Hyland Consultants Ltd
StuctureSmith Ltd

CTV Building Collapse Investigation

Peer Review

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Overview

- Overall report composed of many parts at different levels of detail
  - Extensive Executive Summary
  - Main Body
  - Appendices
  - Supplementary Reports
- Investigation documented extensive data and information
- Investigation was sometimes difficult to follow due to “layering” of information
- This oral report will concentrate on conclusions rather than lack of clarity of the report.

Issues to be covered

- Code requirements for ductility in gravity frame
- Prediction of column “failure” vs. collapse mechanism
- Proposed collapse mechanism
- Primary cause of excessive drifts
- Other issues
  - Block wall on line A
  - Spandrel interaction
  - Elastic and nonlinear analysis
  - Vertical ground motions
  - Exceptionally strong February motions
- Conclusions and Recommendations

Code requirements for ductility in gravity frame

- NZS 3101: 1982 Controlling Code

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Applicable provisions

Secondary Structural Elements

3.5.14 *Secondary structural elements*

3.5.14.1 Secondary elements are those which do not form part of the primary seismic force resisting system, or are assumed not to form such a part and are therefore not necessary for the survival of the building as a whole under seismically induced lateral loading, but which are subjected to loads due to accelerations transmitted to them, or due to deformations of the structure as a whole. These are classified as follows:

(a) Elements of Group 1 are those which are subjected to inertia loading but which, by virtue of their detailed separations, are not subjected to loading induced by the deformation of the supporting primary elements or secondary elements of Group 2

(b) Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to both inertia loadings, as for Group 1, and to loadings induced by the deformation of the primary elements.

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3.5.14.3 Group 2 elements shall be detailed to allow ductile behaviour and in accordance with the assumptions made in the analysis. For elements of Group 2:

(a) Additional seismic requirements of this Code need not be satisfied when the design loadings are derived from the imposed deformations $\nu\Delta$, specified in NZS 4203, and the assumptions of elastic behaviour

Condition 1

(b) Additional seismic requirements of this Code shall be met when plastic behaviour is assumed at levels of deformation below $\nu\Delta$

...

(f) Where elastic theory is applied in accordance with (e) for deformation corresponding to 0.5 $\nu\Delta$ or larger, the design and detailing requirements of Section 14 may be applied, but otherwise the additional seismic requirements of other sections shall apply.

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Condition 3

Summary of Requirements

- Condition 1
  - If structure is elastic under ultimate drifts, no additional detailing requirements apply
- Condition 2
  - If structure is elastic for 50% ultimate drifts or more, Chapter 14 for Structures of Limited Ductility can be applied.
- Condition 3
  - If structure is elastic for less than 50% of ultimate drifts, full ductility provisions must be applied.

Test of Secondary Structure

Table 1. Consolidation and Simplification of Table 13 and 14 of Appendix F

A	B	C	D	E	F	G	H	I
	Column C-1 East West Drifts				Column F-2 North South Drifts			
	EW EQ				NS EQ			
Level	Elastic Deform. Limit		NZS 4203: 1984 K/SM=2.75		Elastic Deform. Limit		NZS 4203: 1984 K/SM=2.75	
L5-L6	0.65%		0.80%		0.62%		0.64%	
L4	0.73%		0.79%		0.73%		0.64%	
L3	0.64%		0.72%		0.69%		0.61%	
L2	0.58%		0.59%		0.61%		0.56%	
L1	0.50%		0.35%		0.55%		0.42%	

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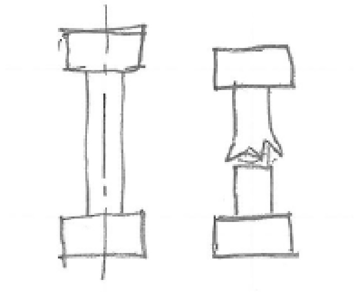
Level	Elastic Deform. Limit	Approx dependable capacity (Col B/1.4)	NZS 4203: 1984 K/SM=2.75	Col D/2	Elastic Deform. Limit	Approx dependable capacity (Col E/1.4)	NZS 4203: 1984 K/SM=2.75	Col G/2
L5-L6	0.65%	0.46%	0.80%	0.40%	0.62%	0.44%	0.64%	0.32%
L4	0.73%	0.52%	0.79%	0.40%	0.73%	0.52%	0.64%	0.32%
L3	0.64%	0.46%	0.72%	0.36%	0.69%	0.49%	0.61%	0.31%
L2	0.58%	0.41%	0.59%	0.25%	0.61%	0.44%	0.56%	0.28%
L1	0.50%	0.36%	0.35%	0.18%	0.55%	0.39%	0.42%	0.21%

Conclusions regarding gravity frame

- The upper four floors in the EW oriented frames and the highest floor of the NS oriented frames were required to be detailed in accordance with Chapter 14, Limited ductility.
- This detailing was not apparent in the design
- The drift capacity of the gravity frame designed according to the Limited Ductility Provisions has not been estimated.

