HEARING RESUMES ON TUESDAY 10 JULY 2012 AT 9.35 AM

MR MILLS:

There's just a matter that Mr Palmer wants to raise with the Commission.

JUSTICE COOPER:

Yes, Mr Rennie.

MR RENNIE:

I think it's actually going to fall to me to raise it Sir. I'm obliged to my friend. Sir, counsel assisting has raised with me the matter of a brief of evidence which we filed, a supplementary brief, yesterday, by Professor Mander.

JUSTICE COOPER:

Yes.

MR RENNIE:

It had been my intention to deal with that when we got to Professor Mander but I'm happy to deal with it now Sir.

JUSTICE COOPER:

Yes.

MR RENNIE:

Professor Mander arrived at the weekend and I met with him on the first occasion and I identified from discussion with him that he expected to orally supplement his file brief to an extent that I considered went beyond anything that would normally be regarded as oral supplementation. As a result of that that brief was prepared. It was approved by him late yesterday and filed Sir. That wasn't intended to convey any assumption at all that the Commission would automatically accept it. It was simply intended to provide the earliest possible notice of its content. I appreciate that that raises issues for the Commission and for other parties. The background as I've put it Sir is simply as I've already stated that these are matters that Professor Mander relies on in relation to the evidence he intends to give and my view as counsel was that that should be put in. Now I appreciate that we need to make a formal application for that Sir.

JUSTICE COOPER:
Yes.

MR RENNIE:
Those are the grounds for it.

JUSTICE COOPER:
Right.

MR RENNIE:
Now there is one associated matter Sir and that is we also filed a PowerPoint from Mr Bradley and, unhelpfully, there are two Mr Bradleys and this is the Mr Bradley retained by us in relation to, well principally in relation to the investigations of the seismic testing sites. That is not new material Sir. That is a PowerPoint summary that Mr Bradley has prepared which is his intended way of presenting his brief if that is convenient to the Commission and, and in relation to that Sir –

JUSTICE COOPER:
I don’t need to be addressed on that. Others have done it in the course of the enquiry and the important thing you have said in that connection is that it’s not new material.

MR RENNIE:
It is not new material Sir and is simply a presentation of that material. In addition Professor Mander relies on some of that material, not only for the supplementary brief but for his primary brief.

JUSTICE COOPER:

Well it was attached to his brief wasn't it?

MR RENNIE:

Exactly Sir, yes.

JUSTICE COOPER:

Well the PowerPoint wasn't but the material on which the PowerPoint is based was attached to his exhibit D I think, yes.

MR RENNIE:

That's exactly right Sir.

JUSTICE COOPER:

So I don't see any difficulty with that. On the other issue you need to be aware I think that when matters are given to us we feel obliged to endeavour to read them even if they're not accompanied with an application for leave because of an assumption that it may be important for the witnesses who are presently giving evidence. Now I will hear anybody who wishes to oppose this second brief. Mr Rennie is there anything else you wish to say about it?

MR RENNIE:

Well the only other thing I wish to say to you Sir is that our overseas witnesses having now arrived I am expecting this to be the last occasion on which I'm having to explain a matter of that nature.

JUSTICE COOPER:

Right, good. Thank you. Is anybody opposed to Mr Mander's second brief being called? Mr Allan?

MR ALLAN:

Sir I haven't read this brief. I don't anticipate that I will be having instructions to oppose it but I'm simply not in a position to advise the Commission right now.

JUSTICE COOPER:

Yes, all right. Mr Reid? Mr Clay do you wish to be heard on this?

MR CLAY:

I'm in the same position as Mr Allan Sir.

JUSTICE COOPER:

Yes well look we will hear it and if there are issues that arise from it that affect other people well we will find a way of dealing with that whether by re-call or video link or whatever. So Mr Rennie I think you've responsibly reacted to this. Obviously it needed to be in writing because it's quite extensive and it refers to references and so on which would have been most inconvenient.

MR RENNIE:

I'm obliged to the Commission Sir and I was simply going to say that in the spirit of the Inquiry if the two of my friends who have reserved their position need to have access or explanation or something we will facilitate that Sir.

JUSTICE COOPER:

Thank you. Thank you very much.

JUSTICE COOPER ADDRESSES MR MILLS

MR MILLS ADVISES JUSTICE COOPER THAT MR HOLMES REQUESTS TO LOG SEVEN NEW SLIDES

JUSTICE COOPER:

Yes.

MR MILLS:

Now I haven't seen them, I don't know whether they involve anything new but, again, I just ask that it be dealt with in the same way. If people have got issues around it then no doubt they'll be raised and I should also note, again on the same basis, that we've got, as my friend knows, further briefs from Dr Heywood and Mr Frost which result from them going out to Burwood after they gave evidence and they are short briefs confirming what they saw there. I don't think they'll raise any difficulties but, again, I just raise it as a matter of formality.

JUSTICE COOPER:

Yes well I think nobody's so far been critical of their methods so it is unlikely that they'll find a basis for objecting to these gentlemen telling us what they saw.

MR MILLS:

Yes I'm sure that's right but I just thought we better keep the record straight on this.

JUSTICE COOPER:

Yes all right. Thank you.

MR MILLS:

Thank you Sir. All right just a few more questions then for you Dr Hyland and Mr Smith and the first one that I wanted to ask you about is an issue that came up in passing I think in an exchange between you and His Honour and this is about these vertical cracks in the lift that were seen by Graeme Smith. You know what I'm referring to don't you, and the note I made, any rate, as I listened to the exchange was that you, Dr Hyland, said that you were interested in that and I'm just inviting you, if you've thought any more about it and whether you have anything more you'd like to say beyond you're interested in those vertical cracks to let us hear from you now and the same applies to you Mr Smith - because these were not things you were aware of at the time you wrote your report were you, either of you?

MR SMITH:
No.

(Justice Cooper addresses witnesses – issues of audibility)

MR MILLS:
Any further comments you'd like to make, any further thoughts you have on the significance of those vertical cracks assuming they were, and I should just note that Mr Coatsworth thought they were construction joints but he didn't view them with the care that Mr Smith did. So for the moment treating them as vertical cracks in the shear core.

DR HYLAND:
So Mr Coatsworth, he's been interpreting Mr Smith's –

MR MILLS:
He has.

DR HYLAND:
– evidence, right, I thought they might have been that. Okay I have been reflecting on it, had the minute from the 22nd of June from the Commission where they talk about that. My thinking is possibly that the cracks may be due to some sort of warping of the, of the shaft from the east end, perhaps moving outwards causing some out-of-plane flexure locally at those locations. When that occurred I'm not sure. It could have been before the drag bars were put in, early on, it may have been after, so that's just -
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MR MILLS:

Yes.

DR HYLAND:

That's my thinking.

MR MILLS:

Any thoughts from you Mr Smith?

MR SMITH:

I mean, I concur with Dr Hyland. I am not, I didn't expect they would be construction joints as we thought the construction joints would be horizontal such cracking of a warping nature is quite reasonable explanation.

MR MILLS:

Yes, and no thoughts on when that might have occurred and whether that tells us anything significant about the building response?

MR SMITH:

Not really, I mean, yeah, no, I'm not sure when they could have occurred.

MR MILLS:

Just on this question of warping, my co-counsel has just passed me a note saying, "Wouldn't warping have affected the lift use?" If that's the cause?

MR SMITH:

Oh, we're talking about, it's interesting it's come up now. I actually telephoned, when we did the survey after the collapse of the lift core and we found some out of alignment, I telephoned the lift company, Otis, I can't recall the name of the person I spoke to but he was aware of the project. I told him about this out of alignment and that we would like to know if he had any record of that being a difficulty during construction. He checked his records and confirmed there was no record of anything there and so that, that would, I guess, cover your question about the warping that that there wasn't any record of them having difficulty with the alignment of the lift, whether it was warping or leaning from that point of view so I do have experience on other projects where taller buildings have been built somewhat out of alignment and it has caused difficulty for the lifts so we would expect that to have been recorded if that had been the situation during construction.

MR MILLS:
So you're saying that this is something that's occurred subsequent to construction:

MR SMITH:
After construction, yes.

MR MILLS:
And with the degree of warping that you're referring to that might explain the vertical cracks, would you expect that to have been such that it would also have affected the operation of the lifts?

MR SMITH:
No, I wouldn't think so.

MR MILLS:
I want to ask you now about an issue that Graeme Frost raised and it was touched on in a question that was put to you by my friend, Mr Rennie, and this is one of three theories that he, I think they're not put any stronger than that, that he referred to at the end of his evidence and this is the one that relates to what at least I call his wings theory, the jaws theory, the jaws around the connection between the column and the beam and because he has put a lot of work into the work he's done and because he said in the course of his evidence he'd like to hear the views of others on this, I'd just like you to take a look at this, and I'll just bring up the drawing which I think is the one that will remind you of this, and if I've got the number right, it's BUI.MAD249.0371A.2, it's a drawing that we've had before during the course of his evidence. Yes,

that's it there. Now I think you know that in his evidence he said that, I think he went so far as to say that they didn't see a single beam where those curved ends, the jaws if I can use that terminology for the moment, had not come off. Are you aware of what he said then about what he thought that might mean? I can you to it in transcript and refresh your memories if you, or give you information about what he said if that's necessary, would that be helpful? Or do you know what he's –

MR SMITH:

I think you're talking about, I don't know if I've got a mouse on here, these diagonal cracks here? So breaking off at this location?

MR MILLS:

Correct, and the thoughts that he had on it, I'll just perhaps remind you of them because I've got the transcript here. I'll bring it up if we need to. But what he said, he's referring to that sketch there, and he said, "It demonstrates to me a possible failure mechanism," and this for the Commissioners is page 38, beginning of page 38 of his transcript which is TRANS20120627.38 and it follows on from that, just so that you've got a note of this. He says, "If the unconfined wings at the end of the precast beams split off, these are potential crack lines I've shown here where those wings could break off, as I postulate was a very likely scenario if the building was subjected to very high vertical accelerations during the February earthquake. The concrete in the compression zone between the bottom of the narrow, remaining section of the beam and the column in-fill concrete would have been under much greater compression than ever anticipated in design," et cetera, then he says, "In this sketch," and this is the sketch that we've got up on the screen right now, "I have shown some red arrows indicating the direction of the compression stresses that would be operating in the bottom of the beam where it met, where the beams met at a column joint. In most buildings where you have pre-cast you would typically have square ends," and so on. Then he says, "Where we have these curved ends on the end of the pre-cast you can see these compression loads can't transfer straight across, especially when we have a very smooth concrete surface…" "So the combined effect of these compression loads coming in from the beam being met by compression loads coming out radially is that you end up with very high forces trying to split these sections, these wing sections off and there's no reinforcing in those wing sections to keep them confined. So I think there's a very strong possibility that a pulse vertical acceleration would create a slightly higher moment, or a much higher moment, temporarily at that joint and could easily be sufficient to break those corners off at which stage they have very little capacity left". Then he goes on to make the point that he looked at over 50 of these in the building, sorry, there were over 50 in the building, he looked at at least 20 or 30 beams and, "every beam specimen we found had no wings". And then of course he goes on to postulate which is what I'm inviting you to give any views you have on that once those wings had come off the capacity to transfer loads was hugely diminished and he said at the end of his evidence when he was listing three possibilities that it occurred to him about collapse after being on site doing all this forensic work that this is the one that he lent towards as a cause of beam column failure. So I'm just inviting you, if you've got any views on that, to let the Commission have the benefit of them?

MR SMITH:

I think the, we had the view we were aware when we were looking at the photographs of, well I should say I was aware, just keep it to me at the moment, I was aware when I looked through the photographs after September that even if there had been some damage to beam column joints we may not have seen it because of this effect that the beams in effect masked the joint so that we could not see the interior of the joint but having said that, you know, there was no damage reported and there was a limited number of beam column joints that were photographed or in that report of CPG. The effect you're talking about now is something I specifically talked to the guys at Compusoft when they're carrying out this further non-linear analysis trying to model beam column joint behaviour that they did recognise this effect of these beams circling the column and in effect reducing the joint zone so they are attempting to take account of that in this further analysis so you know I saw many beams of these internal beams. The other point that I noted I could see this semicircular, can I have the mouse back please? Yeah, look I, I didn't take careful note of whether the crack was here or further out but I certainly saw this semicircular surface here and the bottom bars of the beam projecting largely undeformed from their original shape so that they'd pulled out of that joint without deforming which was an indication of a weak joint.

MR MILLS:

And I take it you would agree with what Mr Frost was brooding on, which is that once those wings came off the available connection, the capacity to carry loads across is significantly reduced?

MR SMITH:

The contact area is reduced so the stress goes up, yes.

MR MILLS:

Dr Hyland? Anything you'd like to add to that?

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DR HYLAND:

Well I guess just the issue I guess is that the vertical support in there would've been still there with the bars going into the, into the beam column zone, if he's just talking about a vertical pulse sort of thing. So there was still the concrete that was in there and the reinforcing steel. We didn't see the reinforcing steel with significant bending in it. The hook still seemed to be as if they were stressed if you like, they were just still as, as they were, so they didn't appear to have been significantly deformed. You know, they weren't sort of bent up, kicked over, that you might expect if, if you did have that vertical pulse. The shear reinforcing in the beams normally stops a certain distance from the face of the column just as of, as of right, as normal so the shear capacity is, would be as, as assumed I guess. The spalling of concrete is, is known to occur in seismic frames. I guess in this case the frame wasn't designed for seismic purposes so, there's that, but normally a bit of spalling isn't considered to be a major structural issue, you know, so.

MR MILLS:
I think he's more concerned about load transfer than structural issue per se, at least as I understand his evidence?

DR HYLAND:
Yeah I don't see there's a big change in load transfer. I could understand there being a bit more concentration perhaps into the, into the zone where the, where the actual reinforcing steel went in. But then, you know, usually there's, there's a, the depth of zone, the compression zone in the concrete that you're relying on is reasonably small and it can cope with a little bit of, a bit of movement so, I mean I think it's an interesting observation, yeah.

MR MILLS:
Well I don't think I've got anything more for you. Before, as I discussed with you this morning and as I've raised with the Commission, before you're actually excused we have to tidy up an issue about supplementary briefs and so on but I'll do that after I've seen if anyone else has got anything they want to ask?

MR ELLIOTT:
Dr Hyland and Mr Smith, only three short topics. Firstly, you saw the drag bars in place during your inspection of the north core I take it?

MR SMITH:
Yes.

MR ELLIOTT:
How did you become aware that they were put in place during a retrofit?

DR HYLAND:

My recollection is that we weren't expecting to, I wasn't expecting to see any drag bars there. I wasn't, there weren't any on the drawings that I had and so that then led to questions about how'd they get there.

MR ELLIOTT:

Questions to who?

DR HYLAND:

I'll just try and recall the events. I think I may have asked, put a question to Alan Reay on that.

MR SMITH:

I just clarify that I think it was in the same visit to Christchurch, Dr Hyland and I both visited Christchurch to inspect the core and also to meet with Alan Reay.

DR HYLAND:

That was the second visit.

MR SMITH:

The second one?

DR HYLAND:

Yeah. Oh, sorry.

MR SMITH:

I'm just trying to recall, so whether we saw them first or whether we were, Dr Reay mentioned that he had a vague memory of some, some alteration that was done that he didn't have records on at the time we spoke to him but it led me to call, he pointed me to Holmes Consulting and I called John Hare of Holmes Consulting who provided me with their report, so that was the first that I saw of their report.

DR HYLAND:

I had a slightly different, different approach there, you know, because I investigated the call prior to you so.

MR SMITH:

Prior, yeah, okay, sure.

DR HYLAND:

And I, we made inquiry, I made inquiry through Department of Building about it eventually and they put a question to Holmes to see if they could find information about it.

MR ELLIOTT:

Secondly, there's been a great deal of interest in this demolition of the neighbouring building as you know, and I'm just going to refer you to your conclusion in the report which is BUI.MAD249.0189.88 and as that comes up I'll just read out to you what you say in your report about that. "The authors consider it unlikely that structural damage was caused by the demolition sufficient to affect the earthquake resistance of the CTV building." This is the last paragraph, "This is because it is common practice to use such equipment for demolition work like that seen in figure 29 and not to cause any significant structural damage to adjacent buildings." My question just is your conclusion that the demolition was unlikely to cause structural damage, was solely based upon it being common practice for that to happen without causing damage, is that right Dr Hyland?

DR HYLAND:

Well I mean the sort of equipment that you can see in the photo and that's being used there is commonly used in demolition of, of buildings, and you know the, this sort of energies and effects that that sort of equipment causes is, is generally accepted not to have, not to cause significant damage. You can get damage when you know people are digging out underneath or adjacent to buildings and you get settlement and things like that, but it's not normal to expect, you know, serious damage to occur.

MR SMITH:

I've got, I could offer some further information. We were aware that that was a recurring theme from our interviews with tenants that it was obviously a big issue for them. There may be a perception that we didn't give it adequate coverage in our report. Certainly, and perhaps the explanation that, you know, some of the explanation is heightened sensitivity of people to vibrations in that situation which was not intended to mean that they were not real effects, so I certainly, I have felt vibrations from demolitions and I know, you know, we certainly appreciate they are real effects, so they weren't discounted, those tenants' views. The other point I'd make is that I have experienced on a number of times doing what they call dilapidation surveys, so you would inspect the building where construction work is planned on the adjacent site to survey the building for damage before construction and then again after construction, so that any effects of the construction on the adjacent site that have caused damage can be, shall we say the contractor of the adjacent site can be held to repair that damage. So I've done that on a number of occasions. This was a somewhat different situation, I guess we could call the CPG report the pre-inspection because that was prior to the demolition work. The different situation than normal is that obviously recorded quite a bit of internal damage, so you know, it's not the normal situation when you've got a dilapidation survey. I think, we are aware or I became aware when I saw Steven McCarthy's evidence that they used procedures in the demolition that weren't approved, including a wrecking ball which would've caused stronger vibrations, but I still, you know, we still have the view that that did not impair the seismic resistance. It would've certainly led to big vibrations in the building. The main interest for us relating to this is the floors and we've already explained that the floor design was quite a long span for that type of floor system, so was reported even prior to the demolition as being lively, and so vibrations could've been felt, which is not surprising. We are trying to, even our original analysis did a sensitivity study on the floor stiffness. So the floor stiffness is quite important to assessing the variation in column loads that occur due to vertical earthquake accelerations and the stiffness you assume for the floors is quite critical for that. So our original report tested the sensitivity of that, the latest information Professor Mander has a view that the floor may be somewhat more flexible again that what we assume, and so we're considering that further. But the main interest is for the effect on columns, so it does affect how much axial load the columns feel from those vertical vibrations. So that's a bit more explanation than, than what we put in there.

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MR ELLIOTT:

So even though you have referred to heightened sensitivity you weren't saying that these things didn't happen, you accepted that the tenants felt –

MR SMITH:

Absolutely. Definitely.

MR ELLIOTT:

And then you were assessing whether those vibrations may have reflected in damage to the building?

MR SMITH:

Absolutely.

DR HYLAND:

It can get quite, you know the amount of displacement to get a feeling of discomfort is quite small. You know it only can be in the order of a millimetre or two and you know we can be sensitive to accelerations of you know, .25 G you know, sorry .25% of G so quite small accelerations can annoy us and cause us discomfort.

MR ELLIOTT:
Would it have been possible to do some sort of calculation to quantify the amount of energy or shaking produced by the demolition work next door?

DR HYLAND:
Yeah you could, yeah I mean you could look at the dropping a ball six metres, work out that yeah.

MR ELLIOTT:
And could you then take that energy and compare it to how it might affect the building and its components?

DR HYLAND:
Yeah, yeah could be done.

MR ELLIOTT:
But I suppose you are saying that you think you would have arrived at the same conclusion even had you done that type of quantification?

DR HYLAND:
I believe so yeah.

MR ELLIOTT:
Finally your conclusion in the report about the effects of damage after 4 September and after Boxing Day which is also amongst other things an area of particular interest. The conclusion you have reached was that there was no evidence of significant change to the building's seismic resisting capacity from either earthquake. That is right isn't it, that was your conclusion on both fronts?

DR HYLAND:

I think the only new evidence we've had that was relevant was the photograph of the connection of the column to the core which did show signs of that connection failing shall we say, so that had, would have had some effect on the response. In effect we had assumed in our analysis to disconnect that connection anyway because we realised it had limited capacity and you know, so, certainly for the February shaking we disconnect it for our analysis because we anticipated that.

MR ELLIOTT:
So dealing with that latter point, to the extent there was diminished capacity due to that area you have just described, would that have been relevant to the collapse scenario that you have preferred?

DR HYLAND:
Well it wasn't – we don't believe the collapse initiated at that location of the building but it may have affected the overall response to some degree because the north wall is such a key raising element.

MR ELLIOTT:
What do you mean by affecting the overall response?

DR HYLAND:
Well it had a stiffening effect on the north core, that column being connected to it.

MR ELLIOTT:
And that stiffening effect was diminished?

DR HYLAND:
That is correct.

MR ELLIOTT:
Due to the damage?

DR HYLAND:
That's right.

MR ELLIOTT:
What does that mean in terms of the behaviour of the building on 22 February do you think?

DR HYLAND:
Well our analysis assumed it was disconnected so we are, if you like taking a low – depends which way you look at it, lower bound or upper bound on deformations from our analysis but in effect that was what we considered to be the most realistic representation.

MR ELLIOTT:
Well just putting that issue to one side and dealing with what you have said in your report about there being no evidence of significant change based on the evidence you had at that time, we know that Mr Coatsworth took over 100 photographs of cracking and damage and various parts of the building and that Mr Pagan assessed damage which would cost $290,000 plus GST to repair so it seems there was a great deal of damage and cracking to the building?

DR HYLAND:
Yep.

MR ELLIOTT:
But in effect what you are saying is that the presence of a crack doesn't necessarily indicate that seismic capacity of a building has been diminished, is that right Dr Hyland?

DR HYLAND:

Yes it is and I think it is important to realise that the only, in all the inspections only found one column between level 1 and level 6 that had a crack in it, so, and that one was a, Mr Coatsworth felt wasn't significant enough to require the epoxy injection. The other three columns that were damaged were up on level 6. There was the one, C18 which was connected into the core which looking at it now with the connection to the head indicating that it may have suffered some significant damage in September. That's sort of a special case column because it is part of the core. The other two –

JUSTICE COOPER ADDRESSES WITNESS – MICROPHONE

DR HYLAND:

So we had C18 at the time that was damaged and was reported widely. The other column at level 6 was out on line 1 near grid, between grid B, A and B I think out on that side and that was reported by Mr Coatsworth and David Bainbridge, he reported seeing a column around about grid line 3C at level 6 with some cracking in it that he was concerned about. So there were two columns at level 6 that were damaged. Now the thing about columns that are sitting on top of a building is that they respond to the building itself response so they actually have an amplified response because they tend to act as cantilevers off the top of the structure, whereas the ones below the structure are moving with the structure as a unit so it is not unusual to expect to see perhaps more cantilever type damage at level 6 than those ones but in fact we only saw one column reported that had damage or a crack down at level 4 line F4 indicates there really wasn't significant structural damage to those columns.

MR SMITH:

I am happy with that explanation.

MR ELLIOTT:

So what you are really saying is that one can categorise damage cracking depending upon whether it is related to structural elements or not, is that right?

MR SMITH:

Yes I mean there was a lot of damage to non-structural performance in the partitions and that's quite an interesting observation and may be one that you know, could indicate that there may be was quite a bit damping perhaps resulting or some sort of effect of that to the structure, you know, I am thinking about the response of the floor, the vibration of the floor, whether there was something going on there with movement in the floor that may have increased structural, non-structural damage to partitions but it is just thinking at the moment.

MR ELLIOTT:

Mr Smith you referred just now to in response to a question from Mr Mills to a potential to the potential for cracking in the beam, around that beam column joint area, would that not have been visible on a visual inspection?

MR SMITH:

Well it is difficult for us to judge. We can only go on the photographs that were provided and we didn't see any evidence of that in those photographs. It also wasn't raised in – the report by CPG did not identify any cracking there so that's, it was a limited coverage and there was no report of that type of cracking so that is all we had to go on.

