2017

CTV Investigation

Engineering Peer Review

New Zealand

[redacted]

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Peer Review of the Beca CTV Building Collapse - Engineering Opinion Report 15 July 2016

I have been asked by the New Zealand Police to peer review the Beca Engineering Opinion Report, CTV Building Collapse. Beca have carried out an in-depth thorough investigation since their commissioning letter of 14 February 2014. During this time they have investigated a number of matters including a series of full scale tests and a modified computer analysis.

The scope of work is outlined in section 2 of the Police letter of 20 December 2016.

Scope of Work

- *Review background material that will be provided be Beca. It is not envisaged that you would need to obtain information from other sources.*
- *Review findings of the Beca opinion.*
- *Review the Beca opinion in relation to the duty, omission, causation and identified departures highlighted in the legal briefing provided to Beca.*
- *Pose written questions to Beca where clarification is deemed necessary.*
- *Write a report documenting your findings of the review.*
- *Answer questions from Police regarding the findings of your report.*
- *Be available to give expert evidence should there be a criminal prosecution.*

The approach I have taken in reviewing their opinions is to look at the three questions that Beca addressed in their report:

1. *Was there any omission by any individual to discharge their duty?*
2. *Was that omission a substantial and operating cause of the deaths?*
3. *If so, was that omission a major departure from the expected standard?*

Review of information

The information that I have reviewed is listed below:

1. The report of Beca CTV Building Collapse - Engineering Opinion Report, 16 July 2016.
2. Canterbury Earthquakes Royal Commission, (CERC) Volume 6, Canterbury Television Building (CTV).
3. Appendix H - Non-linear Time-History Analysis.
4. Appendix K - Physical Testing.
5. Report of The Department of Building and Housing (DHB).
6. New Zealand Standard NZS 4203:1984. Code of Practice for General Structural Design and Design Loadings for Buildings (known as "The Loadings Code").
7. Code of Practice for The Design of Concrete Structures, NS 3101 Part 1: 1982.
8. Commentary on The Design of Concrete Structures, NZS 3101 Part 2: 1982.
9. I have also spoken to and received communication from [redacted] and [redacted] about the CTV building.

Review of Findings of Beca

Beca's findings are outlined in the section of the report entitled **Summary and Conclusions.**

The methodology of Beca, as described on page 1, appears to be a logical set of steps that they followed to fulfil their brief.

They were aware of the importance of the task, as they pointed out in the following paragraph on page 1:

"The collapse of the CTV building was catastrophic and sudden, with the floor slabs pancaking on top of one another leaving little space in-between. The CTV building was the only modern (post-1976) building to collapse in this manner in the earthquake sequence between September 2010 and February 2011."

[redacted]

Beca Opinions, page 2:

1. *The collapse was initiated by loss of stiffness in one or more beam-column joints resulting in column axial failure, probably in the ground floor of an internal frame, probably on Grid 2. The capacity and stiffness of the beam-column joint regions associated with these columns was significantly reduced by the lack of transverse and shear reinforcement and this was a significant contributor to the collapse. The characteristics of axial failure in these relatively slender columns were such that their ability to support gravity loads would have been completely lost once it occurred. This would be either by progressive crushing/failure of the column concrete and/or sliding of the column to one side once at least one end became effectively detached from the structure. Once collapse of these columns was initiated there was no ability for the gravity loads to be redistributed to adjacent columns with the result the remaining columns were progressively overloaded, leading to the pancaking collapse of one floor on top of another.*

This conclusion is the basis of the Beca report as to what initiated the building collapse in such a rapid fashion. I agree that the beam-column joint reinforcing shown on the Drawings S14 of Alan Reay Consultants Limited did not comply with the NZ Standard 3101: Part 1 : 1982 Section 9: Beam-Column Joints.

In section 9.2 to 9.4 the requirements for beam-column joint reinforcement is given. These requirements apply to all frames regardless of whether they are designed for seismic loading or not. This is a serious omission and would lead to loss of stiffness and possible complete breakdown of the joint under very large rotations on the joint.

2. *If the building had been designed in accordance with and compliant with the codes of the day it would not have collapsed in the pancaking form it did, notwithstanding the level of shaking and number of earthquakes in the Canterbury sequence up to and including 22 February 2011.*

I consider that this is correct, as a building designed to the intent as well as the letter of the standards and code would have had columns with close ties, designed to allow the columns to accept large inter-story movement without brittle failure. Any cracks developing in the beam-column joints would have been limited in width compared with the minimum reinforcement shown on the drawings.

