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Justice Mark Cooper, Chair
Royal Commission of Inquiry into Building Failure Caused by the Canterbury Earthquakes
PO Box 14053
Christchurch Airport
Christchurch 8544
New Zealand

Subject: **Peer Review**
CTV Building Collapse Investigation dated 25 January, 2012
by Hyland Consultants, Ltd, and StructureSmith Ltd.

Dear Justice Cooper:

In accordance with your request, I have reviewed the subject investigation. The investigation includes extension documentation that consists of a three part main report and several supplementary reports. There are also several other documents either referenced in the report or of obvious pertinence to the investigation. The reports and other documents that were available for this review are listed below.

CTV Building Collapse Investigation (Hyland Consultants Ltd and StructureSmith Ltd, 25 Jan 2012))
Part 1, 2, and 3
Canterbury Television Building Site Examination and Materials Test (Hyland Consultants Ltd, 16 Jan 2012))
CTV Building Geotechnical Advice (Tonkin & Taylor, July 2011))
Non-linear Seismic Analysis Report (Compusoft, February, 2012)
Analysis of the CTV Floor Diaphragm Adequacy (Prof. Charles Clifton, November, 2011)
Collapse of the CTV Building in Christchurch During the Seismic Event on 22 February, 2011 (Graham Frost, CPEng, 3 March, 2011)
Office Building, 249 Madras Street: Structural Drawings, 40 pages (Alan Reay Consultants, August, 1986)
Office Building, Madras Street, Structural Calculations, pp G1-G79, S1-S57 (Alan Reay, Consulting Engineer)
Structural Report, Office Building, 249 Madras Street (Holmes Consulting Group, January, 1990)

The primary focus of this review was the three part main report. The other documents were not reviewed in detail but used as reference when appropriate. Documents so referenced are not repeated in the reference list. This review is based almost totally on the documents available for review. I have not performed any significant independent calculations, nor did I ever directly view the collapse debris.

It is my understanding that the primary governing codes for this building were NZS4203:1984 and NZS3101:1982.

The three part main report is organized in logical sections as follows:

1 Introduction
2 Investigation Methodology
3 Description of the CTV Building
4 Earthquake and Other Effects Prior to 22 February 2011
5 Collapse on 22 February 2011
6 Eyewitness Accounts
7 Examination of the Collapsed Building
8 Collapse Scenario Evaluation
9 Design, Construction and Standards Issues
10 Conclusions
11 Recommendations

As indicated by the extensive documentation, the thoroughness and level of effort of the investigation is impressive. Section 6, Eyewitness Reports, and more specifically the backup Appendix A, Eyewitness Summaries, show a high attention to detail in an attempt to 'mine" all available information that could lead to a better understanding of the collapse. Similarly, Section 7, Examination of the Collapsed Building, Appendix B, Photos of Collapse Building, Appendix C, Summary of Site Examination and Materials Testing Reports, and the standalone report, "CTV Building Site Examination and Material Tests" indicate an understanding of the perishable nature of this data and the need to collect as much information as possible. Both of these sections contain data that are used in the report to help back-up the proposed most likely collapse initiation. This was Scenario 1 as listed on page 94 and diagramed in Figure 19 and 20 of the Executive Summary--Column failure on Line F or Line 1 caused by excess drift.

The 11 sections of the main report are preceded by an Executive Summary of unusual length and detail (38 pages). In addition, several of the main sections are backed up by extensive appendices, some of which in turn are backed up by separate stand-alone reports. This "nested" approach was probably decided upon to summarize and simplify the large amount of information gathered and data generated for the investigation. However, this organization often results in important explanations and/or descriptions appearing in four places at different levels of completeness and different levels of clarity. (The four locations include the executive summary, main sections, appendices, and stand-alone reports. Section 5 of the DBH Panel Report represents a fifth location, but that report is not discussed here.) This has made the overall investigation more difficult to follow and complete justification of important conclusions difficult to piece together.

This review is organized broadly into six sections. First will be a discussion of the issue of column (or frame) ductility as required by the applicable design code. Second is a discussion of the use of the parameters that identify the "failure" of individual columns as they relate to global building collapse. I will then propose consideration of a new collapse scenario, not revolutionarily different than those described in the report, but one that may suggest different critical deficiencies. The collapse scenario also needs a rationalization of the large drifts needed to fail column and such a scenario is also given. These four sections include comments and opinions on a broad cross section of the report. However, I also have added a section that contains a few isolated comments on specific report issues. I conclude the review with a summary of key conclusions and recommendations.