MR ELLIOTT:

Do you think, do either of you think the engineers would be assisted in post-earthquake inspections by having any sort of tools to help them carry out inspections in addition to just a visual assessment?

DR HYLAND:

What do you mean?

MR ELLIOTT:

Are you familiar with FEMA 306 and some of the suggestions of various tools there?

DR HYLAND:

Well it depends on what sort of inspection you are doing but for you know a structural evaluation type of inspection if you could, if you have the drawings it is going to be a big help. I think in this case when they did the detailed inspection they didn't have any structural drawings so perhaps didn't point out, you know they weren't aware of perhaps some of the issues that were there like the C18 column beam connected to the north core.

MR SMITH:

I think I am just aware that there was a separate session for discussing those which I haven't looked at all that testimony but I am sure evidence would have come out of that to give you some recommendations yeah.

JUSTICE COOPER:

Mr Allan?

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MR ALLAN:

Sir I have no re-examination but I'm conscious that we're about to embark on a process pursuant to which these men will put into evidence their supplementary briefs. I'm wondering if I need to do anything now to put into evidence the actual reports, the building materials report and the joint report?

JUSTICE COOPER:

Well, all right, yes.

MR ALLAN:

So if I can just, briefly, I'll so this through you if I may Dr Hyland, you've given evidence over the past couple of days now concerning findings and conclusions reached in reports that you have prepared. Firstly a building materials report dated the 16th of January 2012?

DR HYLAND:

Yes.

MR ALLAN:

And secondly a, what's been called the building collapse report dated the 25th of January 2012 which is in three parts, and that is jointly authored by you and Mr Smith?

DR HYLAND:

Yes, yeah, I mean it's just one part here but.

MR ALLAN:

And you've got the copy of that with you today?

DR HYLAND:

I've got those yes.

MR ALLAN:

Yes, all right, if we could put that into evidence please?

JUSTICE COOPER:

I think Commissioner Fenwick has some questions Mr Mills, is there anything else that needs to be tidied up first?

MR MILLS:

Well I can leave the supplementary evidence until after the Commissioners have had their questions Sir.

COMMISSIONER FENWICK:

Well you've given us a lot to think about. There are just a few issues I'd like to take you through. Clearly when you venture into a time history analysis it's the last thing you do. You look fairly carefully at the individual components and analyse those so as you knew where the weak spots where and so where you could concentrate your analysis results from your time history analysis, that would be I assume the approach you would take?

DR HYLAND:

That's correct.

COMMISSIONER FENWICK:

There are just I say three different areas I'd like you just to go through with me. First of all if we could look perhaps at the beam column joints. Could we have BUI.MAD249.049.3.3? You can see that figure?

DR HYLAND:

Yes, yes.

COMMISSIONER FENWICK:

What I've done there is this is a sort of back of an envelope calculation, no level of accuracy in there but just to get a general idea and I'm sure you've looked at this. The column is analysed, I've given the assumptions there, are just based on nominal strengths. I haven't tried to over-strength or anything like this so 35 megapascals for concrete and 380 megapascals for steel and just analyse that sector column, plain sections remaining, plain using the Mander stress strain relationships for that, and you can see there I have identified where the compression is in the concrete due to flexure and axial load and you can locate the compression force coming up, the centre of the compression force in the concrete is 92 millimetres from the outside edge of the column. Now if I can just backtrack for a second. This goes into a, a beam and I have just assumed the beam is a continuous beam. Plastic hinges will presumably form, well, will form in the columns. The answer is would they form in the beam or not, and so, well, the beam is resisting gravity loads and it's resisting seismic actions. Perhaps I should ask you what do you think would happen to that beam? Would the plastic hinges form in the beam or partial plastic hinges form in there? Would the, the moments redistribute or not under those actions?

MR SMITH:
I think, yeah if I'm talking –

COMMISSIONER FENWICK:
Did you look at this?

MR SMITH:
Yes we did yeah.

DR HYLAND:
Yes.

MR SMITH:
First comment is about the, okay, let's talk about the beams. There is definitely some hinging of beams evident in our first nonlinear time history analysis. That was the basis of the report in some locations. Not all locations. So in some cases the furth – definitely beam hinges form adjacent to the south wall because that is very rigid element, the beams are constrained rigidly at their ends at that location. They're definitely beam hinges forming at that location. Other locations it is not immediately apparent. The model did show some beam, beam hinging the way we set it up originally, but that was with linear properties for the joint. So one of the issue we're currently looking at in this further analysis is more trying to more accurately model the joint properties so that we see the distribution between the column and the beam and the joint, and it probably wouldn't immediately come out of our initial, our analysis that formed the basis of our report. I think the comment that I note on

the right-hand side there, you've got, "Moment capacity at base of joint zone approximately 80% of the column."

COMMISSIONER FENWICK:
Well we're coming to that.

MR SMITH:
Yeah, no –

COMMISSIONER FENWICK:
Put that on one side, later on.

MR SMITH:
Okay, so, well it's relevant to, to what I was just explaining, that I think the properties you assume for the column and the joint do influence the beam, whether it hinges or not basically.

COMMISSIONER FENWICK:
Yes. I mean my back of an envelope calculation said actually when you add your live load moments, sorry your gravity load moments to your seismic, it looked to me as though you'd get some yielding in the beams under cyclic conditions, not a whole lot but enough to redistribute quite a bit of the gravity load moment as it swayed backwards and forwards. That's really the point I was getting at.

MR SMITH:
Right.

COMMISSIONER FENWICK:
Now I just don't know, do you think that's rational? That was my calculations, a bit crude but do you think that's a rational assumption?

MR SMITH:

Well I'm saying that we did, in the nonlinear analysis we did, we did see that effect of that redistribution in some areas.

COMMISSIONER FENWICK:

Yes, okay. If you get that redistribution then you're going to get the force, type of forces I've indicated in that as a very crude first approximation of a strut-and-tie, and I mean I'm disappointed I didn't see a strut-and- tie analysis of this in your report in more detail, but I think having said that you can see that there's a tension force in this bottom reinforcement on the left-hand side coming in onto that bar which hooks up, and you've now got to see if the forces can be resisted in that hook bar. It's, the hook is traditionally in the wrong place. Normally we'd say it should be right at the extreme side of a joint, not in the middle of a joint as is located here. That's a scale drawing by the way, pretty accurately where it'd be if it was, followed the drawing, you know, if forgetting about construction errors. So if you've got some tension force coming in on the left-hand side that's got to be resisted by bearing against that hook and that means that if half the forces coming in on one side in tension and half the other side in compression at the bottom there, then you've got to balance that force and that can only be balanced by a diagonal compression force from the compression side. Of course it implies what I've shown there in the green struts coming down is a very unconservative assumption because it implies that you've got sufficient development of that hook with virtually no distance from the end. I mean if you were to apply the code and work out the development length of that hook bar it would be outside the beam column joints, and I'm going to get about two thirds of that in the joints, so but to get the incline compression force, so this is an upper limit possible, what I've shown there. If you then say, well half the force goes on that hook bar and half on the balancing the compression force on the right-hand side then your compression force coming up in the column has got to migrate across the interface. Now does that seem a sort of reasonable assumption to you?

DR HYLAND:

I think the thing is what, it depends on what level of seismic load you're putting into it.

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COMMISSIONER FENWICK:
I'm assuming you are working your column to the stage it's, it's going to form the plastic, plastic hinges. I mean do you people have assumed you're going to get a plastic hinge link, I think you said was it 200 or 400 millimetres, 200 or 300 millimetres down the column?

DR HYLAND:
Yeah, yeah.

COMMISSIONER FENWICK:
Yes.

DR HYLAND:
So it just depends on the level of drift that you're, you're putting on the columns whether you're going to get that –

COMMISSIONER FENWICK:
Sure, it's elastic. You're going to get no plastic hinge but I mean if you –

DR HYLAND:
Yeah.

COMMISSIONER FENWICK:
If we're going in elastic, I mean the amount of curvature involved in reaching this, the stress distribution I presume there, it's based on a, I think the peak value is at limiting .0035 strain in your, theoretical strain in your concrete.

DR HYLAND:

Yeah so –

COMMISSIONER FENWICK:

So it's highly untheoretical at this point, the process, a highly confined zone because of the location of the beam.

DR HYLAND:

I think that thing where I was, where I was finding, if we're talking about drifts of sort of 1% or less then, then we're in a position where perhaps we're not getting that reversing in the, you know through the joint, particularly on the internal columns so.

COMMISSIONER FENWICK:

Have you got some numbers to support that?

DR HYLAND:

Yeah, yeah.

COMMISSIONER FENWICK:

I'd like to see them.

DR HYLAND:

Yeah, no I'll get those to you.

COMMISSIONER FENWICK:

Thank you. If we just carry on with where we were. The compression, if you've got to have half the compression force deviating in the concrete, deviating to the middle to that hook then if you do your sums you find that the, the strength of the column is at least 20% stronger than the strength of the beam column, the joint zone now. I don't know if you accept that thesis, the compression force must migrate across. Do you accept that's a rational approach?

DR HYLAND:

When, when you're getting, when you're getting reverse, when you're getting that reversing behaviour, yeah, I don't have any trouble with that.

COMMISSIONER FENWICK:

Right.

DR HYLAND:

It's just whether we get to that point that's, that's what I was – that's why -

COMMISSIONER FENWICK:

So you think that you may not, may not get to that point.

DR HYLAND:

May not get to that point on those internal columns, yeah.

COMMISSIONER FENWICK:

So what you're saying is that the tension force in the beam will never go into, this reinforcement will never go into tension at the bottom.

DR HYLAND:

Yeah the calculations I did at the, at the low drifts levels were, you were getting, you either weren't getting the, getting the tension from the earthquakes sufficient to reverse the gravity compressing, clamping effect or that the –

COMMISSIONER FENWICK:

I'm sorry I don't follow. Will you go through that a bit more closely, a bit more slowly. Clamping effect, what clamping effect?

DR HYLAND:

The gravity, just the gravity closing effect.

COMMISSIONER FENWICK:

Yes.

DR HYLAND:
The gravity closing moments.

COMMISSIONER FENWICK:
The gravity moment.

DR HYLAND:
Yeah, the gravity moment.

COMMISSIONER FENWICK:
The negative moment over the –

DR HYLAND:
Yeah, yeah.

COMMISSIONER FENWICK:
- support.

DR HYLAND:
So once, you've got to get that, a certain amount of drift before you're going to get, overcome that and then get tensions.

COMMISSIONER FENWICK:
That's of the order of 140 kilonewton metres above the, the centre of the column.

DR HYLAND:
Yeah.

COMMISSIONER FENWICK:
According to my approximate calculation, if you ignore the vertical excitations.

DR HYLAND:

Yeah.

COMMISSIONER FENWICK:

So it may go up or down from that quite appreciably?

DR HYLAND:

Yeah, sure, yeah, yeah. We found, I mean the, the hooks aren't, aren't standard hooks but with the development length but they'll still carry a certain amount of, of tension.

COMMISSIONER FENWICK:

Yes.

DR HYLAND:

Even if they did get a little bit in them.

COMMISSIONER FENWICK:

Okay. Can they carry that tension? How would you balance your forces. If they're carrying tension how would you balance the forces without that inclined compression force near the centre of the member. How, how would that work?

DR HYLAND:

That's, that's right. So, so that's a mechanism, I agree with that, once you get to that point.

COMMISSIONER FENWICK:

So if you accept that then you, you accept inclusion that the, the position of the centroid of your compression force in the column has moved towards the centre of the column and thereby reduced its moment capacity at the interface with the beam column joint.

DR HYLAND:

Yep, yep I think that's right, yep.

COMMISSIONER FENWICK:

So one can conclude from that if you're agreeing that provided you get a excitation which will actually induce an elastic deformation in the, in the beam, if it does you get redistribution, provided you get that then you will have a beam column joint zone which is weaker than the column right at the interface, or between the two, you've got a loss by my assessment, and I believe it's a very conservative assessment, of about 20% of the strength. Do you accept, does that seem a reasonable –

DR HYLAND:

Yeah I mean I've got to go through the numbers. I mean we've got some questions from you to, that we're preparing written answers and –

COMMISSIONER FENWICK:

Okay, right.

DR HYLAND:

So I want to do that for you.

COMMISSIONER FENWICK:

Yes, yes.

DR HYLAND:

I understand what you're saying though in terms of the theory and development.

COMMISSIONER FENWICK:

Yes.

DR HYLAND:

But to actually nail the numbers at the moment, I can't really give you a quantitative answer if that's okay.

COMMISSIONER FENWICK:

Okay.

MR SMITH:

Can I just add, I think, you know, we certainly agree the, the embedment of these bottom bar hooks is precarious in the one that you've shown but even more precarious in, in the ones that don't even cross the centre line.

COMMISSIONER FENWICK:

Of course, yes, I agree with you, yes.

MR SMITH:

Yeah.

COMMISSIONER FENWICK:

So if my, if the thesis of redistribution is correct and those bars go into tension then you cannot have a plastic hinge length of 300 or 200 millimetres in the column?

DR HYLAND:

Yeah if you're getting a hinge in the, in the beams, I'm not totally convinced on that one and …

COMMISSIONER FENWICK:

You don't have to get much hinging in the beam to redistribute your moments. It's a –

DR HYLAND:

Yeah.

COMMISSIONER FENWICK:

A movement I think, I calculated it out at something like a crack, a sum of cracks above the top of about a millimetre was enough to give you that redistribution.

DR HYLAND:

Right.

COMMISSIONER FENWICK:

It's not something you actually see as a plastic hinge.

DR HYLAND:

Right.

COMMISSIONER FENWICK:

And of course it's doubly bad because with the reinforcement you have four 28 millimetre bars but two of them stop right at the column face and two go in. So you've got a, a weak section there. Okay well that, that's what I wanted you to look – if we can just go to the next one which is same series but instead of being point 3 it's point 4. This is just a quick summation of what were the shear stresses in the joint zone and it's a bit different from usual. It's a little bit like a, a pre-stressed unit, so you've got a pre-stressing cable through it giving you compression on each side. In this case you've got axial load which does the same. I just point out if you sum up the forces in this joint zone, I've shown the axial load there as 735 above the storey and 1050 below it with the associated bending moments going with it, assuming the column limits it, then you'll find that the, when you sum it up the, the vertical shear stress through there is, is seven megapascals. Now that doesn't mean to say you've got to have shear reinforcement of seven megapascals. I'm just pointing out that this joint zone is actually pretty highly stressed. It doesn't, don't have to carry the whole lot in shear reinforcement because of the clamping type effect due to the axial load would you give you a, a predominantly diagonal compression force which you will know is a, a bottle-type strut but there's no reinforcement in that joint zone, no shear reinforcement in that joint zone at all.

DR HYLAND:
Mmm.

COMMISSIONER FENWICK:
And as you will know bottle-type struts, especially when you get that level of stress, only work if there's nominal reinforcement which stops the splitting action would you, would you agree?

DR HYLAND:
When you say bottle-type strut, okay, like a –

COMMISSIONER FENWICK:
Strut and tie, standard strut and tie, notation bottle strut.

DR HYLAND:
Bottle strut, okay. So the fact that there was no spiral or, or hoops is critical which –

COMMISSIONER FENWICK:
Yep I mean there were –

DR HYLAND:
I certainly accept that.

MR SMITH:
Yeah, no problem with that at all.

COMMISSIONER FENWICK:
There were spirals shown on the drawing but they clearly could not be fitted.

DR HYLAND:
Right.

MR SMITH:
Yeah, very weak.

COMMISSIONER FENWICK:
And even if they had been fitted it's probably insignificant to satisfy the –

DR HYLAND:
Yes, yes.

COMMISSIONER FENWICK:
– bottle-strut requirement. So there is, you're likely to get quite extensive diagonal cracking through that joint zone.

DR HYLAND:
Mmm.

MR SMITH:
Absolutely.

COMMISSIONER FENWICK:
Well that'd be my, is (inaudible 10:33:41)

MR SMITH:
I agree, yeah.

DR HYLAND:
Definitely no. I mean we don't have any trouble with the –

COMMISSIONER FENWICK:

Yes.

DR HYLAND:

– fragility of these joints at all, no, it's …

COMMISSIONER FENWICK:

Let's move on quickly to the next point then. If this is the case you really for the, for this joint zone to work you're really relying on the tensile strength of the concrete aren't you?

DR HYLAND:

No, well I guess all I'm saying is it just depends on how much drift that, that joint zone had to cope with before it, before it gave up.

COMMISSIONER FENWICK:

If those hooked bars pull, go into tension, what's stopping it pulling out? You've got this very steeply inclined compression force which is really at a crazy angle for that hoop.

DR HYLAND:

Mmm.

COMMISSIONER FENWICK:

So I don't know, don't you feel it's dependent very strongly on tensile strength of the concrete?

DR HYLAND:

Yeah I guess either way I've looked at it as just that, that, that these, these joints pulled, pulled apart consequentially as the, as we got failure out of the, off the east face. So, so they, they were protected to some extent until we got the, the failures which we saw in the column hinging on the east face and these just pulled apart like pick-up sticks once things -

1035

COMMISSIONER FENWICK:

(inaudible 10:35:08) the tensile failure of the concrete wouldn't you expect the joint to pull apart?

DR HYLAND:

Yeah. I think, I mean the, that looked like those joints did pull apart and the precast beams were just seen on the site as sort of just units that'd pulled out so I don't think –

MR SMITH:

So I guess the point you're making we accept for a design we would not normally rely on tensile strength of concrete but this is the only mechanism that was available here because we had no circular hoops so I certainly accept that.

COMMISSIONER FENWICK:

The thesis I'm putting up is probably these joints were a lot weaker than the columns. It looks to me and the calculations I've done and they're very crude and very rough and I accept they may be wrong. I'm hoping you people can support or show me where they're not, that this inelastic deformation was certainly pinched into the joint zones, actually the chance that the column was developing a plastic hinge at all is probably fairly remote so the joint zone's weaker then you can't sustain the actions which would give you a plastic hinge.

MR SMITH:

We have seen some evidence of column hinging but we don't know which columns and where but you know and –

DR HYLAND:

(inaudible 10:36:33) seven out of the nine columns that we got out of Burwood that we were from the exterior of the building had hinging.

COMMISSIONER FENWICK:
These columns I'm looking at here on the interior –

DR HYLAND:
Yeah, so we know –

COMMISSIONER FENWICK:
(inaudible 10:36:50)

DR HYLAND:
Exterior columns did have, exterior columns developed hinging. We don't know about internal ones, whether there were any of those that did or not.

JUSTICE COOPER:
So just to clarify it for me, Mr Smith, the columns that you're referring to as having developed plastic hinges were all exterior columns?

MR SMITH:
Yes I believe so. The interior ones as far as we could tell were largely disintegrated. I mean, there's not much evidence of them.

DR HYLAND:
Well I don't now about that.

MR SMITH:
The joint zones and the hinge zones.

DR HYLAND:
Certainly the beams that we see the joint, they're just pulled out of the hinge, you know, out of that beam column joint zone, yes.

COMMISSIONER FENWICK:

That’s what I would conclude from, I'm indicating here the joint zone would literally pull apart and the beams would fall out and of course the, you end up with a column with the reinforcement sticking up, still there but no joint zone and a column above it. Can we look at perhaps then the, go to the, look at the south wall in a little bit more detail than, again you've probably analysed this more closely but can we have BUI.MAD249.0506.2? Now I did send round, we did send round a week ago a list of questions we had some doubts about whether some of these features had been perhaps considered in the depth that we would hope.

DR HYLAND:

Oh, okay, no, no, we have, yeah, okay.

COMMISSIONER FENWICK:

I'm hoping you've looked at these. Have you had a chance to?

DR HYLAND:

Yes, I've been looking at this in detail too just to answer questions. I haven't finished looking at it, sorry, we pulled ahead four or five days in the hearing and –

COMMISSIONER FENWICK:

Perhaps I can just take you through some of those points then?

DR HYLAND:

Would be good.

COMMISSIONER FENWICK:

Can we go to BUI.MAD249.0493.2 please? So we’re looking at the south wall and that was clearly designed as a coupled shear wall so my first question to you is, would this have behaved as a coupled shear wall, that is forming plastic hinges at the bottom allowing the two walls to rock energy dissipation in the coupling beams. Could that have behaved, did you get this far as looking at that or?

DR HYLAND:

Yes, no, I've been, I've done that. The interesting thing with the coupling beam design in 3101:1982 was that you were just to rely on the diagonal reinforcing to provide the shear in there but if you look at the contribution of the conventional stirrups and the concrete you get a significant increase in the shear capacity of that coupling beam and while there was some cracking in those coupling beams we didn't sort of see what we'd expect the classic sort of, you know, breaking out of the you know the concrete and the diagonals going so I think that you know we're actually were getting a, well certainly at the time of collapse this the thing had maybe developed some yield but it was still fully working as a conventional and a diagonally tied coupling beam so it was actually very strong sort of element. We didn't see at the base you'd expect to see sort of double flexural-type cracking at the base but we just saw it on each end. So –

COMMISSIONER FENWICK:

So the wall instead of rocking independently they rocked as one?

DR HYLAND:

Yes, rocked as one and maybe with a little bit of as it went up perhaps became more of a coupling-type action.

COMMISSIONER FENWICK:

That pretty well agrees with my again very rough back of envelope calculation where it looked to me as though the coupling beams were actually too strong to yield.

DR HYLAND:

I don't know if there's –

COMISSIONER FENWICK:

It's marginal but, just, just, let's go on to the next one, what influence would the floors do you think have had on the performance of those coupling beams? Now I've illustrated that in the diagram there but would you agree that when your coupling beam goes over whether you get yielding or not you get elongation because you cannot actually compress the diagonal compression member because there's a lot of concrete round those bars. You can stretch the steel in tension –

DR HYLAND:

Right, at the base.

COMMISSIONER FENWICK:

But you really you can't get appreciable compression there and the diagonals (inaudible 10:42:23) when you tip it over of course it's longer in plan than it is (inaudible 10:42:29)

DR HYLAND:

Yeah, I mean it's a similar sort of issue you have with you know your eccentric base framed type things are you able to get the vertical displacements and the separation from the slabs to actually let them freely work.

COMMISSIONER FENWICK:

Let's stick to reinforced concrete rather than (inaudible 10:42:49) structural steel?

DR HYLAND:

Sorry, yeah okay.

COMMISSIONER FENWICK:

I know your analogy.

DR HYLAND:

Yeah, yeah, but it's that sort of thing isn't it so that the design model's reasonably simple but perhaps doesn't account for those sort of affects, yeah.

MR SMITH:

So the floor has an additional stiffening effect on the coupling beams, yeah.

COMMISSIONER FENWICK:

Yes, and do you think this would affect the performance of the coupling beams?

DR HYLAND:

I think it would stiffen them up, make them stronger, yeah, yeah.

COMMISSIONER FENWICK:

Well certainly by my back of envelope calculations I agree with you, a very significant increase again rather hard to say how much because is that force going to be sufficient to crack the floor and of course it's not much point in asking now whether you saw there were cracks in the floor there, they're just a year and a half too late to see.

MR SMITH:

There was evidence in one of the CPG photographs of cracking adjacent to the wall of the floor immediately adjacent to the wall, parallel to the wall so we saw a photograph that indicated obviously some –

COMMISSIONER FENWICK:

Parallel?

MR SMITH:

Yes.

COMMISSIONER FENWICK:

Or normal?

MR SMITH:

No, parallel.

COMMISSIONER FENWICK:

So, you've, but in this case of course tension showing in the wall there you would expect it to be normal to the wall.

MR SMITH:

Perpendicular, yeah, so it’s not the same effect but there is an interaction effect that obviously. Whether that was shrinkage cracking before we don't know I guess but I did notice that crack.

COMMISSIONER FENWICK:

Not if you've put this tension into the slab and you know concrete doesn't crack, it’s very high, if the concrete cracks you've still got the mesh. It’s still quite a significant force, much smaller. That’s then got to be transferred to those walls hasn't it by shear, I mean if you got compression here it’s being balanced by tension in the slab, so the tension force in the slab actually has to come back into the wall through shear interface between the wall and the beams looping into the wall.

JUSTICE COOPER:

Do you agree with that?

MR SMITH:

Well I'm just wondering is he going to ask a question following this or –

JUSTICE COOPER:

That’s the question. Do you agree Mr Smith?