Column Failure Modes

Column Failure Modes



Squash Mode: Short stocky columns crush (unusual). Vertical movement

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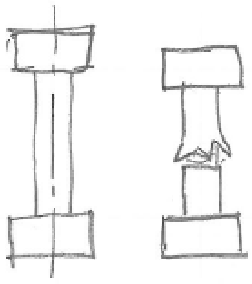

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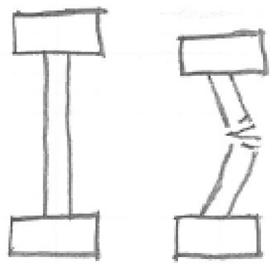

Buckling Mode: Tall slender columns buckle. Essentially vertical movement

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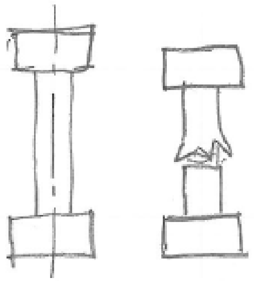

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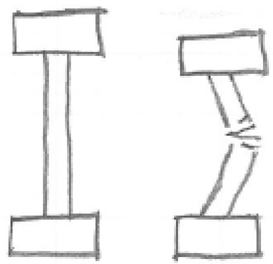

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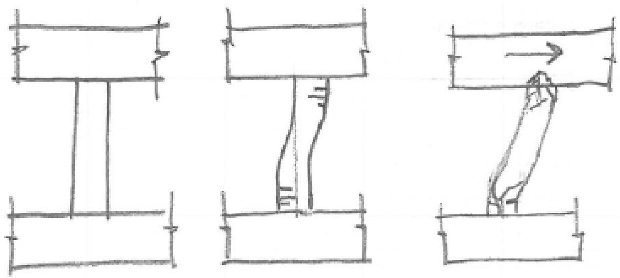
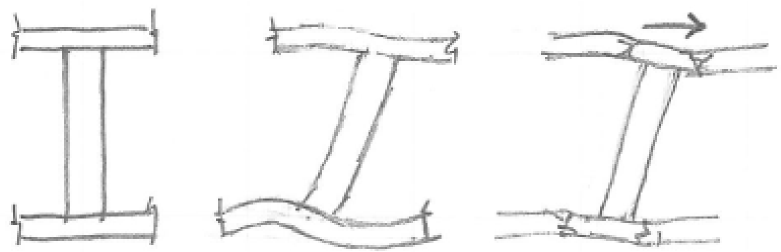


Strong beam/weak column



Strong column/weak beam

Sidesway Mode: Column stays relatively intact. Floor collapses to side

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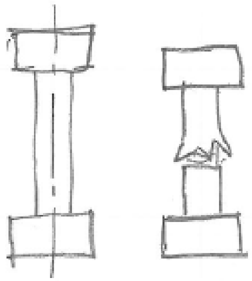

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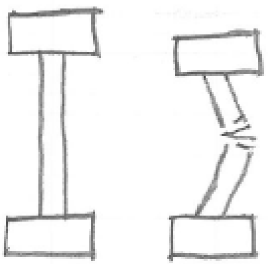

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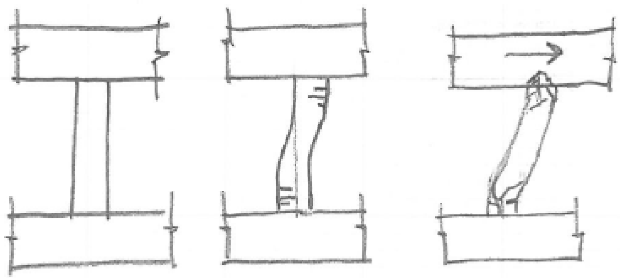