Gravity Frame Ductility as Required by Code

This issue is discussed several places in the overall documentation, but in my opinion is never very clear. The first location is on page 20 of the Executive Summary where some of the general intent of NZS 4203:1984 is stated: "…required that the building as a whole, and all of its elements that resist seismic forces or movements, or that in case of failure are a risk to life shall be designed to possess ductility." However, as also is stated, it may be difficult to interpret the specific applications of this introductory code material to the CTV building except for the commentary suggestion that design of members not required by design to be part of the horizontal force resisting system "should include a check on column and beam adequacy at four or more times the distortion from the specified loading." But as is pointed out in the report, commentary language is informative and not mandatory.

The report then discusses the more specific requirements of NZS 3101:1982 covering secondary structural elements, including a requirement for secondary elements to elastically reach "55% of the ultimate drift." As can be seen in the 3101 provisions that are described below, the requirement of paragraph 3.5.14.3 (f) appears to be 50%. It is unclear if the 55% is a typo or if I have missed something, but the 55% appears several times in the reports. The report then gives a reference to demand/capacity tables (Table 1 and 2, page 26) that appear several pages later in a different subsection as follows: "This is the non-ductile detailing limit in Table 1 and Table 2." This reference is confusing first because the line in Table 1 and 2 is titled, "198<u>6</u> Non-ductile Detailing," (a typo meaning 1982 or 1984?), and secondly because it is not clear if this limit is related to the 50% rule for limited ductility or the fully elastic rule for no ductility. Lastly, the reference is of limited value because it is not clear if any other value in Table 1 and 2 is to be compared with the "1986 Non-ductile Detailing" value to determine the need for ductility.

This conclusion concerning the required ductility of the columns and frames, important for the identification of a potential code weakness that could affect many other buildings in New Zealand, is partially clarified in the main report, Section 9 (page 109), but the distinction is not made between full ductility and limited ductility. Later in that chapter (page 118), the following statement is made with further explanation:

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

Further clarification is finally offered in Appendix F, which includes a more complete explanation of the requirements from NZS 3101:1982, and an application of these requirements to the CTV Building. The important section 3.5.14 of NZS 3101 is paraphrased in Appendix F and is somewhat difficult to understand (although the section is quoted in full in Appendix I). For convenience, I have included the text of the important aspects of section 3.5.14 here:

The requirements for secondary elements are contained in paragraph 3.5.14 under Section 3.5 "Principals and requirements additional to 3.3 for the analysis and design of structures *subjected to seismic loading*." (emphasis mine) Paragraph 3.5.14 states:

> 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 loading, but which are subjected to loads due to accelerations transmitted to them, or due to deformation of the structure as a whole. These are classified as follows:
>
> (a)Elements of Group 1..[section not repeated here]
>
> (b)Elements of Group 2 are those which are not detailed for separation, and are therefore subjected to both inertia loading, as for Group 1, and to loading induced by the deformation of the primary elements.

Paragraph 3.5.14.3 then gives the design requirements for Group 2 elements:

> 3.5.14.3 Group 2 elements shall be detailed to allow ductile behavior 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 $\upsilon\Delta$, specified in NZS4203, and the assumption of elastic behaviour
>
> [(b), (c), (d) not repeated here.]
>
> (e) Analysis may be by any rational method, in accordance with the principles of elastic or plastic theory, or both. Elastic theory shall be used to at least the level of deformation corresponding to and compatible with one-

> quarter of the amplified deformation, υΔ, of the primary elements, as specified in NZS 4203
>
> (f) Where elastic theory is applied in accordance with (e) for deformation corresponding to 0.5 υΔ or larger, the design and detailing requirements of Section 14 may be applied, but otherwise the additional seismic requirements of other sections shall apply.

My understanding of these requirements is that under (a), if the Group 2 element stays elastic under the NZS 4203 projected deformation demand, no ductile detailing is required, and under (e), if the Group 2 elements stay elastic for 50% or more of NZS 4203 deformations, the detailing in Section 14, "Seismic Requirements for Structures of Limited Ductility" is required. Also under (e), "otherwise" (presumably for any limit of elasticity at less than 0.5 υΔ) the additional seismic requirements of other sections shall apply—full ductile detailing.

