HEARING RESUMES ON WEDNESDAY 11 JULY 2012 AT 9.30 AM

JUSTICE COOPER:

Now do you want to ask a question about Dr Frost's, Mr Frost's points that were summarised on the sheet?

MR RENNIE:

Yes Sir, the position is I, the position as I left it yesterday was that the witness would look at that.

JUSTICE COOPER:

Yes.

MR RENNIE:

Just before I ask him about that I've reflected overnight on the Commission's question to me about the putting of Mr Mander's, Professor Mander's evidence to the witness.

JUSTICE COOPER:

Yes.

MR RENNIE:

My focus, I now think erroneously, was on the issue of whether there was conflict between the two of them and there's a statement in Professor Mander's brief about that but I appreciate that not putting the evidence is not helpful to the Commission and my thinking this morning is it may be preferable if I did do that.

JUSTICE COOPER:

Yes.

MR RENNIE:

If I'm taken to have irreparably elected otherwise yesterday Sir.

JUSTICE COOPER:

Yes well I, my question wasn't prompted by anything in particular except that I read Dr Mander's evidence superficially and thought that certainly he emphasises other possibilities –

MR RENNIE:

Yes.

JUSTICE COOPER:

– as causes of building failure which I suppose at a high level are really contrary to what Mr Holmes is saying.

MR RENNIE:

My, my feeling Sir is the Commission may be assisted if I simply put the main points to Mr Holmes.

JUSTICE COOPER:

Yes.

MR RENNIE:

So that he can refer to them.

JUSTICE COOPER:

Yes.

MR RENNIE:

Professor Mander's position as I understand it is he and Mr Holmes are essentially consistent except that Professor Mander in his own view has gone further down the road than Mr Holmes has gone and on thinking about that last evening I realised that I couldn't make the assumption I had made that Mr Holmes would have continued the further distance on the journey.

JUSTICE COOPER:

And I should also say that, I mean my question was prompted by the kind of consideration that I've just mentioned and, of course, I haven't heard your opening. So I don't really know what it is you're going to be submitting other than what one can infer from the evidence that's been circulated. So I really meant, well I've explained why I asked the question.

MR RENNIE:

Indeed Sir, yes and, and of course that, that will be clarified tomorrow.

JUSTICE COOPER:

Yes.

MR RENNIE:

And I think I should say one sentence only today.

JUSTICE COOPER:

Yes.

MR RENNIE:

And that is that our position is seeking to participate in the investigation rather than construct and present a defence, if I can put it that way.

JUSTICE COOPER:

Yes, yes, well that's, I'm sure that's appropriate.

MR RENNIE:

Indeed Sir, thank you Sir.

JUSTICE COOPER:

Thank you.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Now Mr Holmes the first point was Mr Frost's note of the 20th of February.

JUSTICE COOPER:

Sorry, sorry Mr Rennie we'll need to, we'll need to re-swear Mr Holmes.

WILLIAM THOMAS HOLMES (RE-SWORN)

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Now Mr Holmes the first pointed related to a note that Mr Frost prepared. It comprises two pages and, for the record, the reference is BUI.MAD249.494BB. On the 20th of February of this year following his review of the DBH report on page 1 he discusses weak beam column joints, strain hardening in the south wall shear wall and his third point is the lack of confinement at beam ends and in beam column joints and his second sheet is some diagrams explanatory of his points on that. You had yesterday referred to some earlier material of Mr Frost and his evidence which you're familiar with and my understanding was that you'd found that helpful. I wanted to give you the opportunity to say anything additional having seen this note.

A. Yeah I did review this note and I really don't have any comment. I agree I did not check the mathematics but the concept is no different than what Mr Frost was saying before. The strain hardening issue is kind of a new issue but it's really talking about the out-of-plane collapse which is not particularly germane to what we're talking about, unless someone took a sample at that re-bar that's possibly strain hardened and I have no idea whether that happened or not but I agree that if the wall fell out-of-plane perpendicular that bars would have been highly strained and probably strain hardened so I would agree with that as well. The misplacement or the potential misplacement of stirrups in the precast beam so that they were further apart, further away from the column I totally agree that if that was the case that would, that would be a detrimental effect.

Q. Thank you Mr Holmes. Now you're aware that Professor Mander is to give evidence and has filed a brief and then a supplementary brief. I think last evening he may have been provided with the supplementary brief and I apologise for the inconvenience of the timing on that but have you been able to look through those briefs?

A. I have.

Q. Yes.

A. I probably would spend a little bit more time in the future but I have spent considerable time last night.

Q. Yes well I apologise for having created that situation and I appreciate that you have spent that time on it. What I'm proposing to do is to raise with you in series a number of points that Professor Mander makes in those briefs to give you an opportunity to indicate your position in relation to those points should you wish to do so, it's not obligatory, or to add material or whatever you wish to do and then at the end to ask you a couple of more general questions about Professor Mander's evidence.

Now the first point that Professor Mander makes is that he contends that the DBH report neglects the effect of the pre-22 February earthquakes. Professor Mander contends that the structure must have sustained hidden damage in the earlier earthquakes and his own view, as stated in his brief, is he believes that the building should have been red stickered at that stage. Do you have a view on that?

A. I have a very strong view.

Q. And that is?

A. He suggests that the, first he suggests that the building should have been red stickered due to its age and the apparent code level shaking and I totally disagree and I think such a red sticker would be unjustified and impractical. He later says it should have been red stickered when there was reports of lively, or a more lively floor and, all other things being equal, I also totally disagree that that would be the cause of any further investigation of a red sticker. So I have seen no evidence, and of course I wasn't here, I didn't see the building, but I have seen no evidence that would indicate that there was ever knowledge it should have been a red sticker.

Q. Leaving aside the matter of knowledge do you have a position on whether the building may in fact have been, it may in fact have sustained hidden damage in the pre-22 February aftershock?

A. Based upon the evidence I don't think so.

Q. Now Professor Mander supports the DBH report conclusion of exceptionally high vertical ground motions helping to lead to the building failure and he contends that they are primary contributor to triggering the building's failure and subsequent collapse on 22 February. Your position on that?

A. Well he discusses vertical accelerations in September as well as February so.

Q. Yes.

A. As a matter of fact I guess it's the vertical accelerations in September he suggested and the words, he used the word "broken." I have some notes here, broken the slab at the negative moment areas which would be over the beam and I, it's hard for me to imagine how that could have possibly happened. I don't know what broken means. There was fairly

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ductile reinforcing in those locations so I don't agree that that would have happened in September. The vertical accelerations in February were much higher and I think everyone agrees that they probably had some effect, detrimental effect on overall response. I haven't seen anybody come up with a method yet of very definitively combining the vertical effects with the other lateral effects. It's very hard to do in modelling. I'll be very curious to know how this ongoing non-linear time history modelling is going to try to combine all those things. It's extremely difficult to do that but so the amount of additional damage that was caused by the vertical acceleration I don't know but I think it did affect it, yes.

Q. Just pausing on that in relation to what you've said about the current NTHA modelling which is being done, do I take it from your answer that you see that as being an exercise somewhat at the leading edge of the use of that modelling in this type of failure?

A. Well yes it would definitely be at the leading edge. There may be some additional insights gained from that but the possibility of a definitive answer of exactly what happened coming out of that I think is very slim. It would be the same arguments about how things were modelled in the future as they were in the past because there's a lot of different ways to model things and some of the combinations of effects, I know from my work on collapse of concrete buildings in California are simply can't be done yet. They're too complex and the computer modelling combining of effects is just not there yet.

Q. Is that a matter of the design of the software or is it the matter of the computing power required to carry out the (inaudible 09:42:22)

A. Well (inaudible 09:42:22) computer power because you can always wait two weeks or three weeks or however long it takes or get bigger computers so it's becoming less that. It's really becoming that the subroutines that the software that it's not so much the software package. It's a matter of the modules that interact between these various actions within the structure and tell the computer that's tracking flexural or shear loading that there's a different axial load and therefore change what the capacity is and back and forth. Some of those relationships are simply not written up anywhere yet.

Q. So in relation to the caution you've expressed in respect of the current modelling, is that a caution as to the reliability of the outcome or to the scope of the outcome?

A. I'm not sure what you mean by scope?

Q. Well it may not be feasible to test matters which you would regard as relevant to know before you could form an overview of the cause of the failure?

A. Yep, I would be concerned about that and I would –

Q. Yes.

A. – also be concerned that the sensitivity of certain results to assumptions that must be made regardless of how detailed the analysis, is you still have to make assumptions somewhere along the line. I suspect the results would be sensitive to those assumptions and so I think the reliability would be an issue. As I say I'm not saying that there wouldn't be additional insights from the results of this software but again I don't think a definitive collapse scenario will be able to be determined.

Q. Now thirdly, and reverting to Professor Mander's evidence, he accepts that the columns of the building didn't have substantial transverse reinforcing but he does not see that as either a problem nor a cause of failure. Your view on that?

A. Well if in fact the joints sort of fell apart first that would be the case and certainly my position is the joints were the most critical issue. The spiral reinforcing giving some amount of confinement to the column to keep it together while the joints fell apart leaves questions as to how much of the falling apart of the top of the column or the bottom of the column interacted with the falling apart of the joints. So I think for us looking at the situation if there had been more confinement steel we'd be, we certainly could be more sure that the joint fell apart first.

Q. Yes. The matter of the interaction of the perimeter columns with the spandrel panels on the building. Professor Mander's view is that this may have been a contributing factor to the final collapse of the structure but was neither the trigger nor the cause of the collapse.

A. I think I'm already on record as essentially agreeing with that.

Q. The issue of, in Professor Mander's view of the separation of the floor slabs from the north core is described as problematic, but against that he puts the fact that the structure survived the design level Darfield earthquake and many aftershocks after that without collapse?

A. Well first of all I don't agree that the September earthquake was a design level. I characterise that I think in my review as approximately that but there are a wide variety of time histories of shaking that would result in the same spectra. Each of these various time histories would have a fairly significant effect on a response so although we always look at spectra first and say, “Oh that’s near that and it’s close to this,” and so on, that is not a very precise gauge so not only were most of the spectra actually below the code a little bit at the 1 second range if 1 second is a period that’s also problematic as where was the actual period. It’s not totally clear to me if you created a random sample of time histories that would meet the same spectra and you ran a bunch of them that you would find many of them had a larger response than the particular one that hit that building.

Q. Is that a question you therefore feel namely whether that 4 September earthquake, the Darfield earthquake, was or was not at the design level? Is that a question you feel will not be answered ever in this case?

A. Well, I don't think it’ll ever be answered. I think everybody will have an opinion and I have an opinion. I think it was less than design level intended.

Q. Do you regard those opinions of differing persons as competing valid possibilities or would you disregard those who say that it was a design level?

A. Well I would disregard someone who said it was a design level without some indication of why they think that.

Q. Right.

JUSTICE COOPER:

Q. Is part of your reservation because of the duration of the strong shaking Mr Holmes?

A. The duration is one issue. The exact nature of the time history obviously in the field did not affect the building the way a design earthquake should have. Even a well designed building hitting, being hit with a design earthquake should have some damage and there was no damage and I can’t explain that if it was a design earthquake and of course the design earthquake is defined by a spectra but within that spectra as I've mentioned there could be 40 or 50 ground motions that you would need to get the full range of responses that you would expect from that spectra and I, my only explanation of the lack of damage is that not only was this slightly below the code from elastic response spectra standpoint but it probably was at the lower end of the time history signature that caused the response.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. Professor Mander next discusses the asymmetry of the shear wall layout and his view is that the DBH investigation gives excessive weight to that. Your view on that?

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A. I think as a conceptual thing the weight is appropriate, having a very strong tower at one end of the building and a coupled shear wall at the other looks better elastically than it does inelastically. When the coupled shear wall goes inelastic you have a, you have a much more of a torsional issue. Commissioner Fenwick on the other hand has suggested, and I tend to agree with him, that the coupled shear wall probably would not have acted like, or probably did not act like a coupled shear wall, so it was stiffer than, it would've made the condition better actually as far as torsion goes. So I don't think torsion was a necessary characteristic for a collapse.

Q. In terms of symmetrical versus asymmetrical design, and this is not Professor Mander's point, it's leading on from what you've just been dealing with. You have a situation where you have clients and architects who have expectations of a building, and engineers who have to meet those expectations. Is it your view that the engineers should follow the concept or should the concept follow the engineers?

A. Well engineers who have been in a building design business for any length of time know well, and actually joke about, the difficulties that architects give them and our job if you're a good engineer is to give the architect or the developer what they want and still meet what you think is appropriate code design and a good performing building, and I think engineers probably succeed in that with a fairly wide variation. I certainly know in California that you see buildings that seem to be more controlled by architects than others. So my own practice we certainly try to make buildings as regular as possible, and if they're not regular do fairly extreme analysis to make sure we understand what the irregularity is doing to it.

Q. Now, California like New Zealand has to address the reality of the seismic exposure, that's correct?

A. Yes.

Q. Is there an argument that in locations which have that level of seismic exposure it should be a heightened position for the engineer when compared to the architect?

A. There has been, that position has been forwarded, mostly by engineers but it hasn't happened. I think that in general Californian engineers have more power, you might say in the design team than they do, say on the east coast where there is no seismic, because there's simply more freedom that the engineers have. For example a steel building, it is very common to, for the engineer to not design the connections between steel members because it's really a matter of giving some loads and having the contractor or the fabricator do the most inexpensive thing. But in California those connections make a big, are very important to the seismic performance, so we have to detail all those connections ourselves in order to fulfil. Now that's just an example of the fact, well, another, another way to measure that is fees. The structural fees in seismic areas are considerably higher than they are in non-seismic areas because it takes a lot more time to assure a seismic system.

Q. Now next Professor Mander passes to consider the supplementary investigation work, particularly the additional seismic readings in the post-22 February seismic readings obtained by Dr Bradley, and he takes the view that it was not appropriate in the DHB report to disregard one of the four available seismic records pre-22 February and assesses it as being not only a relevant record but one of particular relevance. Do you have a view on that?