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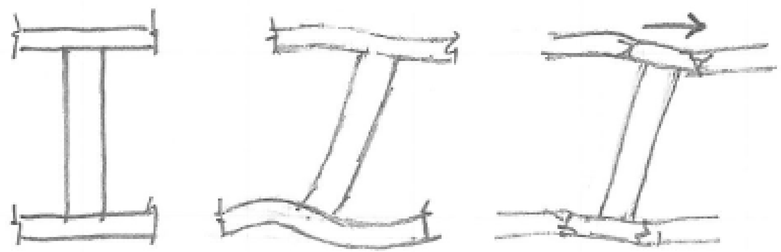

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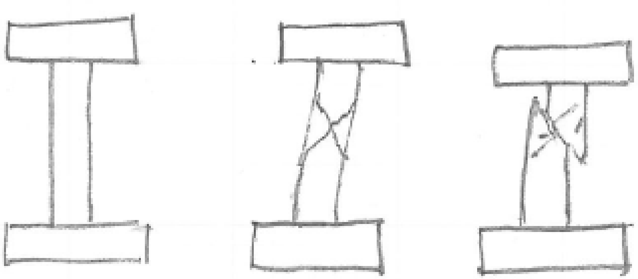
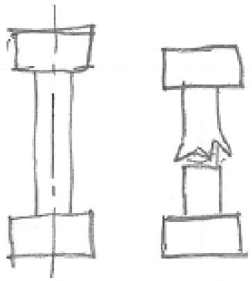


Column Shear Failure: Essentially vertical movement

Note: Flexure-shear mode not shown

Column Failure Modes

Report defines column failure by setting strain limits, which implies strong beam/weak column sidesway mode



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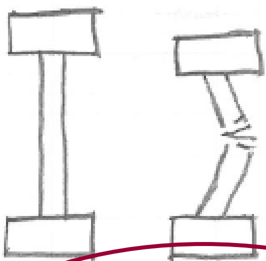

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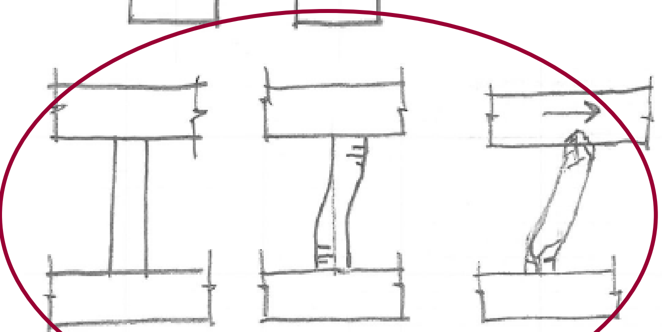

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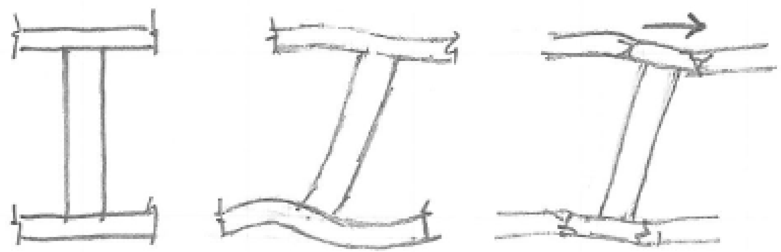

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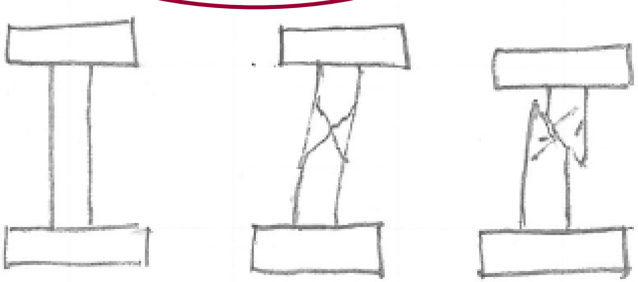

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Building Collapse Mechanisms