The report states that Table 13 and 14 of Appendix F present a comparison of drift demands calculated as per NZS 4203: 1984 and elastic deformation limits for two columns, at grid intersections C/1 and F/2. It is suggested that these drifts can be used to test detailing drift limits as per NZS 3101, discussed above. The comparison shows that the drift demand in the east-west direction exceeds the elastic limits from level 2 to 6 at C/1 and exceeds the limit in the north south direction at F/2 only at levels 5-6. The report concludes on page 257 that the columns at these locations "would therefore have been required to have been designed and detailed using the seismic design provisions of NZS 3101:1982.

However, the report is not specific about what detailing would be required. The governing cells of Table 13 and 14 have been reproduced below as Table 1. Table column B and D are from Table 13, and table columns F and H are from Table 14. I have added table columns C and E for EW EQ and G and I for NS EQ as simple proportions of the data in adjacent table columns for illustrative purposes.

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	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%

Although certain calculated drift demands exceed elastic limits (table column D compared with column B and table column H compared with F), 50% of calculated drift demands (table column E and I) do not. Therefore, in accordance with NZS 3101 3.5.14.3 (f), only Chapter 14, "Seismic Requirements for Structures of Limited Ductility" would be triggered. I assume the detailing requirement would apply to the *frames* , including columns, beams, and joints, in these locations rather than just the columns, as the report suggests. It is unclear what a design engineer would do when only the drifts at the top floors triggered this requirement. It would appear prudent to detail all floors to this requirement, but it is unclear what the standard of practice was at this time or what Councils would require.

The vagueness of these requirements is primarily due to lack of definition of the method to be used to establish the drifts at the elastic limit. The report points this out on page 256 (Appendix F) and suggests that dependable capacity may have been used at the time, primarily because it would be readily available. On the other hand, if use of dependable capacity yielded such low drift limits that a complete revision of the design of the gravity system was required, which in turn would imply loss of economy in the construction of the building, perhaps design engineers would have sought a more sophisticated measure. In the case of the CTV building, using the calculation performed for the report as contained in Table 1, I have estimated the elastic drift limits based on dependable capacity as the Elastic Deformation Limits divided by 1.4, as shown in Column C and G of Table 1. These reduced drift capacities still would not drop below the 50% drift demand limit as shown in Column E and I of Table 1, so Chapter 14, Limited Ductility, of NZS 3101: 1982 could still have been used.

However, it is clear from the reinforcing patterns shown on the structural drawings that the gravity framing in the CTV Building were not designed for lateral loads or displacements. In the investigative repot, the elastic drift limits were apparently established by analyzing column hinging. It is unclear how *elastic* limits would be established for the hooked embedment of the bottom beam bars at columns, or more importantly, for the beam-column joints. Due to the lack of confinement in the joints, particularly on the north and south faces of the interior joints of the beams on lines 2 and 3, it could be argued that the elastic limit would be considerably smaller than for the column hinges.

It should be noted that in NZS 3101:1982, there is also a Section 3.4 "Principals and requirements additional to 3.3 for members *not designed for seismic loading*."(emphasis mine) I suppose designers not familiar with the whole code could have gone into section 3.3 and 3.4 for the gravity frame thinking it was not to be designed for seismic loading and never looked through section 3.5 to find the applicable requirements for secondary structural elements.

Chapter 5, Reinforcement—Details, Anchorage, and Development, of NZS 3101 also has sections for "members not designed for seismic loading" (Section 5.4) and for "members designed for seismic loading" (Section 5.5). It is apparent that the column spiral design for the CTV Building was based on minimum requirement of Paragraph 7.4.7.2 which is referred to from Paragraph 5.4.2.1 applicable for "members *not* designed for seismic loading." The minimum required spiral reinforcement referred to in the report (page 110) is taken from the minimum shear reinforcing section of the code, which may not have been consulting in the design because it was assumed that shear was not a design parameter for gravity columns.