A. I don’t because that’s one area I did not get into the detail. I understand what was done but I did not look at the detail of the results of that analysis.
Q. Professor Mander then deals with some issues of concrete which we’re dealing with collapse essentially today, so I’ll pass over those. He then presents a conceptual building. The red book building which is used for evaluations in university and contends that even that building, notwithstanding its code compliance or perhaps one could say its conceptual code compliance, may well have collapsed in the 22 February earthquake, have you reviewed that?
A. I read that last night. I have not, there isn't enough detail there for, I have not looked at the paper that was written as the basis of that.
Q. And you wouldn’t be familiar with the red book building concept?
A. I am familiar with that concept. We have similar concepts of –
Q. Sorry I was going to say –
A. – of having a standard building that we analyse so.
Q. I was going to say as applied to the particular exemplar that Professor Mander takes?
A. I did not, I do not, I’m not familiar with that no.
Q. Lastly he presents an alternative collapse scenario?
A. Several.
Q. Well he, yes, he strictly he presents an alternative collapse hypothesis and then three scenarios based on the hypothesis, and your view on that?
A. Well they’re all possible, just like all the scenarios that the different opinions have come up. There are four main ones and everyone has a favourite and I think these are, these could be listed there as possible. I could be wrong because there are many pages of descriptions but it appears to me as if they are dependent upon one or more slab, diaphragm being completely free of all shear walls and yet at the same time the same slab is capable of pulling all the columns in the building in one direction or the other to get this two storey buckling. Somehow that seems unlikely to me that that could happen, but it’s not impossible.

Q. Is column buckling in that hypothesis is what he refers to as ULAR buckling?

A. That's a standard buckling but it's not, it's not standard in this case because it's my understanding here you're talking about two levels because it becomes very long. Buckling has a lot to do with the height of the column and the slenderness, so it's unlikely that these columns would have buckled in the one storey mode, but when you get two storeys they get very tall, plus the fact that the joint disintegrates. You have a pin in the middle, and then furthermore, if the slab, the diaphragm is pushing against all those columns you end up, as soon as you get any kind of an angle you actually end up with an additional force that – so all of that is very logical, but as I say the reality of the slab being loose enough to do all that and yet still be capable of pushing all the columns seems a little unlikely to me but not impossible.

Q. Part of the basis on which that hypothesis is developed relates to the collapse of the lower four floors in this six floor building and the lesser collapse on the top two. Leaving aside whether that is a support for the hypothesis do you see that as a significant feature of the collapse?

A. Well there were several witness statements that were referenced. Particularly the eye witnesses in the building. Now I didn't have those, so that's one of the things on my list is to see whether those eye witness statements in fact points to this effect. Now clearly there, I know there were occupants in the building in the top floor that survived and actually rode the building down. I read those accounts, but once it got on the ground how much of the upper two floors were still standing as opposed

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to the pancaked bottom floors is not clear to me, from the pictures. The earliest pictures I could see it didn't appear that I could see any two storeys standing on top of the rubble.

Q. I don't think it is Professor Mander's contention that there were two undamaged or nearly undamaged storeys still standing but that the pancaking related to the lower floors. Is that consistent with your inspection of the pictures?

A. Well it would be logical in any case you have a lot more weight –
Q. Yes.
A. – sitting on the bottom floors and there were certainly no columns keeping floors apart so it is only logical that the bottom floors would be more squashed than the top.
Q. And are you putting it that that would be regardless of the hypothesis adopted to explain –
A. I think so yes.
Q. Now –
A. I would add something on the two storey modes. Dr Mander did suggest that I went beyond the pointing at the joints and suggested that a two storey collapse mode was either necessary or proposed. I can't remember the words he used, and that is not true. I don't think you will see anywhere in my report anything about a two storey collapse other than using the corner at A1 as an example of a joint failure. I never suggested that that had happened anywhere else so that is not a correct interpretation of my report.
Q. To be clear, if he has taken that from your report, it was not your intention that such a meaning be taken from your report?
A. Absolutely not.
Q. Now lastly Professor Mander provided a supplementary brief of evidence which essentially was the development of some of the points of his first brief with additional data and you have had an opportunity to look at that?
A. I did. It is far more technical and mathematically derived and certainly have not had the chance to completely understand the analysis that was run. However it appears as if the results would indicate that, for example there was 10 cycles of code defined spectral displacement in the Darfield earthquake. I find that conclusion hard to believe. I don't know how it was done. I have not analysed any of the time histories myself it just seems hard to imagine.

Q. But if he were to be right on that which is a matter for him to support, would that alter your view on the extent of damage that may have arisen in the Darfield earthquake?
A. Well I would have to apply, I would have to understand how those ten cycles were applied to the building.
Q. Yes.
A. I mean in the end it is always the building it is not the ground motion and you know we use free field motions we call them as opposed to what actually happens in the base of the building, so there has been some discussion of rocking of the tower so that changes the motion so it would take a little bit more thinking than to just say the record shows that there was 10 cycles of this high motion.
Q. Now moving from the specifics to a couple of general matters at the end that I have foreshadowed. A number of the matters that we've discussed this morning have uncertainties attached to them because of the non-availability of for example, the debris site, the north tower and the intact post-earthquake stake and so forth. Are you able to draw any comparison between what happened to the physical evidence in this case and what would be in your experience standard practice in California?
A. I think in California the emphasis and in every earthquake with which I am familiar, the emphasis is recovery and I just think that is always the emphasis. I think the collection and recovery of material to figure out what happened would always be a secondary issue.
Q. I don't think anyone would dispute that but recovery being achieved which of course necessarily may involve deconstruction, debris removal, invasive processes and so forth. At the point which recovery ceases to be the priority there is the question of the preservation of the physical material for the engineering investigation which will certainly follow. Are you aware of any process in California which would require that material to be retained?
A. I am not a forensic engineer by profession but there maybe, Mr Shepherd may know of some guideline in California for that. I, again

in – first of all we haven't had that many collapses in California fortunately. Certainly not that would destroy the evidence in this complete regard. I mean that really makes this significantly different. There were several collapses in the North Ridge earthquake but there were not a lot of victims and the materials sat there, the building and its partially collapsed state sat there for many months so it could be examined so I don't think we have had this kind of circumstance and it would take some jurisdiction or the owner or somebody very quickly to request or obtain some jurisdiction to do that and again I have not seen that happen in the case of earthquake collapses.

Q. The New Zealand response in Christchurch was the creation of a statutory authority with very, very extensive powers but I just tell you that for information.

JUSTICE COOPER:

Well not in the days immediately following the earthquake surely.

MR RENNIE:

Well not in the first days no Sir, but prior to, for example to the demolition of the north tower.

JUSTICE COOPER:

That is true.

CROSS-EXAMINATION CONTINUES: MR RENNIE

A. Well there was some testimony. It was my understanding that there was some requests made and CERA had some other ideas.

Q. I wasn't going there. I was just telling you that that was the New Zealand response.

CROSS-EXAMINATION: MR ALLAN – NIL

CROSS-EXAMINATION: MR ELLIOTT – NIL

RE-EXAMINATION: MR MILLS

Q. Perhaps just one or two questions Sir, to tidy up some loose ends if there are any.

Can I first of all just take you back to the question you were asked about the number of cycles that the building went through in September and what Dr Mander had said about that and your comment at least as I noted it was that that was hard to imagine. I take it, am I right in saying that it is hard to imagine because of your – what you have seen about the physical damage the building sustained?

A. Yes. And even the elastic spectral response and the analysis I have seen that shows that the inelastic response in this particular ground motion unlike some rules of thumb that say that the response, the displacement of the inelastic response and elastic response are the same. There has been some analysis I've seen that shows that in this particular ground motion the inelastic response which is what happens when the damage starts occurring was less than the elastic response.

Q. Secondly, the questions you were asked about the time history analysis and the limitations which I think you acknowledged or referred to in relation to the inputs into that and hence the outputs from it. I just want to ask you the extent to which that flows across into the ability to identify whether there were significant structural weaknesses in the design of the building. Does one preclude the other?

A. No I think the potential deficiencies in the design of the building can only be compared with what the code requires because what the earthquake finds is its own business and the engineer has no way of knowing that so the engineer can only follow the standards and so the deficiencies

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I think can only teach us, or the deficiencies of the ground motion show this can only teach us about the future but any deficiencies in the design of the building stand on their own feet regardless –

Q. Yes.

A. – of what the time history says.

Q. Yes. Now finally just so the late hours you must have burned on getting on top of Dr Mander's evidence aren't lost to the Commission, are there any other points that you would like to make in light of your consideration of that evidence?

A. Yes. There are a couple of comments that I jotted down. Some of that had been covered already by the questions but I'll just quickly –

Q. Please?

A. – go through there, I'll even mention the page number. I quickly found those this morning and the red stickering was already mentioned but on page 5 it suggests that the building should have been red stickered due to its age and the apparent code level shaking and as I've said before I disagree with that. It's impractical method of doing red stickering and it's completely against any kind of rules that we've been using in the engineering profession for a long time.

The document many times suggests that there was a code level test and makes conclusions on that basis, so we've already discussed that. It certainly was approximately a code level as looked upon by the elastic response spectrum but I don't think you can make conclusions to say that that was a code level test and we've already discussed the reasons for that.

We've talked about, a little bit about the vertical ground motions. There was a, there is a statement on page 6 that, "The vertical vibrations from the earthquake prior to February essentially would have broken the fixed end condition of the three span slab." I would interpret that from an engineering standpoint to mean that over each support there is resistance to deflections of the slab. We call it negative moment and New Zealand engineers have a slightly different name for it but it's provided by the reinforcement near the top of the slab and the drawings show a special reinforcement there made of bars which are quite ductile, so it's not clear to me what the reference is that they would have broken those areas. I don't believe that would have happened.

Q. Yes.

A. On page 9 it suggested, "Fatigue-like damage would already have existed in the joint prior to February," and we've talked about the joints a lot. We've talked about the winds and the brittleness of the wings.

Q. Yes.

A. And the smooth precast sections. It's hard to believe that there was 50 such joints in the building and even though each one was not inspected it's hard to imagine that there was much working of that joint that would have caused fatigue without any evidence on the outside shell, any cracks along those wings, any spalling of a little piece of concrete. There's no evidence that such damage occurred in the joints so I would disagree with that.

There is considerable discussion throughout the document, well maybe not throughout, in several places of developer pressure to obtain more efficient construction and the interest rate and the expense of certain details. It is not clear to me how that relates to this investigation. It's still the responsibility of the engineer to meet the code. We all have pressures all the time but we discuss the irregularity pressures from architects. You still have to meet the code.

Q. Yes.

A. So I'm not sure that I agree that's germane.

On page 11 there's a quote that says something like there are many instances of so-called "soft storey effects have been caused by the presence of short shear critical columns". This is a little bit of engineering jargon but I think this is absolutely incorrect statement. Short shear critical columns is almost the opposite of a soft storey effect in buildings that causes damage so I don't know. I absolutely have no idea what the sentence is in there for and I totally disagree with it.

On page 12 it says, "The council permit did not require drag bars," and I again don't know what that has to do with anything. Regardless of what level of review or permit is issued does not change the responsibility of the engineer to meet the code and one of the very first premises of any seismic code is to have a load path. A load path means all the loads can get where they're supposed to be and certainly they have to get to the shear walls so it would seem to me that that is immaterial.

On page 12 it suggests that the P Delta which is an additional load caused in a side sway mode. As the column starts to lean over there's an axial load on this and that axial load tends to push the column even more when it, the column is absolutely vertical there is no P Delta but as it starts to go over there is a P Delta.

Q. Yes.

A. That P Delta is dealt with in most seismic codes as a design issue, maybe not in the mid-'80s but that's not the point. On page 11 it suggests that the P Delta at the perimeter of the building was less because the vertical loads were less and now if each frame were totally independent of one another that would be true but P Delta is normally thought of as a global issue so you have all the columns and all the load going at an angle because they're all tied together by a diaphragm. They can't move independently so the fact that the loads on the perimeter were less does not change the P Delta of the entire storey –

Q. Yes (inaudible 10:17:05)

A. – which would drag those columns over.

Q. Yes.

A. So I disagree with that statement.

On page 13 there's a discussion of the west wall which is the CMU the masonry wall –

Q. Yes.

A. – which we've discussed a lot.

Q. Yes.

A. And there's a statement that says, "The west wall was unfit for the purpose of providing substantial degree of seismic resistance after September." Well it was never intended to provide any seismic resistance so –

Q. Yes, no, I understand that.

A. – it's not, I don't know what that statement is there for.

Q. Yes.

A. Page 38, it says, "The structure was designed as a shear wall building," and it's kind of a partial quote, "and a check was made that the principal gravity load bearing components were not put under excessive side sway displacement." I have seen no evidence that such a check was made.
Q. We'll ask Dr Mander about that.
A. I already mentioned that it's suggested that I propose the collapse rollover more than one storey was needed and I, if that, if I gave that impression I would like to clarify –
Q. Yes.
A. – now that I didn't mean that.
Q. Yes.
A. And then we've already discussed the, my overall review of the collapse scenarios that they are all seem to be possible like others but they do have somewhat unlikely combinations of effects of the slabs that would be required to, for the buildings to fail in that way and that there seems to be some reliance on some eyewitness accounts and I have no criticism of that. I've done the same thing but I have not had a chance to review the ones that are quoted there.
Q. Yes.
A. So I would have to do that before, that might, that might either make my opinion of these collapse modes more favourable or less favourable –
Q. Yes.
A. – or have no effect, I don't know. I'd have to read those to find out.
Q. Yes.
A. So I have some other marginal comments here but those are the ones that, those are the ones that I brought out as that I thought were somewhat germane to the, to my review of this.
Q. All right, well thank you very much, that's helpful.

