ARCL'S INITIAL COMMENTS ON:

(1) DRAFT EXPERT PANEL REPORT (CTV BUILDING SELECTED EXTRACTS), DATED 7 DECEMBER 2011 ("EPR");

(2) DRAFT CONSULTANTS' REPORTS, DATED 5 DECEMBER 2011:

(A) CTV BUILDING COLLAPSE INVESTIGATION ("BCR"); AND

(B) SITE EXAMINATION AND MATERIALS TESTS ("SEMT").

NB: The two draft consultants' reports (A & B above) are discussed together.

Prepared as at 22 December 2011

Reference	DBH Report	ARCL Comment
	General Terms/Glossary/References used by ARCL	
	Code / Code of the day	Reference to the New Zealand Standards applicable at the time of design, ie NZS4203:1984 (Loadings Standard), NZS3101:1982 (Concrete Structures Standard)
	Inter-storey Drift	The difference in horizontal displacement between two floors
PGC	Pyne Gould Corporation Building	Known as the PGC Building
EPR	Draft Report titled: "Structural Performance of Christchurch CBD Buildings in the 22 February 2011 Aftershock, Expert Panel Draft Report, Chapter 5 – CTV Building only", dated 7 December 2011	The Expert Panel Report
BCR	Draft Report titled: "CTV Building Collapse Investigation for Department of Building and Housing", dated 5 December 2011	The draft Building Collapse Report prepared by Dr Clark Hyland and Structure Smith

SEMT	Draft Report titled: "CTV Building Site Examination and Materials Tests for Department of Building and Housing" dated 5 December 2011	The Site Examination and Materials Tests prepared by Dr Clark Hyland
VC2002	Vamvatsikos, D. and Cornell, C.A. (2002). "Incremental Dynamic Analysis" *Earthquake Engineering and Structural Dynamics*, 31(3), 491-514	
VC2004	Vamvatsikos, D. and Cornell, C.A. (2004). "Applied Incremental Dynamic Analysis" Earthquake Spectra, 20(2), 523-553	
MYD	Mindess, S., Young, J.F. and Darwin, D. (2002). *Concrete*, 2nd Edition. Prentice Hall, Upper Saddle River, 384-389 and 466-467	Extract accompanying ARCL's response

	DRAFT EXPERT PANEL REPORT (CTV selected extracts)	
		In general, the expert panel report mirrors that of Dr Clark Hyland's Report. Only specific differences have been mentioned below: All other points refer to Dr Hyland's report.
30EPR (para 3)	There is a reference to eyewitness accounts of spandrel panels falling to the street prior to the main collapse event.	Examination of eyewitness accounts in the BCR do not disclose where this occurred. Given the high importance attributed to the interaction between the columns and the spandrel panel, that this has not been examined in more depth in the report means that no conclusions can be made about it.
31EPR	Site examination and materials testing	We believe there are several issues with the concrete column material tests, summarised elsewhere.
31EPR	Collapse evaluation	1. For the level of importance involved in this forensic analysis, the choice of software appears to be inadequate, as does the apparent lack of statistical evaluation given the wide range of possible input parameters coupled with the high sensitivity of the outputs to the input parameters. There are other more sophisticated software analytical tools (e.g. the "PERFORM" programme used for the PGC building) that could have been used for the CTV Building and that might have provided more useful data for analysis. 2. There is uncertainty regarding the direct use of seismic input data from "nearby sites" to model the CTV Building/site given the variability of earthquake effects over Christchurch and over relatively short distances. There has been no attempt to correlate actual seismic readings with the CTV Building site by, for example, taking site readings over the past ten months. 3. See comments elsewhere.
34EPR	Ground shaking records for analyses	It is noted that several assumptions are made about the site acceleration characteristics, with potentially significant consequences. As noted above, it does not appear that any attempt was made to seismically monitor the site post-February 2011 and then correlate that data

		with other sites which had their accelerations recorded during February. Given the high level of seismic variability over short distances and sensitivity of the outputs, this would appear to be an important requirement that has not been done. Also, no attempt appears to have been made to scale the records up or down and account for the uncertainty in actual site accelerations.