Conclusion
In summary, the gravity frames were apparently classified as members "not designed for seismic loading" (Section 3.4) in the design as opposed to Group 2 Secondary structural elements for "structures subjected to seismic loading" (Section 3.5). Although it is difficult to even apply NZS 3101, Section 3.5.14 to a frame that is not at all designed for lateral deformations, the code drift demand for the building as estimated in the investigation report and shown in Table 1 appears to indicate that for some frames and at some floors, detailing for intermediate ductility would have been required. However, elastic drift limits may be difficult to reliably determine for beam column joints that have no shear reinforcing, no confinement, have embedded hooks from bottom beam bars, and include interfaces with precast concrete. This limit, if determined, may be lower than the 50% of demand drift that allows intermediate ductility and a fully ductile frame would be required. A more logical design solution would be to find a balance between elastic drift limits imposed by detailing and the requirements of NZS 3101 that specify detailing based on elastic limits. This hypothetical condition is of interest only to speculate on the performance of the building under the extraordinary shaking on February 22, given appropriate application of requirements for Group 2 Secondary Structural Element in Section 3.5.14. This performance is not estimated in this review, but such a study is recommended.

"Failure" of Columns as it Relates to Building Collapse
Many sections of the investigative reports include discussion of the characteristics of the building columns. This is logical and justifiable since the behaviour of the columns are central to any collapse scenario. However, the term column "failure" was most commonly used as opposed to column "collapse." One of the few locations where the term collapse was used is on page 14 of the Executive Summary in the statement, "Established methods were used to estimate the capacity of critical columns to sustain the drift without collapse." More common usage is on page 18 where various collapse initiators are described as "Column failure on Line…" Similarly on page 91 of Section 8, Collapse Scenario Evaluation, the discussion of column drift capacity includes,

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

The Non-linear Seismic Analysis Report also focuses on individual column concrete strain limits rather than building-wide collapse mechanisms. For example, on page 41, it states, "The building displacement capacity being defined as the point at which at least three column hinges have exceeded their ultimate plastic rotation limit i.e. the rotation at which a concrete strain of 0.004 would have been reached."

The distinction between column "failures" due to exceedance of flexural rotation capacity and building collapse mechanisms is significant. For example, in the U.S., ASCE 41 [1] was initially considered overly conservative for concrete buildings because deformation limits were generally set at the point when columns (or frames) lost their ability to resist lateral loads. This was conservative for columns and/or frames that were only required to maintain gravity load capacity in buildings because they also had independent lateral force resisting systems (e.g. walls). Elwood, Moehle and others summarized a decade of research on the collapse behaviour of columns as defined by loss of gravity support, and introduced a major revision to ASCE 41 in 2007 [2]. When considering the performance goal of Collapse Prevention for the minimum preservation of life safety in the U. S., columns are considered successful as long as nominal floor heights are maintained, regardless of the appearance of the columns or their ability to resist lateral loads.

Ongoing studies of causes of collapse in older concrete buildings funded in the U.S. by FEMA and NIST [3], [4], has generated further analysis of building collapse mechanisms generated by failure of concrete columns as well as methods to analytically model the mechanisms. However, robust modeling of all of the various column failure types (e.g. squash mode, buckling mode, lateral sidesway, flexural shear mode, and shear mode) is not currently practical. For the non-technical reader a relatively simple description of these failure modes is contained in a Note at the end of this review.

Columns whose failure under drift is flexural hinging top and/or bottom can form potential lateral mechanisms, leading to partial or full building collapse. If such columns are part of the primary lateral load system, and their flexural rotation capacity is exceeded, collapse initiated by sidesway is likely. In similar circumstances, but when interstory drift is controlled by separate elements such as walls or braced frames, sidesway collapse may be prevented.

Columns whose failure under drift is controlled by shear, either immediately or as caused by flexural damage, can collapse suddenly and supported loads will drop more or less straight down. If drifts are controlled by a separate lateral force resisting system, vertical collapse can be delayed for several cycles, even with shear failure, depending on the levels of drift.

Conclusion Regarding Collapse Mechanisms

In my opinion, the CTV Collapse Investigation does not describe a building collapse mechanism, but concentrates on column failures as defined by excess rotation at hinges. Such column "failures" can reduce building lateral and torsional stability and definitely should be avoided, but in the presence of an independent lateral system such as shear walls, are not sufficient to cause a complete and catastrophic collapse such as occurred in the CTV Buildings.

Proposed Refined Collapse Scenario

I propose that a global collapse mechanism was caused more by the degradation of the beam column joints than by column hinging. This is not necessarily in disagreement with the investigative report, but represents a change in emphasis.