QUESTIONS ARISING FROM COMMISSIONER FENWICK - NIL

QUESTIONS ARISING FROM COMMISIONER CARTER – NIL

WITNESS EXCUSED

HEARING RESUMES: 2.15 PM

MR MILLS CALLS
MICHAEL JOHN NIGEL PRIESTLEY (AFFIRMED)

Q. Now Professor Priestley your full name is Michael John Nigel Priestley?
A. It is.
Q. And you've prepared a written brief of evidence which you have with you?
A. I have.
Q. And that's dated the 18th of May 2012.
A. Yes.
Q. And in addition to that you've prepared some PowerPoints that you'll refer to.
A. Yes.
Q. All right. Well I'll just take you through some of your background in that first, in that second paragraph and then just ask you to proceed. You have a PhD from the University of Canterbury.
A. Yes.
Q. You've been involved in seismic performance of structures for more than 45 years.
A. Yes.
Q. You were the head of the then Ministry of Works Central Laboratory.
A. That's not quite right. I was the head of the Structures Laboratory which was a unit of the Central Laboratories.
Q. Thank you.
A. I'm sorry that's a mis –
Q. No, thank you. You were a faculty member at the University of Canterbury from 1976 to 1986?
A. Yes I was.
Q. You were a Professor of Structural Engineering at the University of California, San Diego from 1986 to 2000?

A. I was.

Q. A co-director of the European School for Graduate Studies in Reduction of Seismic Risk, the Rose School, from 2001 to 2009.

A. Yes.

Q. And you hold Emeritus status at both the University of California, San Diego and the Rose School presently.

A. I do.

Q. Thank you. Well then I'll just ask you to start going through your brief of evidence at paragraph 3.

A. Well that's again the statement. Do you want me to read that out, that I've been active in research authoring and co-authoring more than 700 articles and reports, mainly related to seismic design and performance of structures including three major reference books on seismic design of structures?

Q. I think just keep reading so it's in the record thank you.

A. I have received more than 30 awards, prizes or recognitions of my research over the years.

In my evidence I will address the following topics:

my involvement with the Department of Building and Housing, an expert panel,

my areas of agreement with the Department of Building and Housing's consultant's report and the expert panel report of the CTV building,

my areas of disagreement with the reports,

whether the design of the CTV building met best practice of the time,

the state of the CTV building following the 4th of September 2010 earthquake,

the causes and likely sequence of collapse of the CTV building on 22nd February 2011.

I have read the Code of Conduct for Expert Witnesses and agreed to comply with it. I confirm that all of the matters to be addressed in my evidence are within my areas of expertise.

The DBH expert panel.

I was appointed deputy chair of the expert panel assembled by DBH to investigate causes of damage or failure in the February 2011 earthquake of four buildings of major public interest. These buildings included the CTV building. The panel interacted with the consultants appointed by DBH providing guidance and assessment of the work carried out by the consultants. This was a continual process through the development of the consultants' reports and included additional analyses done by members of the panel themselves where appropriate and extensive comments and questions on the final and draft reports prepared by the consultants. Aspects of the consultants' reports were included in the summary report prepared by the expert panel. In some cases the emphasis in the expert panel report differed to a greater or lesser degree from that in the consultants' reports. Comments from the expert panel were essentially the view of individual panel members whereas the panel's report was required to be a consensus document. Inevitably some of the comments or views of individual panel members could not be included in the panel's final report. In the case of the consultant's report prepared by Dr Clark Hyland and Mr Ashley Smith on the CTV building collapse, called the CTV report in further reference from now on, there was some difficulty in reconciling the view of the consultants and members of the expert panel. This is referred to in the expert panel report at page 34, paragraph 4, which notes that although the panel supported the general conclusions reached by the consultants on the reasons for the collapse some members did not agree fully with the conclusions reached in the consultants' report on the identification of critical columns, the influence of spandrels and when in the collapse sequence the separation of the floor slabs and the north core occurred. I was one of the panel members who did not agree with all of the conclusions the consultants had reached on these issues. Because of my known disagreement with the CTV report in several significant areas and because my views also diverge from the expert panel report in some areas I have been asked by the Royal Commission to give evidence to it in a personal capacity and not in my capacity as the Deputy Chair of the Expert Panel. I will endeavour to present my evidence in a way that minimises the use of what are often highly technical terms used by structural and earthquake engineers, however, this cannot be entirely avoided. I also understand that my evidence is expected, well has occurred after evidence from other consultants and the panel's representatives and a number of these terms are likely to have been explained by them. Nevertheless I will occasionally try and make explanations of points that I think need to be made further. My evidence sets out and explains the differences of opinions between my personal view of issues leading to the collapse of the CTV building and the opinions expressed in the CTV report and, to an lesser extent, in the expert panel report. My views were all presented to the consultants who have considered them when writing their final report. However, though I am pleased with changes in emphasis included in the panel report I am not satisfied that all my concerns have been adequately resolved in either the CTV report or the panel report. The purpose of my evidence to the Royal Commission is to provide a clearer statement of my view on alternative collapse possibilities than was possible in the expert, in the panel report I'm sorry.

Areas of agreement.

Before dealing with these issues, however, it is important that I note that there is much in the two reports with which I agree. This is particularly the case with the panel report. Because my evidence deliberately concentrates on those areas where I disagree with either or both of the CTV report and the panel report it could give a misleading impression of my view of the bigger picture conveyed in the reports, particularly in the panel report. The areas where I have little or no disagreement with a report are: deficiencies in the design of the CTV building when considered against the code and best practice at the time, as described in section 9 of the CTV report and page 50 of the panel report.

Q. Now Professor Priestley I'm just going to leave it to you to indicate when you want any of these references brought up. They're there if you want to have them brought up.
A. Do you need to, do you want these put up or not?

MR MILLS:

What's the Commissioners' wish on this?

JUSTICE COOPER:

It's not necessary for our purposes although using your judgement those which are more important could well be brought up for the edification of the people in the gallery watching the proceeding.

EXAMINATION CONTINUES: MR MILLS

Q. Well it might then be useful to bring these ones up because they do, for that very purpose, put the flesh around this of what ones you're agreeing with and what ones you're not agreeing with. So let's do that. So it's the first one there. BUI.MAD249.0189139.
A. So this is section 9, it goes on for several pages.

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Q. Yes it does.
A. – I'm not quite sure that it's going to be useful to –
Q. Probably not.
A. – to go through them, in the sense.
Q. Okay, it's in the report if anyone wants to read it of course. It's a public document.
A. Yep. And I think in fact the next thing, the next item, that's item B here clarifies some of this. The presence of critical vulnerabilities in the design drawings in relation to columns as referenced to on page 35 of the panel report. I don't know if you want to put that up?
Q. Do you think that would be useful, is it something you'd like me to do?
A. I don't actually.

Q. All right then let's just move –

A. I think it's fairly general, these comments that I'm making. Irregularities and lack of symmetry as referred to on page 37 of the panel report. The diaphragm connection as referred to on page 38 of the panel report. The general conclusions as to the factors contributing to the collapse of the CTV building as described under section 5.13 of the panel report, although I disagree about the extent of the contribution of each factor and the likely failure sequence. So I will be discussing that in much greater detail subsequently. The recommendations section 11 of the CTV report and section 5.14 of the panel report. So these are all areas which I am in general, this, in general agreement with. Can we have the first figure please?

Q. Is that the first one in D, 1.16?

A. Figure 1 which is, B, this is of my own ones here.

JUSTICE COOPER:

Q. Yes this diagram is 402.1

A. 402.1 indeed, so my ones are numbered sequentially from 402.1.

EXAMINATION CONTINUES: MR MILLS

A. Are you not seeing anything here yet?

Q. Well no we're not either. Sometimes there's a bit of a delay.

A. There it is. This is just a plan view of the typical building and I just wanted to refresh people's minds on various aspects of it. The north core is this rather large conglomerate or complex of intersecting walls which contains the services and lifts and toilets, stairs and so forth. On the opposite side was the north coupled shear wall which we'll be talking about at various times.

Q. You mean south don't you?

A. South, sorry, my apologies on that. There is the wall which is the west wall here on line A which has a reinforced concrete frame with masonry panels inside which I'll refer to at various times itself, and there is the east wall, the east frame which is on line F itself which has particular spandrels in it, as does this region here, and we'll look at that in just a minute, and of course the internal columns, vertical circular columns at 400 millimetres diameter supporting the floor system which spanned only in this direction. So it's a one way floor system which was connected to the seismic resisting element, the main seismic resistant elements being the north core and the south coupled shear wall itself, though there is some capacity on the east wall as well.

JUSTICE COOPER:

Q. Mr Mills I wonder if it would be helpful if Mr Priestley could be shown the plan that we had prepared which is 0486.1, just to have it in front of him because it's got less information on it Professor Priestley but it, you may find it useful to have it because it identifies the grids.
A. Thank you, that's a much clearer one.
Q. So we've all go that. It has got north at the top of the page which I am told which is really to suit my convenience, but don't let that worry you.
A. It's also convenient. I would point out that this one was, the one that I'm showing was selected by the Royal Commission.
Q. By a lawyer probably?
A. I just gave them and said, I want a figure of this.
Q. That's all right. I just wanted you to know that we do have this plan in case it helped?
A. That's great, thank you, much more useful.

EXAMINATION CONTINUES: MR MILLS

A. Can I go to the next figure though? The, my concerns with the CTV report are undue and inappropriate reliance on elastic response spectrum analysis which I'm just going to shorten to ERSA in future for assessing performance of the CTV building in the September 4, 2010 and February 22, 2011 earthquakes. Misleading information about the, or what I find to be misleading information about the, design seismic intensity for the CTV building. Excessive emphasis on the role of spandrel panels in the east and south faces, this is the east face and this is the south face, and this region here is where the coupled shear wall is, and these are the spandrels which are architectural elements on the lines of the beams on both faces of the building here. In the report these spandrels were, well assume a very high importance to the predicted failure mode of the building itself which I don't believe is appropriate.

Another area of disagreement is the view that the exterior columns were more likely than interior columns to initiate failure, and again it's particular these columns on the east wall rather than the internal ones, or the columns on the south wall.

Rejection of connection failure between the floor diaphragms and the north structural core as a high-probability failure initiator.

Excessive emphasis on torsional eccentricity, based on ERSA analysis.

The modelling of masonry infill panels on line A and the assessment of the effect of the infill on the building performance. You'll recall that line A is the one that's hidden out of here but parallel this is line F, line A is parallel to it on this side here, and there was masonry infill over the lower three levels.

A reluctance to accept the results of the nonlinear time history analyses where these did not agree with the consultants' view of the collapse sequence, and the methodology for the displacement compatibility analysis used in the consultant's appendix F to the CTV report.

My evidence now addresses these issues in more detail.

A, Reliance on Elastic Response Spectrum Analysis, ERSA, and could I have figure 3, the next figure? Some of the significant areas of disagreement stem from the reliance in the CTV report, let me read that again, stem from the reliance the CTV report has placed on ERSA. The initial analyses and development of the consultants' view of factors leading to the collapse of the CTV building, were based on ERSA analyses and eye witness accounts. It is my firm opinion that the ERSA was unsuitable for determining the causes and sequence of the CTV building collapse. It is a design tool intended for determining the required strengths of structural members to satisfy code-specific seismic input. It is not suitable for determining the expected response when assessing a building. It's because of these concerns and, and others that, sorry, it's because of these concerns I and other panel members had about the reliance the consultants had placed on this analysis, the ERSA analyses, that the panel required a nonlinear time history analysis to be carried out. This produced some significantly different results that have not been adequately addressed in the CTV report or, to a lesser extent, in the panel report. And I should amplify this now, that I was not aware and I'm not sure that other panel members were aware, that initially when the consultants started work on the CTV that they started work on a nonlinear time history analysis as well as the ERSA, but that work on this was halted in May at the instructions from Dr Clark Hyland, and this was later reinvigorated in late August or middle of August. The

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unsuitability of ERSA for determining the collapse issues is clearly stated in the recommendations of the NZ Society for Earthquake Engineering (NZSEE) for "Assessment and Improvement of the Seismic Performance of Buildings in Earthquakes", 2006, that is the title of the document. Section 4.3.2(b) (ii) Modal Response Spectrum Analysis (EMA). EMA is just another terminology for ERSA. They are the same type of analysis. And the provisions there recommend against use of ERSA for building assessment where inelastic response is expected. And as I note in my text EMA and ERSA are different terminologies for the same procedure. Problems identified by the NZSEE Recommendations with the use of ERSA for building assessment include:

Inadequacy in determining member inelastic deformation.

Underestimation of higher-mode effects when inelastic response is expected,

and overestimation of torsional response and you can see that these are identified in this slide here. ERSA is a design method for determining required members strengths to code input. ERSA is not suitable for determining expected response to an earthquake and I will amplify that later. These are the recommendations of the society against ERSA inadequacy in determining member elastic deformation, I have just read these three out and these conditions all existed in the CTV building. ERSA cannot predict when during earthquake response the predicted approximate maximum actions would occur and that has some significance in trying to determine a sequence of collapse.

By way of a very brief explanation of what I am referring to: "Member" refers to the various structural elements of a building, for example, the beams and columns. "Elastic deformation" is deformation, or displacement that will disappear if the forces causing the deformation are removed and could I have the next figure please. Right and this is just a very rough sketch here describes a generic building itself being deformed under the effects of an earthquake and these deformations can be considered to be both elastic and inelastic depending on the intensity of the response. Elastic deformation is deformation or displacement that will disappear if the forces causing the deformation are removed. While inelastic deformation is the additional deformation or deflection that will remain after the forces are removed. Under low-level seismic excitation the building deformation is essentially elastic and that's illustrated in here in this rather simplistic graph which has displacement along this axis and some characteristic inertia force which is shown here, the forces, the inertia forces which cause the building to displace. Here, under low level of seismic excitation the response may be idealised as simple elastic, meaning that it loads up a path and unloads down the same path itself and when the earthquake is finished or the series or the section of the earthquake that we are considering at this time is finished the deformation is zero. Under higher levels of seismic excitation the response may be inelastic. Inelastic deformation implies damage to the structure which is expected under high levels of seismic excitation and is accepted under code design and in this case here we are seeing that perhaps instead of finishing at this level the deflections have got rather larger and the structure has got up to close to its maximum strength and deforms essentially at constant strength and then when it unloads it doesn't normally unload down the same line, it unloads to have some residual displacements or residual deformations.