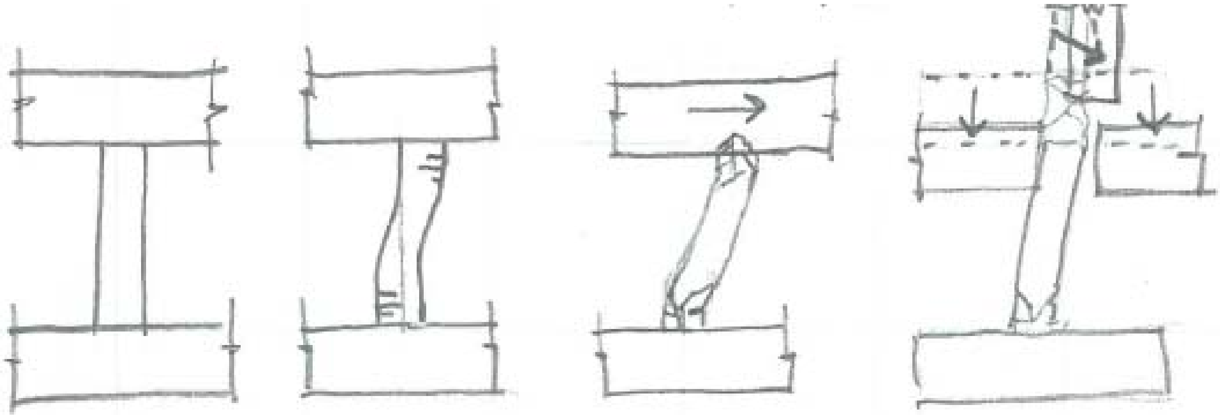
- Studies in the US ([4], [5]) are attempting to identify the <u>most</u> collapse prone older concrete buildings to encourage mitigation. Global collapse mechanisms are identified.
- Local exceedance of "acceptable" strain levels may not be sufficient to cause loss of vertical load carrying ability and collapse, particularly when independent lateral stability is provided (e.g. shear walls).
- Site debris and eye-witness accounts suggest predominantly vertical collapse.
- Vertical collapse modes of squash, buckling, or shear failure not evident and not indicated by calculation.

[4] ATC 78, *Identification and Mitigation of Collapse Prone Older Concrete Buildings*, Applied Technology Council, funded by FEMA, in progress

[5] ATC 95, *Development of a Collapse Indicator Methodology for Existing Reinforced Concrete Buildings*, NEHRP Joint Venture, funded by NIST, in progress.

Most likely collapse mechanism

- Sidesway not evident
- Column "failure" extending into beam-column joints would create significant local instability, loss of overall structural integrity, and loss of vertical support of at least some of the beams, and a mostly vertical collapse.



Report comments concerning beam-column joints

On page 112 of Chapter 9, Design, Construction, and Standards Issues, it is stated,

> It is conceivable that the lack of continuity steel through the beam column joint meant that the beams were unable to cope with much loss of vertical support as isolated columns were damaged and failed. Instead of being able to redistribute some of the load along the frames, the beams may have pulled away from the columns, contributing to the progression of collapse.

However, post processing of the results of the NTHA, as discussed on page 65 of the Compusoft report, indicated that it was likely that many joints would fail. It is stated:

> The trends shown for the demand/capacity vs time of the beam column joints is similar to that exhibited by the hinge formation detailed in Section 10.1.3 above. It should be noted that the capacity of the beam-column joints is sensitive to variances in material strength, and axial load and the ductility demand of the adjacent beam hinges.