1045

MR SMITH:

Okay, I mean well we have explained before that we felt in relative terms there was a better connection of the south wall because we had the whole side of the building slab connected via beams into that wall so there is considerable capacity to transfer shear, whether it's locally at the wall or whether it is further along it would still come back to the wall so –

COMMISSIONER FENWICK:

So are you saying right, well the tension force could be dragged into the beam and then back into the wall?

MR SMITH:

Correct, yes.

COMMISSIONER FENWICK:

Do you agree Mr Hyland that is the route it would go. There is only a limited amount of reinforcement coming out of the wall into the slab so the shear transfer would then go through that beam, is this – and back into the wall, do you agree with that?

DR HYLAND:

Yeah okay if I just back-pedal. So we are saying we've got a zone at the coupling beam where the slab is, so they haven't put reinforce, transverse reinforcing into the slab. There is mesh, there is a bit of mesh there.

COMMISSIONER FENWICK:

Mesh. Nothing else, there is mesh and there are a few 12 millimetre bars coming out at right angles which of course will act as a shear friction of –

DR HYLAND:

Yes.

COMMISSIONER FENWICK:

Diagonal strut action if you like?

DR HYLAND:

At the particular coupling beam they've, it appears they have deliberately put them there which would be reasonable but as an attempt to try to allow the coupling beam to perhaps move or that, so there'd be perhaps a zone there 900 if you took a, I don't know, a 45 degree line perhaps, say there is an influence area of concrete that might have moved up and down a bit. Probably not very much though given the stiffness of these things. And then you have got the shear transfer from the diaphragm into the wall from the bars at 600 centres and then into the beams running east and west of it which are then connected in with the four H24s I think. So there was, there's, I have done some calculations and there's certainly quite a lot of shear capacity there to get into that wall.

COMMISSIONER FENWICK:

So on top of this you have a self strain shear, due to the tension force in the slab, we are then going to throw on the seismic shear which is quite significant coming on to those walls and that has got to be transferred into the walls and as you say, you have just outlined there it a shear transfer from the floor slab into those beams and those beams back into the walls?

DR HYLAND:

Yes.

COMMISSIONER FENWICK:

Can we have – I can't find it now, sorry it's BUI.MAD.249.0493.1. Okay so there you see that the shear force V coming along and if you look at the time history analysis results from Compusoft and I agree there's a question mark over those and we might discuss that later. They are saying that the shear force you transfer in is about 1300 kiloNewtons and it's, if varies plus or minus as you go up but it is pretty well constant and so that force has to be transferred by shear from the floor slab into the beams and back into the wall. Now the question I have is, is that a valid load path? Is that a load path you checked out?

DR HYLAND:
Yes, yes.

COMMISSIONER FENWICK:
It is?

DR HYLAND:
Yes.

COMMISSIONER FENWICK:
The question I have then is, you've got these four 24 millimetre bars that you are signalling which go 75 millimetres into the wall, but what picks up the force from those bars and transfers them further into the wall. At the moment they are just stuck in 700 millimetres in length. So what does the force transfer to, how are those bars anchored?

DR HYLAND:
Are we talking about the south wall or the north core?

MR SMITH:
The south wall, yeah, talking about the south wall yeah. I mean we, those bars were fractured in a number of cases.

COMMISSIONER FENWICK:
Some of them?

DR HYLAND:
Some of them were, yeah and some were still connected to the, the beams were still connected so.

COMMISSIONER FENWICK:

And some were completely disconnected, is that correct?

DR HYLAND:

No, no I think in the examination all the bars were fractured whether that had occurred during the earthquake or during the demolition they were there so I don't think there were any that were pulled out, you know they hadn't de-bonded. So they did have the ability to fully transfer the capacity.

COMMISSIONER FENWICK:

Normally you would expect that steel to lap other steel, wouldn't you but the steel can lap to transfer that force and normally if you put it on one side –

DR HYLAND:

Oh, right, yes.

COMMISSIONER FENWICK:

- you could expect it to tear out a chunk of concrete around the bar. Now what I am saying is was that capacity there?

DR HYLAND:

Yes I believe it was.

MR SMITH:

I had a recollection it was longer than 700 but I mean if we are talking about design shortcomings as opposed to shortcomings in capacity there are other issues such as the connection of the floor starters into the beams was just by a straight lap on top of the beam rather than having any hooks so there's certainly other weak points in that mechanism but I don't know, there wasn't evidence that that had led to the failure but certainly from a design point of view is not desirable.

DR HYLAND:

Yeah I could go back through the, maybe get back to you with that on the observations of those H 24 bars.

COMMISSIONER FENWICK:

The development length is there but what I am saying is the development length but you need to be able to pick that force up otherwise you might get a diagonal pulling out and I certainly didn't read but I may have missed it that all those bars were snapped, I read, as far as I can see some of them had actually pulled out, but I may have got that wrong?

DR HYLAND:

Yeah that is certainly my recollection that they were, you know that they'd either fractured at the ends or –

COMMISSIONER FENWICK:

The main concern I have I guess is, when I look at that wall and the residual you can see that there is yielding at one level which may have been due to a collapse and I think it was, Frost indicated that he thought it was a yield on one side that collapsed and not on the other but I couldn't see indications that that wall had gone to the level of deformation that was being indicated by the time history analysis?

DR HYLAND:

No, no.

COMMISSIONER FENWICK:

Which then made me wonder could that separation occur at that wall, and that is why I am asking did you load track through all these different components and see, because the units have dropped very cleanly at that point?

DR HYLAND:

Mmm.

JUSTICE COOPER:
So did you?

DR HYLAND:
Oh, so well I, my analysis just of, you know if you just analyse the wall as a unit just as a separate unit and apply the, you know the loading standard type loadings to it you are getting it to developing its capacity around about .3% drift. So it is getting to, it is starting to show some level of damage at quite a low level of drift and so my thinking is that it did achieve some, it didn't require a lot of drift to get some damage in it but it still had an awful lot more capacity in it. It could have gone a lot further but it just never got there so it just never got there so it just started to achieve its strength and then something else went.

JUSTICE COOPER:
What level of drift did you say you thought it was –

DR HYLAND:
Around about .3%.

JUSTICE COOPER:
.3%?

DR HYLAND:
Yeah.

COMMISSIONER FENWICK:
Oh I see. I think the non-linear time history analysis had it going much further than that?

DR HYLAND:

Well that is right, exactly and that is part of the, this is the difficulty in trying to interpret the collapse with the analysis and you know...

MR SMITH:

I think to be fair the analysis was done and we had tried to explain that basically with the exception of the drag bars which again were an artificial thing, basically everything was modelled to remain connected throughout the duration of the earthquake and then we would go back through it to identify where, which element we believed failed first so the maximum drifts shown in that analysis actually probably never occurred because it failed well before that point so, but. And I did also show a calibration of the September response with a strain in the bottom bars of that shear wall and it was again, there is a range of interpretations but it was not, shall we say, not inconsistent I believe the response of that south wall in September.

1055

COMMISSIONER FENWICK:

Can we look quickly at the north wall now? So we've got the right figure up. There's the north wall shown on the left-hand side.

MR SMITH:

Yes.

COMMISSIONER FENWICK:

If you imagine the floor accelerating to the west, that's up the floor, then the shear force has to be transferred by the block, the only area where a cell has filled if you like between the fingers of the wall, this is just below the line marked C there, the shear force has to be transferred through there because everywhere else there are gaps.

MR SMITH:

That's correct.

COMMISSIONER FENWICK:

So we have shear going up there and then we have, that force is balanced by the, the north wall on line 5, I'm sorry I've missed the five on the diagram. It's the red wall on the extreme right-hand side. That's as you understand it?

MR SMITH AND DR HYLAND:

Yes.

COMMISSIONER FENWICK:

Now, so this yellow portion at the top with an area that's in-filled is subject to a bending moment.

MR SMITH:

Yes.

COMMISSIONER FENWICK:

In fact the flexural or shear centre of that wall, like a channel, is some way to the right-hand side of that wall outside.

MR SMITH:

Correct, yeah.

COMMISSIONER FENWICK:

So it is quite, quite eccentric and you might normally expect warping to occur. So you've now got bending moment and shear acting at that face and obviously further back from that that section of wall as well. On top of that of course you have got the north south excitation which is going to induce further direct tension on that zone and through the, the drag bars.

MR SMITH:

Yes.

COMMISSIONER FENWICK:

How it will distribute we don't know. We don't know how long the drag bars stayed there.

MR SMITH:
Yeah.

COMMISSIONER FENWICK:
But if one just looks at the east west motion just to start with, you've got a moment and a shear at the section, intersection there of that beam going through there. So that cantilevered element, what's going to happen to it under those actions of the moment and shear? If you look on line C would that top wall which is running in a north-south direction, which is shown horizontal on the diagram, would that tend to pull out? Because you've got flexural tension. What do you see is going to happen as a result of that bending moment?

DR HYLAND:
Okay, is this assuming we've got drag bars or in the areas that we didn't have the drag bars?

COMMISSIONER FENWICK:
In the east-west direction. I don't think the drag bars would do much for you.

DR HYLAND:
Okay I did, I did some cross-section analysis on this with, with the drag bars in and without and –

COMMISSIONER FENWICK:
Well are you saying, sorry I got you wrong, are you saying the shear transfer, the drag bars would transfer shear effectively or they work in tension?

DR HYLAND:

Just in tension, just in terms of your moment couple they, they would be effective in terms of your moment couple. In terms of the shear, the direct shear though, no, I agree you'd be getting the, getting that shear coming in through those beams, so that's part of it and then the direct shear on the, on the face there going into that, into that amenity block area.

COMMISSIONER FENWICK:

So would you agree then that you would have tension, unfortunately not having a mouse I can't point to it, but tension along that line C and below it you'd have flexural tension going through into the base of the, the floor?

DR HYLAND:

Like this, is that what you're saying?

COMMISSIONER FENWICK:

Yes. Flexural tension, and below it.

DR HYLAND:

Yeah so you'd get a, you'd get a –

COMMISSIONER FENWICK:

Somewhere down below as well, further out. That's right. So you're going to have –

DR HYLAND:

Out here.

COMMISSIONER FENWICK:

The tension would be presumably over quite a lot of that, that block?

MR SMITH:

I think the tension, I don't disagree there's tension. I think the magnitude of the tension is dependent on what I understood, what I interpreted, or my assessment was, that the drag bars assist in that in-plane moment to reduce the magnitude of that tension at that line C because it's not only the slab that can take that moment. It's the whole slab rather than –

COMMISSIONER FENWICK:
Right. If we follow your hypothesis then tell me if you've got this bending action then you'd have compression at the bottom of the wall wouldn't you, the bottom level down there, that would go into compression and the next one would probably go into compression.

MR SMITH:
So the drag bar would go into compression.

COMMISSIONER FENWICK:
So the drag bar's gone into – do you agree?

MR SMITH:
Yes, yes.

COMMISSIONER FENWICK:
Okay, right, so the tension's not helping in the drag bars is it because in this particular case they're in compression.

DR HYLAND:
I, I thought –

MR SMITH:
If you're, if you're turning this way, yeah, if you're coming this way yeah, yeah.

COMMISSIONER FENWICK:
Yes, let's just follow that a fraction further.

MR SMITH:
Okay.

COMMISSIONER FENWICK:
If that drag bar on the bottom lip, can you move your arrow, your pointer down to the bottom one.

MR SMITH:
Yep, down here, yep.

COMMISSIONER FENWICK:
If we're now saying that's in compression and we now track where that compression goes, it goes along that wall –

MR SMITH:
Yeah.

COMMISSIONER FENWICK:
Into the vertical wall.

MR SMITH:
Yeah.

COMMISSIONER FENWICK:
What's that doing?

MR SMITH:
Okay so –

COMMISSIONER FENWICK:
No this is carrying on several floors simultaneously.

MR SMITH:

Yeah, yeah. So you're talking about your warping.

COMMISSIONER FENWICK:
You're warping it aren't you?

MR SMITH:
Well you're getting, I think you're getting compression at every level.

COMMISSIONER FENWICK:
All right, you've got compression in those drag bars at every level.

MR SMITH:
Yes.

COMMISSIONER FENWICK:
But that compression force is at right angles to your wall on level 5, the one on the extreme left isn't it? Now if you're saying that this is resisting the moment like compression at the bottom and tension at the top then that's, eccentric action, isn't that doing something to the wall?

MR SMITH:
Well that, that would cause warping stresses and, and –

COMMISSIONER FENWICK:
That's right.

MR SMITH:
Yes, yes I accept that.

COMMISSIONER FENWICK:
Do you think that might account in part -

MR SMITH:

For the cracking?

COMMISSIONER FENWICK:
– for what Smith saw?

DR HYLAND:
Yes, no I think you're right, I think that's, I think you've picked that, I, I didn't realise that at first but yeah now you've put it to us I've been thinking about it.

COMMISSIONER FENWICK:
Now given that you, you would have some compression. I would say it would be very limited because the, the warping strength of that member is fairly low.

DR HYLAND:
Okay, okay.

COMMISSIONER FENWICK:
Would you agree?

DR HYLAND:
Well –

COMMISSIONER FENWICK:
A 300 millimetre wall with reinforcement at, I've forgotten what it was, 400 centres, a 60 millimetre diameter –

DR HYLAND:
Yeah it doesn't, it doesn't take much. It's, from what we've done but the only thing is I guess the, the slab has to stay, the slab's relatively rigid so it will only move as much as it can, you know.

COMMISSIONER FENWICK:
What slab?

DR HYLAND:

The slab. What do you mean?

COMMISSIONER FENWICK:

There's no slab. It's not connected. There's no slab there. The only slab is right where the thing is in there so you might get a bit of reverse flexure at the end but there's no slab is there and we're talking about distances of four, five, six metres aren't we. You know that's the distance between you and me, it's quite a lot of bending.

MR SMITH:

But I think the point about the, if we just move away from the local area around the lift shaft the, and back to grid 4 and beyond is a very rigid slab, there's a whole diaphragm that, obviously if you're pushing that compression force onto that it's got considerable rigidity so.

COMMISSIONER FENWICK:

Can you just use the mouse because I can't find - you're talking about.

MR SMITH:

The drag bar here connected to this wall, connected to this slab, pushing compression along this line here, we've got, we've broken a section but this actually continues on. So actually from, on this line here we've got that rigid body we're pushing against.

COMMISSIONER FENWICK:

I don't think there's any failure, any chance of failure due to that compression force.

MR SMITH:

No exactly. Yeah, yeah.

COMMISSIONER FENWICK:
Might be when it reverses and it goes into tension but not in the compression force.

MR SMITH:
That's right, yeah.

COMMISSIONER FENWICK:
Of course at the top level that compression force you've applied there could well bend that column C18 couldn't it?

MR SMITH:
Yes, yes.

COMMISSIONER FENWICK:
And at the same time as that column is being subject simultaneously to compression and tension as that, because it's looped into that floor isn't it?

MR SMITH:
Yep.

COMMISSIONER FENWICK:
Okay. So that's a partial account of why that column was damaged where everywhere else the column went with the, the floors in this one case it couldn't and so you'd expect that column to be damaged exactly as we saw.

MR SMITH:
That's right.

COMMISSIONER FENWICK:
Do you agree?

MR SMITH:

Yes, yeah.

COMMISSIONER FENWICK:

I mean you've partially told me, you've partially explained that before.

COMMISSIONER FENWICK:

So can I just ask then, we've got this flexure. We know that the forces at this area according to the Compusoft analysis, and I agree there's some questions that can be asked about those and these values may be a bit high but they were looking at a 2000 kilonewton-odd shear goes above that and below it. I've just taken an average sort of value, a 2000 kilonewton shear which gives you quite a substantial shear stress in the zone I'd indicated, and now we're saying it's subject to flexural tension as well? The flexural bending amount of course is 2200 kilonewtons or whatever it is times at least times the distance between the wall and the critical section which is knocking on five metres. That's quite a significant tension you get there. Did you examine this zone and say could you actually, could the reinforcement in that zone actually resist those actions?

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MR SMITH:

I certainly did calculations of that. The limitations, when we came to model it, even though we knew there were limitations physically we could not model a total disconnection of that, it's just, the analysis would stop running so we elected to continue the analysis and then look back at the magnitude of the forces that potentially could exist at that location, and certainly a fracture of that slab and of the drag bars is from the analysis is shown to be a possibility or probability.

COMMISSIONER FENWICK:

But you haven't actually, I mean it would be interesting if you actually went away and looked at these actions and said these are the magnitudes, or perhaps even they're only half the magnitudes. Could that slab have survived that without starting to tear? I'm not saying the whole thing went but could it have survived without it starting to actually fail?

MR SMITH:
Well I have done calculations and I can, I can send you those.

COMMISSIONER FENWICK:
Well I'd like, I'd like to see them.

MR SMITH:
Yep.

COMMISSIONER FENWICK:
That would be quite helpful.

DR HYLAND:
Yeah that's the same issue of, I mean if you apply those loads that we're getting out of the NTHA directly then you'd say these are, this is highly overloaded but again it just comes back to the question did the collapse occur or indicates the collapse may have occurred before it got to those level of loadings.

COMMISSIONER FENWICK:
Can we just look at perhaps some of those forces? So this is ENG.COM0001.69 and when you get there it's table 35 for the north core. Can we concentrate on table 35? Great.

JUSTICE COOPER:
This is from Compusoft?

COMMISSIONER FENWICK:

Yes this is from the Compusoft document yes. So you can see there the different analyses running through and actually those figures don't surprise me. I think you've said before that they were, you felt they were too high?

MR SMITH:
Look I, I –

DR HYLAND:
I do, that's my view. Ashley's maybe not quite as.

MR SMITH:
Well again we, on the basis that we, we remained, everything remained intact through the analysis so it's, it's, I'm not saying the collapse didn't occur and so that may not have eventuated for the same reason that the drift in the south wall. The peak demand was never reached because things fell over before that happened, so the same would ex – the same would apply to this case here. So these are just the maximum values if we had left that analysis running throughout, throughout that, so I'm, certainly the '84 as I understand it well underestimated these forces.

COMMISSIONER FENWICK:
Seriously underestimated.

MR SMITH:
We know, we know that. I raised the point the other day that even a current standard may be short. I want to just await the results of this further analysis before we talk about that further but, so.

COMMISSIONER FENWICK:
Did you actually find anything in the current 1170.5 which would actually tell you how to find the data in forces?

MR SMITH:

Well we, the way we did calculate it was looking at the response spectra analysis, what we call using a section cut function to monitor the diaphragm connection forces for that analysis, so that's the way we interpreted the current standard would assess those actions but, so that's what I'm referring to. I think there's an option if you're looking locally, perhaps you could use the parts option as well but, but I was talking about the response spectra analysis.

COMMISSIONER FENWICK:
If you look through the commentary you'll find that parts and portions is almost excluded from the diaphragm forces as a whole. It mentions everything else but the diaphragm forces.

MR SMITH:
Yeah, so I –

COMMISSIONER FENWICK:
And it was taken out because it's clearly, you know, if you looked at the '84 and earlier codes, the parts and portions is clearly inadequate.

MR SMITH:
Inadequate, yeah I agree with that. So I was referring to a response spectra result when I said that.

COMMISSIONER FENWICK:
And this was an elastic response spectra?

MR SMITH:
Yes it was, yeah.

COMMISSIONER FENWICK:
No reduction for ductility?

MR SMITH:

No.

COMMISSIONER FENWICK:

So you just take the elastic values? Yes, well that would be a valid way of doing it. If of course you look at the ground floor, okay it's not an issue because it's sitting on the ground and we had 0.9 G peak ground acceleration force, it would be 0.9 G wouldn't it? If one looked at one floor up it's probably going to be a bit lower, 0.8 or something like this. If you apply the 0.8 you get about these forces at that level. Then you, it probably goes down and then it starts to go up a bit as you get to the top. So the values here don't look out of the order but there's always, well there will be reservation I'm sure you've got. I mean would you like to comment on, I mean I've got my?

MR SMITH:

I think, I'm looking at it further. The reason it wasn't part of our initial brief to look at how it stacked up against current code, we were looking at the original and so, but I am looking further into this with this further non-linear analysis so I should be able to come up with a comparison.

COMMISSIONER FENWICK:

Yes, how do you feel about these non-linear values, are they – the fact they've come off there, do you think they are realistic or do you think that they may be, I mean what is your feelings about these values?

MR SMITH:

Well I was certainly surprised at the magnitude of these values when I first saw them. I accept there are, it is very much dependent on the stiffness you assumed for the floors and things and I am uncomfortable shall we say making reductions to the floor stiffness because we know in effect that what that is modelling is possibly fracture of the mesh or some other thing so it is not a dependable ductile thing that we are choosing to down rate in our analysis so I do have reservations about it, you know but I think we will compare this analysis with the new one we have got and we may make some judgement from that.

COMMISSIONER FENWICK:
Must be, some of these questions should go to, when we get the time history and analysis back, but the concerns I would have I guess was the way the damping was modelled because a lot of these forces come from higher mode effects and it easy to use a damping that gives you, a model which gives you high damping for the very higher modes and that can give you a falsely high forces but –

MR SMITH:
We've certainly discussed that in detail with Professor Carr so...

COMMISSIONER FENWICK:
Right, yes and the other thing of course is that if the force lasts for .1 of a second it gives you .01 of a millimetre of movement?

MR SMITH:
Yes. That's right it is going to go isn't it.

COMMISSIONER FENWICK:
So that may be high but yep.

MR SMITH:
But I think the point I am coming to is I would like to know that our current standards because we don't normally, we want to ensure that that is never a failure mechanism we want to have a conservative value so...

DR HYLAND:
I thought Andrew King's approach was quite good that Charles Clifton used, seemed to have you know quite a reasonable sort of practical application with, that may be worth following through.

COMMISSIONER FENWICK:

Certainly one of our aims is to look at what we are doing now is it adequate in what we are learning from this and the point about there being nothing in the standard which specifically directs you to as how you can determine these diaphragm connection forces is something which would concern us.

DR HYLAND:

Mhm.

COMMISSIONER CARTER:

Nothing further, I think it would be helpful though if Dr Hyland and Mr Smith could just give us a response to the questions that were sent around –

DR HYLAND:

Yes, no we are doing that.

COMMISSIONER CARTER:

But that has been very helpful to listen to that exchange thank you.

JUSTICE COOPER:

Mr Allan, have you noted the two sets of calculations that are to be provided?

MR ALLAN:

Yes Sir.

QUESTIONS ARISING – MR RENNIE - NIL

QUESTIONS ARISING – MR REID - NIL

QUESTIONS ARISING – MR ALLAN - NIL

QUESTIONS ARISING: MR MILLS

Q. Now I'll deal with Dr Hyland and Mr Clay I think is going to deal with putting Mr Smith's evidence in. Now Dr Hyland you've prepared two further supplementary briefs of evidence?
A. Yes, yes I have.
Q. They're both dated the 24th of June 2012?
A. Yes that's correct.
Q. And am I correct that these are briefs that have been prepared in your personal capacity

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MR MILLS:

They are both dated the 24th of June 2012?

DR HYLAND:

Yes that is correct.

MR MILLS:

And am I correct that these are briefs that have been prepared in your personal capacity not as a representative of the Department?

DR HYLAND:

Yes that is correct.

MR MILLS:

And you have got those briefs with you now?

DR HYLAND:

Yes I have.

MR MILLS:

And they have been signed?

DR HYLAND:
Yes they have.

MR MILLS:
Can I just ask they be put formally into evidence.

EXHIBIT PRODUCED - BRIEF OF EVIDENCE OF CLARK HYLAND

MR CLAY:
Mr Smith, I think you have before you your fourth, fifth and sixth supplementary briefs of evidence?

MR SMITH:
That is correct.

MR CLAY:
You confirm that four and five are dated 25 June 2012?

MR SMITH:
That is correct.

MR CLAY:
And 3 July 2012 for number six?

MR SMITH:
Yes.

MR CLAY:
Those are signed by you?

MR SMITH:
Yes they are.

MR CLAY:

They are your personal briefs of evidence?

MR SMITH:

Yes they are.

MR CLAY:

Those Sir be introduced into – as an exhibit.