	DRAFT CONSULTANTS' REPORT ON CTV	
	Chapter 3: Building Description	
35 BCR	Post occupancy tenancy alteration section. The use of the building includes a language school and formerly included a travel school.	Under more recent NZS4203 codes, a school requires design to a materially higher earthquake standard than "normal" buildings. This has not been picked up when the change of use consent has been granted in 2001. The report makes no mention of this.
	Chapter 7: Examination of Collapsed Building	
55 - 68 BCR		<u>General comment</u>: Whether the damage occurred as a result of shaking, or as a result of collapse, or as a result of the recovery operation, or as a result of debris removal from site, is critical to making conclusions about the damage. Only the first of these is relevant in determining cause of collapse, and in many instances it appears very difficult to determine that portion of the damage that occurred as a result of shaking.
69 - 72 BCRAlso SEMT Chapter 6 and Appendix C		<u>General comment on concrete testing</u>: The claim is made (by implication) that column concrete strength at 28 days may have been 17.5MPa at the time of construction. There is no basis given for this conclusion.
		1. The columns were subject to significant loads, both during shaking, as a result of the collapse, and subsequent recovery and removal operations. They were also subject to fire. Because of this, ARCL suspects it would be extremely difficult, if not impossible, to obtain a core sample that is unaffected by phenomena such as microcracking, which would have contributed to the observed low strength (See photos pp66 to 68).
		2. The worst results appear to be from Column C18 at L1 wall D/E (Refer Table 6 page 91 SEMT, top rows). Refer also Fig. 46 on page 67. This column appears to have

			been subjected to shaking and/or collapse damage and heat. In addition to the microcracking caused by shaking, the reinforcing steel would have heated up in the fire and expanded, causing more damage to the concrete. As well as this, compressive strength reduces significantly when subjected to elevated temperatures (Fig. 17.6, p467 of MYD **attached**). Although the statement is made on p103 of SEMT that "cores were extracted in such a way as to seek to avoid any effect of fire on the concrete properties", it is difficult to see how this could be achieved in practice given the specific exposure was not known. Other samples may have been similarly affected by fire. Based on Fig. 93 in BCR (page 158), the fire appears to have been widespread and the report states that it continued for "several days" (EPR, p30). There is no apparent attempt to analyse the effect of the fire on the concrete tested in circumstances where such fire was widespread through the building, was extremely hot and lasted many days with heat and cooling effects remaining longer.
		3.	Use of Schmidt hammer: Refer p387 in MYD, results affected by surface finish of concrete, moisture content of concrete, temperature, rigidity of the member, carbonation and direction of impact. The general view held by many users of the Schmidt rebound hammer is that it is useful for checking the uniformity of concrete and in comparing one concrete against another, but that it can only be used to obtain a rough indication of the concrete strength in absolute terms. The authors have correlated Schmidt hammer tests with core samples (p 92 of SEMT) but there is still significant scatter of results. Presumably the authors will have carried this scatter through to their conclusions. No confidence interval is given for the reported inferred 28 day concrete strengths.
		4.	Inference on effect of aging (p60 of SEMT): It appears that a reduction factor of 25% has been used to allow for strength-aging effects. It is unclear where the 25% value comes from. Again, there would be significant scatter associated with these predictions, and these do not appear to have been carried through to the reporting of the inferred 28 day strengths.

		5. Strength from core tests are usually lower than those from cylinder tests (refer pp383-384 of MYD) although it appears that some (though not all) influencing factors have been accounted for.
		6. It does not appear that any test has been carried out to determine the composition of the concrete, e.g. petrographic examination, neutron activation analysis, x-ray fluorescence, etc. In the face of clear limitations on the compression and Schmidt hammer testing noted above, such composition testing may provide a useful set of data. Care will need to be taken to obtain identifiable representative samples.
		In summary, the testing results appear to show that the concrete was lower than the specified 28 day strengths. However the statistical error and consideration of the history of the concrete columns would render it very difficult to determine what the actual 28 day strength of the concrete was. On the basis of the inadequacies in testing identified there is no engineering basis to make the conclusions made in the BCR about 28 day concrete strength.
72 SEMT	"Council records show that the structural specification for concrete columns of the Amuri Courts building constructed in 1986 around the same time as the CTV Building, required the concrete to have 28-day strength of 17.5MPa. That specification was prepared by the Design Engineer that designed the CTV Building. It therefore appears that it was not unusual for concrete of that strength to be used in the construction of buildings of this size designed by this Engineer in Christchurch at the time."	In isolation the statement affects columns of unknown load bearing capacity in a completely different building. This is an extraordinary statement that can have no relevance to the CTV building.
	Chapter 8: Collapse Scenario Evaluation	
73 - 94 BCR		General comment: This chapter should probably state more explicitly that the issue of "Demand versus Capacity", in the context of this chapter, is independent of the issue of