I have also mentioned in the previous things, modes and higher mode effects and I would like to discuss those with the next slide too. 'Mode effects' refer to the way a building vibrates in an earthquake. This can change during the course of an earthquake. Buildings respond differently depending on the size and shape of the building and the nature of the ground shaking. The building responds in a combination of the various possible modes of vibration and the characteristics of the modes change if the building responds inelastically to the earthquake. The first or fundamental mode typically provides the largest part of the building response to an earthquake. So here is our building again and this was the type of deformation we showed in the previous slide where essentially the displacements increase as you go up the height of the building and something like a linear although not necessarily linear fashion and this is, normally the structure has the fundamental mode which has the highest, sorry the lowest period of response itself but it is not necessarily the only way in which the structure responds and we are showing here a deformed shape which is more or less characteristic of the second mode of response and you can see that it is very different. Under this type of displacement parts of the building displace in one direction and others in another direction and as we go further up and to the third mode, each of these having a smaller period than the previous one, we see greater complexity in the response. Now with a multi-storey building under earthquake response, the response is always a combination of these but it is also worthwhile noting that the characteristics in terms of the displaced shape and also in terms of the periods change with the intensity. This is particularly true of the first or fundamental mode and less of an issue with the second and third modes, their characteristics do not tend to modify to the same degree as the fundamental mode modifies under inelastic response.

'Torsional response' which I have also mentioned in identifying problems with the ESRA approach, refers to the twisting effect on a building and/or elements of a building. It is greater in buildings that are eccentric, in other words buildings where the strength and stiffness of the lateral-force resisting members, the beams and columns, are not symmetrically located on the building plan which in the case of the CTV building the principal lateral-force resisting element was the north shear core and this was at the extreme north edge of the building and outside the building envelope. So this was a very eccentric type of building. In addition to the problems with ERSA identified by the NZSEE recommendations, the important fact could be added that ERSA provides only an approximate estimate of the peak elastic response of structural actions that is for example the member forces or displacements over the duration of the seismic response and this elastic response assumes that the structure has sufficient strength to respond without inelastic response. I have mentioned that inelastic response modifies the structural performance. It cannot predict inelastic deformations. It cannot determine when these peak response levels occur in the structural elements. In fact, different actions may peak at different times in the seismic response. This is particularly important when interpreting the response from the peak levels. For example, the peak floor diaphragm inertia forces are not normally provided in the computer output from an ERSA analysis. At any instant of the response, they can be determined from the storey shear forces in the members above and below the floor diaphragm being considered. However, the peak floor diaphragm inertia forces during the seismic response cannot be determined from the peak predicted storey shear forces above and below the floor under consideration, since it is unknown, and unlikely, that the peak storey shears above the floor and below the floor will occur at the same instant in the building response.

Further, ERSA is particularly inappropriate when used with recorded response spectra rather than with smooth response spectra. This is because actual response spectra are typically very irregular, with large changes in spectral amplitude for very small changes in effective period, and could we have the next figure please. I think this is, can be illustrated here. I will just read a little more first and then come to this slide. The ERSA assumes that the periods associated with the various modes of response stay constant through the building response, whereas if inelastic deformation occurs, the periods lengthen, and

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particularly the fundamental period, lengthens. The effects may not be significant when related to the design spectrum but can result in very large areas with real spectra as the response ordinate may increase or decrease. Also small areas and small errors in calculated periods have a much great effect on predicted response of a building to a real spectrum than to a smooth response spectrum. So we're looking at that here. We can say here that this is an example, the grey one here is an example of a design spectrum itself and we can see that if the period lengthens by a little bit the response which is shown here, the spectral acceleration response, does not change markedly in any of these three different design ones. However, if you take this extreme example here if you look at the response at this period here, which is about .6 seconds, and then look at the response to maybe .65 seconds period here it's suddenly changed enormously from here to there whereas over the same period shift the design spectrum has changed very little. So there's a real danger in using ERSA when you're actually using real recorded response spectra. As a consequence of these problems with ERSA and the great importance of obtaining analysis of the CTV building that could be relied on in determining the collapse scenario the panel required the consultants to carry out a non-linear time history analysis. This decision was made after the panel had received the first draft of the CTV report. If I recall correctly this was during a meeting of the panel in August 2011. As the consultants did not directly have expertise in non-linear time history analysis a decision was made to employ Compusoft, a consulting firm specialising in high-level structural analyses including non-linear time history analysis. Now I explained earlier that it was, I was not aware that, in fact, Structure Smith which is the company which was looking after the analyses in conjunction with Hyland Consultants had already commissioned Compusoft to do such an analysis but the work had been discontinued at an early stage based on instructions from Clark Hyland. That's just a point of clarification which I don't believe has any particular significance. So interaction between the consultants and Compusoft in preparing the necessary input data and interpreting the results was provided primarily by Ashley Smith of Structure Smith, though considerable input was also provided by panel members including myself. The advantages of a non-linear time history analysis over ERSA include, and could I have the next, you've beaten me to the gun, excellent. So we can just read these. Direct computation of inelastic deformations. Output of time-dependent values of critical results such as the forces applied to the members and the resulting deformations of each of those members. Direct determination of which members or member, member or members first-reach failure thus providing information on where failure might have initiated. The ability to impose orthogonal seismic accelerograms, that is, the recordings of the ground shaking the two horizontal and the vertical direction simultaneously. In other words to feed in the assumed actual effects of the ground shaking on the building to the best of our knowledge with that shaking occurring from different, various different directions itself.

As with all analytical procedures and approaches there are limitations to what can be achieved with non-linear time history analysis, and could I have the next one, and these include the following:

Limited ability to model strength degradation during a failure scenario. So this is quite important. Non-linear time history analysis is extremely useful in determining what may trigger a failure but it can be very difficult to model the actual failure scenario itself in terms of degrading strength.

Limited ability to model shear failure in concrete members.

Limited ability to model interaction between different actions in a member such as moment in shear. Moment refers to the moment, the demands which lead to bending. Shear refers to the, to the demands tending to cause diagonal cracking and I have a slide to follow this but I'll continue, at this stage, just to read the list out.

Problems with modelling elastic and hysteretic damping which is a highly technical issue which I don't propose that we get into at this stage.

In some cases excessive computational time because of the complexity of the analyses and the large number, often many millions, of equations that must be solved. This can lead to instability in the computation making solution impossible. Can I have the next slide?

Okay, just to, to illustrate, did I do that? I probably did. Sorry. Okay it's there now. To illustrate the difference between shear and bending. In the left-hand side what we're showing here is a column from one floor, at the top of one floor to the bottom of the next floor if you like and under bending moment the, the structure bends itself as you can see like this and cracks form on the tension side of the element, often inclined due to, to shear effects as well but you can see that this is a, a shape which has bending to it and the critical areas are at the top and the bottom of the element itself. In shear the structure is subjected to the same forces but the shear forces tend to concentrate the, in the effects of the diagonal cracking itself often forming a brittle failure where the two elements may even disconnect as is shown here with one element moving bodily to the right with respect to the left. So there's a, a different mode of deformation and the fact, the determination as to whether the deformation is primarily flexure or primarily shear depends on the relative strength of the element in bending and shear. The actual deformations will be a combination of flexural and shear deformation but what we try and do in design is to ensure that the shear deformations are not excessive because it's a very brittle failure mode with very rapid reduction in strength.

Results from the non-linear time history analysis differed in many cases from the conclusions the consultants drew from the elastic response spectrum analysis. However, despite the serious limitations of ERSA I have referred to earlier in my evidence and the New Zealand Society for Earthquake Engineering recommendations against its use the consultants have generally chosen to rely on the ERSA results rather than the non-linear time history analyses results where these differ. The principal points of divergence between the results of the two analyses and those where the consultants' decisions not to revise their conclusions cause me the greatest concern. The importance of diaphragm disconnection from the north core as a potential failure initiator, conclusions that the exterior columns on line F were more critical than the interior columns, particularly as a consequence of the spandrels on that line, the significance of masonry infill on line A and the potential for beam column joint failure. The validity of the ERSA analyses using the simplified response spectrum adopted by the consultants is further placed in doubt by the modelling of member stiffness based on the recommendations included in the Loadings Code of 1984 and the Concrete Design Code of 1982 and these were appropriate for considering design at the time but not in terms of, of determining the performance to the real earthquake. In other words they did not consider, they did not represent a state of the art appropriate for 2011, and this is clarified a bit more in my next paragraph where it says the consultants' choice of member stiffness was appropriate for estimating compliance in 1986 with code displacement and strength distribution requirements. However, it was inappropriate for estimating the response of the design structure to the recorded seismic excitations of September 2010, December 2010 or February 2011. Here the aim of the investigation was to determine as accurately as possible what had actually happened and the best state of the art information about how to model member stiffness should have been used.

B. Design Seismic Intensity. The 1984 design spectrum

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used for the purpose in designing the CTV building compared with the actual spectrum calculated from the recorded accelorograms referred to in figure 11 of the CTV report copied as figure 5.9 in the panel report has a number of inconsistencies with material presented elsewhere in the report. If these figures are compared with figure 4.10 of the panel report which purports to show the same information significant differences are apparent. And could we look at the next figure please? We can see here this on the left-hand side is the figure that comes from the CTV report and this one here is the figure that comes from the expert panel report and what I'm asking you to look at is the 1984 code intensity which is shown here in the CTV report by this dashed line here and in the –

JUSTICE COOPER:

Q. That is a purple coloured line?
A. Purple coloured line.
Q. Yes.
A. Thank you. And in the panel report by this blue line here, see there. Now in the CTV report this initial plateau here is at .88 times the acceleration of gravity and the plateau here is about .53 times the acceleration of gravity. In the panel report which corresponds exactly with the code report for 1984 for flexible soils the plateau is at .5G here and at .3G for this longer portion here.
Q. I think you meant to refer to the 1984 code requirements?
A. Yeah, code requirements.
Q. Rather than code report?

A. I'm not sure, occasionally my tiredness may get through and I apologise for that.

Q. That is perfectly understandable.

A. Thank you on that. So this is the code of 1984 code requirement here and corresponds, I have checked, with the, with what is in the code for flexible soils and this value here is 75% higher than this value here. Also 75% higher here with this. I've recently seen comments on the fact of this 75% from Dr Hyland and he claims it's 1.48 or that there is an error in this figure here. I've checked that and there is no error there and I've checked the sums and it is 75% higher, at least as far as I can determine.

"First in looking at this the 1984 design intensity in CTV report figures 11 has an initial plateau at .88G", as I've said, "which is 75% higher than shown in the panel report figure 4.10. The latter is the value shown in NZS4203:1984. The scaling factor of 1.75 corresponding to an increase of 75% has been applied to the entire spectrum. As far as I can understand the reason for this is that NZS4203:1984 required that when ERSA was used for design the base shear which refers to the design lateral force from the earthquake ground motion at the base of the structure should not be less than 90% of the value resulting from an equivalent lateral force or single mode analysis. To briefly clarify this a single mode analysis is a simplified approach which assumes that 100% of the building mass is associated with the fundamental mode of vibration. In an ERSA the responses found by calculating the response of the various different modes," such as I showed you before, 1st, 2nd and 3rd et cetera, "with each within appropriate proportion of the total building mass associated with that mode and then combining these modal responses in accordance with procedures specified in the design code. This generally meant that it was necessary to increase the base shear that had been calculated from the ERSA because ERSA typically produce base shear results that were less than this 90% figure. In the case of the CTV this scaling factor calculated by the consultants was about 1.75 as I've noted above, presumably for the east-west direction of response. I would expect in a different and much lower scaling factor to apply for the north-south response as a consequence of the gradual regularity and the reduced tortional response but this is not discussed or presented in the CTV report." I have more recently seen information from Dr Hyland that in the north-south direction it was about 1.15 so but this information is not in the report and it is not clear when discussing north-south response whether in fact that was used or not or whether the 1.75 scaling factor was used.

"A scaling factor of 1.75 is unusually high implying that the base shear calculated from the lateral force approach using the same member stiffnesses as used for the ERSA approach analysis was about 95% higher than that from the ERSA analysis." I can't check that but it's another that surprises me. "Compounding my concerns about the correctness of the scaling factor used by the consultants the CTV report then applies this same scaling factor to the design response spectrum, as I've shown here." So it's been applied here to the entire spectrum. "In my view this is incorrect. It should only have been applied to the base shear and not to the design spectrum. The scaling of the spectrum implies that the design spectrum is a function of the building's characteristics which it is not. At the best I find this confusing. What, in my view, is an inappropriate scaling of the design spectrum also results in a false comparison with a recorded earthquake intensity implying that the recorded intensity was not much higher than the design intensity." In this I'm referring to the February 22nd event. "Secondly, the spectrum representing the average February 22nd 2011 response in the CTV report differs from that in the panel report figure 4.10. The reason appears to be that a different set of accelerograms was used for the consultants' ERSA studies than the generally accepted four accelerogram sets described in the panel's report. The accelerograms are the recordings made by accelerographs, the instruments which were used to measure the intensity of the ground shaking. Four sites were initially selected as those most likely to represent the ground shaking in the Christchurch Central Business District and agreed by the expert panel. The spectra used by the consultants ERSA studies also differs from the spectra set used in the consultants' non-linear time history analyses studies and in appendix E in the ERSA page 233 of the CTV report it is noted that the response spectrum used for ERSA was the average of the Westpac Building, the CCCC, the CHHC and the police station accelerographs." I won't bother to put the full names of these. "Note that the Westpac building and the police station accelerograms are not included in the context report summarised in section 4 of the expert panel report and part of the reason for this is that the Westpac and police station recordings were recorded in the basement of buildings and will be contaminated by the characteristics of the building response itself. They were not free field records which is why they were not used in the panel report. I understand that the consultants' ERSA spectrum is the average for a given period of the maximum ground shaking from the two components of the four accelerograms regardless of the direction of ground shaking. The spectrum so calculated was then applied independently to both the north-south and the east-west direction. It appears that no combination of east-west and north-south response was attempted provided the evidence from ERSA and to a lesser extent NTHA showing that the tortional eccentricity of the CTV building resulted in significant north-south response from east-west excitation and this makes it rather more important to combine the effects of east-west and north-south excitation together.