Other evidence of joint failure

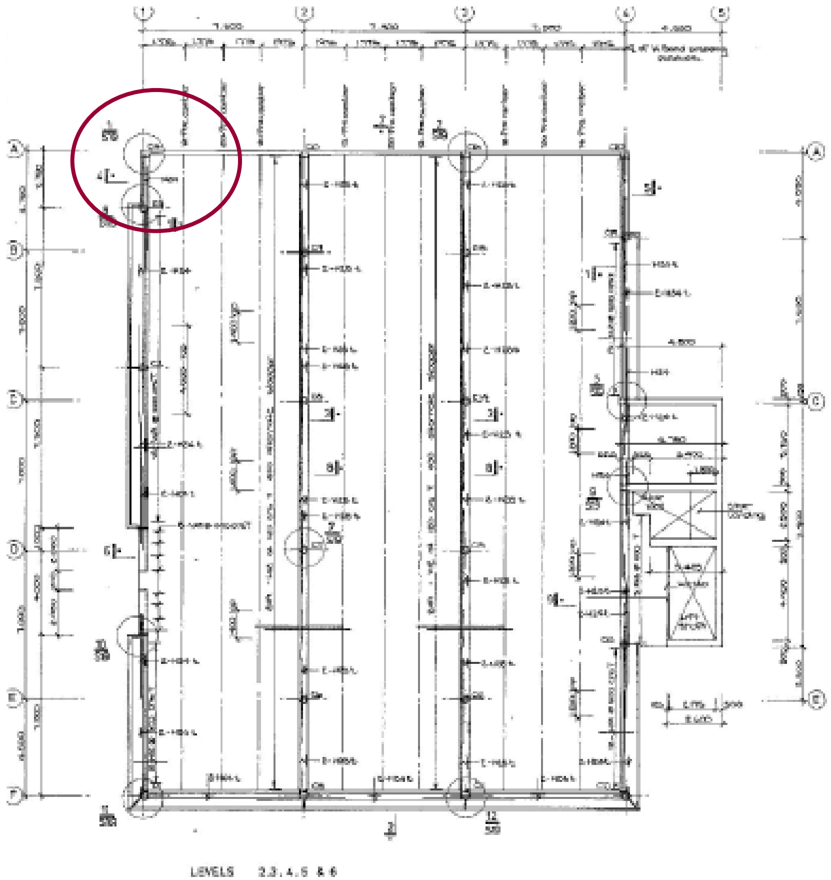
- Graham Frost CPEng, spent five days assisting the rescue and recovery at the site and sent a short summary of his observations to DBH:

  "The evidence found during the rescue and recovery efforts at the site suggests that the collapse mechanism (/initiation??) included the very brittle/non-ductile failure of the beam-column joints…

  "And while most beams survived the collapse intact (except for their ends), no intact beam-column joints were found…."

Other evidence of joint failure

- Eyewitness 16 from Appendix A of Report: On the outside of the building, at the lower level at the corner A-1.



Eyewitness 16

"I remember looking up and seeing the building pretty much right above my head, so it had obviously swayed from side to side. I threw my workmate off the machine and as I was jumping, I had to push myself out of the way of the falling corner pillar." (Southwest) "Just out of the corner of my eye I saw the concrete spit out the corner. The pillar came down and brought the machine down to the ground and buried the wheels. ."

He described seeing the column fracture. "It buckled out. It had cracked and the two bits held still by the steel had spat out, and obviously as the weight got too much, it broke and came down. This was in the middle of the column, between floors. It 'kicked out' in the direction of Les Mills. I

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Two story buckling of corner column?

I believe that his description of the buckling, "…the two bits held still by the steel had spat out…This was in the middle of the column, between floors." was actually a two story buckling of the column at 1A when the joint at level 4 had broken up and released the column outward. There was no reason for the column to buckle between floors, particularly here because the axial loading was low and there was no possible spandrel interaction on either face of this column. But the description of the two bits held together by the steel sounds exactly like other columns at the site with rebar exposed in the joint region. This particular column was unique in the building (see Detail 1 on Sheet S19). It

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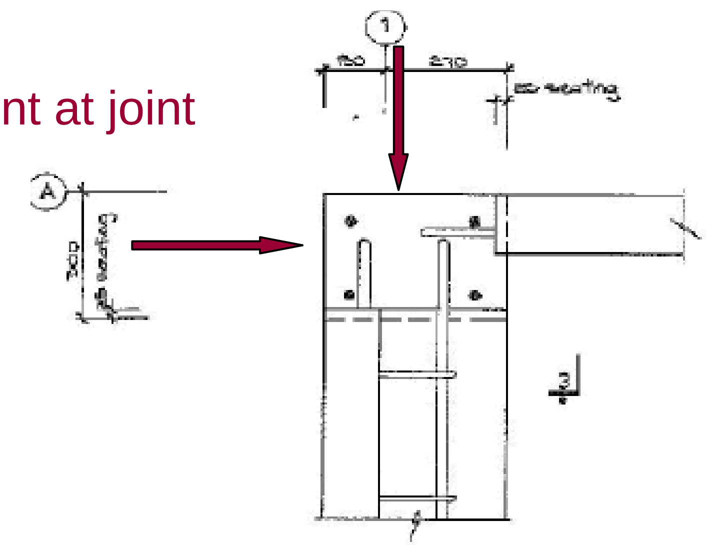
No confinement at joint