		whether the design complied with the code of the day or not. "Capacity" deals with the actual (probable) strength, while design deals with the safe (dependable) strength. It should be noted that the dependable strength is determined based on the design earthquake accelerations, which were considerably lower than the actual earthquake accelerations experienced on Feb. 22.
73 - 94 BCR		General comment: One question that remains unresolved (whether this chapter should seek to answer it or not) is this: *Would the building still have collapsed had the earthquake accelerations experienced at the site been limited to the Peak Ground Acceleration that the NZS4203:1984 design code is based on*? Scrutiny has been applied to the design of the building, and to a lesser extent, to current and previous design codes. However the fact remains that the earthquake of Feb 22 contained acceleration demands well in excess of the design code accelerations, and it appears that no real attempt has been made to quantify this effect, at least in a probabilistic sense. To determine the effect of the magnitude of the accelerations, a suite of simulated earthquakes could be run which are scaled to the acceleration on which NZS4203:1984 is based. A statistical analysis would then determine the likelihood of the building collapsing under a design-code level event. The ultimate test of compliance is performance under the design loads, which may have been demonstrated in the earthquake on 4 September 2010.
77 BCR	"The analysis assumed that records from nearby sites were applicable."	It appears that efforts were not made to get a more accurate picture of actual site ground motion characteristics. In particular equipment for seismic monitoring should have been placed on the site to collect data for correlation with the seismic data used. The absence of such data could be an important inadequacy in use of the simulation data.
77 BCR	"Because of the possible uncertainties in the levels of actions from the analyses, such comparisons were taken as indicative only."	There are established methods for taking better account of uncertainties in time-history analyses. For example, the incremental dynamic analysis procedure (Vamvatsikos and Cornell, 2002 and 2004 – see reference above) takes into account variability in earthquake record type and intensity. The same principles could then be used to account for other uncertainties, e.g. strength and stiffness of building elements, critical dimensions, loading variations etc. The advantage of these methods is that they involve fairly intense statistical post-analysis to determine more likely scenarios. In the case of the CTV building, the type of

		forensic investigation would justify a more rigorous approach to dealing with the uncertainties than the approach adopted in the report. While we can never be entirely sure of the *actual* collapse scenarios, and the approach taken has identified *some possible* collapse scenarios, it appears that no attempt has been made to determine which were *statistically more likely* to have occurred. As a result, some other possibilities may have been missed, while some possibilities identified may turn out to be of limited relevance.
78 BCR	"Observations after the 4 September Earthquake and inspection of structure remnants after 22 February Aftershock indicated that there had been contact between the columns on the north, east and south faces and the Spandrel Panels."	The draft report does not state how the authors determined that such contact had occurred after the September 4th earthquake.
79 BCR	"Because it was not possible to know what the gaps were, various levels of interaction between columns and spandrel panels were considered."	The drawings state 10mm gap either side. This is the most likely gap that would have occurred. A more rigorous statistical approach would have been more valuable in evaluating the effects of the spandrel panels.
79 BCR	"In fact it must be recognised that the possible existence of low concrete strength, and/or greater than assumed interaction with a spandrel panel could mean that a column in another location could have initiated failure."	A more rigorous statistical approach would have been more useful in evaluating these effects.
80 BCR	"Estimation of the actions from the NTHA and ERSA on the Drag Bars attaching the floor slabs to the North Core was subject to some uncertainty."	
80 BCR	"However, it was found that, for the 4 September Earthquake, the analysis indicated severe damage in the plane of the walls on Line A if the masonry was fully restrained by the concrete header beams and columns. Photographs of the walls and statements by	These two statements appear to be incompatible. On one hand, the report appears to be claiming that if the walls were connected, they would be severely damaged, which they weren't. The logical conclusion from this would be that the walls weren't connected (as per the design intent). In the second statement, the report appears to be claiming that the lack of damage in the rest of the structure was due to the presence of the masonry wall stiffening it