Q. Now –

A. Yes.

Q. – at the bottom of the previous page the word was 'despite' I think you said 'provided'.

A. Thank you, despite, yes, no attempt was –

Q. Is despite what you mean?

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A. Yes, yes. I'm glad one of us is awake.

Q. (inaudible 15:05:02)

A. The role of the spandrel panels on the east and south faces on the CTV collapse.

In an earlier draft of the CTV report interaction between the spandrels and the columns was stated as the definitive cause of column failure. In other words the definite initiating action in the collapse. This is still the only mechanism discussed in detail in both the CTV report and the panel report, with a series of figures describing the sequence of failure (panel report figures 5.14 to 5.16 and CTV report Figures 17 to 19). These figures show column hinging at or above the top of the spandrel panels and could I have a look at figures 11 and then figures 12.

Here we see the first of these figures. This is from the CTV report, figure 11 itself and there – 17, yeah, as we can see here there is hinging. This is the initial hinging or the deformation of the building as the intensity of shaking increases and you can see that initially there is the potential for hinging noted at the top and the bottom of particular columns. It is then shown that the deformation increases to make contact with the spandrels itself and that hinges form at the top of the spandrels itself in this case itself. Now part of the analyses done with the elastic response spectrum analysis was done on the assumption that perhaps these spandrels were placed without any gap with the columns on at least one side due to difficulties in placing and therefore this gap as shown between the column and the spandrel might not have existed and a lot of play is made in the report on the significance of this to the failure. Could I have the next slide, or just before doing that, you will note here that this is shown as hinges forming at the top of the spandrel itself. Next slide.

This is the next slide from the CTV report figure 18 and you can see here that a rather different picture is shown here. The gap is shown here but when a contact is made, the hinge no longer forms at this location but higher up the column itself. Now this in my view is an impossible situation because of the fact that the moment at this location here is very much higher than at this location which is meant to be at the top of the lap splice of the vertical column reinforcement. Now I believe it is impossible for a plastic hinge to form at this location given the shape of the bending moment diagram which would initiate from contact with the spandrels at this location. So I do not agree with the conclusion implied by these figures itself.

My calculations indicate that it is unlikely for a number of reasons: The permit drawings show an intended gap of 10 millimetres between the columns and the spandrels. If as-built this gap had existed, the interstorey drift prior to contact with the spandrels would have been sufficient to cause the columns to fail without any influence of spandrel/column contact. I do note from having read Mr Harding's evidence that he believes that gaps were provided or exceeded in accordance with the construction approach that was adopted. The analyses in the CTV report supporting the mechanism of collapse were based on a zero gap between columns and spandrel, on one side not on both sides. The strength and stiffness of the spandrel/column contact were assumed in the CTV report to be infinite, meaning that if contact between a column and a spandrel occurred there would be no deformation of the spandrels or of the connections of the spandrels to the beams and that there would be no failure of the spandrels, or the connections prior to column failure. And by infinitely stiff it is like coming up against the most rigid thing that you can imagine if the column hits on to it, it doesn't deform at all it stays exactly where it is. The strength and stiffness, yeah I have – sorry. I am on to D. My calculations on the strength of the connections between the spandrels and the supporting beams indicate they would fail at a level of column/spandrel contact force about 20% of that required to induce hinging at the top of the spandrel. My calculations for the capacity of the end diaphragm of the spandrels to transmit the column/spandrel contact force, using an analysis technique called yield-line theory, indicated an even lower capacity than the value corresponding to the connection capacity. Could I have the next slide please.

This just shows the calculations but it is not very important. What I want to illustrate to you is this is an end-on view of the spandrel. So this is the outside of the building, this is the vertical face of the spandrel, this is the horizontal face and at each end of the spandrel there is a vertical diaphragm which is this piece of concrete connected to the elements here and then these locations, R1 and R2, are where the spandrel is connected to the beam below it. So those are the two fixing points and there is another two fixing points at the other end. Now, this diaphragm here is rather flexible and not particularly strong because there was very little reinforcement in it and it was only 100 millimetres thick and if you look at the way this is likely to fail it implies what are called yield lines, developing where these ragged lines are here and here itself and you can calculate what the strength of the structure would be associated with these, the development of these failure planes itself. And this comes out to be rather small and it is lower than the connection capacity. Photos of spandrels on the ground – I should mention that there has been no dispute with Mr Ashley Smith who has seen these calculations and agrees with this yield-line calculation.

Q. I don't think you have read G yet.

A. What is that?

Q. I don't think you have read G yet, I think you were skipping down to H.

A. No I don't think so, I am still about to yes. Photos of spandrels on the ground after the collapse indicate failure to the end diaphragm, consistent with what the yield-line theory would predict. So you can see here this is now the spandrel itself having fallen down and you can see that this, there should be an end diaphragm in here. It has failed along the plane there and the plane there. We can’t tell whether this other internal one failed but it does have the same shape of failure as we would predict from the yield-line theory. So In this figure the end diaphragm has been completely broken off the spandrel, and the small portion remaining is consistent with the predicted failure pattern.

Point G.
I have seen no evidence of column failure occurring at the level corresponding to the top of diaphragm. Photos of column failures tend to indicate failure at the top of the splices, probably due to high vertical compression force. The moment at the top of the column bar splice the location implied by CTV figure 18 (which is the one where we saw the second of the two figures, where we saw the hinge forming some significant distance above the top of the spandrel), the moment at the top of the column bar splice is much lower than at the top of the spandrel and as a result is not a critical location for formation of a plastic hinge. However, it is a weak location for column compression failure. The CTV report figure 17 shows a plastic hinge forming at the top of the spandrel. However, figure 18 shows the hinge forming some distance above the top of the spandrel. The panel report does not refer to, or include CTV figure 18. The CTV report refers to a line F column failure, probably induced by spandrel contact, as the preferred collapse initiator. However, my calculations indicate that the failure of the F line columns would not result in sufficient load transfer to overload the E line columns to the extent necessary to cause compression failure. These calculations show that the beams connecting lines E and F including full contribution from the slabs between columns would fail as cantilevers at less than 60% of the full axial force on line F. As a result the full axial force on line F could not be transmitted to line E. Now I have since seen, my calculations were based on rather simple back of the

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envelope type calculations. I've since seen more detailed calculations by Ashley Smith who comes up with a value of 70% rather than 60% and I accept his value as being the absolute ultimate capacity of that. However, in my view, though that increases the axial at maximum possible axial load on the E line columns, I still feel that it's unlikely that

that would have caused a sequential failure. If the line F columns had failed this would in my view have resulted in a different pattern of failure to that which is known to have occurred – possibly protecting the interior of the building from failure and leaving more cavities and voids in the building that might have provided safe areas for occupants of the building. It would not have led to the almost, not necessarily have led, to the almost complete concertina effect that occurred, without other collapse initiators occurring. And I should again emphasise that this is supposition on my part. We don't know this for definite, for certain. I also note that the columns on line E have lower axial load due to gravity effects than columns on line D, which, as I note later, are in my view more likely to have acted as a collapse initiator than the line F columns. The comparatively low gravity load on the E line columns made them less susceptible to failure due to the limited possible load transfer from the F line columns. Though that still could have occurred if the displacements had been sufficiently high. The Hyland thesis involves collapse as a result of an east-west lurch and identifies the spandrel–column interaction as the initiating event. Based on the ERSA, this east-west lurch involved high torsional response, with high displacements of F line columns in the north-south directions as a consequence. The nonlinear time history analysis did not predict such high torsional response. However, a combination of east-west and north-south response is predicted by the nonlinear time history analysis to induce significant north-south response on line F.

D, exterior columns versus interior columns as collapse initiators.
And this is C, figure 16 if we can? It seems like we've got the wrong figure there?

Q. It will come eventually. I'm confident of that.
A. I'm not sure quite where I'm meant to be at the moment.

JUSTICE COOPER:

Q. 4202.16 I think we're looking for?

A. Yes I'm wondering if, yes I am, yes. Could I have figure 16?

EXAMINATION CONTINUES: MR MILLS

A. And I'll come to this shortly but what it shows is the inter-storey drift of different columns on two different lines. This is the columns in blue are on the F, oh, sorry on the D line which I believe is more likely to have been an initiator of failure than the F line which is shown in the orange and yellow dots shown here. The numbers here, L4, L3 and so forth refer to the, the level of the column itself. So there's a level 3 column, level 2 column and level 1 column and so forth. The same here, level 5 column and the D line level 4, level 3, level 2, level 1, and they are showing the drift capacity as a function of the axial load, and so they're identified here with the expected axial load and from that we can determine what the expected drift capacity would be. Analysis results (CTV report appendix D; panel report, figure 5.13) indicate that drift demand/capacity ratios at level 3 were higher for internal columns D2 than for the column F2 that the CTV report identifies as the initiator columns. The estimated demand drift is listed in CTV tables 1 and 2 as 1.9% for both line F and line D the exterior and interior columns. Under gravity loads the drift capacity from this figure as shown here at level 3 is shown - under gravity load the drift capacity from panel figure 5.13 is 1.3% for Column F2 and 1.09% for Column D2. These drift capacities are taken from figure 5.13, and also differ from those listed in CTV tables 1 and 2. For some reason this is not shown in the CTV report itself, I think the numbers are correct. The numbers in tables 1 and 2 are a little coarse and I believe have not been interpreted correctly. The resulting demand/capacity ratios are thus 1.46 and 1.74 for F2 and D2. Now the magnitude of that number. Remember it's the ratio of the demand to the capacity. So a number of 1.46 means that the calculated demand is 46% larger than the calculated capacity. So if we compare those two numbers for column D2 the demand is 74% higher than the calculated capacity, whereas for column F2 it's 46% higher, indicating on that basis that column D2 is more susceptible to failure. However if we include the effects of maximum vertical accelerations, the difference between the F2 and D2 demand/capacity ratio increases. And they will also show in this figure here. So if we're looking at level 3, this is the value that we've been looking at before without including vertical accelerations. If vertical accelerations were included the drift capacity drops, the axial load increases to this value here and the drift capacity reduces, and you can see that the drift capacity has reduced from about 1.25, sorry about 1.3 down to about 1.15. If we look at the column on the D line, D3 at the same level, the increasing axial load is significantly higher, it comes to this point here, about 2100 or 2200 kilonewtons and you can see that the capacity in terms of drift has dropped very significantly, much more so than the F line columns so here –

JUSTICE COOPER:

Q. Sorry, the drift capacity for the L3 plus V. Did you say 1.95 or 1.15?
A. I said the drift itself was about 1.15.
Q. Does it look like that to you?
A. This is –
Q. Or more like –
A. – L3 plus vertical –
Q. Yes.
A. – if I move across to there it's between the 1 and the 1.2, slightly less than that, so I would say 1.15. Am I missing something?
Q. Well, I would be very surprised. It's probably me. But if you look at the scale?
A. Yes, this line here is 1.2 correct?
Q. That would be about halfway?
A. No it's not halfway between. Down here is 1 and there is 1.2 and I would say that's about a quarter of the way down from 1.2 down to 1 which would give us 1.15.

EXAMINATION CONTINUES: MR MILLS

A. And you can see then for the D line column with the vertical, increase in vertical load, it has now dropped down to about 0.73, a very much greater reduction itself. So if we consider those effects, for F2 the demand/capacity ratio increases to 1.65, so that's 65% above capacity, but the D2 value increases to 2.44, or 144% above capacity. So we can see that the internal columns are much more susceptible to the influence of high vertical accelerations than the exterior ones and to my mind, considering that we know there were very high vertical accelerations at the site, and the influence was considerable in terms of the gravity loads, that that makes again the interior columns as being a higher probability initiator to failure than the exterior ones. Because of the high gravity loads on the interior columns they were also more vulnerable to low concrete compression strengths. I am aware that some of the evidence the Royal Commission will hear is critical of findings made in the CTV report about concrete strengths and I don't wish to enter into that argument. The point I am making here is that, if concrete compression strength was low, it would have made the interior columns

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more vulnerable to failure than the exterior ones. The demand drift for D2 is based on the CHHC record, whereas the demand drift for F2 is based on the CBGS record. The non-linear time history analysis indicate that the CCCC record is more critical for east-west response which is what effects the internal columns. Can we look at figure 17. So in this figure here what we see are the drift demands from non-linear time history analysis line F on the left and line 1 on the right. Now we don't have a direct comparison for line 2 in the interior columns but it would be similar but less than the line 1. Now we can see on this one here that if we are looking at line 1 here the CCCC record which is this dark blue purple one provides the largest drifts in that particular direction and also in the opposite direction the CCCC is the green one so this is the drift in one direction of response, this is the predicted drift in the opposite direction of response. And whereas we see that what we have been using for the D2 drift is the CHHC which is this value here. If we had used this one then the difference between the line 2, sorry the D line drifts and the F line drifts would have been very much greater so this would again have increased the difference and given an indication that perhaps the interior columns were more susceptible to failure than the exterior ones.

It is also notable from this that this is actually shown as the drift for line 1 because we don't have a direct comparison for line 2. I have mentioned that line 2 will be similar but somewhat smaller. But line 1 is the line, the south most line which has the shear core in the, sorry the coupled shear wall in it but also spandrels and you can see that the drifts under this direction sorry, the drifts on line 1 are very much bigger than the drifts on the F line itself from the non-linear time history analysis. So it would seem to me that if you were going to predict the spandrels had a strong influence of failure on the failure or that if the exterior columns were more susceptible to failure than the interior columns then you would have to make the conclusion that line 1 was more critical than the F line itself. This conclusion has not been drawn in the CTV report.