As previously discussed, the report, including the nonlinear time history analysis, emphasizes drift controlled column failures as defined primarily by concrete strain limits. The extent of beam hinging is not discussed thoroughly. Simple capacity calculations indicate that the columns would tend to hinge first for displacements in the main direction of frames (e.g. NS at column F3, EW at column D3). However, there is little evidence from the rubble that beam elements remained intact at column locations, indicating that column hinging was accompanied by joint degradation, regardless of which one happened first. The degradation of the joint region could have been sudden and complete, due to the lack of shear reinforcing or confinement and the presence of hooked bottom beam bars in the joint, particularly at the interior columns on lines 2 and 3 and on line A.

In this building, joint failure would lead to a global collapse mechanism far more directly than column hinging failure because 1) essentially all elastic and plastic moment capacity at the joint is lost resulting in extreme local and global instability, 2) gravity support of beams by the columns could be lost, and 3) the joint could come apart, leaving only the weak floor topping to hold the floor plates together. The areal photo attached as Figure 1 shows a large NS oriented fracture in the exposed slab directly east of the collapsed stair. This could have happened during the collapse but it may indicate that the beams along lines 2 and 3 lost their ability to hold the building together.

The proposed change in emphasis represented by the proposed collapse scenario is important because column hinge failure would logically lead to a sidesway collapse, and all evidence points toward a more vertical collapse mode. The classic vertical collapse modes of columns, including squashing or shear failure, are not evident, nor indicated as probable by quick calculations. However, the three possible results of joint failure listed above could certainly result in sudden and mostly vertical collapse.

The more or less vertical collapse is also consistent with eye witness accounts and with the folding over of the front coupled shear wall into the center of the building—with only a slight tilt towards the east (as shown in Figure 1), and several of the lower floor slabs leaning against the tower (Figure 165, page 267 of report), indicating that line 3 probably collapsed before these slabs lost vertical support from the tower along line 4.

As previously stated, the vulnerability of the beam column joints is pointed out in the investigative report, just not emphasized. On page 14 of the Executive Summary, "Lack of ductile detailing in beam-column connections" is listed as a critical vulnerability. On page 93 of Chapter 8, Collapse Scenario Evaluation, beam-column joint capacities are discussed as a vulnerability, but "Given the greater uncertainties with analysis of the joints, and given the results that had come out of the column analysis, it was decided that limiting the analysis to columns would be sufficient for the purposes of this investigation."

Figure 1. Aerial Photo from Southeast (probably taken at the same time as Figure 6 in the investigative report) Source: Google images

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.

The potential importance of the joint performance was somewhat de-emphasized in the NTHA (nonlinear time history analysis) as stated on page 201 of Appendix D: "5. It has been assumed for the purposes of the NTHA that beam hinge formation is not limited by the capacity of the beam bar end anchorages or beam-column joint shear capacity." 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.

There is considerable photographic evidence of almost universal joint failure, although it is seldom evident when the photo was taken and how much recovery efforts contributed to the state of the rubble. However, there is at least one knowledgeable description of the condition of the structural elements soon after the collapse during the recovery efforts. This comes from Graham Frost, CPEng, who spent five days assisting with the rescue and recovery as a USAR Engineer. Mr. Frost apparently felt his engineering observations of the collapse were important and sent a short description to the Department of Building and Housing. Excerpts from this write-up follow:

> 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 the total loss of slab moment capacity associated with the slab failure between the profiles steel soffit…and the 200 mm thick composite concrete floor slabs…And while most *beams* survived the collapse intact (except for their ends), no intact beam-column joints were found. Similarly, all interior beams and most exterior beams had completely separated from the concrete floor slabs.
>
> Other important observations include:
> 1….
> 5. The formed surfaces to the inside of the shell beams are smooth.
> …
> 7. Formed block-outs at the end of the shell beams and interior "half-beams" are also smooth.

> …
> 10. The sides of the precast beams appear to have "exploded" away from the column core at the joints.
>
> 12. The floor slabs were found leaning against the main core and slightly higher against the coupled shear wall—suggesting that collapse of the floor and beam elements started near interior columns before the north and south wall strong elements.
> 13. This was also supported by the fact that most of the slabs in the SW corner of the building appeared to have dropped with very little horizontal displacement. The slabs and beams in the NW corner ended up several metres north of the original building line—suggesting rupture of the floor plates at a very early stage of the collapse.