JUSTICE COOPER:

What happened to 1, 2 and 3?

MR CLAY:

Well Sir I have discussed that with Mr Mills and one I think has gone in and it has been decided to put the other two in at the end when the evidence is given.

JUSTICE COOPER:

Okay.

MR CLAY:

It is more germane to that issue Sir.

JUSTICE COOPER:

Very well. Yes, well I have nothing to add, thank you very much.

WITNESSES HYLAND AND SMITH EXCUSED

HEARING ADJOURNS: 11.17 AM

HEARING RESUMES: 11.35 AM

MR ALLAN CALLS

ROBERT DAVID JURY (SWORN)

Q. Good morning Mr Jury.
A. Good morning.
Q. Now you're here giving evidence today because you were a member of the expert panel that was charged with overseeing amongst other things the preparation of the report that's been subject of discussion over the last few days?
A. That is correct.
Q. Now your own expertise or your current role, perhaps I start with that, is you are a technical director in the discipline of structural engineering at Beca Carter Hollings and Ferner?
A. That's correct.
Q. You've got over 30 years' experience in the field of structural engineering consultancy, in particular relating to the performance of structures in earthquakes?
A. That's correct.
Q. And over that time you've received several Excellence Awards for projects with which you've been involved including the Sky Tower up in Auckland?
A. Yes.
Q. And you've authored, co-authored over 40 technical papers on various issues relating to structural engineering and you were a member of the Standards Committee that developed the current New Zealand Loading Standard?
A. That's correct.
Q. And its predecessor?
A. That's correct.
Q. Now Mr Jury, in order to walk us through your role in the panel and the panel's deliberations have you prepared a presentation to assist you with that?
A. I have.

Q. Yes and we've got the first slide up there I see. If you're able to just speak to this presentation for us please and I may have questions for you as we proceed?

A. Thank you. Well as counsel said I was a member of the expert panel that overviewed the production of the CTV building collapse report. I'm here today as one of those panel members and although my own views may come out as part of my presentation I am trying to be objective in presenting this on behalf of the panel.

Q. Is the scroll perhaps the -?

A. The scroll is the one? Oh, great, now we're right. The content of my presentation will cover the membership of the panel, its roles and responsibilities as was set out in terms of reference, how it functioned and how it deliberated and that may provide a bit of understanding in terms of the result of the panel's deliberations. I will present the key findings in relation to CTV because this panel report also covered the other building collapses as well that were investigated by the Department of Building and Housing. I will present the conclusions that were in the panel report. I will also attempt to give some discussion and make some comment on the differing views because there were obviously differing views, both within the investigating consultants' deliberations but also deliberations of the panel and the interaction of the panel with the investigating consultants. And finally I will present the recommendations of the expert panel in relation to the issues as they came out on CTV.

The panel was appointed by the Department of Building and Housing. Its main objective was to produce an overview report of the building investigations for, as I mentioned before, the CTV building, the PGC building, the Hotel Grand Chancellor building and the Forsyth Barr stair collapses. It was required to address matters relating to the investigations in quite a wide framework but also quite a restricted scope as well which I will outline, and it also had part of its objectives too to indicate issues that might be considered by the Department of

Building and Housing in its role as a regulator, in other words, changes to standards, codes, that sort of thing, and issue advisory notes.

The panel was made up of 11 individuals. They came from a wide range of areas of expertise. The panel was led by Sherwin Williams who was a construction law expert. The deputy chair was, position, was taken by Professor Nigel Priestley, a leading authority on earthquake design and structures and he's presented at this Commission before. Dr Helen Anderson was a specialist with knowledge in seismology issues. Marshall Clark was a specialist –

JUSTICE COOPER:

Marshall Cook.

EXAMINATION CONTINUES: MR ALLAN

A. Marshall Cook, sorry, was the specialist, had specialist knowledge of architectural building design for earthquake. Peter Fehl had specialist knowledge of construction and construction industry practice. Peter Millar specialist in knowledge of geotechnical engineering practice. Professor Stefano Pampanin, a professor at the University of Canterbury here, specialist leading authority on earthquake design and structures, and George Skimming with specialist knowledge of territorial authority roles in building procurement, et cetera, and also one of each of the investigating consultant teams was also appointed to the expert panel. That included Dr Clark Hyland representing the Hyland and Smith grouping. Adam Thornton who looked into the Hotel Grand Chancellor. And myself who looked at the Forsyth Barr stairs and the PGC building collapses. Those 11 people comprised the panel.

The panel activities were project managed by the Department of building and Housing and its project manager Dr David Hopkins. Dr Hopkins also fulfilled the role as the principal editor of the panel report. The panel and its terms of reference was charged with providing guidance and direction to assist in achieving overall objectives that I've already outlined. It was required to advise on the scope and extent of the investigation but not necessarily provide professional advice on that investigation. It was required to monitor and review the consultants' approaches, their investigations, the data and the data that was inputs and also the outputs. It was also required to recommend to the department any changes in scope necessary to address the matters for investigation and that occurred on a couple of occasions with the CTV investigation. It was also required to review and approve the consultants' report and then finally produce an overall, overview report covering all the matters. It's important to recognise though that it did not fulfil the function of a peer engineering peer reviewer and it was not required to carry out its own calculations but no doubt, and I know that I did myself, carry out calculations to investigate various aspects.

In terms of the process of the panel it met seven times between March and October 2011 and had other supplementary meetings also after that date. The meetings were run formally and minutes were taken. Each meeting included a presentation from each consultant followed by discussion. So this was quite important because over that period between March and October the investigations were developing.

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Early on in that process there was very little written down but as each meeting was held and the investigation was progressing those results were presented typically in a PowerPoint format to the overall panel and, and discussions ensured from that. Over that period also panel members corresponded freely via email and I think you've seen the evidence of that in the emails that are on the Commission's website. It was important to recognise that all panel members were given the opportunity to contribute and did so and from their different standpoints. It was probably true that the technical specialists in structural engineering probably participated more in the technical areas but all panel members did contribute and the panel members were also given the opportunity to comment on the consultant's report which came in various drafts, in terms of the CTV building came in various drafts to the panel.

The expert panel did rely on the investigating consultant material. So their analyses, their results, the, those results were relied on by the panel. That's not to suggest that we thought that there was any doubt because we didn't but we were commenting on the inputs but we accepted that the outputs were, were accurate given those inputs. An iterative process was used to prepare the panel report in that drafts were prepared with comments fed back and adjustments made and new drafts circulated. So it was a very iterative process. Those report drafts were sent to every member of the panel and the overall objective of the panel report was to achieve consistency where it was possible with the investigation report.

Q. Is that a function Mr Jury of the responsibility or role that the panel had in reviewing and approving the consultant's report?

A. That is correct. I think the, the panel was well aware that it would have to approve finally, as one of its requirements under the terms of reference, it would have to approve the investigating consultant's report.

Q. I believe yesterday it was described by my friend as an alignment process. Is that how you would characterise it?

A. I think that's a very fair summary, yep.

Q. Thank you. Please continue.

A. Thank you. Once the findings of the investigation had been agreed, and by that I mean that the investigating consultant had met all the requirements of the expert panel, the panel did meet to discuss overall conclusions and recommendations and in the case of the CTV building they mainly came out of the CTV investigations and then the final panel report was approved by all panel members and I think that's, that's an important point to make.

The key findings that are reported in the expert panel report are, there are a number of them and I'll quickly go through those.

We found, based on all the evidence that had been presented to us via the investigating consultant's activities, that the damage during the 4th of September and 24th of December 2010 earthquakes was unlikely to be a significant contributor to the collapse on the 22nd of February.

That the shaking during the 22nd of February 2011 earthquake was stronger than design levels and by implication also that on the, it was stronger than design levels.

That the columns and beam column joints should have been detailed for ductility.

The columns did not meet minimum requirements for shear. I didn't mention in terms of the terms of reference when I was going through that slide that one of the requirements in the terms of reference was that we did not go into the issues of culpability or liability but we were very interested to understand what had happened with the design and where that might have impacted on, perhaps, reasons for the performance of a building.

We noted that the centre of stiffness of designated primary seismic resisting elements was significantly eccentric to the centre of mass.

We noted that there were, that the ties between the floor diaphragms and the north wall had been retrofitted after construction but only on levels 4, 5 and 6.

We also noted that in its pre-September condition the building was calculated to achieve somewhere around 40 to 50% of new building standard.

There was no evidence of liquefaction within the site or a significant movement in the foundations.

And we also noted a number of issues relating to construction that had the potential to introduce weaknesses into the building and they included:

low concrete strength in some columns,

non-roughened construction joints,

poor connection of some beams to the north core on some levels and non-achievement of intended structural or non-structural separations in several areas.

Those findings very much align with the report of the investigating consultants.

Our conclusions based on all of that are encapsulated on this slide and they are:

That collapse occurred because shaking caused forces and displacements in a critical column or columns sufficient to cause failure

And that once one column failed other columns rapidly became overloaded and failed.

We might fall into, or we might come under some criticism for the very general nature of this conclusion but I think it is a direct result of the number of scenarios that have been hypothesised already in our report, in the expert panel report and also by a number of others that it's not absolutely clear the exact sequence of the collapse but in some respects it's a bit like the fact that you die when your heart stops but anything can cause that to happen. The main cause of the failure was the loss in the columns in our view.

Q. The report, the expert panel report I am talking of now Mr Jury does contain a section headed up, "Possible collapse scenario," and in that section you note, the report notes that several possible scenarios were identified. That scenario 1 involving initiation of failure along line F was a strong possibility. Is that the scenario 1 that was referred to as that scenario 1 by Drs Hyland, or Dr Hyland and Mr Smith who have just given evidence?

A. That is correct. I would say also in comment that there is not much difference between any of the other scenarios in terms of when they might have occurred. To have the columns fail they need axial load and they need drift and all of those scenarios involve axial loads in various forms, some more, some less and, but they all involve drift and you need one or other or both of those to have failure.

Q. Thank you.

A. Factors that may have contributed or did contribute to the failure include:

the high horizontal ground accelerations on the 22nd of February,

exceptional vertical ground motions potentially, of course these were not recorded exactly at this site but there are a number of other records around the city that would attest to these.

Certainly a lack of detailing of columns and beam column joints.

High column axial stresses. Perhaps not exceptionally high unless you bring all those issues that could lead to axial load, high axial loads into play at one time.

Potentially low concrete strengths in critical columns.

Interaction between columns and spandrel panels, certainly a potential issue, and quite important perhaps for scenario 1.

Separation of floor slabs from the north core as a potential issue.

The plan irregularity.

Influence of the masonry walls on the west wall.

And perhaps quite importantly the limited robustness and lack of redundancy in the whole structure once things started to happen.

1155

Q. By "redundancy" do you mean availability of alternative load paths?

A. Alternative load paths and also particularly for vertical gravity loads. Once the columns' capacity have been exceeded there was nothing effectively to separate the floors and the floors came down.

It's of interest to note that out of all those potential issues there are a large number that relate to potential vulnerabilities in the structure and those I've highlighted in this slide in red, and all of these in combination on their own or in combination could be potentially major issues in terms of evaluating a potential collapse sequence.

There were differing views as I alluded to earlier on in my presentation, and I think in terms of emails that were presented yesterday at the

Commission, some of those were quite forthright. I think though that they indicate robust discussion amongst the panel and also with the investigating consultants about the conclusions that they were coming to in terms of collapse sequences. But these differing views included what was the most likely initiation of the collapse, the validity of modal, the response spectrum results and predicting the performance during earthquakes, and the relative importance in weighting of the identified potential contributors, particularly the influence of the spandrel panels and the time of any – timing of any separation of the floor slabs from the north core. I would perhaps add to that one comment that has been brought out in evidence which was certainly an area of differing views and that was in relation to the need for calibration of the analysis results, particularly the response spectrum results. I have read Professor Priestley's evidence and I agree or accept that the points that he's outlined cover these items and generally convey the issues well on the differing views that were expressed at the time. My own view, for example, on the collapse sequence probably tends more to scenario 2 rather than 1 but I accept that there is a lot of evidence around that could attest to any one of these scenarios.

Q. Mr Jury you said in a statement of yours at 31 May of this year that a final version of the panel report was prepared, that all members of the panel were prepared to accept as reasonable, so the report as finalised reflects resolution of these issues giving rise to differing views?

A. That is correct. I think a lot of effort went into preparing a report that all the panel members could accept and that would explain some of the careful wording in that report, to in order to accommodate all those views.

Q. Yes I was going to ask you if that had implications for the wording of the report?

A. I don't, I don't believe that anybody's views was compromised by those wording, particular wordings, but they were quite carefully crafted in order to get acceptance from everyone.

Q. Thank you, I interrupted you.

A. I think too that I alluded to before that we have presented a number of collapse mechanisms and I guess anybody, any engineer looking through the evidence will have, has the ability to come up with their own examples, or own scenarios of what might occur, and there have been others presented in evidence as well that I have seen. I think it's important though to concentrate on the conclusions and the findings, particularly those that we've presented in our report, because I don't believe that they are necessarily affected by any particular scenario that's presented.

Q. So all, just to be clear then, all expert panel members agreed with the conclusions in the report?

A. That is my understanding yep, and I guess there will be more evidence that comes out of the investigations that are currently underway to extend a nonlinear time history analyses and those things which will either sway the opinion to one or other of the potential scenarios.

Just thinking about these three bullet points I've got up here in terms of differing views. I've already indicated that my, my feelings more go towards the scenario 2, and I have listened over the last few days at the evidence given particularly around the beam column joint stiffnesses and what have you, they, and the strengths. The beam column joints have the potential to be a collapse initiator but they also have the potential to change the stiffness of the frame once the integrity of the joint may have been lost and I –

JUSTICE COOPER:

Q. Just remind me about scenario 2 Mr Jury?

A. Scenario 2 relates to the internal columns carrying most of the vertical load but that perhaps having a, being an initiator though a failure of the column and I just offer that perhaps a failure of one of those internal columns at the ground floor level A, the junction with the foundation where the drifts and the time history analyses to date certainly show drifts sufficient to cause failure are perhaps, is that those columns are perhaps less determined by the stiffness of the joints.

EXAMINATION CONTINUES: MR ALLAN

A. So the recommendations that were outlined in the expert panel report were that the DBH, Department of Building and Housing should take action to address a number of issues.

These included a review of design allowance for irregularity. I know a number of the technical panel members are concerned about the allowances that we have for irregularity even in our current standards and I suppose it would be possible to suggest you should not have irregularity but it would lead to some perhaps uninteresting buildings, or should not have, we should always aim for regularity and achieve regularity but it's not always possible so there needs to be tools available to do that and we query whether the current requirements are adequate.

We also recommended that there be some investigation made to identify other existing buildings around with perhaps this non-ductile gravity column issue. I think as Dr Hyland and Mr Smith pointed out there are a number of issues related to the 19 – the codes that were in place in the 1980s when this particular building was designed. Those requirements were tidied up and made less ambiguous in the 1990 versions of the particularly the concrete code, but there were a number of buildings, a large number of buildings constructed in that 1980s period where some interpretation could've been made to the detriment of those buildings perhaps in allowing those, allowing columns to be detailed without the confinement that we would now expect. Those investigations, some investigation has already been done by the Department in that, around that area.

And also we recommended that identification be made of existing buildings with columns affected by part height spandrel panels. I think one of the areas of differing view was in terms of the effect that the part height spandrels may have placed, may have played in terms of affecting the column performance, particularly on the exterior faces. But there is no doubt that panels, spandrel panels affecting columns is a really, a real potential vulnerability in other buildings potentially.

We also recommended that design procedures should be developed or reviewed in relation to connections between the floor slabs and structural walls. There was no doubt in the expert panel's mind that the connections between the, particularly the north core and the slab but also potentially the south wall and the main floor system had no, had very little capability to transfer forces should they be required to.

And we also recommended that review be carried out of measures to improve the confidence and design and construction quality.

And that is the end of my presentation.

1205

Q. Thank you Mr Jury, now you appear to have a copy of the expert panel report with you and that is dated February of this year?

A. Yes.

Q. And there are some introductory chapters but the chapter that sets out the panel's views in relation to the CTV building is found at chapter 5 of that report?

A. Yes.

Q. If we could put that into evidence please, thank you. Now one of the points of difference that you referred to in your slide covering this was the validity of modal response spectrum analysis in predicting performance during earthquakes. You went on to state that as far as you understand each panel member agreed with the conclusions in the report. At what extent then do those conclusions turn upon the validity of modal response spectrum results?

A. I believe it was always the expert, the members of the expert panel's view that the response spectrum analysis techniques were primarily to determine whether it was possible to design this building in the way it was designed in the 1980s and so it was more around what an engineer might have done in the 1980s in order to design this building. It was extended by the investigating consultants to attempt to explain other issues but I believe it was always the expert panel's view that elastic analysis like these were quite limited in explaining some of the issues and that it would be better if not essential that analyses of the non-linear time history nature also be conducted to test some of these issues. I don't hold the view that is expressed in Professor Priestley's evidence that undue reliance was placed on the results of the ERSA analyses in either the expert panel's report or the investigating consultant's report because I think an awful lot of discussion went on in terms of preparing the report to make sure that the various forms of analyses were given their appropriate level of importance if you like in terms of the conclusions that were reached.

Q. The non-linear time history analysis being the essential analysis under-pinning the conclusions in the report, as opposed to the ERSA result?

A. Yes I think both provide some input into the final conclusions but I think the greater weighting has been placed on the drifts that occurred that result out of the non-linear time history analyses and I also accept the point that Mr Smith made that these analyses, while they attempt to get the closest we can possible get to reality they will always fall short because no matter how much accuracy you put into the analyses there are always question marks about the role of certain assumptions in the final outputs.

Q. Indeed. Now in that respect both Dr Hyland and Mr Smith emphasised the importance of reconciling analysis results with witness testimony and the debris observed after the earthquake, aftershock. The expert panel placed similar weight upon those other inputs into your thinking?

A. Yes I believe we did. Quite a lot of store in coming up with the scenario 1 as the primary means of collapse was the fact that did or did not the slabs detach from the north core and certainly the evidence of the – the photographic evidence and the evidence of investigators was that maybe it did not. There was a lot of discussion around could the slabs have detached but not necessarily collapsed under gravity load and I still think that is an unanswered question but it does suggest that for one reason for going with scenario 1 is that the evidence that we currently have would suggest that those slabs did not detach at levels 4 through to the top of the building. I believe that the expert panel members also well they looked at the, particularly the plan area, plan of the north core and its attachment to the slab and notwithstanding calculations that were carried out, wondered how it could still hang on.

CROSS-EXAMINATION: MR REID – NIL

MR RENNIE:

Just by way of clarification Sir, the witness also has a lodged written brief, where I understand this to be a presentation of that and I am assuming that is before the Commission as well?

JUSTICE COOPER:

Yes we have it here, I am assuming that is to be treated as evidence, to be taken as read.

MR RENNIE:

That is what I am assuming Sir but it is already in if I can put it that way.

JUSTICE COOPER:

Yes and it has been spoken to rather than read out.

MR RENNIE:

Thank you Sir.

CROSS-EXAMINATION: MR RENNIE

Q. Mr Jury, you mentioned that you were here yesterday and I think some days before so you have heard some of the evidence already?
A. That is correct.
Q. And one of the matters that you have mentioned related to a series of emails and you commented on the tone of that but I am actually going to

ask you about the issue in that which related to the cracking of the slab, you recall those?
A. I do yeah.
Q. If we just have the first brought up which is BUI.MAD249.094B.31. Now this is at October 2011, this is I think about a week before the final formal meeting of the expert panel?
A. That is correct.
Q. Yes, and am I right in understanding that after that meeting on the 20th of October the interaction between the members of the expert panel basically went on by emails, phone discussions and so forth?
A. That is correct.
Q. And am I also right in understanding that the way the time sequence ran on the tasks for the expert panel the CTV building was the last in point of time to be considered because the investigation was last in point of time to be carried out?
A. That is correct.
Q. So to put it simply by 14 October your attention was, as a panel, was essentially now centred only on the CTV building?
A. That is correct, and the panel report.
Q. And overall panel report, certainly, yes. But the process that had been established for the expert panel essentially relied upon Dr Hyland and Mr Smith to locate and present the information and then the panel would, as it were overlook it but not separately enquire or test?
A. That is correct.
Q. Now the hypothesis that you were raising in this email which is in fact to Dr Priestley was that the slab may have been cracked in the Darfield event or possibly even before that?
A. That is correct.
Q. Yes and that hypothesis you referred a short while ago in your evidence to that saying it is in your mind still an open question as to whether the slab did crack at some point?
A. Yes and perhaps one that will never be determined fully.
1215

Q. Yes, and to be clear, do I have it right that the slab cracking which you were thinking of could be a cracking of any of the slab floors in the building although your attention was particularly focused on 4, 5 or 6?
A. My question was relating to 4, 5 and 6 but probably primarily relating to the area of the slab adjacent to the north-western area of the core.
Q. Yes.
A. And perhaps around the drag bars.
Q. Yes, but to be clear you were contemplating there may be a floor on which the slab had cracked or that all floors had a cracked slab?
A. A difficult one because if shrinkage cracking had been present then it had to be caused by something and it had to be caused by restraint and the only restraint that I could see at that time was the restraint between the west wall which may not have been fully separated and the north wall and there would have been restraint in the concrete, causing cracking in the concrete.
Q. And the leaving aside the obvious question of looking at a slab and seeing a crack, the indirect indications of a cracked slab would be accounts from persons present on the floor or secondary signs in such things as floor coverings?
A. That is correct but none was noted.
Q. Well that's really the point I was coming to because more accurately I suggest none was reported to the panel, is that the position?
A. That's probably more accurate, yes.
Q. Yes, yes. Now if we can next go to BUI.MAD249.0285.10? Now this is an interview conducted with a Mr Godkin by Dr Hyland and Mr Smith.
A. Yes.
Q. And it's unlikely that you will have seen this before?
A. I have seen this before, well I'm not sure exactly this version of it but I'd seen his, his evidence and also the recounting of that in the investigating consultant's report. I think it was in there too.
Q. Well let's take him stage by stage. Dr, try again, Mr Godkin has given evidence to the Royal Commission and so you're telling me that you've seen his brief of evidence?

A. I've seen his brief of evidence. Yes.
Q. Were you here when he gave evidence?
A. No, I wasn't.
Q. No. Now the next thing is you've put to me your belief that Mr Godkin's evidence was also to be found in the eyewitness accounts in the investigation report. Is that what you've just put to me?
A. I would reflect on that, because I'm not certain. I'd certainly read it in the last few days (inaudible 12:18:26)
Q. I'm not digging a hole for you at all, to the contrary I'm going to invite you to have a look at the left-hand corner of the page in front of you?
A. Oh right.
Q. And you will see that this is in fact an interview which took place on the 9th of February 2012?
A. Yes, I think I've read this on the Commission's website.
Q. Yes.
A. This particular (inaudible 12:18:44)
Q. Yes. So in fact and I can tell you if we scroll back to page 1 which I don't think we need to do it is expressly stated in the heading that this interview took place on the 9th of February 2012. Now without going into a detailed review of the pages I can tell you but I can verify this if I need to that the eyewitness reports in the building collapse report consist of four interviews and they are, that is to say four interviews of persons inside the building and having now heard seven people from inside the building give evidence to the Commission, even though the building collapse report doesn't name the four people who were interviewed it's possible to cross-match them and work out who's there and who's not, and Mr Godkin it appears was first interviewed on 9 February 2012 by Dr Hyland and Mr Smith. So what he told them could not form part of the material which came to your expert panel, could it?
A. No.
Q. No. Now Mr Godkin says a number of things about a hump in the floor and if you look down to the bottom of the page that I have referred you to he says in relation to the hump that it was on the south side of the

hump that developed from the floor and he then says that it got progressively worse, see that?