Right. So both the CTV report and the panel report imply that line 1 was less critical than line F (CTV report p95 “Critical Column Identification, first paragraph, final sentence) though the reasoning is not clear to me and I do not agree with it. The reasoning as I understand it was that despite these very large drifts somehow the coupled shear wall was going to protect the columns from failure and I can't see the logic of that but I may be missing something.

If an interior column failed it would tend to induce lateral catenary action, as a result pulling other columns towards it. Very little displacement would be required only about 30 50 millimetres to cause failure of the adjacent columns. Failure of the F line columns would not result in lateral catenary forces to other columns. I will explain this in a figure in a few minutes. This is because the F line columns are exterior columns and the horizontal forces required for catenary action cannot be sustained in the east-west direction. Can we have figure 18.

So catenary action is just a chain. Very simply if you want to have a chain hanging between two points itself then there are reactions provided at both supports. Here these are rigid walls and it requires both vertical and horizontal reactions itself. The chain itself pulls on the support with a horizontal force in this direction here and a vertical force down there. Now if there was no support at this end itself, you can see that this catenary action whereby this hangs cannot occur and therefore these horizontal forces can't occur so this what would happen if you took that external support away. The chain would just collapse itself. But how does that relate to the building itself? If we imagine here we have an interior column which has failed itself. If it fails the beam does not have enough strength to support the load that it carries by simple flexural action and it will start to sag very significantly. In sagging it then tends to act as the catenary like a chain itself placing both vertical and horizontal actions onto the next point of support which is the next column itself. So these forces here have to be transmitted, have to be resisted and they will tend to create both additional vertical forces which make these columns more susceptible to failure and let's say if the displacement is in this direction when this column fails the additional force associated with this support of the catenary action will provide additional tendency for this column to fail. However, if we have the exterior column failing itself then what will happen as we have seen from the calculations the capacity of the beam here is insufficient to support the load as a cantilever and this will just fail and support on here. There is no catenary action, no horizontal force will be added to the E line column adjacent to this so it does not significantly increase the tendency under lateral forces for this column to fail. So it may not be a terribly significant point but it is just one of many of the points which persuade me that the exterior column is not the most critical one.

JUSTICE COOPER:

Q. I think we can take the balance of the paragraph 51 as read –

A. Thank you.

Q. Because essentially you have told us what it says.

A. Thank you. These results indicate that the interior columns were significantly more vulnerable to failure than the exterior columns, particularly when the effects of the recorded high vertical accelerations are considered. This conclusion differs from that in the CTV report, but agrees with the conclusions reached in the statement of evidence of Graham Frost, which I have read. Mr Frost is a CPEng who acted as risk manager for the USAR and police teams during rescue and recovery work, immediately following the CTV collapse. His interpretation of the collapsed state of the building was that collapse of the floor and beam elements started near interior columns.

HEARING ADJOURNS: 3.33 PM

HEARING RESUMES: 3.51 PM

EXAMINATION CONTINUES: MR MILLS

Q. You're at paragraph 53 as I think you know.

A. Yeah I was on point e., that's right.

Q. Yes.

A. Connection Failure Between The Floor Diaphragm And The North Core. And the figure of 19 is already up. You may well have seen this before from the CTV report. The connection between the floor slabs and the north core as designed and permitted was clearly inadequate to achieve sufficient connection. It should be noted that most of the weight of the building comes from the floor slabs. As a consequence most of the forces induced by an earthquake on a building result from the floor slabs themselves. For these forces to be resisted by a building they have to be transferred to the lateral force resisting members. In the case of the

CTV building this is primarily the north core which is now shown in an upside down configuration in comparison with what the chairman would like. I apologise for that. But we know that the connection between the floor slab, shown up here, and the webs of the core here was particularly poor. This force transfer involves high tension and compression forces and also bending moments. If the connection between the floor and the lateral force resisting members does not have sufficient strength, particularly tension strength, the connection will fail and the earthquake forces cannot be resisted. The consequence is greatly resisted displacements of the floor including the columns which support the floor making structural failure more probable. In the case of a building supported by frames designed to provide the lateral resistance of the building the connection between the floors and the columns is rather straightforward as it occurs at many locations. In the case of the CTV building the connection is primarily to the webs of the north core in very localised positions. These positions where connections were made thus need to be designed for very high tension forces. This high tension force capacity was not provided. In particular, the lack of design connection between the floor slabs and the north core webs on lines D and D/E, that’s this line here and this line here, the connections here between the floor slab and these webs was, was very remarkable, no sorry, the lack of the design connection at these locations was very remarkable. These locations had essentially no tension capacity and would fail under comparatively minor seismic loading placing additional demands on the connection between the floor slabs and the north core between lines C and C/D. This is line C and this is line C/D here. So here the connection was better but if these failed here and here then there would be the potential for a failure to occur between the slab and this core itself which did occur. The eccentricity of the connection as designed was particularly susceptible to failure under east-west response due to the eccentricity of the lateral forces from the north core. However, eccentricity was also present in the north-south direction and would have been exacerbated by failure of the connections of the floor slab to the north core at lines D and D/E. This serious inadequacy was noted sometime after the construction by Holmes Consulting Group and partially remedied by Alan Reay Consultants' installation of drag bars but for some reason, that was not clear to me, these were only installed in the higher floors. They were not installed at levels 2 and 3 and this design inadequacy has been fully discussed in the CTV report. Without drag bars installed at levels 2 and 3 the eccentricity discussed earlier would have been present from the start of the earthquake response, placing additional tension forces on the slab connections at line C and C/D as previously discussed. In my view drag bars as designed for the CTV connection retrofit were a poor alternative to a properly designed connection involving a greater contact area between the floor and the webs of the north core. The method of connection requiring overhead grouting would have been very susceptible to instalment problems. In other words it would have been hard to rely on the integrity of such a connection. The non-linear time history analysis results indicate that failure of the drag bars between the floor diaphragms and the north core on lines D and D/E would occur before column failure though at a rather similar time. Can we have a look at figure 20?

And here we see in this particular case the column failures are shown, the column displacements and failures are shown by these lines here as a function of time and the diaphragm disconnections are shown here. So the diaphragm disconnections are occurring at an earlier stage than the maximum displacements here though here's the column capacity at what is a conservative estimate of column capacity here and here. I believe that these lines here should be ignored as they rely on spandrel interaction and we see here that the failures are occurring before that. The drag bar failures are occurring before the diaphragm, before the column failures.

The calculated capacity of the drag bars that the CTV report assigns to them is, in my view, unrealistically high. It is based on the shear capacity of the bolts connecting the drag bars to the floor and implies perfect placement of the epoxy in pre-drilled holes and infinitely strong concrete surrounding the bolts. Calculations based on concrete compression strength of twice the actual recorded concrete strength in the building indicates the bolts would fail in flexion, not shear, at about 50 to 60% of the strength used in these figures implying still earlier failure in fact. And I have viewed a report by Beca Consultants, commissioned by the Royal Commission, which supports these values using a different type of analysis but the values are about 50% of the values, sorry the Beca values are about 50% of the CTV report values for line D and about 70% for lines D/E. Similarly the non-linear time history analysis results predict that the floor diaphragm failure adjacent to walls on the lines C and C/D would occur prior to column failure. The critical section's about 1.2 metres away from the wall, from the ends of the wall, where the H12 bars terminate and all of the strength is provided by the mesh.

JUSTICE COOPER:

Q. Just H12?

A. H12 at that location.

Q. What's H12?

A. H12 is, sorry, high strength 12 millimetre diameter bars.

Q. Thank you.

A. If you want to, if we could go back to the previous slide.

Q. Number 19?

A. Number 19, yes. I think that's, yes, shown here. This location, this shows failure location of slab disconnection in the region of C, C/D to, to D here and you can see that the, the failure occurs at about 1.2 metres out.

Q. Yes.

A. There were these saddle bars as they're called, 12 millimetre diameter high strength bars coming from the slab here and terminating at this region. Once you got past there, there was only, that's the H12 saddle bar ends stick out about 50 to 120 millimetres. You can see that the failure occurred right at that location itself.

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Q. Thank you.

A. So to read that sentence again, the critical section's about 1.2 metres away from the ends of the wall where the H12 bars terminate and all of the strength is provided by the mesh. The floor would almost certainly have cracked at this location by shrinkage effect, but if not based on the non-linear time history analyses the uplift of the wall ends due to flexure during response displacements to the north, would have been sufficient to crack the concrete at the termination of these H12 saddle bars. My own calculations indicate that the resistance to slab fracture adjacent to the north core flange on line C – that's at the line that we've just seen in the previous slide – up to midway between line C and CD and including dowel action of the anchorage bars of the bends on, perhaps we could go back to 19 again, because I think it would clearer, we could see it – so I did some calculations of the capacity of this failure surface including the mesh bars here and also the capacity in this region in shear of the mesh bars in this location and at this location here a failure between the beam here and the wall flange here because there are H24s, 24 millimetre diameter bars passing from the beam into this region here and if you make the rather dubious calculation that in fact these could fail in flexure or fracture in tension, then you get an upper bound idea of what the capacity of that would be to tie the slab into this web as well. So the calculation I did included that component, included the tension capacity of the bars through here and included also the shear capacity of the mesh in this location over here, and my calculations indicated that the capacity of this indirect tension would be about 800 kilonewtons. If we can go two slides on to figure 21 we see here the forces here, I'm saying that these are the connection forces between grid C to CD and we compare this with the capacity that I calculate of about 800 kilonewtons and you can see that 800 kilonewtons is at about this level here and you can see that it's several levels we would expect to have failure occurring. So figure 21 shows the Compusoft non-linear time history analyses results in the, I'm sorry I should have pointed out that tension is down on this side and compression is up here, so we should be looking at the lower half of this one where you can see that many of the slabs would be predicted to fail. Figure 21 shows the Compusoft non-linear time history analysis results for the connection force between the north core and wall C. Maximum tension forces exceeding 2000 kilonewtons or 2.5 times the calculated slab capacity are shown to have occurred frequently. I have not included any contribution of the Hi-Bond steel trays to the tension capacity of the slab as the Hi-bond was discontinuous at the beam support on line four with very short seating. In addition the statement of evidence of Graham Frost reports that he observed total bond failure between the slab and the Hi-Bond. His statement says that he found not a single section of slab to which the tray deck was still attached. If either the drag bars at the upper level or the concrete slab failed the displacement to the floor under inertia effects would have been greatly increased as discussed earlier. I note that the Hi-Bond trays and the supporting beams would have provided adequate vertical support for the floor until the columns failed due to excessive displacement. In other words, even if that failure plane occurred we would not suddenly get the floor collapsing because it was still supported by the Hi-Bond trays and by the beams themselves. So it would require some significant displacements before that would occur. Thus under a moderate earthquake such as the September 4th 2010 earthquake the slabs, (there's an error in my writing there, that word displacement should be slabs), might not fall unless the maximum displacement response exceeded the available seating length of about 400 millimetres.

Torsional eccentricity.

The CTV report based on ERSA studies emphasises the high torsional eccentricity of the building and the exaggeration of this eccentricity by the concrete infill wall at the lower levels of the west frame, that's frame A of the building. The prediction made in the CTV report based on ERSA was that displacements on line F would be increased by the effect of infill on line A. The non-linear time history analyses indicated the opposite trend. I again note that advice of the New Zealand Society for Earthquake Engineering guidelines thought that ERSA is inappropriate for assessing the response of buildings, particularly when torsional eccentricity exists.

And the next point I want to talk about is the modelling of masonry infill on line A. I also believe that the influence of the western masonry infill wall on line A has been exaggerated in the ERSA. The design drawings for the building specified a gap between the panels and between the panels and the columns and the use of flexible sealant in these gaps. However on the basis of a reflection by Mr Fortune, one of two men working on a scissor lift on the west side of the CTV building on the 22nd of February, it is claimed in the CTV report that the infill panels, three per bay, were constructed with full contact between the panels and the columns and with full contact between the panels. This is despite the fact that it would have been very difficult to place competent mortar to the outside header joints adjacent to the columns from the inside of the building, particularly for the upper courses of block work. At the time the CTV building was constructed there was an adjacent building which made its access from the outside of the wall impossible. The consultants and the panels also had available to them a statement by a Mr Coatsworth, the engineer who carried out a post-September inspection of the building that the required flexible sealant was correctly placed on the inside header joints. In the course of preparing my evidence I have also been advised by counsel assisting that there is now another witness who says that there were gaps at the top of the concrete sections underneath the beams. My view is that it was more likely that there was intermittent mortar on the outside header joints between panels and columns and beams but this would not have been sufficient to lock up the panels. I also note that the top course of blocks under the beams was not grouted and in fact could not have been. This was confirmed by Mr Fortune who said that they found that the top course in each section was hollow and it's also confirmed by the evidence of Mr Harding. Despite these points and the fact that even if grouted the vertical joints between the panels would not have been able to transmit vertical shear, significant vertical shear stress, the frame on line A was modelled in the ERSA as a fully competent monolithic wall for the lower three storeys, with a modulus of elasticity of 15 gigapascals. Information provided in the CTV report on the torsional response of the building is based on this assumption. Now since then I have seen a response from Dr Hyland that in fact the panels were modelled as separate panels but as uncracked panels and he has also said that the fact that there were diagonal cracks noticed in these panels proves that they developed their shear strength and did not fail in flexure. I would dispute that. I don't believe that's a valid interpretation in that we would expect with a flexural failure of the panels based on crack section properties that there would be inclination of the cracks due to shear. So I think that that's an incorrect assessment and the fact that he has modelled them as uncracked in flexure and shear and essentially fully locked in at the top makes them much too stiff. In the non-linear time history analysis carried out by Compusoft and StructureSmith the panels were modelled with my advice as individual flexural elements as an upper bound on the stiffening effect and compared with analyses where the infill panels were ignored as intended by the designer and I should mention that the upper bound flexural elements that we had assumed that the top course was fully grouted so that there was not a length of 200 millimetres of bar which would be very flexible in terms of

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the connection to the top. So I believe that the upper bound value in the, in a Compusoft model was extremely upper bound indeed.