Another interesting eye witness description of failure mode comes from Eyewitness 16 as documented in Appendix A. Eyewitness 16 was working on the CTV Building at the time of the earthquake. He was facing the west side of the building near the SW corner (lines 1 and A). He and his workmate were using a scissor lift working on the west face. He was in the lift about 3 metres up when the shaking started. His description was as follows:

> "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 remember I was still looking at the corner of the building at the time….I turned away to the right to throw my workmate off the end of the machine, then I turned back to make sure nothing else was coming and that is when I saw the corner –sticking out around 300mm. It let go—and came down when I was jumping out."

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 was lightly loaded but had only 4-H20 vertical with R10@250 mm ties (none presumably in the 550 mm tall joint, based on all other columns in the building). It supported an L shaped precast beam from line 1 with top steel bent down and bottom steel bent up, and only the slab edge from line A at level 4, 5 and 6 (See Section 7 on Sheet S15). The joint region was therefore unconfined on three faces, and the description of the column kicking out—but at a floor level—seems likely.

It should be pointed out that the column joint condition at 1A is not the only "non-standard" condition in the building. Columns 2A and 3A are also unique, particularly at levels 4, 5, and 6 where there is no edge beam on line A, and where the beams on line 2 and 3 vary in depth from 550 mm at line B to 350 mm at line A (See Beam 05 and 10 on Sheet S20). Similarly on the east end of lines 2 and 3 at line F, the beams in the last bay supported on line F also vary in depth from 550 mm at line E and are reduced to 350 mm at line F. However, this condition differs from line A because the precast beams are returned to full depth about 500 mm from the column (See Beam 01 and 06 on Sheet S18). These spacer blocks of concrete simplifies the temporary support of the precast beams on line F as shown on Detail 12 of Sheet 19. However, no reinforcing is specified for this block of concrete and it may well have been unreinforced.

<u>Conclusion Regarding Collapse Scenario</u>
The purpose of attempting to describe the actual collapse mechanism is to enable identification of the key vulnerability. If the columns in the CTV were better detailed, but the beam-column joints were not, the building probably still would have collapsed. On the other hand, if the beam column joint was improved, both for shear and confinement and to better tie the beams to the columns, the building may not have collapsed so completely--with the important proviso that the lateral loads were adequately transferred to the lateral load resisting elements (the walls).

What Led to Drifts That Caused Collapse
Professor Charles Clifton performed force based calculations of the connection interface between the diaphragms and the NS walls of the north tower, based on independently estimated floor inertia loads. He concluded that excess deformations in the columns, probably at level 3, were caused by a diaphragm disconnection at that level.

However, the main report suggests reasons why this may not have occurred in Appendix G. Appendix G reports on calculations that show that the lateral disconnection strength along the slab remnant path (that portion of the slab remaining on the tower) was greater than along other paths at Level 4, 5, and 6, indicating that lateral disconnection from tension did not take place at these levels; the reasoning being that such lateral disconnection would have taken place along the weakest path. However, I see no mention made about Level 3, where the retrofit drag bars were not placed. The primary argument against a level 3 disconnection is apparently the configuration of floor slabs from the lower levels leaning against the tower as seen in Figure 165 on page 267 of the report, suggesting a collapse in the center of the buildings (e.g. line 3) prior to failure of these floor slabs at the face of the tower, which would create the configuration shown in Figure 165.

However, the leaning configuration of the lower slabs could be explained by a disconnection at level 3 that caused a sudden increase in drift ratios in level 2-3 and/or level 3-4 frames, which could have initiated the collapse. Presumably the disconnection at level 3 would also cause a gravity collapse at the face of the tower, but that collapse could have been arrested by the level 2 slab. After the complete collapse occurred at line 3, the slabs could have taken the configuration shown in Figure 165. Alternatively, the disconnection at level 3 could have allowed large NS lateral movements without a complete gravity collapse along the face of the tower, also creating the configuration shown in Figure 165.

For consideration of the possibility of a diaphragm disconnection, the lack of damage due to NS loading in the north tower is compelling. I was interested in attempting to corroborate the Clifton conclusion based on probable diaphragm movements. Without performing independent calculations that would develop directly certain parameters of interest, I estimated these parameters from the data available in the report. It appears little lateral resistance was given by the tower before the general collapse occurred. Therefore the probable drifts at each level represented by the damage level of the NS walls of the tower are of interest.