Design loads would have been much larger and the south wall would have withstood greater loads before any hinging took place. The connection to the north wall would have been designed with sufficient strength to connect to the main building.

3. *The sudden pancaking of the failure occurred because the primary seismic structure had insufficient strength and stiffness to protect the primary gravity frames as detailed, and as a consequence the primary gravity frames were not sufficiently detailed for flexure or ductility to sustain the deflections imposed on them.*

I agree with this statement. The concept of the CTV designers to isolate the gravity load structure and treat it as a structure not required to be detailed for ductility was not appropriate for this building. The seismic resisting elements, consisting of the two walls, allowed for large movements between each floor during the earthquake. This resulted in the columns, which had limited reinforcement, exceeding their capacity for movement and therefore they failed. This movement was further amplified by the eccentricity in the building stiffness.

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4. *The primary seismic structure (i.e. the shear walls and their foundations) initially survived the earthquake, damaged but still standing, apart from the South Shear Wall which ultimately failed in its weak direction after the collapse of the floors because it lost north-south support provided by the floor slabs and the North Wall Complex. The intent of the codes of the day was that pancaking collapse would not have occurred in the level of shaking experienced on 22 February 2011, although the building could be heavily damaged. It is our opinion that, had the whole building complied with codes and practices of the day, the building, though damaged, would have had sufficient resilience not to collapse suddenly and in a pancaking fashion. All other buildings in Christchurch of a similar age with similar structural systems that experienced the 22 February 2011 earthquake had shear walls that provided more effective protection of the gravity structures and the gravity structures were detailed in more tranverse steel that provided greater ductility.*

This is a similar conclusion to item 2. The south wall was overstressed during the earthquake and yielded causing a plastic hinge at the lower level (considered to be level 2), thus resulting in excessive deflections which the primary gravity structure could not sustain. The rear of the north shear wall, placed outside the main envelope of the building was long and thus very rigid. It was not attached to the building sufficiently and thus was not effective in limiting east-west deflections (and possibly north south deflections also). The designers did not take these movements into account when deciding that the gravity structure could be designed with no ductility and very limited non-compliant beam column reinforcing.

5. *There is no evidence that the collapse was triggered by ground subsidence/settlement.*

I have seen no evidence that the collapse was triggered by ground subsidence. The deflections of the structural ground beams could have resulted in greater inter-story deflections than that calculated assuming a rigid base to the walls thus giving a false impression of the stiffness of the building.

6. *The original 1986 structural analysis and design of the building was undertaken using appropriate codes and analysis techniques but contained a number of errors in relation to the application of the codes. There was also a significant mathematical error.*

   The significant errors included:

   (a) *An incorrect calculation of the building's natural periods of vibration in both the east-west and the north-south directions leading to an under-calculation of the seismic design load.*

On page S1 of the seismic calculations (Reay Consultants 10/6/86) a period of vibration of the building was assumed of 0.7 seconds. This was an approximate first estimation typically carried out initially to arrive at a seismic design load for the building. The horizontal load coefficient Cd derived, assuming this period of 0.7 seconds, was 0.1. That is to say the weight of the building would be multiplied by 0.1 to obtain the horizontal load to be applied in order to design the building to resist an earthquake.

This value is later modified by the usually more accurate computer analysis. The results of the computer analysis, as shown on page S15 of the calculations, gave a period of 1.06 seconds in the X direction (north-south) and rather surprisingly the same 1.06 seconds in the Y direction (east-west). As a result the design load for the building is reduced by multiplying by a factor of 0.712. That is a reduction to 71% of the 0.7 second initial value, based on Fig 3 in NZS 4203:1984.

In general, the longer the period of a building the less the design load applied, in the case of Christchurch for flexible soils, when the period is greater than 0.7 seconds. The other question is how a longer period of 1.06 was obtained from the computer analysis for both directions. The east-west direction would be expected to be much stiffer than the north-south direction based on the length and size of the shear walls in each direction and hence the periods would be expected to be different.

(b) *Incorrect reduction in the analysis results for design of the primary seismic elements by including an extra unjustified 0.8 factor, leading to a further lowering of the seismic design load compared with that required by the code. In combination with 6a), the net effect was that the North Wall Complex (in the north-south direction) was 25% weaker and the South Shear Wall (in the east-west direction) 40% weaker than required by the code.*

A reduction factor of 0.8 has been applied, as can be seen on page S17. There are two applications of the factor to different coefficients. I agree that they should not have been applied.