Analyses based separately on flexural action and on sliding shear produced similar additions to the lateral strength of the A line structure. My view is that the flexural model adopted by the nonlinear time history analysis was an upper bound on strength. Displacement capacity would have been underestimated as it was assumed that the upper course was fully grouted, with no slip between the top of the panel and the beam above. It seems clear on the evidence that this was not the case, and I've also seen a photograph after failure which indicates that the panels, well you cannot see any sign of mortar on the vertical cor – the vertical edges of the panels themselves. I believe that they were not grouted.

Q. I could just let you know on that that when Dr Heywood and Mr Frost went out again to Burwood –

A. Sorry, what?

Q. Dr Heywood and Mr Frost went out again to Burwood after they had given their evidence, took another look specifically for that and have now given us two further short briefs in which they included saying that they found sealant on some of the rectangular columns.

A. But not, yeah, the figure that I'm talking about which shows no mortar on those things is, I've lost, I've put a thing through it, it's BUI.MAD249.0222.2 which is there, that's the fellow. Yes, as you can see here, here is a vertical, I mean this would have been presumably in contact with this line here, well adjacent to it, because you can see no sign of any mortar, nor can you see any sign of mortar on this vertical plane here, so I think that we can conclude that it was built rather similarly to intended.

Q. If you like I'll bring up for you the photographs that have come in with that most recent additional brief so you can have a look at it given that –

A. Okay.

Q. – they have been out. I thought that you might like to see it?

A. Yes thanks.

Q. This is BUI.MAD249.0520.2 and this is from their most recent Burwood visit.

A. Yes.

JUSTICE COOPER:

There is a rumour to your right Mr Mills that you may have quoted the wrong number I think?

MR MILLS:

BUI.MAD249.0520.2. If it's wrong it's not me.

EXAMINATION CONTINUES: MR MILLS

A. Well shall we move on, perhaps we don't need to, or do you?
Q. We can pause a minute. I think you'll find the photos interesting.
A. Yeah, sure. I'll find them interesting whether they confirm or deny my theory.
Q. I think they're confirmatory of what you're saying.

JUSTICE COOPER:

Well shall we come back to it? Unless we can have an estimate of?

EXAMINATION CONTINUES: MR MILLS

Q. It sounds like it's coming. There it is.

WITNESS REFERRED TO SLIDE – PHOTOGRAPHS

A. Okay so this is the sealant that –
Q. Yes.
A. – I'm seeing down in the bottom here.
Q. Yes and they've also noted the top one.
A. Oh, yeah okay. Yep, okay, thank you. My next point is related to perception on my part, anyway, that the consultants and probably particularly Dr Hyland had a reluctance to accept the nonlinear time history analysis. I've already discussed several areas where the consultants have preferred to accept the results of the ERSA analysis over the nonlinear time history analysis results. These include potential for failure of the floor diaphragm north core connection to act as a failure

initiator, relative importance of columns on lines 1, D and F as potential failure indicators, extent of torsional response in the February 22nd earthquake response, and the influence of line A infill on the CTV response. In addition to these should be added the importance of beam column joint capacity to the performance of the CTV building. This is only briefly discussed in the CTV report. The lack of ductile detailing in beam column connections, principally as a consequence of there being no transverse reinforcement in the joint, is identified on page 14 of the CTV as a critical design weakness, without further comment. On page 93, the potential weakness is repeated, but claims there that there are uncertainties in the assessment method and that limiting the assessment to column capacity (i.e eliminating the beam column joints as a potential failure initiator) and I quote, "Would be sufficient for the purposes of this investigation." This explanation is repeated in appendix D nonlinear time history analysis on page 228. The Panel report does not include the beam column joint failure as a possible contributor to the CTV report, and that can be confirmed on page 53 and 54. However, the Compusoft analyses described in the Compusoft report on page 65 and appendix B pages 85 to 89 examine the predicted performance of the beam column joints in considerable detail. In other words they had a particular model in their analysis which enabled them to assess whether or not joint failure would occur, and this was based on a principal tension, principal compression stress model. The assessment shows that 18 joints were at capacity between 2.3 and 3.6 seconds after shaking began, which is well before the largest displacements of the columns occurred, and that a further 26 joints, a total of 44, became overstressed between 4.5 and 5.7 seconds. The assessment draws attention to the detail of the beam bottom reinforcement which is hooked up into the joint rather than being continuous through the joint. Transfer of the tension in these bars under positive moment would have created additional tension outside the hook, increasing the probability of joint failure. These points are not included in either the CTV or the panel report, despite being listed in the Compusoft report.

Displacement capacity in the ERSA analysis, which refers back to figure 1, I don't think we need to see it again. The methodology described in the CTV report for estimating the drift on line F in CTV report appendix F is inappropriate for response estimation for the a number of reasons. They are, 1) the analysis is based on the response spectrum analysis. I've mentioned that this is inappropriate before and I don't think we need to repeat the reasons. The inter-storey drifts were calculated as the difference between maximum storey displacements although the drifts in floors do not necessarily occur simultaneously. The maximum displacements are based on member

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stiffness values recommended in codes current at the time of the building was designed rather than 2011 state of knowledge values." That particularly relates to the north core and the south coupled shear wall. The stiffness in frames and lines 1, 2, 3, 4 and F were not included in the analyses. And the results of the analyses incorporating these errors were taken to be the true displacemernts and then applied to an inelastic model of frame F using different member stiffnesses. And I don't believe that the methodology is such that you can get reliable results from such an approach.

Whether the design of the CTV building met best practice.
The CTV report refers to a number of code compliance issues with the design. I have not directly compared the detailing of reinforced concrete members nor their connectivity with code design requirements in place in 1986 in any great detail. However, in my view it is clear that in 1986 the date on which the structural drawings were submitted for permit many of the details included in the building would fail a test of best practice to current state of knowledge."

Q. Can I just pause there before you go on to go down your list of that?

A. Yes.

Q. And just ask you where does this concept of best practice, the current state of knowledge, sit in your view in relation to the obligations that the design engineer has?

A. Well in my mind it’s extremely important. We are obliged to design to codes and it is recognised that codes provide a minimum level of safety and we also know that codes are, can lag behind the current state of knowledge. So if there is information that is available but is not codified then I believe that the designer has a duty to incorporate that information. It may not be a legal requirement but I am sure that the public expects us to use best practice not just to design to the code.

Q. Thank you.

A. And I would say that that has always been taught in my knowledge in structural engineering at the universities. This is a well established principle that you don't just take the codes.

Q. Thank you Professor.

A. “The aspects which I feel did not pass a test of best practice include the lack of ductile detailing for the columns.”

Q. And again I'm just going to ask you to pause for a moment if you wouldn't mind?

A. Yes.

Q. I just want to ask you a couple of questions about that. Do you have an opinion on why there was no ductile detailing in the columns?

A. Yes, from, at least I've perceived a probable reason. In reading Mr Harding’s evidence it’s clear and he states that the columns were designed as pin-ended columns. So a pin-ended column when it’s displaced laterally would not develop any forces and therefore would not be required to be detailed as a ductile or limited ductile column. The requirements I understand in the code were that if the, were the columns had to remain elastic at the predicted response displacements of the building. If they didn't do that then they would have to have ductile detailing. Although he may have conceptually considered that they were pin-ended they weren't detailed as such. The column reinforcement was continuous through the joint and properly lapped so that there would not be any possibility of a pin connection with no moment capacity forming and also the beams were continuous through the joint, so they were detailed as moment resisting. The connections between the columns and the beams and the columns and the column above were detailed as moment resisting with the result that if the columns were displaced laterally they would develop forces and if those forces were sufficient to exceed the calculated strength then they would need to be designed and detailed with ductile detailing which would mean a great increase in the amount of reinforcement. So I suspect that those calculations weren't done. Rough calculations that I've done would indicate that using what I understand to be the recommendations of the codes of the time, I'm not trying to do a detailed comparison here, is that the columns would be modelled as having the full uncracked section stiffness if they have significant axial load which they certainly did and under those circumstances the displacements required to develop the strength would be quite small, only about well less than 12 millimetres.

Q. Yes.

A. I've not done any detailed comparison of that with the predicted displacements but I understand that predicted displacements were much larger.

Q. Yes.

A. So (inaudible 16:25:48)

Q. I just want to ask you one further question around that before you go on if you don't mind?

A. Yes, sure.

Q. Now I don't want to get you into an analysis of the code –

A. Please don't.

Q. – I'm going to ask you a question that will assume certain matters and then ask you to just comment on that, and the, what I want to put to you is something that was dealt with by Mr Jury when he gave his evidence on behalf of the panel, and he had a list of key findings and one of them was, it’s directly related to the point you were just making, that the columns and beam column joints should have been detailed for ductility?

A. Mmm.

Q. Now in his case he was stating that as a code requirement but I just want you to assume for the moment that it is a code requirement?

A. Yep.

Q. And then ask you what you think the performance of the building would have been likely to have been in February if it had had the full ductile detailing in that column and beam column joint that the code would have required if that was the requirement that applied here?

A. I'm a bit reluctant to answer that because without knowing exactly what, how much transverse reinforcement is required I can't make a calculation, but I did do some calculations which if I recall correctly were with the amount of transverse reinforcement that would have been required for a fully ductile, upper limited ductile –

Q. Yes.

A. – and the increase in displacement capacity was very substantial. I can't state definitively that it would have avoided collapse though I believe it almost certainly would have if that had been the case, but that's a bit of a wishy-washy answer. I would prefer to have the data with me to compare that. Certainly I can state that the displacement capacity would have increased beyond the level of displacements that are predicted by the non-linear time history analysis and perhaps that's the safest thing –

Q. Yes, I think that's –

A. I can say that definitively.

Q. Thank you, that's useful.

A. Okay, I –

Q. I interrupted you, you were at B on your list?

A. Yep, what's that sorry?

Q. I said I interrupted you –

A. Yeah, I was up to –

Q. – when you were at B on your list?

A. – I was up to point B which is really related to the first, the excessive spacing of transverse reinforcement in the column which is greater than the column, half the column diameter which would be considered to be an absolute maximum I believe. "The excessive cover of reinforcement of columns resulting in inadequate compression strength of the concrete core in the event of spalling of the cover concrete," and I would point out that there is nothing, the cover was 50 millimetres. That means that there is 50 millimetres of concrete outside the core and there is nothing particularly wrong with that except for the fact that the columns were very small diameter so that if the displacements got to the stage or the vertical load got to the stage that the cover concrete started to spall then you would very rapidly get a great reduction in the axial load carrying capacity of the columns themselves and they would tend to fail just under straight vertical load. "Very high levels of axial compression in the columns". I view the levels as being very high, I note that under the design dead and live load some of the columns were at the absolute limit that would be permitted by the code at the time I understand. From the, that's from the report.

Q. Yes.

A. "Lack of transverse reinforcement in the beam column joints as well as in the columns. Poor connectivity between the precast beams and the columns. Lack of adequate connection between the floor diaphragm and the north core on lines D and DE. even when the Alan Reay Consultants were informed of this deficiency by Holmes Consulting Group in 1990 when that firm carried out due diligence for a prospective purchaser the serious problem was in my view only partly rectified by Alan Reay Consultants." They may well say that they were not needed

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at levels 2 and 3 and I haven't seen the sums to support or deny that so I want to make sure that that's known. "Of particular concern to me is the poor detailing of the columns combined with the high axial load levels. A textbook by Professors Park and Paulay, 'Reinforced Concrete Structures', published by John Wylie in 1975 clearly identifies this as dangerous, and I give the location, section 6.4, pages 217 to 221. This book was published some 10 years before the CTV building was designed and was widely referred to by New Zealand designers using reinforced concrete as “The Bible.” It’s inconceivable in my view that Alan Reay Consultants was unaware of this information and, again, I, I believe that designers have a duty to design not only to the code but also to the state of accepted knowledge applicable at the time of the design and, as I said, certainly the public expects us to design to that standard.

The State of the CTV Building following the 4 September 2010 Earthquake.

It’s entirely possible that a partial floor diaphragm, north core connection failure could have occurred in September 2010 and non-linear history analysis results indicate the possibility of drag bar floor diaphragm failure under the 4 September earthquake”. It’s sort of tenuous. It could have occurred or it might not have. “The displacements the building experienced in the September 2010 earthquake would not have been sufficient to cause complete failure of the building. Because the Hi-Bond trays and the east-west supporting beams would have continued to support the floor it’s conceivable that separation did occur but it was not picked up in the post-earthquake inspections and it was the reason for the, it may have been the reason for the increased flexibility of the building that was noted by many of the occupants. The separation may have been difficult to observe during the post-September 4 inspections. The inspections, the investigators would probably not have known about the drag bar installation and hence would not have paid them attention”. Certainly their report I understand doesn’t mention them. “If fracture of HRC mesh in the floor had occurred this might not have been visible because of floor coverings or it may have been construed as shrinkage cracks as only, crack widths of only two millimetres are required to induce mesh fracture”, and I base that two millimetres on observed behaviour in the Clarendon Towers building where after the September earthquake the floor coverings were lifted and crack widths of two millimetres were observed with fractured mesh underneath them. So even with less than that they may have actually fractured.

Q. Yes, thank you.

A. So it's not a theoretical value but based on experience. "It should be noted, it also should be noted that low crack widths in columns after September 22nd noted in the CTV report as an indicator of near elastic response can be misleading. During the earthquakes the crack widths may have been very much larger but due to the high vertical loads on the columns these cracks could almost completely close up when the shaking associated with the earthquake ceased". So I think this is a difficulty in assessment of reinforced concrete buildings that may not have been fully appreciated in the past and I have given evidence on this point earlier.

"Causes and Likely Sequence of Collapse of the CTV Building on 22nd February 2011.