	Eyewitness 16 found no damage or spalling."	up, suggesting that it was connected. There appears to be inconsistencies throughout the report as to the actual contribution the masonry walls made during the earthquakes.
81 BCR	"This suggests that the masonry walls, at least for the 4 September level of shaking, were considerably stiffer than assumed in the NTHA analysis and that the response of the structure to the ground motion may have been significantly less than indicated by the ERSA and NTHA using full ground motion and spectral acceleration records."	
81 BCR	"The graph indicates the response in terms of horizontal acceleration for varying structural natural periods of vibration. Low-rise buildings generally have low periods and tall buildings having [sic] higher periods. The fundamental vibration modes of the CTV Building corresponded to values around 1.0 second. "Refer also Figure 39.	The period of the building is highly sensitive to model inputs, and as shown in Fig. 39, the acceleration design spectra (particularly around the 1.0 second mark) is highly sensitive to the period. In addition, the structural period changes during the earthquake as the building softens due to damage. Again, it is important to consider a range of fundamental structural periods and obtain an expected range of accelerations based on statistical analysis.
83 BCR	"The computer analyses found that if the full September earthquake record had been applied to the models they predicted severe damage to the masonry infill wall. This indicates that the real building response to the September ground motion was less than that indicated by the use of the full record in the computer model. It also indicates that the response of the building to the February Aftershock was also less than that predicted by the computer models using the full records."	The alternative that the walls were not connected to the concrete frame appears to have been discounted by these two statements.
84 BCR	Fig. 40	From the computer analysis, some columns appear to have reached at least 1.0% drift during the September earthquake. The analysis appears to indicate that the columns would have failed had there been no gap to the spandrel panels. As there appears to be no failure, the

		gap appears to have been in place as per the construction drawings. In addition, the report states on Page 78 that a concrete strain of up to 0.007 could be justified. This does not appear to have been built in to the graph. There are other factors that may influence the drift capacity which do not appear to have been considered.
85 BCR	"The demands represent values derived from the full ground shaking record. If it happened that the building response was less than calculated, the plotted displacements would be less. This could be due to the CTV site not experiencing the full ground motions recorded at other nearby sites or because the response of the building was not as great as the analysis determined. Note that a reduction of about one-half on the 4 September displacements would mean that they did not exceed the minimum assessed yield capacity of around 0.6%.	It is not clear as to the basis for the suggestion being made that the ground accelerations were lower than at nearby sites. There appears to be no attempt made to correlate the site acceleration conditions with nearby sites. One-half seems to be a very steep reduction in acceleration when comparing to nearby sites - is there any basis for this value?
86 BCR	"It is important to recognise that the expectation of design standards in construction is that even at the attainment of the maximum drift levels there should still be a low probability of collapse occurring."	The code of the day, NZS4203:1984 (and incidentally the current loadings code NZS1170:2004) do not require checks beyond the design basis earthquake. If the expectation of design standards is that there is a low probability of collapse at "maximum drift levels", then this lack of checking in the design codes is a serious deficiency. We understand some overseas earthquake codes now require these types of checks.
86 BCR	Table 3	The computed drift demand given in the table for this indicator column, based on code provisions, is 0.7%. Based on the 4th September event, the computed drift demand was 1.0%. Owing to the fact that the columns did not fail under a drift demand of 1.0%, it could be reasonably assumed that the column was capable of reaching a drift of 1.0%.
88 BCR	"The authors believe that based on their investigation the following specific deficiencies in critical components contributed to the collapse: The specified gap between the precast concrete spandrel cladding units and the	The gap specified in the structural drawings was sufficient to allow a column drift well in excess of the maximum specified in the Code of 0.83%. Hence it should be noted that the deficiency is not one of *design,* and the drifts imposed on the 22nd of February may have