(c) *Failure to calculate correctly the building deflections under code-specified loading, particularly for loading in the east-west direction. This may have resulted in a lack of appreciation by the designer of the deflections required to be sustained by parts of the structure, which, in turn, led to the errors described in (d) and (e) below.*

I think that this is an important and critical point. If the designer(s) had checked the predicted deflections of all the frames correctly, then the gravity load bearing structure would have been seen to require some degree of ductile detailing. The members such as the beams and columns would have been much more likely to have been able to sustain the large inter-story deflections.

(d) *Non-compliance with the code requirements for the spacing and amount of specified transverse reinforcement for beam-column joints, resulting in a lack of ductility/resilience.*

The standard applicable at the time, NZS 3101:1982, gives formulae for calculating the amount of shear steel in a beam-column joint. These columns are subject to reversals of moment even if they are not regarded as providing seismic resistance. These reversals of moment will induce tensile forces in the beam-column joints which require reinforcement to resist them. There is a requirement, under clause 9.4.8 of NZS

4203:1982, that beam-column joints with circular columns have stirrups at a spacing of 200 mm maximum. The stirrups specified on the drawings are at 250 mm centres. This does not comply with the code of practice.

(e) *An incorrect and not justified to regard the beams, columns and joints in the building as being required to resist only gravity loads. In doing so, the designer concluded incorrectly they were not required to be designed for seismic frame action or for ductility. This caused a lack of resilience in those elements which were then unable to sustain the required seismic deformations while maintaining their primary role of supporting the gravity loads.*

I agree that this was a fatal decision that the designer(s) made or overlooked. The designers(s) failed to appreciate that the building was flexible and therefore the gravity elements would need to be designed for ductility. In other words, gravity elements that possessed the ability to accept large deflections without failing.

(f) *An incorrect units' conversion leading to a gross understatement of the shear to be transferred from the floor slab to the North Wall Complex for loading in the east-west direction, which led to an under-designed connection that did not comply with codes of the day. This gave rise to the potential for the floors and North Wall Complex to separate with the result that the building would not behave as the codes intended.*

The calculations of Alan M Reay Consulting Engineer on page S57 for the concrete shear stress v_i show a design shear force of 30,000 Newtons (30 kilonewtons) for line 4, which is the line that contains the connection to the large northern shear wall. This underestimates the load by a factor of 10. The *maximum shear to side walls* mentioned at the head of the page is 300 kilonewtons. 300 kilonewtons is 300,000 Newtons not 30,000 Newtons, the value used in the calculation of concrete stress.

(g) *Failure to check the connection of the floor slabs to the North Wall Complex for loading in the north-south direction, leading to a connection that did not comply with the codes of the day. This gave rise to the potential for the floors to detach from the North Wall Complex with the result that that the building would not behave as the codes intended.*

I agree with this conclusion. The lack of significant connection to the north shear wall was an important oversight in the design of the seismic resistance of the building.

7. *Apart from the lower design loads arising from 6a) and b) above, the primary seismic structure comprising the shear walls and their foundations, was designed and detailed generally in accordance with codes and practices of the day.*

I agree that the two shear walls were designed with ductile detailing and that the foundations were designed to resist the uplift and base moments generated by the shear walls. I understand that for the south shear wall, diagonally reinforced connecting (coupling beams) beams were designed with sufficient strength to cause the south wall to act effectively as a single wall, rather than two separate walls connected by beams that yield under earthquake loads.

8. *The transverse beam-column joint steel specified on the drawings in the primary gravity frames did not comply with the code requirements. Away from the joints,*

the transverse column steel was light (i.e. small diameter and large spacing), but compliant with the minimum requirements of the code in the critical areas. Our investigations have shown that the requirements of the 1982 code, albeit light and less than required in later codes, would have been sufficient to prevent the building collapsing in the manner it did in this earthquake, if the significant design issues identified (understrength and non-compliant shear walls, and non-compliant beam-column joint transverse reinforcement) had not been present.

I agree with the first statement that the reinforcing in the beam-column joints did not comply with the code. Mr Harding stated that he assumed the columns were pin-jointed at their ends. This erroneous assumption, in reality, means that he assumed that they would not attract moments due to the building swaying in an earthquake. In fact they were fixed at each end and therefore generated moments as the building swayed.