A. I do.

Q. So that at this point he's discussing a post 4 September and pre Boxing Day scenario?

A. Okay, yep.

Q. Now if we can go to the next page –

A. Can I just, to clarify?

Q. Yes.

A. Is he talking about a hump at the ground floor? I think when I read it –

Q. No, this is level 4.

A. Level 4, okay thank you.

Q. Because just to quickly cut through to give you a fair view of what I'm asking you about. The Commission has heard from three witnesses. The other two were Mrs Aydon and Ms Brehaut. Now they didn't talk about a hump but they talked about a post 4 September and post Boxing Day experience and especially post Boxing Day that pens and pencils placed on desks would no longer remain in place and had to be retained with rubber bands and blu-tack because otherwise they would roll across the desk and fall on the floor, but their evidence didn't specifically address this matter of the hump that Mr Godkin is talking about. Now if you again go down to the fourth entry on the page I've now taken to you, Mr Godkin says, "The hump got progressively worse and the hump there was what staff and myself went to Brian about," do you see that?

A. Yes.

Q. Now Brian was the manager, sadly we're not able to have his account of what was involved and it then, the discussion then runs on down through there indicating as I put it to you that this was a shall we say a distortion to try and use a neutral word in the floor part of it covered by carpet, part of it covered by vinyl. Now I'm not trying to cut you short, you're welcome to read through the next two or three pages if you would like to do that. But the point I'm putting to you is, is this the kind of eyewitness evidence which you had in mind in suggesting that this question of the slab break had to be considered?

A. Certainly, certainly it would be, yes.

Q. And in terms of the level of significance that one would give and what we have here in essence is Mr Godkin's account, two people supporting part of it, an account of deterioration in the sense of a greater slope and greater problems, a significant change after Boxing Day. Are those matters that you would give weight to in considering the slab breach issue?

A. I believe so but I think there was also other evidence too, particularly by the quantity surveyor and those that were inspecting the building for damage and repair. Look I'm just not certain whether this this hump was covered in that investigation or not so you'd have to consider all the evidence that you had but certainly this evidence would send you on a path to go and look into those issues, yes.

Q. The evidence of the inspection by Mr Coatsworth who was the independent engineer and of the quantity surveyor who accompanied to him related to a period before the Boxing Day quake.

A. Okay.

Q. And the evidence of Mr Godkin in relation to the nature of the hump and the evidence of Mrs Aydon and Ms Brehaut in relation to the rolling pens, the blutack, et cetera, indicates a more obvious inclination after Boxing Day and Boxing Day seems to have been the start of the pen and blutack problem, would that again suggest an investigation as to whether the Boxing Day earthquake had had some impact on the integrity of the slab?

A. I think all evidence has to be taken on its, on the words that are stated.

1225

Q. Yes.

A. The Boxing Day earthquake though was significantly less in shaking generally than the September and I, I get the impression maybe that you can confirm that this hump was visible after September and got worse on Boxing Day?

Q. That's what Mr Godkin says.

A. So it wasn't picked you by the quantity surveyor and the independent engineer in their investigations. I'm asking the question really?

Q. No I can tell you it was not picked up at all. Mr Godkin, the evidence from Mrs Aydon is that Mr Godkin had one of those senses of acuity to level such that some people have who go around lining up pictures on the wall and that sort of thing because he had identified a separate bulge in a window which she stated she was unable to see until he pointed it out to her and, and I can't assist you as to whether that was a relevant factor to what we're talking about but just so you're aware of what we're talking about in terms of the level of perception. Now in all events the rejection of your slab breach proposition on the 14th of October 2011 seems to have been the end of any attempt to investigate that. Is that correct?

A. I suspect so. I mean the difficulty was that the calculations for that particular connection were showing that it had, had quite a lot of capacity.

Q. Yes.

A. My, my immediate reaction prior to, to that was that the tear that you can see in the photographs of the slab in that area could indicate a tension failure in the concrete at the end of the starters coming out of the north wall, or the slab of the north core but the calculations that had, were done around that sort of suggested that that probably wasn't the weak point from a tensile point of view. So you were then left with trying to decide that if you were going to continue with the argument of failure of the drag or the connection between the north core and the slab that there had to be some deterioration of that connection prior. So when that was I guess considered and discounted that discharged that view from my, from my concerns and at that time too we weren't reliant on the separation of the north core from the slab or the, any of the failure

sequences that were put forward. It was just an exasperating [*sic*] circumstances if it had occurred.
Q. Sure.

JUSTICE COOPER:
Exacerbating perhaps?

MR JURY:
Exacerbated.

CROSS-EXAMINATION CONTINUES: MR RENNIE
Q. The 14 October proposition that you put to the investigators was one that if I read your email correctly Dr Priestley also considered merited consideration.
A. Yes I think in, in reading Professor Priestley's evidence he still has the view that the separation of the north core from the slab was a primary, or potentially a primary initiator. I, I, I don't need that to be comfortable about the other scenarios and I'm, but maybe Professor Priestley will be asked that question also but whether he is reliant on it, but I don't think I'm reliant on it for any of the other scenarios.
Q. My industrious junior has passed me a note slightly correcting what I told you about Mr Coatsworth which, he points out that in paragraph 101 of Mr Godkin's, is that right, 101 – 10, 10 of Mr Godkin's evidence Mr Godkin told the Commission that he had told Mr Coatsworth about the hump so I think probably it would only be fair to Mr Coatsworth to assume that he gave some attention to the issue pre-Boxing Day. Now what we've just been discussing indicates the difficulty of obtaining a settled 100% confident outcome of why the building collapsed doesn't it?
A. It does.
Q. Yes and yet the impression one takes from the process that was being run by the Department of Building and Housing was that they wanted it

brought through to a report which stated the reason why the building collapsed. Was that your perception of it?

A. No I think, I think the, the Department as, as was the expert panel was very keen to make sure that no possible reasons for the collapse that could not be absolutely ruled out were not continued through to the final report. So people could take positions on which scenario they felt was more valid than others, taking everything into account, but it was important to make sure that each of the potential views that had been expressed and hadn't been discounted were carried through into both the consultant's report and also the panel report. So I don't, I don't believe that the Department ever indicated to, certainly that I'm aware of, ever indicated a desire to, to follow a particular line. The panel members certainly set out what they thought were minimum requirements in terms of the panel report.

Q. Now the, the unusual feature I suggest of the Department's process was that it was running in parallel with the Royal Commission process, seemingly seeking to obtain answers to largely similar questions. Is that a matter that was taken into account by the panel in considering how and when to report?

A. I, I believe that we, we were engaged before the Royal Commission had, had necessarily been promulgated or, or appointed but it was around, it I recall correctly it was around the time when the possibility of a Royal Commission was certainly out there. I believe that the, the process the Department entered into in terms of commissioning us to carry out investigations was based on their perceived need for capturing and dealing with these issues but certainly once the Commission had been appointed it was always of our view that this report would perform, would form part of the evidence that went to the Royal Commission. So in that respect we were mindful that the Royal Commission was proceeding on a parallel path and, and in fact I guess while we were preparing this they were undertaking their own investigations from various parties as well.

Q. It was a close run thing in the sense that the panel in the investigation were announced on the 6th of April and the Royal Commission on the 11th. It wasn't that far distant.
A. Mmm.
Q. To what extent therefore would you say that the expert panel's report is essentially information you gathered to assist the Commission, or did it have some other purpose?
A. No I think the, one of the purposes, or one of the purposes was to present recommendations but I think that was primarily to inform the Department about issues that they might take up –
Q. Yes.
A. – in the future in terms of making sure that the regulatory environment did cover these issues that might come out of these investigations.
Q. Well that's clearly a separate and proper purpose for the Department to consider isn't it?
A. And I, I suspect, look I don't, I wasn't in the, in the Department so I don't know what their thinking was at the time but I know that early or late in, late in February, early March there was quite a, an urgency in order to get a, a report underway and particularly to collect information that might otherwise be lost.
Q. That's 2011 you're talking –
A. That's 2011, yes, yeah.
Q. Thank you. This morning counsel assisting produced what are known as reply briefs from Dr Hyland and Mr Smith and there's a provision, or statement by Dr Hyland in one of those which I'm just going to read to you to give you an opportunity to comment on it because it relates to the purpose or process of the panel and how it operated and I'm just going to read it out to you. I'll have it put up on the screen as well and take a moment to think about it and then you don't have to comment on it but I'd like to give you that opportunity. The reference is WIT.HYLAND.0001.1?

1235

Q. And just while that's coming up Mr Jury and it, well there it is with us now, this is the way in which Dr Hyland has replied to a number of statements in Dr Priestley or Professor Priestley's brief, and if you look down to C14 which is the third commentary box you'll see a paragraph which reads as follows, "The panel report draws on the building collapse report with some modifications by panel members. Nigel Priestley was vice chair of the panel, the chairman Sherwin Williams was a non-technical chair. Nigel Priestley's approval was apparently required for the reports to be accepted by the Department. No separate investigation was undertaken by the panel. I understand that the DBH project manager, David Hopkins, largely drafted the panel report." That's what Dr Hyland is saying about Dr Priestley's evidence.

A. I would be prepared to make some comment. I think I've already said that Dr Hopkins was the editor and I think that that's the function that I saw him fulfilling. Whether he drafted sections or not, the, the draft reports were made freely available and were commented on. Somebody had, some editor had to put the report together to start with and to allow others to comment on it. I don't think Nigel, Professor Priestley's approval was only part of the process that I've outlined for the expert panel. The expert panel's terms of reference required each member to effectively approve the consultants' report, so that was part of that process. I didn't see anything else in terms of my experience.

Q. And in terms of your own extensive experience and your participation in the panel do I take it you're well satisfied that the expert panel report is the report of the members of the panel and not anybody else?

A. Yes I am quite.

Q. Now I'm now going to take you to a brief separate point and the reference for this is BUI.MAD249.0494A.40.

WITNESS REFERRED TO DOCUMENT – EMAIL DATED 22 DECEMBER 2011

Q. And if we just go to the next page for a moment, I'm sorry must be the one after perhaps. I'm sorry it must be, no go back and I'll deal with this first. Now this is an email which you on the face of it won't have seen because it relates to comments that you made on the 22 December 2011 submission which Alan Reay Consultants Limited made in respect of the December version of the draft report, do you see that?

A. Yes I can see that.

Q. Now you will see that what Mr – what Dr Hopkins is saying is that you've made comments, you made them on the basis that the document would be sent to ARCL, Dr Hopkins says, "But it won't be." Now firstly did you know that your comments were not sent back to ARCL for their consideration?

A. I know through a meeting that was held in December that some discussion was made on how the expert panel and how the investigative consultant should respond to comments made by Alan Reay Consultants but, look to be honest I don't recall what's in this attachment. If I was able to see it I'd be able to comment on it.

Q. I'll take you to it, it's got a slightly different reference, it's 494AA.21?

WITNESS REFERRED TO DOCUMENT – EMAIL ATTACHMENT

Q. Now, there's a number of pages of this and I think all that we need when it comes back is a simple illustration. Because what I'm asking you about is process not the substance of the points. Now this is a document which you will see is headed, "Comments on ARCL responses, summarise HCL/SSL responses 10-1-12," do you see that?

A. Yes.

Q. Now this is late in the process of the final preparation of the building collapse report and in turn the expert panel report isn't it?

A. Yes.

Q. Late in that time period?

A. Yes.

Q. Now the draft – the first occasion on which ARCL saw a draft report was on the issue on 8 December of the report and the version that it was then in, did you know that?

A. I don't recall whether I knew it or not sorry.

Q. Were you aware that between the February earthquake and that time ARCL had made repeated requests for copies of information, access and so forth to enable it to participate in this evaluation?

A. I was vaguely aware of those request but I never participated in the final decision of whether to provide or not.

Q. No, and I'm not suggesting you or the panel did, that was the Department, did you know that the Department consistently refused to provide that material through 2011?

A. I believe I'm aware of that, yes.

Q. Yes. Would you agree that that was contrary to the normal practice in the engineering profession that where one engineer's work is being examined that engineer will be consulted in relation to the investigation of him?

A. I, I believe that that probably is an important premise, but at the time the report had not necessarily reached its final conclusion, so in that respect I think the, both the panel and the investigating consultants needed to get their reports completed and they, where necessary they took into account the comments that had already been made but they weren't necessarily bound to respond back with individual answers or provide information, I guess.

Q. Well I'm not going to labour the point with you because it's clear from what you've said that it wasn't the panel which took a view on how there should be involvement with ARCL, it was the Department. But if we move to December, the 22nd of December when the responses come back in, ARCL having had only just over two weeks to prepare those in relation to all this work, the evaluation which then took place appears to have included comments by you which Dr Hopkins decided would simply not go back because that was his decision. It sounds as if that's contrary to the process that you were expecting by the time the report was in final draft stage?

A. I guess I wasn't, I didn't have expectations to that. I guess from my point of view whether the comments were sent back or not depended on whether it was considered overall necessary to do so. I just don't recall either why Dr Hopkins made the comment that my comments had been prepared to send back to Alan Reay Consultants because it wouldn't have been in my mind in the time, other than preparing them in a form that could go back, rather than maybe more, be more basic responses perhaps.

Q. Well we haven't actually got a page with your comment on it if we, if we just scroll down we'll find some Word format, if we just go a page or two I think it is from memory, 31 will do it?

1245

JUSTICE COOPER:

Mr Rennie, I hesitate to interrupt you but I am wondering how far this is going and for what purpose? This is a Commission of inquiry into the reasons for the collapse of the CTV building. It is not an inquiry into the processes that were followed by the Department of Building and Housing when it was carrying out its quite separate investigation of the building. Now there may be a point at which this has a relevance if it is able to be brought to some particular issue canvassed in either the consultant's report or in the panel report which is germane to the collapse of the building but you haven't arrived at that point and I don't know whether you intend to.

MR RENNIE:

Well Sir I accept Your Honour's guidance, what I am endeavouring to deal with in the summary form here is that the parties that I represent are presenting evidence which is a challenge to evidence which has been put before the Commission in, as a result of an investigation which did not engage with the parties that I represent and that is the single point that I am after because it is not a matter of what we have put forward previously being discarded in that investigation, it is a matter that we are having to put it forward now. Now all I am seeking to do is make that apparent, I am not seeking to waste the Commission's time by, as I put it yesterday, arguing about a BCR instead of arguing about the issues but equally of course we have a situation where notwithstanding all the evidence that we filed these witnesses have come forward and described it as if none of our evidence was relevant or should be taken into account and that is what I have to deal with. I mean this witness has presented a whole series of conclusions.

JUSTICE COOPER:
But the normal way that is done is to say that experts retained by your clients are of the view that A, B, C and D occurred, what do you think of that but you are a long way away from running that sort of line of cross-examination.

MR RENNIE:
No Sir, no well I mean it goes beyond that, we will be calling Dr Reay. We will be calling Mr Smith from the office and separately –

JUSTICE COOPER:
Yes but presumably to address substantive error in the analyses presented by other engineers including Dr Hyland and Mr Smith and the present witness so that is – well I don't understand what this is about because if your point presently is simply that Dr Reay was not consulted about the report until it reached the point where it was a final draft on which those involved in the Department of Building and Housing inquiry in whatever capacity were happy for there to be consultation, if that's your point we understand that. We also understand, because it is set out in Dr Reay's evidence that it was considered that the time given for that response was far too short. We understand that too but is there anything else you want to develop our understanding about in this way.

MR RENNIE:
Not in this way Sir, no.

JUSTICE COOPER:
Well you can rest assured that we are very conscious of the points I have just made.

MR RENNIE:

Yes I was anticipating a line of approach to our evidence that it is patently clear I don't now need to anticipate –

JUSTICE COOPER:

Which is what? Do you want to make that express because –

MR RENNIE:

You can't say it now because you didn't say it then.

JUSTICE COOPER:

No. I hope it I apparent, it should have been from what I said yesterday I would have thought. We are conducting our own inquiry and we will make our decisions –

MR RENNIE:

Exactly Sir.

JUSTICE COOPER:

- on the basis of the evidence we hear. You couldn't – Dr Reay could not possibly be criticised on the basis of not being able to play a more active role in the Department's investigation than he was allowed to do.

MR RENNIE:

I am obliged Sir and the only associated point which the witness has acknowledged, although it is hearsay is that the Department wouldn't make the data available so we started late and we can say that, we say that in our briefs and I am just being far too cautious and in the process of that have managed to be disrespectful in ways –

JUSTICE COOPER:

No, no, no –

MR RENNIE:

- I regret that.

JUSTICE COOPER:

No there is no suggestion of that either.

CROSS-EXAMINATION: MR RENNIE

Q. Mr Jury I will now reconnect myself to my notes and move on. The panel did take an active role in identifying the types of testing which should be carried out in the investigatory phase, didn't it. It commented on the use of ERSA it suggested further NTHA analysis, those are two examples?
A. In terms of the analysis yes, in terms of collecting the samples and what have you probably less because that course had already started by the time the panel sat.
Q. No I was addressing really the analyses to be used, the programs to be preferred and the extent of the work to be done?
A. I think initially it was reactive, we got a bit more proactive as the process moved on.
Q. Now we have ended up in the situation where not only very properly but indeed much to be welcomed we have further NTHA analysis being done and you'd have some awareness of that?
A. I am aware they are being done, I don't know the detail.
Q. The situation therefore in that sense is that the moment our knowledge is not as far advanced as it may be by that further analysis?
A. Mhm I have my own personal view on that I could let you have that.
Q. Well, I walked into it, if you want to express it?
A. My own feeling is that analyses could go on forever on this building. Sooner or later each investigator has to decide whether he is prepared to live with the conclusion that I put up on the slide and I have not seen any indication that that is not a primary conclusion that would resolve out of any analysis and it is possible to go on refining but I think it

becomes a diminishing returns exercise perhaps but it does not preclude the fact that something could be done that would come up with some totally earth shattering result that would change everything.

Q. Is that particularly focused on the understanding of the engineering issues in the failure of the building?

A. I think it revolves around the very detailed discussion and comments that we made around the drift because it, I mentioned before to get column failure you need axial load and you need drift or one or other or both of those aspects and as the investigation developed we were at first at the point of some of the analyses trying to decide whether there was enough drift to indicate the columns had failed. Earlier analyses to we were uncertain about the axial load situation but those analyses all developed both the ERSA and the time history analyses and other analyses that were done, all developed to the point where I don't think anybody who was involved in the process, the investigating consultant or the expert panel were in any doubt that enough drift, enough axial load in combination could occur to fail the columns.

Q. That would leave the issue though would it not as to whether this could have been found earlier at an earlier stage by inspection or after an earlier earthquake as still being highly relevant?

A. It may be relevant I think the observations of the engineers and others who walked through the building after September and are important in that regard, I mean it's, it has been suggested that minor cracking could be indicative of major damage but those who are looking at buildings after, immediately after earthquakes are looking for the signs that damage, significant damage has occurred. It doesn't preclude the fact that it may have occurred and you don't pick it up, certainly.

1255

CROSS-EXAMINATION: MR MILLS

Q. Firstly I'll just take you to a point that my friend raised with you about the evidence of this hump and I just want to put this fully so that you can then see if you've got any different views on it. It's actually referred to in two places. First of all as my learned friend said it's referred to in Mr Godkin's evidence. It's referred to in two paragraphs, paragraphs 10 and 11 but then it's actually also referred to in Mr Coatsworth's evidence. So I just want to tell you what the combination of that is and you can tell me whether you want to add anything or change anything to what you said before about its relevance. So first of all Mr Godkin's evidence, and he's referring pretty clearly to the inspection that was being done at the time by Mr Coatsworth. He refers to it as the second inspection and he says, "I was present for the second inspection that took place in late September," I won't read all of it, "I walked around the floor with John Drew and the engineer in late September," that's when David Coatsworth did his inspection, "And pointed out the various cracks and the hump in the floor and then he goes on in paragraph 11 to describe that hump and says that he drew it to the attention quite specifically of the engineer, as I say I don't think there's any dispute, that'll be Mr Coatsworth, and he describes the effect of the hump. He said it was sufficient to cause a pencil to roll across the receptionist's desk which did not happen before the September earthquake and then he says, "The engineer said that all concrete buildings hump between the supporting beams that hold the floors up when the concrete dries over the support and the engineer told me the building was doing what it was meant to do following an earthquake." So that's Mr Godkin. Now the equivalent to Mr Coatsworth and he says in paragraph 71 of his evidence, "As I have previously mentioned I remembered talking with some of the staff at Kings Education," and of course Mr Godkin was at Kings Education, "About the deflections in the floor and walking over the floor to see what they were talking about. I noticed the high points over the beams and the sags in between but I would have expected to have seen more significant deflections if the floor had yielded." Now, again, I don't think there'll be any dispute that what he's describing in his more technical engineering language is what to Mr Godkin is the hump.

A. Mmm.

Q. Now is there anything more or different that you would like to say about that issue now that I've put the rest of that evidence to you?
A. No. Thank you, thank you for that. I, I was envisaging when a hump was being talked about, about a discontinuity in the floor perhaps -
Q. Mmm, yes (inaudible 12:58:48).
A. – creating a kink in the carpet. So it is a different connotation in terms of a hump caused by the slab being held up by the beam and the slab deflecting in between the means, it’s support.
Q. Yes.
A. That is a much more gradual hump.
Q. Yes. So do I take it from that that having heard that evidence that you wouldn't have attached any different significance to it to that which Mr Coatsworth attached?
A. No and it, and it certainly wouldn't be the discontinuity in failure of the slab that was referred to in that email that was –
Q. Yes.
A. – that I had -
Q. Yes.
A. – written.
1300
Q. The next thing I want to take you to is evidence that’s going to be given by Dr Reay, one of the briefs that he’s intending to give, it’s his second statement of evidence and again, just for reference. I won't ask for it to be brought up but so the Commission can have the reference it’s WIT.REAY.0002.1 and the part I just want to take you to, maybe I will have this brought up actually. It’s at page 5 of that reference, WIT.REAY.0002.5, and the reason I'm bringing it up it’s where he comments on the DBH report and of course for that reason it’s directly relevant to matters that you're involved in. I won't take you to all of this. He’s got a number of points that he makes in here about limitations and complaints about the DBH report. Really the only one I want you to look at and I invited you to comment on is the paragraphs 20 through 24 under that heading ‘Non-linear time history analysis’ and in particular

you'll see if you glance through that and I'll give you a moment to do that then I might ask you more about it after the adjournment but you'll see that he's strongly advocating that further tests need to be done in order to be able to, so it would seem, reach any confident views about what had occurred, and you'll see that reference there to a "shaking table reduced scale physical model experiment on a six degree of freedom shake table," and I'm assuming you understand that because I haven't got a clue what that means and then he goes on in paragraph 23 to refer or at the end of 22 referring to it being done, "either by development in New Zealand," and I take it from that we don't have those facilities right now, "or done abroad in either the United States or Japan," and then he goes on to explain what he thinks that would achieve. Now I'll give you a second just if you need to –
A. Mmm. Yes, okay.
Q. All right, now I'm just inviting your thoughts on that and it may be that a comment you made a few moments ago in response to a question from my friend, Mr Rennie, about analyses can go on forever might be equally the answer you'll give me here, but do you have a view on that and in particular the thrust of that which is as I understand it is to say that unless you do these things you don't, you're not really in a position to form confident or sufficiently confident views about the collapse scenarios?
A. Yes I think there are a couple of points that are raised here. One is about the effect of the various earthquakes in sequence. That's something that can be relatively easily addressed and I believe is being addressed in the new analyses that are being carried out. I think there will be as many questions out of those series of analyses as there are for the current series. The need for shake table testing of the building. I mean that's the, that's the next degree of sophistication in terms of analysis. A real life but still scaled version of events.
Q. Mmm.
A. I don't personally believe that it would, it's necessary to carry out such analyses or such testing to confirm or otherwise a scenario. I mean I

suspect having done that test you would still be limited by the assumptions and you'd still have question marks about what actually initiated, you'd be able to see what initiated collapse in the tests but you putting that into reality and comparing it with evidence of witness statements and all the other evidence might be quite difficult. That correlation exercise.

Q. And am I right that to do this just as with the non-linear time history analyses that the accuracy of what comes out is heavily dependent upon the accuracy of what goes in?

A. That is correct. You would be, if you were going to test it for destruction you would only have one record that you would test it to, well unless you built many models.

Q. Mmm.

A. And then you'd have trouble knowing whether they were all the same, it's just there are variations and uncertainties involved in all the parameters so.

Q. And what you'd be trying to do here am I right would be to replicate all of the members and the strength and all the components of a building built in 1986?