It will be apparent from my evidence that my views on the critical weaknesses of the CTV building in general align closely with the panel report and albeit to a somewhat lesser extent with the CTV report. My areas of disagreement relate primarily to the relative importance given to the many weaknesses in terms of their potential to act as failure initiators. My view is that too much emphasis is given to the failure of F line columns as the failure initiator and too little on other possibilities. My view is that the columns on line F are unlikely to have acted as the failure initiator but note that I am not saying that they didn't". I'm just saying it is unlikely. "A more likely sequence is that failure of the floor diaphragm north core connection would have occurred early in the response to the February 22nd 2011 shaking, closely followed by distress of a number of beam column joints" and this is based largely on the non-linear time history analysis. "This beam column joint damage would be initially concentrated in the bottom region of the joint and the consequent spalling of the concrete would reduce the column capacity to support the vertical loads and the lateral displacements. The diaphragm north core failure would increase the displacement demands on the columns and failure of internal columns due to a combination of large displacement, spalling of concrete and high vertical loads including vertical acceleration effects would cause, result in explosive failures of the columns and of the joints".

And that ends my evidence. However, I don't know how the, the Commission wants me to deal with this. I received on the 28th of June a request to consider various aspects and I have prepared a response to those. Would you like me to go straight into doing that?

JUSTICE COOPER:
Have you conferred with Mr Mills about this?

MR MILLS:
No we haven't, no.

PROFESSOR PRIESTLEY:
No.

MR MILLS:
I'm really in your hands I think as to how you want to do it.

JUSTICE COOPER:
Q. What's the subject matter of this?
A. This, it comes under your signature Sir.
Q. That, it is -

COMMISSIONER FENWICK:
Q. You've actually responded quite adequately to several of those already, the beam column joints –

A. I have, I have got some other more detailed ones, but if you're happy with no complaints -

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JUSTICE COOPER:

Yes, no that's, I just wanted to know what it was. That's, that's very good of you. Thank you. So if you can –

MR MILLS ADDRESSES JUSTICE COOPER

PROFESSOR PRIESTLEY:

It's about 100 pages of response… not really at all. Some of the questions I couldn't respond to because I didn't feel I had –

JUSTICE COOPER:

Q. You just take us through it as you would wish to. Thank you.

A. There are just over three pages.

Q. Thank you.

A. I perhaps will read the questions first because I haven't incorporated. The first question, or the first series of questions relates to the south wall on line 1 and it's stated in the request for information that the wall appears to have been designed as a coupled shear wall. "Would this wall have behaved as a coupled shear wall in the Canterbury earthquakes?" So it's point A. "In particular would the coupling beams have yielded with plastic hinges forming in each of the walls?"

Q. Now just, just pause for a moment Professor Priestley. We can actually display this because we've given it a number and it's MAD249-

A. Okay.

Q. - .0506.2.

A. That would be good because then it would save me reading them out.

Q. So the one that you've just read appears to be paragraph 2A.

A. Since I've read that out perhaps I can continue to –

Q. Yes.

A. – read my response, yeah, okay, that's great. So I'm responding to point 1A here.

Q. Point, it's paragraph 2A I think.

A. Yeah, it is sorry, I'm sorry it's line 1.

Q. Thank you.

A. Thank you. My calculations indicate that under large inelastic response the wall would have behaved as a coupled wall in the Canterbury earthquake – in the Christchurch earthquake it should say. If the coupling beam capacity is based solely on the diagonal reinforcement, ie. the additional horizontal basketing reinforcement is ignored, the moment capacity of the coupled wall is less than the wall capacity based on the assumption that the entire wall behaves as a unit. That is, that plane sections remain plane across the full five metres of the wall itself. My calculations also indicate that the wall shear capacity was sufficient to allow flexural capacity to be developed and that sliding shear would not be an issue despite the high tensions developed in the tension wall. Yield displacement corresponding to development of wall-based plastic hinges, and I note that spandrel hinges would be expected to form earlier. The yield displacement at level 6 based on a bilinear approximation to the moment curvature response would be about 188 millimetres corresponding to an average inter-storey drift of about 38 millimetres. It's of interest to compare this with the displacements predicted by the non-linear time history analysis which result in maximum displacements for the three records considered of between .3 metres and .5 metres corresponding to displacement ductility demands of about 1.6 to 2.7 related to the roof level which is not strictly correct but they're similar values. Local ductility demand on the spandrels would be expected to be larger. The condition of the wall remnants would indicate that the ductility demand was less than this, particularly in the spandrels. There are a number of possible reasons for this. One, foundation compliance may have increased the yield displacement reducing ductility demand. Two, failure of the floor slab, south wall connection may have occurred at an early stage reducing displacement demand of the wall, though I think that’s unlikely as I'll explain later. Three, failure of the floor slab north core connection may have modified the south wall response in a manner not predicted by the non-linear time history analysis. It’ll be noted that the response to all three records indicate failure of all core connections by 3.1 seconds or

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earlier into the response. At this stage maximum east-west displacements of the south wall were predicted to be between about 60 millimetres and 200 millimetres depending on which record is considered. It is not clear to me that the east-west degree of freedom of the connection between the slabs and the north core was released when fracture of the connection was predicted in the non-linear time history analysis. I think that that is the case that it wasn’t released, it was only the tension capacity. If it was not released then displacements on the south core subsequent to diaphragm disconnection are likely to be overestimate. In other words it would act as the stiffest thing remaining and will get more swinging at the other end of the floor slab on the beams associated with it. So that is not incompatible. So that’s the best answer I can give to point A.

Point B. “What influence would the floors in the building have had on the behaviour of the south wall?” I say that is partly answered in my previous one. Also under north-south response, oh, I have got it written twice there. I will have to figure out what I meant. Now under north-south response, response to the north reduces the axial load supported by the south wall due to deformation of the slab response to the south increases the axial load on the wall but these are rather minor things and I don't think they are what Commissioner Fenwick was questioning.

Point C. “Was there an adequate load path to transmit the inertial forces from the floors into the south wall?” Regarding floor inertia force transfer into the wall, this is principally provided by eight H12 bars acting as shear dowels. It is not clear that this is sufficient to transfer expected inertia forces to the wall particularly when higher mode effects are considered. However there are H24 bars from the peripheral beams on line 1 that are anchored into the walls which can act as collectors and the fact that the wall is captured by the beams at each end provides compression force transfer.

Point D. "How did the design of inertial forces between the wall and the floors compare with the corresponding design actions calculated from the codes?" I am not able to answer that so I will skip that one if I may.

The next relates to the north wall and if we can see there the first question yeah, thanks this is point E. "Given the lateral force resistance in the east-west direction what level of ductility would be appropriate in designing the wall and the inertial forces generated between the wall and the floors?" I had some problems with this and the description of the question is, is in my view not strictly correct as it implies separation of the north core into four walls, no sorry I should at the top, this is not in relation to E but it is in relation to the top, it says, "In this wall complex there are four walls that can provide lateral force resistance in the north-south direction and one wall on line 5 to provide lateral force resistance in the east-west direction." And my response to that is the description in the question is in my view not strictly correct, that it implies separation of the north core into four walls acting in the north-south direction and one wall acting in the east-west direction. I suspect that that is not the intent. it is just how the wording comes out. In fact all walls contribute to both the north-south and the east-west direction. I note from Dr O'Leary's evidence which I have read that he also separates the components of the north core into walls acting in the north-south and the east-west direction. This causes me to wonder if the same artificial separation was made in the design process resulting in an artificially low estimate of the east-west flexural strength with consequence of high over strength factors, and I suspect this may have been the case. Point E then given the lateral force –

Q. Well before you move on, how do the north-south parts of this web structure act to resist east-west motion?

A. They act as flanges to the wall, so if you just take the wall on line 5 by itself –

Q. Yes?

A. There is a certain amount of reinforcement in it but it is not divided. It is not separated from the flange that is adjacent to it so if the wall is in tension on that side it is also pulling up the flange immobilising the reinforcement in that as well. So a design, and I will mention this I think in more detail later, a design based on ignoring the north-south wall when you calculate the amount of reinforcement required in east-west wall, in the wall on line 5 would be very conservative and safe in terms of the capacity, but in terms of determining what other forces might occur such as the shear forces that are carried by the webs and the connection forces elsewhere, it's very, very non-conservative. Now, I'll discuss that later. Okay, so point E –

COMMISSIONER FENWICK:

Q. The thought that anyone would do that would not have crossed my mind when I wrote that question.

A. What's that sorry?

Q. The thought that anyone were to do that, I said that it would not have crossed my mind when I did write that question.

A. No I didn't think it would, that's why I said I didn't believe that this was the intention. But I do believe that this may have been the case and anyway, I'm not going to go any further into that.

"Given the lateral force resistance in the east-west direction what level of ductility would be appropriate in designing the wall and the inertial forces generated between the wall and the floors?" And I said that the discussion that I've already said about the separation into separate north-south and east-west walls is relevant to this. My answer is based on the north core acting as a unit not as individual wall elements.

Simple hand calculations I made in August 2011 indicated a maximum strength of the north core in terms of base shear strength of about 12,800 kilonewtons corresponding to a base shear coefficient of about .37G. The value from Compusoft's pushover analysis was about 10,000 kilonewtons, or about 20% lower than I calculated, corresponding to a base shear coefficient of about .29G. The discrepancy between the two values can be explained by the coarseness of my hand analysis, all reinforcement was assumed to be yielding at probable strength, and also to the influence of foundation rocking which modified and reduced the response of the wall in the nonlinear time history analysis. Assuming a fundamental period of one second the 1984 design elastic acceleration response ordinate is about 0.43, hence the expected structural ductility demanded co-level response would be about 1.2 to 1.5. Even under the Lyttelton earthquake response spectrum, ductility demand of the north core would be predicted to be low. Less than three, and that is with the structure remaining completely competent.

JUSTICE COOPER:

Q. Now, you're using terminology that we haven't been using, is Lyttelton the February?

A. Sorry, the Lyttelton is the February earthquake yes. This however is not the design value but an assessment value. The question posed in E is I believe ambiguous in that the actual strength would not be used in choosing the design ductility, instead the design ductility would be chosen and then the strength conservatively matched to that. Since an ERSA approach was adopted for the design the base shear strength would have been based on 90% of the 1.0 second value using total building mass, reduced by a ductility factor of about 5, or it could have been. In other words about 0.08G. If the north core had been subdivided into separate north-south and east-west elements as discussed above, then the strength would've been greatly underestimated, although this would be conservative for design resistance, it would be non-conservative for reactions that, or actions that required to be capacity protected such as shear in the wall itself and also connection forces between the slabs and the wall itself. The higher values corresponding to an assessed strength, a ductility of about 1.5 as above should've been used. It's not clear to me that ARCL, that's Alan Reay Consulting Ltd, had specifically considered the load, oh, sorry I'm onto the next point, F.

Sorry, which is: "What was the load path for shear transfer between the floors and the wall complex?" It's not clear to me that Alan Reay Consulting Ltd had specifically considered the load path for shear transfer between the floors and the north core. Given that the core was concentrated near the centre of the building, the inertial forces from the outlying sections of the floor would need to be transferred by truss action, implying diagonal compression forces and a collector tie along line 4. This does not seem to have been considered. The eccentricity of the lateral resistance in the east-west direction would also require moment as well as shear to be transferred across a rather small interface between the slab and the north core. A lack of adequate connection between the slab and north core at lines D and DE has already been discussed at some length.

Point G. "Would the wall complex

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warp under the action of this shear transfer? Can you account for the observed vertical cracking in the wall complex?" Now I have some problems with this just due to my own ignorance and I know Commissioner Fenwick knows a lot more about this than I do so my answer may not be right. Section warping of the north core would have been largely restrained by the slab provided the connection between the slab and the core remained competent. Once drag bars or other connectors failed at least partial section warping would be possible. This should I believe be picked up in the nonlinear time history analysis. Note that restraint of the section warping would result in additional stresses in the walls. I'm not familiar with the vertical cracking in the wall complex and I haven’t seen this in any figures so I can't actually comment.

COMMISSIONER FENWICK:

Q. I can tell you if you like –

A. Yeah, I’d be interested yeah.

Q. There were two cracks observed in the – Graeme Smith travelled up in the lift.

A. Yeah.

Q. After the September earthquake and found two cracks running the full height of the wall measured between .5 and .8 millimetres of width and they were in the space where the lift went up towards the wall on the west side of that gap. He said about a metre and 1.5 metres from that wall and the cracks went the whole way. So tended to indicate there had been some bending. They were only visible on the inside, not the outside.

A. Only on the inside, so they were opening of the flanges, yeah. Well I sort of think I’ll stick to my saying that I can't answer that.

H. “What other structural actions are associated with shear transfer from the floor into the structural wall complex?” I'm not quite clear what this question is examining but I think my answer to F at least partially addresses this.

And I. “Is the detailing of the junction between the floors and the wall complex adequate to resist the shear force and the associated actions?”

Q. You've already answered that.

A. I think I have, yeah. I just, yeah, I think I don't need to go any further into that. I do have -

Q. No.

A. – some comment that just says that note that conventional capacity design theory would predict that the connection forces in the Lyttelton earthquake would be no larger than under the much lower design intensity as it would be constrained by the flexural strength of the north core which would be (inaudible 16:52:44) in both units. More recent investigation in the capacity design forces would indicate that higher connection forces would be expected under Lyttelton earthquake as a result of increased higher mode response which would not be limited by the wall flexural strength.

Point J. "How does the predicted magnitude of shear force transfer between the floors and the wall complex correspond to the corresponding design value found from the two code levels?" Again I'm not going to answer the code things but I believe I've already answered this. The predicted magnitude of connection force should be related to the actual expected flexural strength of the north core, not the minimum required flexural strength of the wall corresponding to code required lateral strength. This is a fundamental aspect of capacity design which was required in 1984. Using the approximate values listed above the difference between the 1984 required strength and the strength based in the Compusoft pushover analysis is a factor of 3.6. So if it was only designed for the connection forces corresponding to the code required level, then it would be severely undervalued.
I think that that it's – you know the rest I've got is not relevant, and leave it at that, thanks.

HEARING ADJOURNS: 4.55 PM

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