Beca considered that their investigations showed that the columns, with their minimal ties, would have been able to prevent building collapse if the shear walls had been stronger and compliant and the beam-column joint steel had been to code. I understand this is based upon their theoretical analysis incorporating the non-linear time history analysis to obtain building deflections of the columns. These deflections were then applied to the columns to ascertain whether they failed or not.

I would be cautious of this conclusion as the deflections obtained in the complex analysis define the movement with limited accuracy (estimated as ± 50% by Compusoft) especially when the building is experiencing large deflections during the earthquake input, such as the February earthquake.

9. *Although the transverse column steel in the CTV building complied with the minimum requirements of the concrete code it was very light compared with general practices of the day. We have found no other similar sized buildings of the era in Christchurch with as little transverse steel as was specified in the CTV building columns and beam-column joints.*

I agree with this statement. I have never seen such light reinforcement in a New Zealand building before. I remember the first time I saw this reinforcing of 6 mm bars spaced at 250 centres thinking that is was very difficult to understand how an engineer trained in New Zealand could specify this reinforcing when earthquake engineering was ingrained into us as students at university.

10. *The poor and non-compliant retrofitted connections between the North Wall Complex and the primary gravity structure, if present in an otherwise compliant building, would not have led to the collapse of the type that occurred in the 22 February 2011 earthquake.*

I understand that this opinion is based upon the Beca analysis and I think that is most probably correct. It is possible that in the north south direction the north return shear wall connections could have been overstressed and separated from the floor structure during the February 2011 event, thus causing an amplification of the movement of the building. The NLTHA showed a cycle of overstressing of these connections.

11. *The known construction errors (lack of transverse steel through the beam-column joints, changes to some beam reinforcing end anchorages, potentially low strength concrete, un-roughened precast to in-situ concrete joints), if present, in an*

otherwise compliant building, would not have led to the collapse of the type that occurred in the 22 February 2011 earthquake.

I would agree with this statement. This opinion of Beca is based upon their test results and computer analysis. If the shear walls had functioned to their full code design strength and been connected to the floor slab adequately then the deflections would have been less and the building may have survived.

12. *The structural design was carried out by Mr David Harding, an employee or Dr Alan Reay. According to Dr Reay's evidence at the CERC, neither he, nor any other engineer in his practice, or outside it, provided any oversight or checking. Mr Harding was a senior engineer and employee of Dr Reay, but inexperienced in multi-storey design. The stated lack of oversight by Dr Reay, or any form of review or checking by another experienced person, was contrary to accepted practice of the time. Our experience and the conclusion we reached following discussions with other practitioners of the day was that a principal of a design company, partner of a partnership, or sole practitioner with staff, would oversee a design such as this (often completing the concept design themselves), or organise another senior employee or external consulting engineer to review it. In our opinion, relevantly experienced oversight or review at concept stage would have questioned the low lateral seismic displacements and/or the decision not to detail the primary gravity frames with ductility. To appoint a newly employed senior engineer without experience in similarly sized structures and rely on him to ask questions if needed, was not in accordance with generally accepted and expected practice.*

I agree with these statements. My own experience at the time in a small design practice was similar to that outlined by Beca. I was involved in all the aspects of the design process from initial client contact, preliminary design, and then, if the detailed design was handed to a competent staff member, reviewing the design before it left the office. It is most irregular that a small practice with only two engineers, as was the case in Dr Reay's practice as I understand it, that no review of the design was carried out by Dr Reay or a senior person appointed by him.

13. *We agree with the CERC report conclusion that the review undertaken by Holmes Consulting Group in 1990 for a prospective purchaser of the building was "never intended to be a full peer review of the design of the building". The assessment did not identify that the gravity structure (i.e. the primary gravity frames) should have been detailed with ductility. This is not surprising given it would take an analysis of the whole building (both primary seismic and primary gravity structures) to confirm the requirement, and the short time available permitted a relatively superficial review.*

I think that this conclusion is reasonable. Holmes carried out a pre-purchase review only. I have not seen the scope of this review but they did identify that the connection to the shear walls and especially the north wall were *"insufficient"*. They however stated that *"the shear walls themselves appear to have been generally well designed to the requirements of the current design loadings and material codes."* In fact the shear walls were not designed to the appropriate loading values from the loadings code. The report did state that the shear walls "*appear*" to have been generally well designed. I would agree that the detailing of the shear walls was in general well designed, as an inspection of the drawings would indicate. They (Holmes) would not have picked up the design loads without a complete three dimensional analysis. This analysis is both time consuming and expensive, and in my experience not typically carried out for a pre-purchase review, especially at that time, i.e. 1990.