A. And scaling it down –

Q. Yes.

A. – too which there are real problems with trying to scale down such testing but they, all these, all these techniques and what have you just hopefully lead you to a better understanding perhaps but none of them I don't think will ever give you a categorical answer.

Q. So you don't have the reaction to this that absent that? You don't have confidence in the conclusions that were drawn by the department?

A. No, when I first read it which was before today I thought not, not really necessary in my view.

HEARING ADJOURNS: 1.05 PM

HEARING RESUMES: 2.19 PM

CROSS-EXAMINATION CONTINUES: MR MILLS

Q. Just like to ask you a question if you feel able to deal with it about one of the other briefs of evidence that's come in and it's the first of the two briefs of evidence that have come in from Professor Mander, have you read that?

A. I have, yes.

Q. You'll be aware that as part of that evidence that he's put forward an alternative collapse scenario, I think it's fair to describe it as, you've read that?

A. Yes.

Q. Do you feel able to comment on that in any way?

A. I think I would, along with any of these scenarios, would say that they have to be added to the list of potential scenarios but my own personal view would be that there are probably more obvious collapse scenarios than necessarily that one.

Q. And is it also your view that irrespective of what scenario we're dealing with that there are certain facts about that building that have to be addressed?

A. Yes, yep.

Q. And by that I mean a number of the individual structural facts that you have referred to in the report, is that what you're agreeing to?

A. If by facts you mean the evidence, the observed evidence?

Q. Yes.

A. Yes that'd be correct, yep.

Q. Yes, that would be a better way of putting it, observed evidence, yes. All right. I just want to ask you a few questions now about the page of your overheads, your power points that deal with what's called 'key findings'. I just want to clarify a couple of issues around that. There aren't pages on it so I can't give you specific guidance but it's the key findings page that has six bullet points on it beginning with 'Damage during the 4 September 2010 earthquake'. Now there are three bullet points in there I just want to take you to briefly. The first one says, "The columns and beam column joints should have been detailed for ductility," and I

just wanted to clarify the meaning of ‘should have been’. Is that a reference to should have been to comply with code?
A. That's correct.
Q. And then you also say, “The columns did not meet minimum requirements for shear,” and again, is that the minimum requirements of the code?
A. That's correct.
Q. Then finally you say, “Ties between the floor diaphragms and north wall had been retrofitted after construction,” this is the drag bar issue, “but only on levels 4, 5 and 6,” and I wondered if you had a view, one you feel able to give an informed opinion on, about the decision not to also install drag bars on levels 2 and 3?
A. I think the engineers who retrofitted those drag bars obviously came to the conclusion that the load demands, the requirements on the attachment of the north core to the slab at those levels, were at a level which meant that the current, the existing capacity of that connection was sufficient. I am, I guess, and it’s hard to separate out hindsight from what you would have done yourself but I think that the ability of the connection as it existed on all levels was more contrived than you might have otherwise expected so it needed a very definite attachment which it didn't have.
Q. What do you mean when you say more contrived?
A. Well it relied on relatively few starters coming out of the beam on line 5, is it? The one that runs –
Q. Line 4 I think.
A. – line 4 the one that runs through the tips of the north core. It relies on the connection there over a relatively short length of the slab between the first, the northernmost wall, sorry, the westernmost wall and the next wall in from the north core which is a relatively short length of slab in which to connect the wall in and I think this has already been talked about today, no definite means of transferring any tension loads arising from north-south swaying from the slab directly into the core, into those walls that are running north-south.

Q. Yes.

A. I mean certainly in modern construction, even with the current code and it's obvious perhaps deficiencies in terms of estimating the forces, the level of forces that you'd be required to design for would have required some very specific provision of drag steel from the core to the slab.

Q. On every floor?

A. On every floor, yes.

Q. Now just one final thing I want to ask you about while you're here, not directly dealt with in your evidence but taking advantage, I think, of your ability to comment on this. Would I be right that in your years as a structural engineer that you've had a substantial level of experience in training new engineers? Young engineers?

A. That'd be correct, yes.

Q. And that wouldn't necessarily mean new out of graduation but new to particular areas of structural engineering?

A. That is correct.

Q. Has that been only at Beca's or has that included other practices?

A. My entire career has been with Beca.

Q. Is there a general expectation and I suppose culture at Beca's about how mentoring and training is to be done and is required for people assuming new types of structural engineering work?

A. Yes, I believe so. It was instilled into me very early in my career and has been all the way through really the need for review of work that others are doing, across all levels and particularly those that are inexperienced in particular areas, there is a need for review of more senior people over those aspects, yes.

Q. So you wouldn't allow someone who had had no previous experience in designing a multi-level building to work without some close supervision?

A. Certainly within our organisation that'd be correct, yes.

Q. All right, thank you very much.

CROSS-EXAMINATION: MR ELLIOTT

Q. Mr Jury, first some questions about the demolition at the adjoining property. Just refer you to the section in the panel report dealing with that BUI.MAD249.0192.32, and that's on page 30 of the panel report, and I'll just read that out as it's coming up, so the heading is 'Demolition of neighbouring building,' it says, "The building next door to the CTV building began to be demolished almost immediately after the 4 September 2010 earthquake. Demolition continued until the week before the 22 February 2011 aftershock. Demolition work caused noticeable vibrations and shuddering in the CTV building which was a significant concern to the tenants. The view of the investigation team based on a general description of the demolition operation and photos of the demolition process was that the demolition would have been unlikely to have caused significant structural damage to the CTV building." Now that section of the report refers to the investigation team's view but was that also the view of the panel about that issue?

A. In terms of the panel I don't recall that ever being queried by anybody within the panel. It would certainly be my view but in terms of the whole panel I don't think it was ever discussed in great detail or even, or queried really. I personally agree with the comments made by Dr Hyland and particularly Mr Smith regarding the amount of energy involved in demolition and certainly it creates vibrations but not of a sufficient magnitude to really result in large amounts of damage.

Q. As I understand it part of the panel's function was to approve the consultant's report?

A. Yes.

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Q. Are you saying that that section of the consultant's report was not –

A. No, no that's -

Q. - approved by the panel in that way?

A. No, sorry if I gave you that impression. No what I was saying was that it certainly would be my impression that this clause was approved along with everything else but I was just saying it didn't come under a great deal of discussion.

Q. I see.
A. In particular.
Q. Did you hear the evidence this morning from Dr Hyland and Mr Smith –
A. Yes.
Q. – when asked about this point?
A. Yes, yep.
Q. And as I understand their evidence it was that they made the conclusions based upon their perception of common practice and did not include any consideration or quantification of actual energy release on the adjoining site or energy impact on the CTV building. Are you still content to endorse that conclusion without that type of further analysis?
A. Yes I am, yep.
Q. Secondly, I asked Dr Hyland and Mr Smith about their conclusions on the state of the CTV building post-September and post-Boxing Day. Your slide which Mr Mills referred you to, and if that could be brought back up please, BUI.MAD249.0503.9. You say there that one of the key findings was that damage during the 4 September 2010 and 24 September 2010 earthquakes was unlikely to be a significant contributor to the collapse on 22 February 2011. That's a little bit different to the way the consultant's report was phrased which talked about diminished capacity I think but, again, did the panel approve and endorse the consultant's findings that the effects of September and Boxing Day were such that the seismic capacity of the building was not significantly diminished.
A. Yes this, this was quite an important conclusion to come to and it did receive a reasonable amount of discussion with, within the expert panel. The expert panel concluded or really agreed with the investigating consultants in terms of diminished capacity but that was, that was on the basis of the, the evidence that they had collected, you know the photographs of the damage, the reports from the, from the other parties who had been in and inspected the building.

Q. Did the panel consider these issues as if they were discrete events or did they consider the possibility of cumulative impact of a number of events?

A. I think in the back of the minds in terms of the discussions as I recall them they were around the idea of whether the earlier events could have affected the result in the subsequent event, the February event. So that was on the minds of, of the panel, or the technical members of the panel certainly, the structural, technical people and I know that we, when we analysed the, the PGC building and also the Forsyth Barr building we did put the, we did run them consecutively. They were much easier analyses to run than necessarily this one but we did just run them just to see if it made any difference for those particular examples. I didn't perceive the need to do that for, for the CTV building based on the levels of damage that were recorded.

Q. So was it the panel's position that either as a result of the effect of individual earthquakes or as a result of cumulative effect of earthquakes in neither case there was no diminished capacity for the CTV building prior to 22 February?

A. That's correct, yeah.

Q. There's been discussion and I think there will be further discussion about whether the September earthquake could be classified as being a design level event or above or below or in the vicinity of design level. I was going to ask you one or two questions about that and it may assist if we refer you to the graphs in the panel report and I may not have the right reference for this, I don't, but it's on page 23, BUI.MAD249.0192.25. [Sorry Your Honour I think my document may have been superseded by a later reference]. Mr Jury I should ask would that graph assist in discussion about –

A. Yes it would.

Q. Good. Thank you. So this comes from a section –

JUSTICE COOPER:

So is that the number you read out or is it a different number?

MR ELLIOTT:

No Your Honour it's a different number so I'll –

JUSTICE COOPER:

So just read that into the record if you would.

MR ELLIOTT:

BUI.VAR00056.25.

JUSTICE COOPER:

Double zero I think.

MR ELLIOTT:

Double zero. Thank you.

CROSS-EXAMINATION CONTINUES: MR ELLIOTT

Q. Mr Jury these graphs appear in the section of the panel's report entitled, "Comparison with Design Levels," and is it right that the purpose of these graphs is to compare the shaking experienced in the September and February earthquakes with the design level of loading required under various codes?

A. That is correct, yes.

Q. Now on both of these graphs each of the codes has been represented by a line showing the design level of loading expected for particular buildings of different periods. Is that right?

A. That's correct.

Q. And then the earthquake forces experienced during the two earthquakes from each of the stations around town are plotted on each graph. Is that right?

A. That's correct .

Q. And then there's an area of grey shading in which, which is intended to encompass that range of shaking as recorded at the different stations.

A. It's intended to indicate diagrammatically what might be a reasonable range, yep.

Q. Just dealing with one preliminary issue, is there a reason why at a period of point 5 on the lower graph the shaking for a 2010/2500 year event is different to the upper graph?

A. I noticed that when you pointed out this diagram, when I first looked at it. I, I don't understand why there would be a difference. They should both be the same.

Q. They should be the same?

A. Yep.

Q. Do you know which is correct?

A. No off-hand I couldn't do that but it would be a relatively simple thing to confirm which one was correct.

Q. So just looking for you to assist us in, in this discussion of a design level earthquake. So I'm taking the top graph now, if that top one can be enlarged please. Thank you sir. So referring to the top graph which relates to the 4 September 2010 earthquake. The blue line represents the applicable loadings under the code relevant to the CTV design period. Is that right?

A. That is correct, yep.

Q. And the red line relates to the 2010 design for a one in 500 year event. Is that right?

A. That is correct.

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Q. Using that graph to assist us can you make any comment about whether or not, considering the CTV building, 4 September was a design level event or not?

A. I think in general terms the September event in terms of both its magnitude or extent of shaking as represented by the response spectrum and its duration would suggest that it was less than the design event. It might have been considered by some to be close to the design event but I think typically less than the design event and that is notwithstanding that some of the traces went above the line.

Q. I was going to ask about that. You are talking about a period of one, are you, for this building?

A. Yes well certainly not in the longer period, certainly not in the two second on where you do have a bump that is quite considerably above the design values but in the short period end less than one second. The intensity of shaking inquired by that spectrum is at or slightly low in general terms than the blue line if that is what you are asking me.

Q. So considering a period of one –

A. Of one second, yep.

Q. Do you say that the shaking at that period is at, above or below design level?

A. I would say generally below but it is close.

Q. And given that the grey shading extends both above and below that line, how do you explain the answer you have just given?

A. I think I have yet to see any records that would fill in the grey hazy line if you like in that one second area. I think that was just diagrammatically intended to convey the impression that this is generally where it might have been but it is not only the extent of shaking as measured by the spectral acceleration there are also the other aspects like the duration of shaking and other aspects. Other aspects might also be these were peak values what happened in the overall record. They all go to determining where the design was. In terms of actual spectral accelerations you could say it was on the blue line or the blue line represented pretty much what happened.

Q. Is it right that the 2010 one in 500 year event represents a level of loading not too different to the 1984 design level event at one second, at the period of one second?

A. In terms of design given the same levels that of activity, the same level provision for that, yes they would be very similar. I mean the red line is definitely above right in the very short period end but to all intents and purposes about the same.

Q. Can you explain what the meaning of this other higher line is, one in 2500 year event?

A. Well if the correct number is the one in the bottom graph that is probably the one we should be comparing but that is effectively a lower level of risk, shaking of a lower level of risk and in design terms that is used for the design of more important structures that in typical structures so the CTV building was a typical office building that would have been designed for the ultimate limit state at one in 500 but if it had been a hospital that was required immediately the event then it would have been designed for a one in 2500 year event at the ultimate limit state. So it represents a degree of shaking that has less, that is less likely than the one in 500.

Q. Has the panel produced a similar graph for the Boxing Day event?

A. I don't know whether the panel did but it was certainly, it was presented in our reports on the PGC and the Forsyth Barr stair, we produced the, we reproduced the Boxing Day event. It is a much lower amplitude when you look at the response spectrum event and very much focused towards the short period end than these other plots for either September or February events.

Q. I may be putting you on the spot here, but at a period of one second are you able to say whether the shaking in the Boxing Day aftershock was above, at, or below the 1984 design level?

A. Yeah I think I can go from memory and say that I would be almost certain it was below, quite a long way below at one second.

Q. Do you – again you may not be in a position to answer this, but are you aware of any earthquake between 4 September and 22 February that produced shaking at a period of one second at or near the 1984 design level?

A. No I don't think I can answer that, I don't think I have seen all the results from all those records. There were a number but my understanding was that Boxing Day was the more significant of all those aftershocks.

Q. Turning to the bottom graph which relates to the 22nd of February. Again considering a period of one second are you able to comment upon whether the shaking experienced at that period was at, above or below the one in 2500 year event level?

A. I think in general terms above. Certainly there are some structural periods where the response was less for some of the records but I think in general terms above the 2500 year return period shaking levels as represented by the spectral accelerations but once again this earthquake had less duration than one might have expected a design earthquake to have so it certainly met, or it well exceeded the design values or spectral acceleration but its duration was not as much as we might have expected a design earthquake to have.

Q. So how might that difference in duration effect the way a building is affected by shaking?

A. Well I think we feel that the effect of duration is affected by the level of ductility you might provide in it so it's an intangible thing in terms of design to allow for the duration of shaking but it's, we think, we believe that the buildings if they are redundant and resilient that they will be able to sustain high duration of this design level. So many cycles at or slightly less than the design value.

Q. Just turning back to your key findings. You have said that one of the findings was that columns and beam column joints should have been detailed for ductility and you have also said columns did not meet minimum requirements for shear in the CTV building. If you keep those in mind, and then one of the conclusions you state is the collapse occurred because shaking caused forces and displacements in a critical column or columns sufficient to cause failure. Firstly, by failure in that context do you mean failure such that pancaking resulted?

A. That's failure as we are implying it is here, we are saying loss in vertical load carrying capacity.

Q. Complete failure?

A. Yep.

Q. The consequence of which there can only be pancaking effect?

A. That is correct, yeah.

Q. Can you express a view on whether if the columns had been designed for ductility and/or the beam column joints had been designed for ductility whether the columns would still have failed in that complete way on the 22nd of February?

A. My own view would be that if both the columns and the beam column joints had had the degree of confinement that maybe they should have had by the codes of the day my feeling would be that this building would not have collapsed the way it did. I have reflected on, in terms of differentiating the column, beam column joints and the columns. Once again my own view would be that if the beam column joints had been confined and not the columns the building still would have collapsed. The other way round I'm not by any means certain. I think if the columns had been confined and the beam column joints hadn't, I think the building probably wouldn't have collapsed also. So the critical thing for me was the columns, confinement of the columns. That's not to say the beam column joints couldn't fail. Fail in terms of softening off, not being able to carry anymore lateral load, not providing the stiffness for the beam column joint et cetera. But I think that there was enough in terms of the large bars passing on the bottom steel into the joint region that wouldn't necessarily have led to every beam column joint losing its vertical load bearing capacity, and therefore leading to pancaking. That is a personal view.

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Q. Is that something which the panel considered?

A. I don't recall discussion on that.

JUSTICE COOPER:

Mr Rennie, one or two things have come up which hadn't come up previously. I will give you an opportunity to ask further questions should you wish to.

MR RENNIE:

I was particularly conscious of a couple of matters after lunch that I do want to ask our experts about Sir. I'm not in a position to ask a question immediately.

JUSTICE COOPER:

In whose questioning?

MR RENNIE:
Most particularly counsel assisting.

JUSTICE COOPER:
Well that is all we have had since lunch, was it Mr Mills or Mr Elliott?

MR RENNIE:
No I'm sorry, I was distinguishing Mr Elliott from Mr Mills in that regard.

JUSTICE COOPER:
Yes, and what was the distinction?

MR RENNIE:
I, when I used the expression "counsel assisting" I meant senior counsel.

JUSTICE COOPER:
Yes, yes I know Mr Rennie, but I'm asking you in whose questioning did the issue arise upon which you want further time?

MR RENNIE:
In lead counsel assisting.

JUSTICE COOPER:
Mr Mills'?

MR RENNIE:
Yes, yes. I'm not anticipating I need more than, you know, two or three minutes just to check a couple of things in my mind Sir.

JUSTICE COOPER:
Yes, well would you like to do that now?

MR RENNIE:

If that's convenient to you Sir?

JUSTICE COOPER:

It will be thank you. We will adjourn briefly whilst you do that.

HEARING ADJOURNS: 2.52 PM

HEARING RESUMES: 3.03 PM

MR RENNIE:

Thank you Sir, I'm obliged. The actual outcome of that is that the point that I have in mind for Mr Mills' questions, I'm now in the position that I don't need to proceed but I have had brought to my attention one matter relating to Mr Elliott's questioning, so I will deal with that.

JUSTICE COOPER:

Yes.

MR RENNIE:

There is one other aspect to it Sir, and that is that I would regard it as a virtual certainty that Mr Jury will not have read Professor Mander's second brief which came in yesterday and which of course contained some further developments of the graphs that Mr Elliott was asking about, and I don't see myself as being fair to the witness to spring that on him, and having got myself in that situation Sir, I am just going to have to do what I can with it, in terms of calling that witness in due course.

JUSTICE COOPER:

Yes well there may be a world of meaning and what you have just told me but you could always ask Mr Jury if he has read it.

FURTHER CROSS-EXAMINATION: MR RENNIE:

Q. Well I will ask Mr Jury, have you seen Mr Mander's brief of yesterday with the additional seismic material?
A. No.

JUSTICE COOPER:

Well there is nothing we can do about that.

FURTHER CROSS-EXAMINATION CONTINUES: MR RENNIE:

Q. Mr Jury, in answer to Mr Elliott there was discussion between you and him on the matter of the building pancaking which I take to be a direct vertical collapse as opposed to a building moving in a sideways direction, is that right?
A. I think it's, I think I had taken by his meaning it could be a bit of both, mainly vertical but would be associated with a bit of lateral drift.
Q. Yes the, a building which is swaying in an earthquake would normally be expected to fall outside its boundaries in the direction at which the collapse coincided with the sway, would you agree?
A. I would think so, yes.
Q. Distinctively in the case of this building, that virtually didn't happen did it?
A. Well as I understand it the building did collapse over Madras Street. It did extend to the edge of the pavement on Madras Street.
Q. Well it extended on the basis of the photographs and I can take you to those if needed, to the footpath and to some extent slightly encroaching on cars parked alongside. I think one of the witnesses said it was a metre but in fairness I don't think that was a realistic estimate. I think it was probably in excess of two metres looking at the photographs.
A. Yes, I –
Q. That's the order of degree, does that fit with your understanding?

A. That would fit with my understanding yeah.

Q. Yes. So the competing theories in this case as to the likely causation of the collapse, the column theory essentially involves a sway concept, would you agree?

A. I agree, I think I said in my evidence that requires the axial load and drift.

Q. Yes.

A. You don't get enough just out of axial load with no drift, you need both.

Q. But, so in that sense you would disagree with an alternative scenario of a pancaking variety which essentially relates to beam column joint failure?

A. Yes I have difficulty with that because the, I think that even the joint failure requires some lateral movement to dislodge the bottom bars of the, of the beams out of the joint. Up into, yeah.

Q. Yes, the extent to which these beams were engaged in this joint was a fairly short distance?

A. A fairly short distance but certainly into the middle region of the column in those central columns anyway.

Q. So in attempting to identify a preferred collapse scenario to achieve the ultimate location of the debris which is shown from the photographs almost entirely within the boundaries, we either have a pancaking scenario or we have a swaying scenario where the collapse point happened to match a sway which was more or less at the perpendicular at the time?

A. No I think we've got to bear in mind the context with the sway. It's only a relatively small dimension of sway that we might be talking about. Less than a hundred millimetres that we're talking about in terms of the drift required, so, so it's a relatively, you could say that's almost down on top.

Q. Yes, do I understand though that you don't discount a pancaking approach, your preference is for sway?

A. That's right. I mean I can conceive that failure of beam column joint could lead to the column being displaced and passed, and the floor passing down beside it or round it. I'm not sure that there's evidence to suggest that that happened in this case but.

Q. We're essentially engaged in what we call, could be called the "forensic pathology of buildings" in this case aren't we?

A. That's correct.

Q. Yes and to an extent that involves looking for clues and then applying weight to the judgment about that clue, and then aggregating that to form a preferred conclusion?

A. Absolutely.

Q. And different engineers may reach different views?

A. Absolutely.

RE-EXAMINATION: MR ALLAN – NIL

QUESTIONS FROM COMMISSIONER FENWICK:

Q. Yes, there were three issues I went through with Mr Smith and Dr Hyland and you will have heard the responses and you've already given me the response about the beam column joint theory, you say it's inappropriate. Now what I'd like you, if I've got your interpretation correct because you said if the columns were confined and the joint wasn't, you think it would have survived?

A. That's, that's a bit of surmising on my part, but yes that's the view I expressed, yes.

Q. Could you tell me why do you put stirrups or spirals in a column? For what reason do you do this?

A. You put, you put the, in the column you put the spirals in to confine the concrete to ensure that you at least maintain some semblance of the concrete to maintain the vertical load carrying capacity and to stop the bars from buckling.

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Q. Now if we go into the beam column joint why would you put spirals in the beam column joint?

A. For the same reason.

Q. Right, so you are going to stop the bars from buckling, why do the bars buckle in the columns but not in the beam column joint?

A. Well it does depend on the configuration of the joint but if you only have beams coming in from two directions you could certainly get column bar buckling in the joint.

Q. How many directions of beams come in the –

A. Into the interior –

Q. – columns?

A. In one direction.

Q. So that the bars and the joints could have been susceptible to buckling?

A. They could have been yes.

Q. What about the confinement of the concrete, could that have been susceptible as well?

A. Yes certainly.

Q. So if I am getting your interpretation right you put the reinforcement in the columns to confine the concrete and to stop the bars buckling. When you come to the joints zone you put the spirals in for exactly the same reason?

A. Absolutely.

Q. The only difference being that in the joint zone you have a very much higher shear as well don't you?

A. That is correct.

Q. So in fact this does not imply that perhaps for where you have got beams coming in two sides, would you not say, we've got the same conditions but you have got a very much higher shear. Wouldn't this now tend to indicate perhaps that the beam column joint is more critical than the column?

A. I think where my thinking is coming to is what would cause failure. So what would lead to vertical load carrying failure and I think that it is almost definitely the case in the column that if you had failure of the bars of failure of the concrete that you'd get collapse. I am not so certain in my mind that the failure in the joints necessarily leads to the same situation as quickly as the columns so that is all that I'm coming from. I would never suggest that you didn't confine the joints or the columns but...

Q. So when a column fails due to lack of confinement of bar buckling, what happens?

A. When the column fails due to lack of confinement?

Q. Lack of confinement and buckling of bars, usually the two go together?

A. Yeah, yep.

Q. So how does it actually fail?

A. Well I – well you'd lose the cover concrete, so you lose the area of concrete that you would normally expect to be carrying vertical load.