Drifts at each level that may cause collapse of columns would seldom occur at the same instant during the earthquake. The "sets" of drift shown in the simplified analysis below are listed at each level for convenience and do not suggest a complete deformed shape of the structure. However, given a rigid or near rigid diaphragm, the drifts at any level can be used to estimate relative movement of that diaphragm.

It appears that the damage level of the NS walls in the north tower was less than the damage anticipated by code (at ULS demand). In Table 2, column B contains drifts estimated for the original design, probably with a fixed base assumption. Column C in Table 2 lists NS ULS drifts from the report (Table 14, page 258). I presume that the drifts in the lower floors in the report are greater than column B due to the modeling of flexible foundations. Because the structure survived the Darfield event with very little damage, drifts estimated for those motions are also of interest. These were found, based on the NLTH analysis, in Figure 125 (page 209). The NLTH results are considered high in general, but drifts shown here also include disconnection of the tower at the lower floors in the model, so they are an upper bound for low levels of damage. Based on these data points, I have estimated a general level of drift that could be associated with the damage levels observed in the NS walls of the north tower in February and these drifts are shown in column E of Table 2.

From data in the report, we can also obtain drifts at which columns could fail, leading to the progressive collapse. In Table 14 of Appendix F, drifts to cause failure (as defined in the report) of column F2 in the NS direction are given. I will assume these drifts are the same or similar to drifts that would lead to general instability as previously described. These drifts are shown in table column F of Table 2.

It can be seen by comparing table column F with table column E of Table 2 that the drifts of the diaphragm along the face of the tower implied by column collapse are considerable larger than those implied by the damage. These rough estimation lead me to conclude that one or more diaphragms must have disconnected relatively early in the shaking, unloading the tower, leading quickly to the collapse and not allowing further lateral loading to significantly damage the tower.

So I conclude that although the building experienced torsion from several sources, it is unlikely that the failure drifts of columns could have been reached without either severely damaging the tower, or having the diaphragm partially or completely disconnect from the tower at a lower level.

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%

Assuming the figures in Table 2 are in the right range, the only other explanation of large drifts in the gravity columns without damage to the north tower is a general failure of the diaphragm not directly related to the connection to the tower. Such a failure could have occurred because the NS cross building tie was weak in the midspan region of the slabs, and the EW cross building tie could have been compromised by joint failure, either of which could have cause a breakup of the diaphragm.

Other Comments

Concrete block wall on line A

The concrete block wall on line A was clearly intended to be isolated from the structure. However, even if built as detailed, interaction would have occurred at large drifts. However, the severe torsion resulting from initial full engagement of this wall would have put very large demands on the NS walls of the north tower—demands that would seem to conflict with the weak wall to diaphragm connections as well as with the moderate damage observed in the NS walls (see discussion above). There also could have been moderate interaction until the building started to collapse, at which point the walls became fully engaged and were damaged.

Spandrel interaction

Little evidence was given to indicate that the spandrel panels and/or the connections to the slab were strong enough to cause column failure. However, construction tolerances definitely could have caused early interactions at some columns. As the drift increased, particularly after collapse began, the spandrels and columns would have impacted in many places and could shown local damage. But I don't think the spandrel interaction was a cause of this collapse. Such interaction was not necessary for the collapse to occur.

Exceptionally Intense Ground Motion

The characteristics of the exceptionally intense ground motion on February 22 are not discussed in detail in the reports, although they are acknowledged as a contributor to the collapse (page 1 of the Executive Summary). They are also discussed in relation to the elastic response spectra analysis performed as part of the investigation.

It must be remembered that the building survived the September 4 shaking with little apparent damage. The nature and extent of the collapse in February indicates an extreme brittleness in the structure that was triggered at some intensity between the two shaking levels. This performance highlights the issue of the need for a better understanding of the resilience and toughness of our buildings beyond code specified drift levels, particularly older ones.

Elastic Response Spectra Analysis

Figure 11 of the Executive Summary shows average response spectra from three nearby sites and compares them to the code applicable to the building. This is useful to emphasize the unusual intensity of the shaking. However, Appendix F contains several tables of comparative interstory drifts from the various events calculated by ERSA. Similar comparisons can be made directly from the spectra and the purpose of these analyses is unclear. On the other hand, the ERSA performed for code defined spectra was useful to check several different parameters of the original design.