14. *The Christchurch City Council (CCC) process for confirming the adequacy of the structural engineering aspects of building permits often relied on the designer and included only spot checks otherwise. The other designers we spoke to who were practicing in Christchurch at the time said they would not rely on the building permit checking process as a review of their design.*

I accept that this was the practice at the time in Christchurch and would reinforce the opinion of Beca that Dr Reay should have either checked the building himself or arranged for it be checked by an experienced person. The practice at the time in Auckland was for the Auckland City Council to concentrate on checking the philosophy of the seismic design of the building. An engineer there called [redacted] was well known for his interest in seismic engineering. However we did thorough checking within the firm because we did not have confidence to rely on the council engineers to examine every design detail and check it's compliance with the New Zealand standard at the time. This applied to the seismic detailing as well as we considered that the council were mainly concerned with the broader picture rather than the fine detail.

15. *Other factors have been raised, such as the cumulative effects of multiple earthquakes and demolition of the neighbouring building. In our opinion these factors were not of sufficient severity to be substantial contributors to the collapse.*

I would agree that the demolition of the neighbouring buildings was not a factor in the collapse. The December earthquake may have caused some structural damage that was not obvious from a visual inspection however I agree with the Beca conclusion that they were not of sufficient severity to be a substantial cause of the collapse.

Dr Reay's Involvement, page 5:

Alan Reay – practicing as Alan M Reay Consulting Engineer, ARCE and later a director of ARCL (post 1988): ARCE agreed to design the building. As Dr Reay was a sole practitioner, this agreement was directly with him.

It is Beca's conclusion that:

1. *Dr Reay, as the person who undertook to design the building, omitted to discharge his duty to allocate appropriately experienced personnel to the design, checking and review process of the building structure.*

2. *This omission was a substantial and operating cause of the collapse, and*

3. *The omission was a major departure from the expected standard.*

I agree with Beca's review of the responsibilities of Dr Reay. I think that it summarises the responsibility that he had as a specialist consulting engineer with a PhD in seismic engineering to provide a design that met the codes and standards of the day well. I think that both the design standards of the time, NZS 3101: Part 1, and the loadings code, NZS 4203, gave guidance on design that was quite specific and a person trained as a professional engineer would be able to follow these rules and recommendations.

[redacted]

I consider that Dr Reay's failure to ensure that the design was carried out to the codes and standards of the day was a major departure from the design standards of the day, and that this failure contributed to the collapse of the building.

David Harding – employee of ARCE and designer of the CTV building:

I agree with the finding of Beca on the shortcoming in the role that Mr Harding played in the design of the CTV building. Mr Harding made significant errors as outlined by Beca. He assumed the gravity structure was not required to be designed for ductile actions as well as underestimating the design lateral loads. I consider that this, together with the other errors, was the reason the building collapsed.

I agree with the Beca conclusions:

1. *Mr Harding, as building designer, omitted to discharge his duty in relation to the design of the CTV building as the design did not comply with generally accepted practices and standards of the day.*

2. *The omission was a substantial and operating cause of the deaths, and*

3. *The omission was a major departure from the expected standard.*

Geoff Banks – director of ARCL and designer of the drag bar retrofit in 1990/1991:

I agree with the conclusion of Beca as to the role of Mr Geoff Banks. In hindsight it is perhaps easier to expect Mr Banks to have designed the connection to be stronger. At the time there were possibly pressures on him to reduce the costs of the retrofit, as it may have been considered a design error and as such would have possibly been at the cost of Alan Reay Consultants.

The tie was only of minimal strength for the loads generated by the design earthquake.

It is Beca's conclusion that:

1. *Mr Banks omitted to discharge his duty in relation to the design of the strengthening works, but*

2. *The omission was not a substantial and operating cause of the death.*

I agree with the first conclusion and also the second. If the retrofitted ties had been of sufficient strength not to fail then they would have helped reduce the deflections of the building in the February 2011 earthquake. However I do not consider that this would have prevented failure caused by the other short comings of the building design.

Graeme Tapper/Bryan Bluck:

I accept the findings of Beca on the responsibilities of the CCC at the time.