Q. So when you have a column which is 400 millimetres wide and the spiral is 300 millimetres wide, even though the spiral is at 250 centres it is almost the diameter of the spiral apart, you'd only lose the cover concrete?

A. No you could lose all the concrete. I think the bars, the column could explode quite dramatically I would think and I suspect the issue in terms of the beam column joints really was, I mean all the drift calculations that have been calculated in terms of what drift and what capacity that system had were based on an integral beam column joint I think so the issue then becomes if the beam column joint does have a shear failure, loses integrity in terms of being able to carry the moment through the joint what degree of restraint does that provide to the column and therefore how does it affect the drifts and the capacities to resist drift, I think that's certainly an issue.

Q. You don't think the beam column joint could lose its concrete just as we were discussing before?

A. Yes I think it could and maybe it did with the loss of those wings off the precast beams certainly.

Q. If it lost its concrete what would happen?

A. Well in my –

Q. Would it carry the reinforcing – would it carry the axial load on the remaining bars?

A. Well I think the – I would have expected if the columns had, if the beam column joints had failed and that led to the collapse I would have expected those bars that were running off the bottom of the beams to have been bent in at least some of those samples that were collected as it tried to bypass the column if the column was still in place.
Q. When you looked at all the pictures did you see any interior beam column joint that had any concrete in it?
A. No because I don't think we could recognise any of those interior columns from the debris.
Q. But you did see several columns with gaps in them where the beam columns –
A. Yes, mainly perimeter, I think mainly perimeter columns.
Q. So quite a few beam column joints entirely lost their concrete?
A. Yes I agree with that, I think it is difficult though to determine whether they were initiators or whether they were consequential failures of the whole lot coming down. I would expect that as a frame collapse with the level of confinement that this frame had right through the whole length, part of the column that you would expect it to come apart as you witnessed but it is very difficult to say what was the initiator.
Q. Yes. So let me try to summarise. I think what you are saying is that the concept I had that the beam column joint was weak and the column is not one that you would support because you –
A. Yep, yeah, no I find that quite reasonable.
Q. But you don't think that could lead to collapse because it has got to be in the column?
A. I think I would put the column failure higher than the beam column joint even though it had less strength but that's –
Q. But if it has less strength we have also got to acknowledge it couldn't develop a plastic hinge?
A. Couldn't develop, well –
Q. Because the plastic hinge would have been pushed into the joint zone, wouldn't it?

A. It would have been but that has to be counted against the evidence that showed at least some plastic hinging in some columns wherever they may have been.

Q. After it had failed –

A. And mainly the exterior frames, certainly not the interior frames.

Q. Yes, so if didn't have a plastic hinge, or had a very much reduced plastic hinge the deformation calculations which were based on someone's analytical programme which assumed that it was a plastic hinge length of 200 millimetres which is quite an arbitrary figure, wouldn't have existed would it, so you couldn't have actually got that amount of deformation out of the columns that has been assumed in the analysis, is that a correct assumption?

A. Yeah it is a slightly different calculation but I would propose that the hinge that was occurring immediately above the foundation was much less affected by the joint behaviour than the rest of the columns so failure in that ground floor which is still one of the scenarios, you know mid to low, height of the building, is still quite a possibility even if the joints had lost totally their lateral load carrying integrity.

Q. Sure. What would you say to between levels 2 and 3, one storey up?

A. Well I think that is another issue isn't, the effect of the free diaphragm or potentially free diaphragm when things start to happen in those areas may well have exasperated the drifts in those areas. Can't say for certain but it could have done as well but still with the column joint I would say if the column had lost its integrity the column joint had lost its integrity, yes the columns would have found difficulty to get enough drift to fail them.

Q. I really appreciate your comments. Look just a couple of other issues I just might quickly take you through I don't know if we need to pull up the – perhaps we will pull it up at any rate. BUI.MAD249.0493.1 I would just like to look at the south wall quickly and you followed the discussion on this, the tracking of the loads through the floor slab into the beams and back into the wall and we talked about whether the right-hand side, the two 24 millimetre high strength bars were effectively anchored into the wall or not. So you agree that was probably the general way in which the load was tracked into the wall?

A. I think that is the way it was intended to track, yeah.

Q. Is there anything else you want to comment about that discussion we had or agreement we had? I have got one more question after this but is there anything else you want to add to what we have already talked about the potential failure modes of the shear transfer into the beam and then the beam into the wall and the anchorage of the beams into the walls and the failure of the walls to behave as, actually as a coupled shear wall is intended to?

A. Yeah well the connection of the slab into the beam line on, get the numbers right, level 1, line 1.

Q. Level 2 I suspect –

A. Sorry line 1.

Q. – is the one you are referring to. I was hoping you would bring that one up –

A. Yeah, it's very light isn't it in terms of the way that's connected in. I totally concur with the comments regarding the development of these H24 bars into the walls. It's a common issue I think that engineers assume that by developing the bar into the wall they will get good connection.

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Q. Yes.

A. In this case I think the evidence probably was that there was ability to transfer some load from this but I, I agree it wouldn't be good practice to not have the drag bars running right through effectively.

Q. On, on level 2 there were stairs cut through. So a section of the slab adjacent to the south wall was removed. Would this, do you think, have had any effect on the shear transfer?

A. I think the, the response for that is yes, must have, must have had some effect. I think if, if, if the earthquake had been solely in the east-west direction maybe you would have got some shear transfer along the exterior beam line and the push would have helped but I, I think because the earthquake is multi-directional you can't rely on that. So any openings down that line would have to effect the ability to transfer, specially when the anchorage of the slab, steel into the beam line wasn't necessarily as we might do today.

Q. Yes. Well thank you now just quickly onto the north wall and the effect that the shear transfer is limited to that one bay where the toilets were. Is there anything you want to add to the discussion we've had there about the strength of the floor slab on the left-hand side of that beam and there in flexure and shear and possibly in direct tension due to the north-south simultaneous action.

A. The photographs of this north core wall after the event and the bit of slab that was left up in the air there would be exactly as I might have thought it might have looked if it had a tension failure.

Q. Yes.

A. Particularly at the end of the starters.

Q. Yep but that's, that's (inaudible 15:22:22).

A. But the calculations done sort of suggested that there might have been a reasonable amount of capacity there but I, we, we concentrated on this area quite a bit before we, we put it to one side and we put it to one side I think because the, we managed to convince ourselves that there was enough drift without it to, to lead to the scenarios that we presented but -

Q. Yes.

A. - I'm, I'm still, I still am doubtful that, that we would expect such a detail to work in every earthquake.

Q. So some tearing along that line would actually relieve the loading –

A. Yes.

Q. – and reduce the stiffness wouldn't it?

A. Yes.

Q. You say it may not have been necessary for that to trigger the failure but it might or might not have?

A. That would be my feeling, yeah, yes certainly.

Q. Thank you very much. It's great to have ideas bounce back. Really appreciate it. Thank you.

QUESTIONS FROM COMMISSIONER CARTER – NIL

QUESTIONS ARISING: MESSRS REID, RENNIE, MILLS AND ELLIOTT, ALLAN - NIL

WITNESS EXCUSED

MR MILLS CALLS

WILLIAM THOMAS HOLMES (SWORN)

Q. Now Mr Holmes I look here to see that your full name is William T Holmes and I should know what that T stands for but I don't.

A. Thomas.

Q. Thank you. And you are a senior consultant with Rutherford & Chekene a consulting engineering firm in San Francisco?

A. Yes.

Q. You've been in practice for over 45 years in all aspects of design structures but with a particular focus on protection from earthquake effects?

A. Yes.

Q. You have a BS in Civil Engineering from Stanford and also an MS in Structural Engineering from Stanford?

A. Yes.

Q. You are a registered civil and structural engineer in both California and Tennessee and most relevantly to the role that you're here for you have been appointed as the peer reviewer to the Royal Commission on these issues of structural engineering applicable to the CTV building.

A. Yes.

Q. Well as I understand it you've got PowerPoints that you're going to take us through and some additional ones that came in today.

A. Yes.

Q. And I'll get you to integrate those as you please to do. So I'll just leave you to go through that.

A. Okay my presentation today is pretty much following my written report from I believe April 30th is the date and that report included discussion of the organisation and clarity of the collapse report and I've specifically mentioned that there were many levels of discussion on different items including the executive summary, the main body and appendix and then supplementary reports. Many subjects were covered in all four of those places and sometimes it was very difficult to find which layer of information had the key information. So it was probably due to the reformatting of the report over the many months of development but, nevertheless, it made, it was somewhat difficult for the reader in my opinion but giving, having said that this report will really concentrate more on the conclusions rather than a lack of clarity although most of the discussion about the code is really a result of my trying to clarify for myself and perhaps for the Commission what I, what I think the code said regarding the ductility requirement for the gravity frames but it really, I'm not really saying anything different than what the report said. It's really more of an explanation.

So I will cover some code requirements in my opinion for the ductility of the gravity frame.

I'll discuss a little bit what I consider a column failure versus a collapse mechanism and some work that we're doing in the US right now on, on these issues.

And I propose yet another collapse mechanism which is not that different than many others that have been discussed in the last few days. It may have been a little different in April but it doesn't seem to be that much different now.

And I will also discuss what I think may have been the cause of excessive drifts. It's another, one man's opinion.

And then there are other somewhat controversial issues, the block walls, spandrel interaction, the elastic and non-linear analysis, vertical ground motions, and, and I will again point out that the February motions were exceptionally strong.

And then I'll talk about my own conclusions and recommendations which I included in the report which are similar to but not the same as the panel.

The code requirements in my opinion for ductility of the gravity frame, and of course I have never designed to New Zealand standards, all my experience is in the US, but I think I can read a code. So NZS 3101:1982 seem to be the controlling code in this regard. [Somehow the mouse is not going forward. All right now I click it, is that right?] The work that we're doing in the US right now is concerning a collapse of older concrete buildings and we're doing that work because we have many, many older concrete buildings and most of our standard evaluation methods will fail or they will fail to pass –

Q. (inaudible 15:30:18) page 20, is that what you intend?

A. I'm on page 20?

1530

Q. See, on the right-hand corner there.

A. Don't know how it got to be 20. I am –

Q. The (inaudible 15:30:35) really is where you want to be rather –

A. Yeah, I don't know how I got to, the mouse is not doing. I've skipped a whole bunch of things.

Q. Yes, that's what we thought.

JUSTICE COOPER:

Q. You actually had begun to address the code requirements.

A. Yes I know and I have several slides here and they don't seem to be...

Q. You need to get back to number 4 I think?

A. Yes I do.

Q. That's where you were and then there's another one –

A. Sorry, I don't know what happened to the mouse, jumped around.

Q. The mouse sometimes is difficult to control. Others have experienced that so it's not a special talent of yours.

A. Thank you for pointing that out. I must be trying to subconsciously get over this particular section or something.

MR MILLS:

I was going to say just, give that there's been a break and I should have said this before that the arrangement with Mr Holmes is that as far as this material on the code issues is concerned that if there are any questions arising from it, from any counsel, we will arrange for Mr Holmes to come back in again to the code section of the hearings so that he doesn't need to be examined on them now.

JUSTICE COOPER:

Is that as you understand it Mr Rennie? You're hearing that for the first time?

MR RENNIE:

I've just grasped it Sir but I entirely agree.

EXAMINATION CONTINUES: MR MILLS

A. Hopefully there will not be a lot of questions because I am not saying anything that wasn't in the report, I'm just trying to clarify it. So this is the sort of index of this code and there is a section that has been mentioned in passing before that says, "Principles and requirements for members not designed for seismic loading," and then there's another section that says, "Principles and requirements for structures subjected to seismic loading." Now when I read this I said well it's possible for an incorrect, I want to point out, it would be possible if someone said the gravity frame is not designed for seismic loading because only the walls are, to go into section 3.4 and design the gravity frame. I don't think that's correct but I was just raising the possibility that there may be some confusion. I would think an engineer designing such a building would recognise this but there is this potential confusion. I also want to point out that I believe that the spirals that have been much discussed in the columns pretty much exactly comply with section 3.4 rather than section 3.5 so it is possible that that section was used.

So if we go into section 3.5 you do find a section specifically concerned with secondary structural elements which I believe are the applicable provisions for the so-called gravity frame. I'm only calling it the gravity frame because that's its primary purpose. A gravity frame could very well have to have drift superimposed on it and so design. So if we go into this section you will see that there are several groups defined. They are not part of the primary seismic force resisting system but group 2 elements and group 2 elements have been discussed by several in testimony previously but not explained perhaps in this detail, so it says, "Elements of group 2 are those which are not detailed to separation of the structure and are therefore subjected t both inertial loadings as for group 1 and to loadings induced by deformation of the primary elements." So that's drift, so elements of group 2 are those elements that are not primarily part of the seismic force resisting system but have to go through deformations and that certainly would apply to the so-called gravity frames in this structure.

So if we then go to the requirements of group 2 elements we find the first condition says that if those frames, gravity frames can go through the maximum imposed deformations required by the code and stay elastic there are no requirements, no additional requirements that's condition 1, and I have to admit this little section 3.5.14.3 took me a long time to go through and figure out exactly what it meant but I think I understand it now.

Condition 2 says that if the frames stay elastic for only half of the maximum deformation required by the code then you have to use section 14 which is limited ductility requirements which are something less than full ductility so I wanted to clarify all this because there's been a lot of discussion about whether the frames had to be ductile or not and it's not black and white. It can be not ductile, limited ductile or fully ductile.

The last condition is that you don't stay elastic for half of the maximum deformations in which case condition 3 applies which the additional seismic requirements of other sections which is full ductility.

So in summary condition 1 says if the structure is elastic under the ultimate drifts there are no additional detailing requirements. Condition 2 says if they're elastic for 50% of the ultimate drifts you can use limited ductility. And condition 3 says if the structure is elastic for less than 50% of the ultimate drifts full ductility provision must be applied. That's my interpretation. As far as I can tell from discussions with others this is correct interpretation. Now it took me a long time to find in a report where there was tests of this and this was discussed in a testimony of Dr Hyland and Mr Smith. I have tried to simplify the table in the report from tables 13 and 14 in appendix F so it can be seen a little bit more easily and there was some discussion about column B and F in this table that this was a liberal interpretation trying to see if there was any way that the structure could be shown to not need ductile detailing. So these are the numbers from the report, these are not calculations I have made. So you can see that the column B is less than the full drift shown in column D and for column F2 the drift at least at the top levels shown in column F is less than the full drifts shown in column H, and again I say that this is, I just copied and pasted the tables from the report and cut out much of the information that was there that was confusing to me before. So you go further and there was some discussion about maybe the elastic deformation limit is not the appropriate, appropriately conservative enough and maybe some kind of approximate dependable capacity should be used so I've estimated what that was and then in column E and column I, I have divided the maximum deformation in half to see if it meets that test, so 46% does meet all of those tests so therefore it's in condition 2 in my opinion which would say that these frames at least based on these two indicator columns, and I suspect the other ones would be similar, would have, according to the code would need to be detailed for limited ductility, chapter 14. There was no indication to me that that was the case the way they were detailed.

So my conclusions at least based on that table solely literally at the upper four floors in the east-west oriented frames and the highest floor in north-south oriented frames were required to be detailed in accordance with chapter 14. I think in my opinion if an engineer found this based upon one indicator column they probably would have gone and checked many other columns and I think in the end they would have concluded that all of the gravity frames should be designed for limited ductility provisions.

1540

Now if they were, I looked at chapter 14 and I re – there are, there is some vagueness in that chapter, and since I was not practising in New Zealand in that decade I really don't know how exactly an engineer would've provided for limited ductility in accordance with those provisions, and I have not estimated what the drift capacity of the gravity frames would have been under those provisions.

Now the next thing I want to talk about a little bit is, I started, I jumped ahead before. We are doing work in the United States trying to understand how older concrete buildings collapse because we have figured out that the jurisdictions who control buildings will probably not develop requirements to mandatorily fix old concrete frames, because there's too many of them, they're too expensive to fix, the owners are powerful, there's a whole bunch of reasons why it probably won't happen, and in addition our evaluation requirements in the United States are very conservative, so almost all the buildings would fail and therefore have to be retrofit at a very high cost. The Federal Emergency Management Agency is sponsoring several projects to develop ways of putting all these older concrete buildings into different bins, or different groups of buildings that have a very high probability of collapse, and others that have a lower probability of collapse, or that perhaps will only be significantly damaged so that they life safety issues of the older concrete buildings could be addressed more easily by local jurisdictions without requiring extensive nonlinear time history analysis of every building, which is also a non-starter because it's too expensive from an engineering standpoint. So they would like a, FEMA would like a simpler evaluation technique, and that's what myself and others are working on, and in doing that you have to start thinking about collapse, specifically, and what controls collapse mostly of course is columns, so you start looking at what are the failure modes of an individual column and the first one you can talk about is a squash mode which is a short stocky column would simply just crush vertically. There have been tests of these kind of things. It's somewhat unusual in, in real buildings to ever find this.

There is a buckling mode where a tall slender column would buckle to the side. You would essentially get vertical movement under such a failure, perhaps a little bit of a side movement, but mostly vertical. Both of these modes would give you a vertical movement in failure which will become significant in a minute.

Then there's so called side sway mode where the column or most of the column stays relatively intact and the floor, upper floor will collapse to the side, rotating around the somewhat intact columns. There's two versions of that. One is what we engineers call strong beam weak column where you have a big fat beam at the top and the bottom which does not yield or form plastic hinges, which forces the plastic hinge into the column. That's on the left.

And then you have what we prefer, a strong column weak beam situation where the column is strong enough when it rotates at the end to form plastic hinges in the beams, and the beams are less likely to fall off or collapse in this circumstance, so the situation on the right is much better in terms of collapse than the situation on the left. Although the situation on the left, the side sway flexural yielding of a column in the collapse world is considered relatively benign. Of all the different collapse modes, the one on the lef – the one on the right we like, the one on the left is still not all that bad.

And the column shear failure is the last one which is the, in our opinion, the worst failure mode. You get a shear failure in the column and the column will collapse vertically along one of these diagonal lines and so this is really the bad actor.

So there's several tests that we like to look at in buildings. Number one is the column, what we call "shear critical". Can the movements that are formed at the top and the bottom under drift force a shear failure in a column? If that doesn't happen then we start looking, is it a strong bon – beam, a weak column or a strong column week beam collapse, and so on. The upper two modes of failure don't, do not come into play in seismic very often. So it's really the side sway mode and the column shear failure that we are concerned about. We have actually run many, many analysis of buildings with varying varieties of strong beam weak column, and strong column weak beam and varieties of shear strength in the column to try to understand at what point and what combination of these various parameters could we think a collapse is highly likely, in which case that is one step at least in giving the engineer a way to separate the really bad buildings from the other buildings.

So the report defines column failure by setting strain limits which has been discussed at some limit and to me this implies a strong beam weak column side sway mode of failure, and that, I have circled that and I think my interpretation of the report is that all of the so-called failure limits and triggers were talking about this particular failure mode.

HEARING ADJOURNS: 3.47 PM

HEARING RESUMES: 4.05 PM

JUSTICE COOPER:

Q. Yes Mr Holmes, thank you.

A. Okay. I had just gone over various column failures and I'd already discussed these projects that we're now looking at in the US concerning collapse mechanisms. They're referenced at the bottom. They're also referenced in my report. They're ongoing. There is no report on these yet because the studies are ongoing.
In my opinion the, the local exceedance of acceptable strain levels as used in a report may not be sufficient to cause loss of vertical load-carrying ability and ensuing collapse, particularly when independent lateral stability is provided from shear walls.

The site debris and eyewitness accounts as discussed many times suggests predominantly vertical collapse but the vertical collapse mode for columns that I discussed which would include squash, buckling or shear failure were not evident and if you do just a little bit of calculation on the columns they are not indicated that those are probable collapse modes.

So I was looking for yet another way that you could have this building come down almost vertically so quickly and I concluded after looking at lot at the drawings that it would possibly be likely that the beam column joints failure would do a lot of bad things besides just cause a lack of moment capacity at that location, because if the beam column joints sort of fell apart it's very likely that the beams would fall off their supports and I will comment about something that Mr Jury said. He said there were no bottom bars bent up that would be required in order to have that failure mode but if you had some plastic hinging at the top of the column in addition to the joint failure you have a little triangle at the top of the column and these beams, those hooks would have easily fallen, fallen down. So all it wants with a beam column joint falling apart you have two-storey instability from the columns and you have beams collapsing. So it seems to me that would be one way of explaining how a collapse mechanism that could occur almost vertically and, and very quickly, and a side-sway mechanism I don't, I don't think can explain that. It could be, it certainly is a failure of the column but is not a collapse mechanism.

Now there were some comments in the report concerning beam column joints. They've been discussed. There is one point where it says: "It's conceivable a lack of continuity steel through the beam column joint meant that the beams were unable to cope with much loss of vertical support as isolated columns were damaged and failed. Instead of being able to redistribute some of the load along the frames the beams may have pulled away from the columns contributing to the progression of collapse". And this is sort of a, of a progressive collapse issue. We structural engineers look at progressive collapse meaning that if you get one failure somewhere in a structure it could very well lead to collapse of the whole structure so the collapse would progress through the building. So there was some discussion of the importance of the beam column joint and then a later and another point of the report suggests that "the trends shown for demand capacity versus time in the beam column joints is similar to that exhibited by the hinge formation detail in section 10.1.3". So they're suggesting that the capacity of the joint is similar to the capacity of the columns as they defined it. But at that point they explain for a variety of reasonable reasons that they did not consider the performance of the joints in their models.

Now this, I'm going back to my report. Graham Frost has testified. I'm just going back to what I wrote back in April that Graham Frost had sent a short summary of his observations to Department of Building and Housing and I in the report quote his summary of what he reported in more, way more detail in his testimony but it certainly talked about his opinion that his observation showed "very brittle non-ductile failure of the beam column joints" and that "no beam column joints were found". He reiterated this in greater detail several days ago.

Other evidence of joint failure, and I put this in a report and there was perhaps an interpretation that I was suggesting this was the initiation of the whole collapse. That was not my intent. My intent was to indicate that perhaps there is one example of what seems to be an obvious joint failure in the building which could indicate that other joint failures could occur. This is somewhat unique. Columns on A, line A were somewhat unique compared to other columns in the building. They were small, square columns and I think they had extremely vulnerable beam column joints. This was an eyewitness that was working at the building and as the earthquake started to occur he had to jump from his machine and he says, "Just out of the corner of my eye I saw the concrete spit out the corner. The pillar came down and brought the machine down to the ground and buried the wheels," and by describing what he saw he said, "The column buckled out. It had cracked and two bits held still by the steel had spat out and obviously as the weight got too much it broke and came down," and he said, "This was in the middle of the column between floors. It kicked out in the direction of Les Mills."

Just highlighting a couple of things. The columns that were held together by two bits sound a lot like some cases in the building where the joints had come apart and there were two pieces of column that were held together. So I don't think that this particular case as he suggested was between floors. I think it was a two-level buckling or the joint had come apart and the slab which should hold everything together was very poorly connected to everything in that corner and you ended up with that corner having no attachment to the building and when the joint fell apart the column at that corner buckled outward. That's the way I would interpret this. Again, this particular column was unique in the building and I'll show it right here. This is a plan. This is line 1 and this is a very unique case on line A where the beam is simply a little sort of piece of wall and there's virtually, the corner of any building with poorly reinforced joints is the most susceptible because there is no confinement on two adjacent faces so this concrete under any compressive load will buckle out, will spall outward and this whole thing will move outward. So the joint kicked out and it, if you assume this case he would have seen these bars with no concrete around them with a relatively intact column above and a relatively intact column below. It, it meets his description almost to the T other than the fact that he thought this was in the middle of a column between floors. Now this is an example of, of a case where I think the joint failed.

Now why is it important that we try to figure out what actually collapsed in an area - well I think there's many reasons we want to identify the predominant vulnerability. There may be other buildings like this in New Zealand or around the world and we would like to know what really should we be looking at and unlike Mr Jury I'm, I'm saying I think if the columns had more confinements but the joints were the same I think the building probably still would have collapsed. On the other hand if the beam column joint was improved both to provide minimal confinement and to better tie the beams to the columns but the columns were the same I think the collapse may have been partial or localised particularly if lateral stability from the north tower was maintained which is a big if, but if all of those things would have been true I, I think the collapse would have been avoided or far less intense.