	perimeter concrete columns did not allow for a minimum seismic gap to be maintained."	exceeded the design drifts.
88 BCR	"The 400mm diameter concrete columns and the beam-column joints were not designed and detailed for seismic requirements."	See below. This statement is misleading if this was not a requirement at the time.
88 BCR	"Based on statistical analysis of the column concrete test results a significant proportion of the columns were likely to have had concrete strengths less than what was specified. The distribution of concrete strengths was also less than would have been expected when account is made for the increase in strength with time expected for concrete of that age. This reduced the redundancy of load carrying capacity of the columns."	There are several issues with the concrete strength tests carried out - see elsewhere. The report appears to be over-stating the effects of the lower concrete strength on the performance of the columns.
88 BCR	"Non-seismic detailing to the slab, beams, columns and beam-column joints meant that these elements broke away from each other once columns began to lose load carrying capability. There was very little ability to redistribute load by secondary structural mechanisms such as catenary action once collapse initiated."	The reference to "non-seismic" detailing relates to the Code of the day. The ability for redistribution of load to secondary structural mechanisms was not a specific Code requirement.
	Chapter 9: Design and Construction Issues	
95BCR, 47EPR		This chapter outlines a number of potential non-compliance or design issues. Some of these issues are being clouded with performance on the Feb22 earthquake or updated engineering knowledge. For example, some of the detailing we now know performs extremely poorly however it was acceptable under the codes at the time of design. Therefore these items should not be considered a design issue.
95BCR,	"The building as a whole was found to have satisfied	

47EPR	the building inter-storey drift requirements."	
95BCR, 47EPR	"The displacement compatibility analyses showed that the drift capacity of the Line F columns at dependable strength was less than the K/SM factored inter-storey drift limit of 0.83%. This meant that the columns could not be detailed on the assumption of elastic behaviour…"	The code required the interstorey drift to be limited to a maximum of 0.83%. The actual drift of the building determined by analysis was less than this 0.83% limit. Even if the columns did not have sufficient strength to achieve the drift limit of 0.83%, this is irrelevant to the code requirements. It is the drift determined by analysis based on the code earthquake loads that is important in the design of the columns.
96BCR	References to codes and standards dated 1990, 1987	These codes are dated post design and construction of the building (1986) therefore are not applicable to the design.
96BCR	"The actual as-built gap to the Spandrel Panels either side of the columns may have ranged from 0-22mm…"	It is unlikely that a 0mm gap would have been provided. The contractor would have centred the spandrel panel, and if the gap between the column and the spandrel panel was insufficient, the spandrel panel would not have fit. Also these elements were an important architectural feature.
97BCR	"The beam-column joints… had very little spiral reinforcing… This level of detailing is indicative of the joints having been considered to satisfy only the non-seismic design requirements of the concrete structures standard…"	If the columns can be shown to have remained elastic under the code loads and therefore able to be detailed with standard provisions, then the beam-column joints are fully compliant and they are not a design issue.
97BCR, 47EPR	"The main seismic resisting elements were not located symmetrically about the centre of mass. The centre of stiffness of the designated primary seismic resisting elements was significantly eccentric to the centre of mass."	Eccentric structures were permitted at the time of design (and still are).
98BCR	"The design calculations that were provided did not include displacement compatibility analysis of the secondary beam and column frames."	The calculations provided are not necessarily the complete set of calculations for the design.

98BCR	"Infill walls conforming to the requirements of Group 1 elements were required to be separated from the structure by twice the K/SM factored inter-storey drift displacements (NZS3101:1982, cl. 3.8.4.1(a))."	It is ARCL's understanding that the Code does not require separation by twice the K/SM factored inter-storey drift displacements. The code required separation to allow for the K/SM factored inter-storey drift displacement (ie the building structure movement) plus the displacement of the separated element (ie the masonry wall movement) plus construction tolerances. The code clause referenced does not exist in our version of the code, however the likely clause being referred to is cl. 3.5.14.2 (a)
98BCR	"Drawings indicated that the top course was to have no gap between it and the underside of the concrete header beams and be fully grouted."	This appears to be a drawing omission, however as noted in the report the calculations show the intent was that the top course of block masonry was left unfilled, and workers including eyewitness 16 (see Figure 117) indicates that this was the case.
99BCR	Robustness	While this paragraph is all technically correct, it is not a design issue and more an issue with the standards of the day.
99BCR, 48EPR	"Roughening of internal surfaces of some precast shell beams and not others indicated."	The specification required all surfaces inside the stirrups, and those against which concrete is later to be cast, to be fully roughened (refer ARCL specification 3.12, 3.6). The specification was to be read in conjunction with the drawings.
99BCR, 48EPR	"No starter bars were shown extending out of the precast beams on Line 1 and 4 and into the slab."	Starter bars were shown into the beams, refer drawing S15, plan and sections 1&2, H12@600. These bars did not extend into the precast shell elements however they did extend into the beam elements. The H12 bars had sufficient length into the beam to full develop and lap with the beam reinforcing.
99BCR, 48EPR	"The required concrete 28 day strengths were not shown on the drawings, but were stated in the specification."	It was (and still is) common practice to generally specify the concrete strengths in the specification and not on the drawings. The spec is to be read in conjunction with the drawings. This is not a design issue.
99BCR, 48EPR	"The gap between the Spandrel Panels and the columns was not identified as a minimum gap for seismic separation purposes."	A gap was shown on the drawing. It was the contractor's responsibility to build to the drawings.
100BCR,	"The IEP indicated a large range of potential	It is worth noting that if the drawings were not reviewed, an IEP could yield the building to be at 77% NBS. This shows this method is highly unreliable unless further review of the