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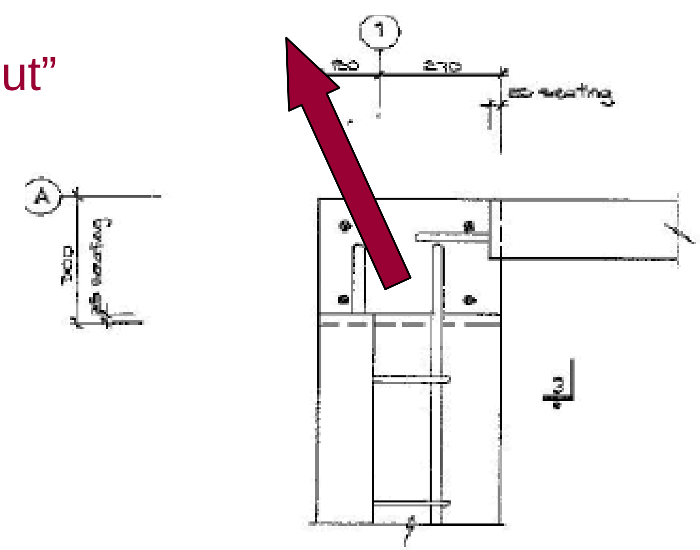
Joint "kicked out"