Bill Jones/Gerald Shirtcliff.

[redacted]

I agree with the Beca findings and conclusions on the roles of the employees of Williams Construction.

Further Comments on the Beca Report:

The CTV building failed as a result of a combination of adverse factors that all contributed to a building which suddenly collapsed during the February 2011 earthquake.

I think that the conclusion reached by Beca in the Summary and Conclusions (item 1, page 2) that the cause of the failure to be non-compliant beam-columns joints on grid line 2 is a limiting conclusion. I am concerned that this could give the impression that this error was solely responsible for the collapse, and that had the joint been designed to code then the building would have possibly survived without complete collapse.

I think that the structural failure of the CTV building was due to a combination of several shortcomings in the building design.

The mistake in estimating the design loads on the south shear wall by the designers led to the south wall forming a plastic hinge under the earthquake loads with reported failure of the reinforcing steel. The deflection of the building under earthquake loads once this occurred, because of the dramatic increase in the eccentricity of the building, resulted in the building swinging about the north wall. This led to larger inter-story movements in the floors amplified on the south part of the building and the formation of hinges in the columns. They were not detailed to remain elastic under such movements and a progressive failure of the load carrying capacity occurred with sudden collapse resulting.

This is described in detail in the body of the Beca Engineering Opinion Report, in section *12.2.7 Possible Collapse sequence*. I think that section *12.5 Cause of Collapse - Conclusion* better sums up the reason the building collapsed, which was that the primary gravity system was not detailed for ductility and could not sustain the lateral deformations.

The primary cause for the catastrophic nature of the collapse, and the large number of fatalities, was the decision by the designer to assume that the primary gravity structure was fully protected by the primary seismic structure and did not need to be designed for seismic action.

There a number of design issues that are debated but that I think are significant and had an influence of the failure of the building.

In my opinion the eccentricity of the CTV building was a significant cause of collapse. Although the clause in the loadings code, 3.1 SYMMETRY, stated:

"*The main elements of a building that resist seismic forces shall be, as near as practicable, be located symmetrically about the centre of mass of the building*".

This statement was ignored by the designers of the CTV. Beca, and others, have stated that this statement did not strictly apply, as other buildings had been designed in Christchurch with similar eccentricities and these survived without collapse. I accept that, but in my view the symmetry clause should have been taken as a warning that the design of an eccentric building required that extra care be taken if you were going to ignore this clause.

The following clause in the loadings code, 3.2 DUCTILITY, states:

"*The building as a whole, and all its elements that resist seismic forces or movements, or that in the case of failure are a risk to human life, shall be designed to possess ductility*".

This clause was also ignored by the designers, as the central columns were, in the case of failure, a risk to human lives. The fact that this clause was ignored by the designers was the major cause of the building failure as Beca pointed out above.

Interpretation of the clause 3.8.1.2:

"*Computed deformations shall be calculated neglecting foundation rotations.*"

Many engineers take this to mean that the deflection of the supporting soil and its effect on foundation rotation should be ignored, but that the foundations themselves should be included in the model. If the foundations, as detailed by the CTV designers, had been modelled in their 3-D ETABS analysis, then the inter-story deflections would have been greater and the need to reinforce the gravity structure for ductility would have been more obvious.

Appendix H Nonlinear Time-History Analyses

This analysis is the present 'state of the art' in creating a mathematical model of the building structure and understanding how it responds to the recorded ground motions of the Canterbury earthquake recorded at a nearby station. However there are still limitations as to the accuracy of applying the results to individual members.

Summary

My comments on the Beca Engineering Opinion Report are listed in my report above, and are related to each item as set out in the Beca Summary and Conclusions section. In summary:

- I agree with the Beca report findings in relation to the responsibility of Mr Harding and Dr Reay, plus the other people mentioned who were involved.

- I agree that the mistakes identified by Beca, including the failure to follow the code and standards of the time and the shortcomings in the design, led to the catastrophic collapse of the building with the loss of life.

- I consider that the Beca Report explored the various design shortcomings and mistakes thoroughly.

- Item No.1 in Beca's Summary and Conclusions concentrated on the beam-column joint failure on line 2, level 2 as initiating the failure of the building. Much of the NLTH analysis and the full scale testing was undertaken to confirm this conclusion. I consider that this initial failure of the columns on line 2 was probable but that other areas of the structure would have likely failed as well.

[redacted]