Nonlinear Time History Analysis

The insights potentially gained from a non-linear time history analysis would be expected as part of such an important collapse investigation. However, there is a huge range of options currently available for modeling concrete structures. Typically, more detail involves more time for creation of the model and interpretation of the results. More detail in modeling also has a large effect on computer run time. The balance between the expenditure of resources and the development of useful results is often hard to define. Certainly, the model used could have been more detailed, and could have included degrading column hinges, consideration of axial load on capacities, inclusion of specific model for the beam column joints, and a variety of failure modes in the diaphragm and its connections. However, the results can be no better than the components of the analysis and, in any case, the input ground motion is the biggest unknown. The P 695 studies [6] in the U.S. have documented significant differences in response of structures to different earthquake records, even when scaled to the same intensity levels. The same studies also estimated uncertainties in predicting collapse from other sources, including modeling.

Vertical Ground Motions

It has been noted that exceptionally high vertical ground motions were recorded in February. The report includes a study to estimate the effects of vertical motions on the response. Although the NLTHs were run with the three components of motion simultaneously, the effects of axial load on column failure were not considered directly.

However, based on review of the step by step results for drift and axial load for one column, it was noted in the Compusoft report that maximums did not occur at the same time (Figure 56, page 70). However, it was concluded in the main report (page 25, 101) that drift capacities could have been reduced in the 25% range due to changes in axial loads from vertical motions.

The mitigation of these effects due to non-concurrency and short duration of loading is not discussed, but the report concludes on page 220 (Appendix D) that the "vertical earthquake effects in combination with the column actions resulting from lateral drift are significant and may have contributed to the column failures…"

Conclusions and Recommendations

The exact set of deformations that instigated the collapse may never be known, even with far more sophisticated modeling, because of the many important parameters which can only be estimated, including:

- The exact ground motion demand
- The drift at which the joints would start to degrade
- The strength and stiffness of the connections of the diaphragms to the walls of the north tower
- The strength and stiffness of the diaphragm itself
- The extent of interaction of the block wall on line A and the resulting torsion.
- The effects of vertical ground motion on critical components

Based on this review, it is my judgment that the most important seismic deficiencies in this building were the brittle gravity frames and the poor diaphragm, particularly the connections to the north tower walls,

What lessons can be learned and applied to the general building stock of New Zealand—and elsewhere? There are certainly building characteristics that can be described for engineers performing seismic evaluations, or even searched for in Council files. These include:

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.

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.
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.

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

If you have any questions regarding this review, please contact me.

Respectfully submitted,

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Note on Column Collapse Modes

In simple terms, a column can "fail" in its ability to resist axial loads in several modes as described below:

a. Squash mode: Caused by a high axial load for which the materials of the column are overwhelmed and "fail" without the introduction of flexure of lateral drift.

b. Buckling mode: Caused by an axial load that causes lateral instability of a column with a high slenderness ratio (the ratio of the column length to the column radius of gyration—a measure of width or thickness. Long columns susceptible to buckling required special design procedures.

c. Flexural sidesway mode: Caused when columns lose rotational restraint at the top and bottom to form a mechanism that allows lateral movement without additional lateral force. Loss of rotational restraint can come from hinging (inelastic rotations for which additional force is not required to product additional rotation) in the column, hinging in connected beams, or failure of the joint connecting beam and column. Simple mechanisms can be formed within one story or complicated combinations of hinges can create mechanisms involving multiple stories. Sidesway is normally also associated with a P-Delta phenomena in which the axial loads in the columns ("P") are laterally off set from the support below ("delta") creating an additional lateral force that in turn tends to increase delta. At some point the increase in lateral loads from P-delta cannot be resisted and the structure collapses sideways. However, if an independent lateral force resisting system such as walls, braced frames, or moment frames can withstand the P-delta forces, no collapse will occur even with a mechanism in some of the frames of the building.

d. Flexural shear mode: Caused when flexural actions—normally at the top and bottom of a column and developed due to lateral drift--exceed the elastic capacity of the column and the ensuing damage leads to a shear failure of the column. Shear failure causes diagonal sliding planes to form in a column that leads to essentially vertical collapse.

e. Shear mode: Caused when lateral drift causes shears that exceed the shear capacity of the column, leading directly to relatively sudden vertical collapse.