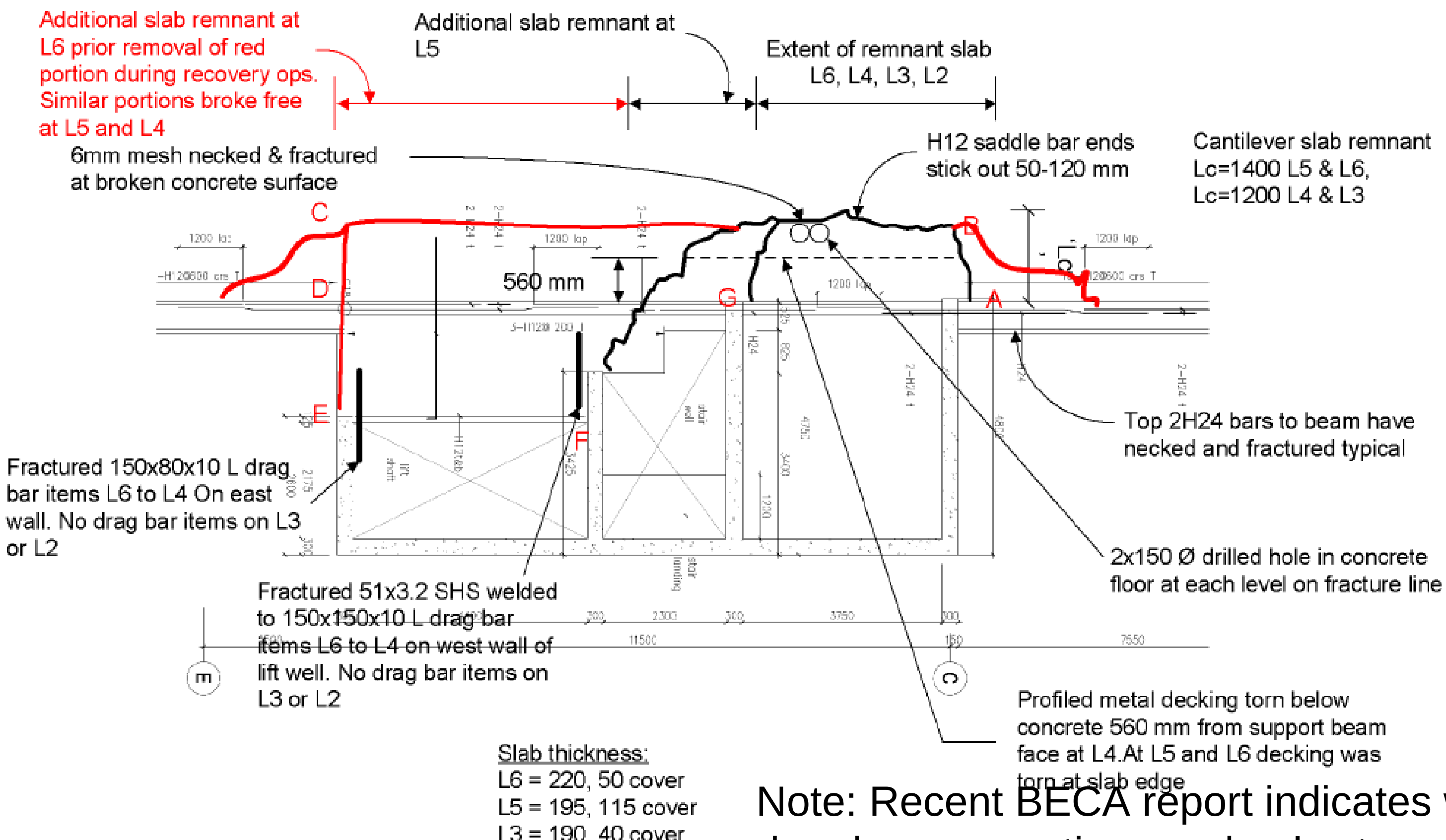


Importance of more specific collapse scenario

- Identification of predominant vulnerabilty:
  - If the columns had more confinement, but joints the same
    - the building probably still would have collapsed.
  - If the beam-column joint was improved both to provide minimal confinement and to better tie the beams to columns, but the columns were the same,
    - Perhaps collapse would have been partial or localized, particularly if lateral stability from the North Tower was maintained.
- The predominant vulnerability is needed to find other vulnerable buildings in New Zealand (and elsewhere).

Primary cause of excessive drift in columns

- Report suggests that upper level slabs failed from vertical movement, not tension, indicating interior collapse prior to disconnection:



Note: Recent BECA report indicates weaker drag bar connections and only strengthens this argument.

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- Disconnection at lower levels considered unlikely due to slab positions in Figure 165, again indicating collapse away from the face of the tower at line 4

Figure 165 - North Core slabs leaning against the North Core showing that their collapse occurred after collapse of the Line 3 frame

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- Disconnection at lower levels considered unlikely due to slab positions in Figure 165, again indicating collapse away from the face of the tower at line 4
- However, the configuration of slabs in Figure 165 can be explained in two ways:
  - Slab at level 3 disconnects leading to large drifts in middle floors that initiate collapse. Slab at level 3 also collapses vertically but is arrested by the slab at level 2. Floors proceed to collapse ending in configuration shown in Figure 165.
  - Slab at level 3 disconnects but does not completely lose its gravity support at line 4 (face of tower). Middle floors suffer large drifts initiating collapse, eventually leading to configuration in Figure 165.

Additional argument for slab disconnection

- North tower relatively undamaged, indicating even less drift than anticipated by code design.

Table 2. Maximum estimated NS drifts in NS walls of north tower and "failure" drifts

A	B	C	D	E	F
Level	NS drifts at North Tower				Col F 2
	Code NS drifts from original calcs	Code NS drift from report (Table 14)	Approx NS drifts from Darfield (Figures 125, 126)	Maximum estimated drift consistent with Feb damage level	NS Failure Drifts
L5-L6	0.80%	0.60%	0.90%	0.50%	1.58%
L4	0.70%	0.60%	0.80%	0.50%	1.45%
L3	0.60%	0.60%	0.80%	0.40%	1.30%
L2	0.40%	0.50%	0.75%	0.30%	1.20%
L1	0.20%	0.30%	0.60%	0.20%	1.15%

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L4	0.70%	0.60%	0.80%	0.50%	1.45%
L3	0.60%	0.60%	0.80%	0.40%	1.30%
L2	0.40%	0.50%	0.75%	0.30%	1.20%
L1	0.20%	0.30%	0.60%	0.20%	1.15%

- Tower drifts estimated from damage patterns are completely inconsistent with column failure drifts, even with torsion.

Additional argument for slab disconnection

- North tower relatively undamaged, indicating even less drift than anticipated by code design.

Table 2. Maximum estimated NS drifts in NS walls of north tower and "failure" drifts

A	B	C	D	E	F
Level	NS drifts at North Tower				Col F 2
	Code NS drifts from original calcs	Code NS drift from report (Table 14)	Approx NS drifts from Darfield (Figures 125, 126)	Maximum estimated drift consistent with Feb damage level	NS Failure Drifts
L5-L6	0.80%	0.60%	0.90%	0.50%	1.58%
L4	0.70%	0.60%	0.80%	0.50%	1.45%
L3	0.60%	0.60%	0.80%	0.40%	1.30%
L2	0.40%	0.50%	0.75%	0.30%	1.20%
L1	0.20%	0.30%	0.60%	0.20%	1.15%

- Tower drifts estimated from damage patterns are completely inconsistent with column failure drifts, even with torsion.