1615

A. For this predominant vulnerability I think is needed to find other vulnerable buildings in New Zealand and elsewhere.

Now what was the cause in my opinion of the excessive drift in the column. There has been endless discussions of whether the slab pulled away from the north tower. I find the reports rationalisation of why – one of the reasons why they think it didn't are quite compelling actually. They have done extensive calculations on what they observed was the failure surface which is the red line but they have found a far weaker tension failure surface, that where no failure was seen so if it was a pure tension failure of the building pulling away from the tower their reasoning is why didn't it fail along the weakest plane and I have to agree with that argument. I think it is very strong. I have a few comments about that later.

The other argument for no detachment from the tower is that the photo showing all of these slabs piled up would indicate that the centre of the building collapsed first and the slabs stayed intact – attached for some time and were not part of the collapse mode.

I think the configuration of the slabs in that photo I just showed could be explained in two other ways. The slab at level 3 which did not have any retrofits disconnects and leads to large drifts in the middle floors that initiate collapse. The slab at level 3 also collapses vertically but is arrested by the slab at level 2 and then as the collapse progresses the floors proceed to collapse ending in that configuration shown in figure 165. That is one scenario.

The other scenario is that the slab at level 3 or in fact partially at level 4 or 5, disconnects from a tension standpoint and causes increased drifts but does not completely lose its gravity support at the face of the tower and the large drifts cause the collapse away from the tower and eventually leading to this configuration at 165 so I don't think that that photo is definitive in terms of preventing a theory that some of the slabs detached.

Another argument for slab disconnection and I am surprised that there hasn’t been more discussion about this that the north tower was relatively undamaged. And I don't know how the drifts that occurred that were needed to so-call fail the columns could have occurred without detachment because that implies that the same drifts or similar drifts because there was torsion going on would have occurred in the tower and I am not going to go through every one of these columns but in E is what I think the maximum estimate drift that was consistent with the reported damage level of the tower was and of course it is a drift that each floor. The north-south failure drifts of the column as reported and it has been discussed in a testimony it may be as low as 1% but they were failure just calculated that I took from one of the tables of 1.58453020 significantly bigger than what, bigger than the damage level of the tower. The only way you can explain this is if the slabs, one or more slabs detached from the tower very early, caused a large drift, caused the joint and/or column failure. Otherwise these big drifts, even with a lot of torsion could not have been put into that tower without a lot of damage, a lot more damage.

So in the report I suggested the tower disconnected at the lower level early in the shaking initiating collapse before significant lateral load was transmitted to the tower. I think another acceptable or reasonable theory is that there was partial disconnection at the higher floors that allowed increased drift of the columns causing failure and then eventually ending up with the situation at the site and the photos as shown.

The other issues that seem to be discussed a lot: the concrete block wall in line A, it was clearly intended to be isolated but would have interacted at large drifts even if built perfectly. There has been some discussion about whether the detail could be built. The severe torsion created by significant early interaction would have put large demands on the north-south wall of the tower, that was mentioned in a report. It would have put large demands also on the columns but this torsion also would have put large demands on the north-south wall of the tower in the connections because any torsion in the diaphragm would also be putting a twist on that tower, so it would also put large demands on these weak connections but again that was not indicated by damage to the tower so it was also mentioned in a testimony yesterday that the concrete block wall when included in the models actually sometimes (inaudible 16:21:24) causing larger drifts and sometimes helped causing less drift, so in my opinion a concrete block wall is not a major player in, for the collapse.

The spandrel interaction. Subsequent to my writing the report there has been more evidence presented but in the report there was no systematic evidence in a report to support this interaction theory other than it was suggested that this interaction caused the, could have helped cause the failures on a perimeter of the building but calculations or other evidence really was poor. As I say there has been some discussion subsequent to that but under my particular favourite scenario of the joints it doesn't affect that collapse mechanism.

Other issues, the elastic response spectral analysis, was preformed for the code defined spectra and it was useful to check the original design. The purpose of that ERSA for using spectra from other shaking actually occurring in a CBD was unclear to me as several people have said. If you want to compare linear response you can just look at the spectra, you don't have to do an analysis. So since the structure seemed to be highly non-linear particularly in February this analysis I agree was not very useful other than for to check the original code design.

The non-linear time history analysis, the insights from such an analysis is normally very useful. In this case a much more complicated model probably in my opinion would have been required to reasonably predict the response in collapse including degrading column hinges which vary with vertical load. The more explicit modelling of joints, the failure modes in the diaphragms and the more realistic modelling of the connections to the north tower, or lack of connections and some amount of calibration between input predictions and actual response. The non-linear time histories other than trends did not calibrate with observed damage very well so you could learn certain things from them but the absolute numbers that were output did not calibrate well and I understand perhaps, well I know that more, such analysis is being pursued right now. At the time the cost and benefits of more complex models must have been weighed and either due to time or resources more advanced non-linear time history was not pursued.

The vertical ground motion was not directly considered in a linear behaviour from lateral loads it was done simultaneously and there was some post-processing which indicated a potential non-concurrence of maxima although those things are so random it probably would have been concurrence of maxima somewhere along the line. It is interesting the report concludes that the axial loads from the vertical ground motions could have reduced the drift capacity of the columns by up to 25% which is significant.

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So even with the analysis that was used, there is some indication of vertical ground motions could have had an effect on the columns.

And of course the exceptionally intense lateral shaking in February has been discussed over and over. In the end those intense motions clearly did something that did not happen in September and it would appear as if this building had a characteristic of extreme brittleness. It may have been nearly elastic in September and once it went past its elastic state it had several very brittle elements so anything over September or any, or certainly twice September which was what happened in February, caused this complete collapse.

My conclusions are that the exact set of deformations that instigated a collapse will probably never be known, even with more extensive modelling. Again, I wrote this several months ago before I knew there was going to be more extensive modelling, but I think my opinion is still the same. I think that whatever model anybody comes up with, no matter how sophisticated, there will always be things to debate and particularly the ground motions. The drifts at which the joints will degrade, the very complicated modelling of the joint I think particularly with the precast and cast in place combination, the strength and stiffness of the diaphragm I know was a big variable being considered but there still is unknown of what is the right one. The connection of the tower is very complicated to model, particularly on those levels where there was no retrofit done, and the extent of interaction of the block wall, the effect of vertical ground motions on the critical components and the potential concurrence or non-concurrence with maximum drifts.
So in the end I think like most folks, whether they are advocates of which of the four collapse scenarios, that my judgement in the case of the brittle gravity frames and the poor diaphragm connections were probably the most significant contributors to this collapse.

So what were lessons to be learned from this? I think that other people, including the Department, has suggested that we, that you have to look at other brittle gravity frames and in New Zealand I think that’s also true in the United States. We had I think particularly at this period we had probably lesser requirements than was in your code in terms of deformation compatibility of gravity frames and in passing I would note that if you use our most sophisticated analysis techniques in the United States, which would be contained in ASC41, the very small drift limits on gravity frames is pretty much what always controls our retrofits. We either have to increase the ductility of those columns or we have to put in massive shear walls to keep the drifts down to what we thing are now appropriate drifts. I have not evaluated the gravity system that would've resulted from the application of limited ductility but it would seem to me that New Zealand should certainly look at how that requirement was triggered, and if so what deformation limits it would provide. The configuration of the beam column joints in this building are primarily a result of the use of precast shell beams and starter beams. The use of precast in this area may also be a cause to require review of drawings of buildings that used precast in this way, particularly in the light of the suggestion by engineer Frost of the round columns and the wings which I'll discuss in a minute.

I think the diaphragm issues, we call the floors "diaphragms" in the US. They are a large part of our design, they have been for a long time. We always put in very specific collectors, even in the '80s or the '70s I think if you'd have seen the north tower there would've been substantial reinforcing sticking out of all those walls into the diaphragm. That's pretty much would've been standard procedure.

Q. When you say the "north tower" you're referring to shear core?

A. Yeah, the north tower shear core. There is the wing walls are coming out perpendicular to the building and we would've seen bars coming out of those walls into the slab to take the load back into the walls.

As far as lessons, I've seen other buildings of different eras in Christchurch that have in incom – that in my opinion have incomplete diaphragm designs or lack of collectors. There have been several people suggesting that the loadings of diaphragms may have been inadequate and are still inadequate. My bigger concern is how much attention engineers gave to diaphragms at all, forgetting about what the loading was. If the loading only determines how big the bar you put in there is, but I have seen several buildings of the '80s and '90s that indicate to me that for many engineers it must not have been a high priority, or it must not have been on their list of, of design issues. That's just my observation from several buildings. And also the adequacy of the design diaphragm forces should be reviewed. That's already been mentioned by others.

Interaction of non-structural walls is another issue and the potential precast spandrel beams. Both of these things are known deficiencies that engineers that evaluate buildings look for. If there are non-structural, particularly block or masonry walls that could prevent the frame from moving, that is certainly something to look for, and similarly short columns caused by either precast beams or some other infill between columns is also a known deficiency. Whether or not either of these things had anything to do with the collapse in this building is not the point. The point is that these are deficient – seismic deficiencies that certainly should be on a list of things to look for.

I also recommend reviewing current procedures for evaluating the adequacy of drift tolerance for gravity frames. If an engineer were given this building, how would they evaluate the adequacy, or how would they evaluate the drift capability of this particular gravity frames? What kind of modelling assumptions would they use that would lead to the drift demands they checked? And also I think the possible effects of vertical accelerations on brittle components is a big missing piece in the US codes and the New Zealand codes. Engineers really don't know exactly how to, how to deal with vertical accelerations, except for certain specific elements like cantilevers and very long spans. That's what is in our code.
Probably need to think about a multiplier on ULS drifts to establish evaluation for drift demands of these gravity frames. Since they are brittle it may be prudent to put a little extra safety on your evaluation by checking them for higher drifts than you might expect. Such a multiplier would be essentially setting the rarity of the ground motion for which

collapse should be prevented, and in new buildings this is easier to do 'cos it's not, it doesn't cost much. For retrofit or for checking existing buildings it's a very, very important parameter that has a lot of money associated with it and a number of buildings that need to be retrofitted would be definitely affected by what this number was. So I think it's a policy issue that should be established with communitywide input to some sort of acceptable risk. So the acceptability criteria also for the drift in these older concrete gravity frames need to be procedures for calculating such acceptable drift need to be developed.

That's the end of my presentation of the formal report. I do have some additional, a few slides that I've developed in the last couple of days. They're, shall I go into those now or do you want to stop here for a minute?

1635

Q. No, continue thanks.

A. Okay, I'm just calling this additional thoughts. I don't want to be redundant but I wanted to emphasise that the report discussion of the diaphragm tension failure planes shown believe in my opinion is a strong argument. That is on the upper floors the tension failure plane observed in the field is not the weakest and is therefore not likely. However, if there was some partial disconnection along one of these planes those two things could still be true, so complete, this argument is very good for a complete tension failure. It does not necessarily rule out a partial loosening which would have caused additional drift in the gravity frame.

Again, I think there's been very little discussion about the light damage observed in the north tower. I want to re-emphasise that. This is what really led me to looking for a failure mode other than proposed. There had to be some way that the load was not transferred to the tower otherwise there would have been more drift. So the question is could the 1 or the 1.5% storey drift which was affecting the column failure could have occurred in the north-south oriented walls of the tower without more damage. I don't think so, which leads me to believe there was some disconnection.

It’s not clear to me exactly how engineers at the time would have implemented the requirements of limited ductility that I think were indicated by the drifts so it’s difficult to speculate on whether such a structure would have survived or not, if it had had those requirements.

I note in my report that Graeme Frost initial letter to the Department of Building and Housing, and I used some of that information but his testimony in the last few days was far more detailed and convincing concerning the lack of intact joints observed, the smooth precast surfaces and lack of bond with the cast in place concrete and the potential failure plane of the precast wings under compression. Now he suggested that vertical acceleration would have caused this compression but drift induced bending would also cause increase in this compression as well.

So I've taken his sketch here which has been shown recently today and I just visually wanted to see what this would look like if under his suggestion so I then taken the drawing of the same joint from the building and I have simply blocked out the wings being gone. And now I think if any engineer came upon the situation in the lower right they would be very concerned and say this is not a very good joint and I'd probably do something about it. So you have to follow through a little bit visually I think to see what some of these theories mean so if the wings in fact had broken loose I think you would have a very dangerous condition. Those are my additional thoughts.

Q. The one further thing I have contemplated doing is there’s been a, if I can find it, there’s been a further brief came in from Mr Smith which commented specifically on your evidence. Have you had a chance to look at that?

A. I did and I tried to answer some of his issues as I went through –

Q. All right.
A. – I may have not answered all of them but I tried to point out certain lack of clarity perhaps in my report.
Q. All right, well look what I thought I would do, just to give you the opportunity to respond specifically to the points that he's raised to take you through it. I think there's on my count any rate about five or six paragraphs, maybe six or seven, in your actual brief which he's commented on and so I thought I'd take you to those and if you've got anything more you'd like to say in response I imagine the Commission would like to hear it now so I'll get them brought up and we can look at them and you can then tell me whether you've got anything more to say.
A. There are similar comments from Dr Hyland.
Q. Yes. Yes, so I thought we'd just deal with Mr Smith?
A. Okay.
Q. I think that covers them off well enough. So it's WIT.ASMITH.0004.3 is the first page of it. And the, as you know Mr Holmes, the first comment is on page 7 of your peer review report where he quotes from it and then he goes on to say, I think over in paragraph 12, that he can confirm, "The approximate column drift capacities from the non-linear analysis of the columns at grids F2," et cetera and then he goes on to describe what he's done and I think ultimately –
A. I looked at this and the way I interpreted his comments, and I may have misinterpreted them, but in Item 8 he is suggesting that the following quote is something I am saying?
Q. I think he is.
A. The fact of the matter is this is a quote from their report so it's not something I was saying. I was taking that quote out of the report and then commenting on it.
Q. Right.
A. So any comment that he was making on the content was their, he was commenting on their own report.
Q. And how did he do the second time round? All right, well then there's nothing that he's said about what you've done. So let's go to paragraph

19 see if he managed any better this time. He refers to the bottom of page 8 continuing onto page 9 of your report and then says at paragraph 20 that he disagrees with the conclusion?

A. I'm sorry, where are we now? On 20.

Q. If you go to 19?

A. Oh, 19, okay.

Q. He identifies the section from your report which at paragraph 20 he then says he disagrees with?

A. Mhm. Well that's a professional disagreement probably. I mean that has to do with side sway versus vertical collapse and the whole analysis using a side sway failure of the top and bottom of the columns I don't think describes what actually happened very well which is what led me to look for something else and the something else I came upon was the joint failure which not only would cause failure of the vertical load carrying capacity to the columns but would also likely let beams fall off the column supports causing for a very sudden and mostly vertical collapse. So that's my best guess of what happened and if Mr Smith believes that the side sway failure is the valid one then I guess it's a disagreement.

Q. Mmm, so I take it that there's nothing that he's said in paragraph 20 which I think is further attempt to set out the point of disagreement. There's nothing in there that raises anything you haven't thought about in the report you've already done?

A. I don't think so.

1645

Q. All right. Then the next one is paragraph 21 of his brief where he refers to page 9 of your report. And then you'll see down at paragraph 22 he says, "I accept William Holmes' comment that the potential failure of beam column joints was not given adequate emphasis in the Hyland Smith Report," accepts it's a further viable collapse scenario but then says at paragraph 23, "However, I do not agree," and he says that he believes the columns may have been critical, as explained in paragraph 20 above, comments on the time history analysis and goes on through at paragraph 27 restating I think what you're already aware of which is that he didn't consider it necessary to further investigate whether the columns or the beam column joints may have failed first but still maintaining his view that he thinks that it was the columns that went first. So nothing in there that's new to you?

A. No there isn't. I'm probably particularly sensitive to the difference between failure of a column or frame to resist lateral loads as opposed to resisting vertical loads because I was involved in a development of the predecessor to ASC41 and we had very, very small drift limitations for concrete frames because the researchers, mostly researchers who had helped us develop those rotational limits and drift limits had been testing lateral load-resisting frames all their careers and to them when you pushed on a frame sideways and it stopped taking load and started going horizontally that was a failure, and for a lateral load carrying element that's true. But many, many buildings that we were evaluating in California had shear walls. So they still drifted, so there still was a problem with the frames but they wouldn't just keep going horizontal because the shear wall was holding them up and keeping them stable. So what we were concerned about was the ability to carry vertical loads. So I talked to Dr Jack Moehle at the University of California maybe 15 years ago and he actually started a programme to test columns like this with a big vertical load on them and sure enough he found that the limits on columns that had been used widely before that, which were very small drift limits, maybe .3 to .5% drifts, these columns would continue to carry vertical load out to maybe 1 to 1½% drifts. So that led to a, a lot of studies by Dr Ken Elwood and others to, to develop data bases of columns that had been tested all over the world to try to come up with vertical load failure drifts rather than lateral load. So I just have this particular history that makes me sensitive to side sway collapses and the inability to resist lateral loads being different than the inability to resist vertical loads.

Q. Yes. Thank you for that. I think the next one that he's picked up is paragraph 28 of his brief where he refers to pages 13 to 15 of your brief and this is on the question of the floor diaphragm disconnection and you'll see that he then refers to the time history analysis that's been run down at paragraph 29 and says that the conclusion that you've reached is not consistent with what he had found from the non-linear time history analysis and says that in his, what they found there was that foundation rocking could be compatible with larger column drifts than those that you had calculated and, again, says, "My conclusion was that column failure could have occurred with or without diaphragm connection failure," and then comments on the fact that if there had been large storey drifts due to disconnection of lower floor diaphragms he would have expected to see more out-of-plane damage in the collapsed south wall. So anything knew in that?

A. Well it's my understanding that there was some investigation of potential rocking at the base of the north wall and there, there was no particular indication there was a lot of rocking, although it was hard to see. Large rocking of the tower in fact would have, would be an explanation of what he is suggesting. So that is an open question.

Q. Yes.

A. How, how much the, how much the tower rocked.

Q. All right and then finally, and this doesn't call for comment from you, he just says at the end of his brief that he doesn't disagree with your overall conclusion that, "Based on this review it is my judgement that the most important seismic deficiencies in this building were the brittle gravity frames and the poor diaphragm, particularly the connections to the north tower walls." So there's nothing more in that I need to take you to. The only other thing that I should have done before is just to confirm your actual brief of evidence as opposed to your PowerPoints and get that put formally into evidence and that brief of evidence is dated 30 April 2012.

A. Yes it is.

Q. And you've got that in front of you now do you?

A. I do.

Q. It's been signed I think.

A. It was signed, yes.

Q. And I'll just ask that that be put formally into evidence.

JUSTICE COOPER:

Thank you. Yes. Mr Reid.

MR REID:

Yes Sir. I don't have any questions at this stage of the witness.

JUSTICE COOPER:

You will in due course?

MR REID:

I will be in due course, yes.

JUSTICE COOPER:

Possibly.

MR REID:

Yes.

JUSTICE COOPER:

Mr Allan.

MR ALLAN:

I was rather hoping Sir you might call on one of the other counsel at this stage to enable me to, just to digest the import of this evidence and reflect on that.

JUSTICE COOPER ADDRESSES MR RENNIE

CROSS-EXAMINATION: MR RENNIE

Q. Mr Holmes I have two questions for you. You referred to Mr Frost's material. I'm not sure whether you were here when I was asking a day

ago about a post-building collapse report comment that Mr Frost provided in respect of three elements which he felt should have been discussed in the report in more detail. Were you here when that happened?
A. Well I did watch the testimony.
Q. Yes.
A. The full testimony.
Q. Yes.
A. So I must have seen that, yes.
Q. On the 20th of February 2012 Mr Frost wrote an email to Dr Hyland.
A. No that one I have actually not seen.
Q. That you have not seen.

MR RENNIE:
I now have to confess Sir that our best combined efforts of the three of us haven't yet found it in the transcript record but it, it may be a matter that we could find overnight if the witness is going to come back and if not it may still be -

JUSTICE COOPER:
Just a minute.

MR RENNIE:
I was going to say if not Sir it may be that we just forgo what Mr Holmes' review of that.

JUSTICE COOPER:
Well what is it you're looking for?

MR RENNIE:
I'm sorry it's just been found Sir. It's not on the website but it's on the Commission's system we've been advised Sir.

JUSTICE COOPER:

Right, so we can deal with it.

CROSS-EXAMINATION CONTINUES: MR RENNIE

Q. So MAD249.0494BB. It only has one page I think unless it's got the email on top and the page next to it. That's it. Could you just look at that Mr Holmes. This is Mr Frost discussing, as you will see, three matters. Weak beam column joints, strain hardening in the south wall shear wall and lack of confinement at beams ends. You've not seen this before?

A. I have – no.

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Q. No. Well it's probably unreasonable to expect you to comment on it immediately but if we can organise a copy of it for you and subject to the Commission's view it may be that you will have an opportunity to comment on it. The only other question that I wanted to ask you related to your, much earlier in your evidence your discussion about the buckling of columns, and you showed modes of buckling. They related to buckling on a single floor as depicted –

A. Yes.

Q. – by you. Was it your intention that they should be considered as confined to a floor, or is it the position that what you were referring to could apply to several levels of the building at the same time?

A. It could, it could apply to several levels of the building at the same time, presuming that the centre floor, let's say the joint disintegrated and you have almost no column or a pin occurring there, but the second requirement for that to occur is there be no lateral support for that middle floor because it would have to move sideways to have all of the columns buckle. In a shear wall building, unless there's detachment, it comes back to the detachment issue, that would not happen. I mean so theoretically it could happen. Certainly it could happen without shear walls because there would be no support other than the frame itself, so –

Q. Or post-detachment?

A. Yes.

Q. Are you aware of any research into multi-level collapse of that kind?

A. Well, when you run frames through many, many ground motions that has been done with something called incremental dynamic analysis, a procedure developed at Stanford University which is now widely used in the US for research, you find many, many failure modes, side sway instability modes, and I have a perfect slide to, that I was trying to get to show this group from that but my virtual private network in my office computer doesn't work the last couple of days so I haven't been able to get it, but the fact of the matter is some of the modes, if you put pins in the top and bottom of columns, or you put sometimes pins in the two beams on either side, either of those causes an instability and sometimes you end up with a two storey mech – what we call a mechanism. Less frequently you end up with one floor going in one direction and the other two floors going in the other direction which would be the, would be the two storied buckle. That is somewhat infrequent based upon the studies that I have seen.

Q. And in relation to this building in depicting a single storey mode, was it your intention to say that that is the mode we should consider, or should we consider both single level and multi-level modes?

A. I was showing that to differentiate what I thought were vertical column collapse modes versus side sway column collapse modes and considering the fact that the evidence seems to show a vertical collapse mode, you know I looked at the vertical collapse mode which would be shear and buckling and squash and I said none of these seem reasonable so at that point I was looking for another mechanism that would cause a vertical collapse and I conclude it was the joints.

JUSTICE COOPER:

So the issue based on that document that is displayed, would you like to pursue that in the meantime?

MR RENNIE:

No I am not seeking to cross-examine on it Sir but given the weight that this witness attached to Mr Frost's other comments it appeared to me that the commission might be assisted by this witness' comments on that document given that it was Mr Frost's considered view after the issue of the report but beyond providing the opportunity to comment it wasn't my intention to ask further questions.

JUSTICE COOPER:

Well I think as you said probably unfair to ask him to comment seeing it on the screen for the first time so that can wait to tomorrow. Now it is back to you Mr Rennie but you are not putting to Mr Holmes any of the evidence that your – any propositions that you wish to derive from the evidence of Dr Mander.

MR RENNIE:

That has not been my intention to do so Sir, no.

JUSTICE COOPER:

So I think the best course will be for us to adjourn to 9.30 tomorrow, you can ask that additional question Mr Rennie based on that material and then Mr Allan you I will turn to you and Mr Elliott you may have some matters.

MR HOLMES:

So I can get a copy of the document?

JUSTICE COOPER:

Yes that will be arranged Mr Holmes so that you can – somebody will arrange it.

HEARING ADJOURNS: 5.00 PM

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