48EPR	performance with a lower bound of 44%NBS."	drawings and basic analysis is carried out.
100BCR	"It is important to clearly communicate the specified requirements in a manner that is easily interpreted by on site construction personnel. Placing the concrete strengths on the Drawings is the best way for this to be communicated."	It was (and still is) common practice to generally specify the concrete strengths in the specification and not on the drawings, where there is unlikely to be any misunderstanding of the requirements. The contractor would be expected to seek clarification if required. Strength is not the only criteria for concrete manufacture and the specification is a critical part of the construction document.
179BCR	**Appendix D - Non-Linear Time History Analysis**	This appendix outlines the detailed analysis carried out to undertake the "post-mortem" analysis of the building and why it collapsed.
179BCR	"The 3D model... was created using SAP2000 finite element program."	This program is commonly used by industry around the world and is recognised as being one of the leading software programs for practitioners. However there are more advanced programs available that would be more appropriate for detailed forensic analysis. For example there is a program called Ruaumoko, which was developed by a professor at Canterbury University, is extensively used for research applications in NZ and in many places around the world. Another is the "PERFORM" programme which is distributed by one of the Department's consultants, Compusoft, and was used by a separate Department consultant for the PGC building analysis. It is unclear why the PERFORM programme or similar was not used for the CTV building simulations.
180BCR	"The basis of the non-linear analysis is reported in more detail in the referenced 'Non-linear Seismic Analysis Report' by Compusoft Engineering."	We have not been given this report so it is difficult to ascertain the assumptions used in the analysis.
181BCR	"Material strengths were taken as the average values from tests… Average concrete strengths for columns were taken as equal to the specified 28-day strength + 2.5MPa."	These assumptions are not consistent. They are using test data typically, then specified strengths rather than test strengths for columns. They have added 2.5MPa to the 28day strength to allow for the aging effect. The NZSEE guidelines recommend adding 50% to the concrete strength.
181BCR	"Foundations were modelled with non-linear soil springs with stiffnesses evaluated by T&T."	These values evaluated by T&T are not given in the report.

182BCR	"For columns, rigid plastic interacting M-M hinges were used, calibrated for the average gravity axial compression action on the column."	The axial load interaction should have been considered. The level of compression on the column varies throughout the earthquake. The strength of the column is dependent on the axial load. It is a key parameter, especially when coupled with the effects of vertical accelerations.
182BCR	"The drag bars were modelled using fuse tension links incorporating 2mm initial slip in connections.. At actions equal to the calculated limit state capacities of the Drag Bar and its connections the fuse links would disconnect."	This is a critical element in the model and the behaviour of the building. The modelling should have allowed for variations in the assumptions, eg what if it was 1mm initial slip rather than 2mm. 2mm would be an upper bound as holes in the steel for bolts are oversized by 2mm to allow some tolerance. Similarly, the limit state strength of the elements could have varied to some degree, which could significantly affect the behaviour. A more rigorous statistical approach would have been useful to evaluate the effects of the assumptions made.
183BCR	"The upper bound stiffness and strength of the masonry infill was modelled…"	There is no indication that the most probable or lower bound values were considered in addition to the upper bound.
185BCR	Earthquake records (CCCC, CHHC, CBGS)	In addition to these records, it would have been prudent to scale the records up and down to allow for any variation to the actual CTV site compared to the sites where the seismic recorders were situated.
187BCR	"The maximum storey drifts predicted by the NLTA for the 4 September Darfield Earthquake event are around 1.1%."	The building did not collapse in the September earthquake, and based on the analysis reached drifts of 1.1%. This drift exceeds the maximum drift allowed for in the code, and well exceeds the design level drift. The columns did not fail in this event, nor did they exhibit any significant damage, therefore this is evidence that the columns may have been capable of sustaining a design level event relative to the original code of the day.
194BCR	Assessment of floor diaphragm connections	This page notes a number of key points. The model is predicting the failure of the drag bars or diaphragm ties very early. Based on the physical evidence this is discounted. They mention the analysis is very sensitive to the assumptions made, however it does not appear that a full statistical analysis has been undertaken to account for such variation.
196-197BCR	Vertical accelerations	Significant variation of the axial load was determined from the analysis running only the vertical accelerations (+/- 80%). The graph in Figure R shows the impact of the axial interaction. All points to the right hand side of the curves are potentially in disagreement to