- Tower disconnected at lower level early in shaking, initiating collapse before significant lateral load was tranmitted to tower.

Other Issues

- Concrete Block Wall on line A:
  - Clearly intended to be isolated, but would have interacted at large drifts even if built perfectly.
  - However, severe torsion created by significant early interaction would put large demands on NS wall of tower—and weak connections. Not indicated by damage to tower.
- Spandrel Interaction:
  - Systematic evidence not included in report to support significance of this interaction.

Other Issues

- Elastic Response Spectra Analysis
  - ERSA performed for code defined spectra useful to check original design
  - The purpose of ERSA performed using spectra from nearby shaking is unclear.
    - Linear response comparisons can be seen from the spectra
    - Structure was highly nonlinear so analysis not very useful.
- Nonlinear Time History Analysis
  - Insights from NLTH normally useful.
  - In this case, much more complicated model would be required to reasonably predict response and collapse
    - Degrading column hinges and variation with vertical load
    - Explicit modeling of joints
    - Various failure modes in diaphragms.
    - Extensive calibration between input, predictions, and actual response.
  - Costs and benefits of more complex models must be weighed.

Other Issues

- Vertical ground motions
  - Effect of vertical ground motions not directly considered in nonlinear behavior from lateral loads
  - Post processing indicates non-concurrence of maxima
  - However, the report concludes that axial loads from vertical ground motions could have reduced drift capacity of columns by up to 25%.
- Exceptionally Intense Lateral Shaking
  - Elastic response spectra indicate shaking considerably more strong than assigned to the CBD by code
  - The CTV building survived the Sept shaking with little apparent damage
  - The collapse in February indicates an extreme brittleness in the structure, triggered at some intensity between those experienced in Sept, 2011 and Feb. 2012.

Conclusions

- The exact set of deformations that instigated the collapse will never be known, even with more extensive modeling, due to contributions that can only be estimated.
  - Exact ground motion demand
  - Drifts at which joint would degrade
  - Strength and stiffness of diaphragm and its connection to the tower
  - The extent of interaction of the block wall on line A.
  - The effect of vertical ground motions on critical components.
- Judgment indicates that brittle gravity frames and poor diaphragm and connections were most significant.

Lessons learned

1. Brittle gravity frames
    a. It appears that for this building, if NZS 3101:1982, paragraph 3.5.14 was checked, the solution would have resulted in a requirement to apply the requirements of only Chapter 14 Limited Ductility. I have not evaluated the gravity system that would have resulted from such an application, and, in fact, the detail of the requirements may be open to interpretation. I recommend that designs of this era be reviewed to see if this requirement would commonly be triggered, and if so, whether the resulting deformation limits would be adequate.
    b. The configuration of the beam-column joints in this building are primarily a result of the use of precast shell beams and starter beams. The use of precast in this way in this era may also be cause to require review of drawings.

Lessons learned

2. Diaphragm issues
    a. Potential issues with the use of relatively thin toppings with mesh reinforcing have been highlighted in several buildings.
    b. The lack of collectors to the north tower has been discussed at length. It is unclear if this design was common at the time and something that needs systematic checking. However, I believe several other buildings of different eras have been discovered in Christchurch that have incomplete diaphragm designs or lack of collectors. The state of the practice over the last 25 years in this regard should be established to better direct the investigation of older buildings.
    c. The adequacy of diaphragm design forces should also be reviewed.

Lessons learned

3. Interaction of "nonstructural" walls or other elements.
    a. The construction details of the block wall on Line A had little tolerance for error and even if constructed perfectly may not have sufficient clearances to prevent interaction that would not be considered in design.
    b. Similarly, the precast spandrel beams also may have interacted with structural response.

Additional Recommendation

I also recommend reviewing current procedures for evaluating the adequacy of drift tolerance for gravity frames. Several aspects of this procedure need review to assure evaluations identify dangerous conditions:

- Engineering modeling assumptions that lead to drift demands
- The possible effects of vertical accelerations on brittle components.
- The need for a multiplier on ULS drifts to establish evaluation drift demands. Such a multiplier would essentially set the rarity of ground motions for which collapse should be prevented. This is a policy issue that should be established with community-wide input.
- Engineering acceptability criteria for drift in older concrete gravity frames of various configurations