September 30, 2009 Pariaman Earthquake and Damage to Large Scale Buildings. 2nd International Conference on Earthquake Engineering and Disaster Mitigation (ICEEDM-II 2011), Surabaya, Indonesia, Institut Teknologi Sepuluh Nopember / Indonesian Earthquake Engineering Association.