		the modelling assumptions. There are a large number of points where this is the case. The full axial-moment interaction should have been modelled.
		The bending strength of the columns is dependent on the axial load. Generally the strength decreases with a reduction in axial load. With the vertical accelerations recorded as high as they were, it is possible that during a cycle of negative vertical acceleration, the axial load in the columns was effectively reduced to a minimum level which reduces the column strength significantly making them far more susceptible to collapse. Refer to several of the eyewitness accounts that describe significant vertical accelerations and jolts.
198-203BCR	Assessment of Critical Columns	Generally they have found the capacity of columns was not exceeded during September and was exceeded in February. The Line F Columns tend to be critical. In the time history analysis, the diaphragm disconnection occurs prior to the columns failing, which raises the questions of how applicable the models are and how is the variation in assumptions accounted for.
203BCR	"Figure W … 4 September.."	It is assumed this figure is referencing the 22 February event, not the September event, as the capacity of the columns has been significantly exceeded.
203-204BCR	Beam Column Joints… "Given the greater uncertainties with analysis of the joints, and given the results that had come out of the column analyses, it was decided that limiting the analysis to columns would be sufficient for the purposes of this investigation."	The draft report appears to have neglected a potential failure mechanism. A conclusion of column failure has been reached yet beam column joint failure has been discounted without a full and thorough assessment.
204BCR	"It has been difficult to reconcile the damage predicted by the analysis with reports of damage by others… The analysis generally indicated a higher level of damage that what was reported."	The analysis has not produced results consistent with recorded events.
205BCR	"Vertical accelerations alone were considered not to have caused columns to fail… However when combined with lateral drifts, vertical accelerations	The aspect of vertical accelerations does not seem to have been investigated and modelled as thoroughly as it could have. Column axial load and moment interaction appears not to

	certainly could have contributed to column failure."	have been fully modelled which it should have.
207BCR	**Appendix E - Elastic Response Spectra Analysis**	
207BCR	"The axes of the instruments are very close to N-S and E-W."	This is not entirely correct. Some of the instruments are, some are not. Referencing the GNS website, the CCCC instrument is orientated at N26W/N64E, the Westpac Basement is at N45W/S45W. The remaining instruments are within 3 degrees of being NS/EW
210BCR	Figure 112 shows the CTV building with a period of 1.0sec	The period of the building is highly sensitive to modelling assumptions and differing assumptions will produce a different period.
212BCR	"Main assumptions in modelling were as follows: Upper bound soil stiffness, as recommended by T&T."	Although a sensitivity analysis was carried out based on the range of expected soil stiffnesses, only the upper bound is presented. This was to achieve a conservative estimate of the natural periods and base shears. There is no reference to codes, knowledge or standards at the time of design.
229BCR	**Appendix F - Displacement Compatibility Analysis to Standards**	
232BCR	"The bending moments and shears determined from the plane frame displacement compatibility analysis at this drift level were found to exceed the elastic limits for bending at levels 3,4 and 5 and at levels 4 and 5 for shear…. The columns therefore did not appear to satisfy the conditions of NZS4203:1984 to allow them to be detailed with non-seismic detailing."	The resulting bending movements and shears are dependent on the assumptions made in the analysis. Different assumptions may show that the detailing requirements complied with the standard. The reference to NZS4203 appears to be incorrect in this context; the likely code they should be referencing is NZS3101:1982.
235BCR	The ERSA drift calculations have been calculated using the difference in displacements between levels. The error they say is small.	Correct procedure for ERSA is to use the drifts directly from the ERSA and not from the difference in displacements between the two levels, due to the combination of maximums from each mode and the loss of sign when combining the maximums. While the difference is not expected to be significant the correct procedure should be followed.