**COMMISSION RESUMES ON THURSDAY 5 JULY 2012 AT 9.30 AM**

**COMMISSION RESUMES: 9.38 AM**

**BRIAN KEHOE (RE-SWORN)**

**CROSS-EXAMINATION CONTINUES: MR PALMER**

Q. Mr Kehoe, yesterday you finished up by telling us that you considered that there would be nil loss of capacity to the CTV building after the 4 September earthquake and the aftershocks that followed that before the inspection by Mr Coatsworth on the 29th of September, do you recall that?
A. Yeah
Q. How confident are you of that conclusion? Are you 100 percent confident?
A. I'm very confident of that conclusion based on the evidence that Mr Coatsworth has presented and the state of the building when he saw it.
Q. I take it from that you mean that the level of cracking and damage that he's depo – or given evidence about, would not cause alarm in your eyes?
A. That's correct.
Q. By the time Mr Coatsworth inspected the building on the 29th of September the building had undergone, in his words "a design event" and would you agree with him in that respect?
A. Based on my understanding of the ground motion from the 4th of September earthquake and the design code that was used at the time, I believe that statement is accurate that the level of ground motion was close to, or if not, at the level that would have been designed for.
Q. In respect of the CTV building?
A. Yes.
Q. Did you read the tenants' observations of the building in the period after the 4 September earthquake in their evidence?

A. Yes.

Q. Their evidence of changes in the building that they noticed, what they called generally as increased liveliness, the vibrations, some of the damage they observed, particularly to the internal finishings.

A. Yes I do.

Q. If the forces that were unleashed by the earthquake that caused that damage and those changes to the building caused no loss of capacity to the building what were they doing to the building?

A. I don't understand your question.

Q. Well the building was visibly affected by the 4 September earthquake and aftershocks, your evidence is that that would have caused no loss of capacity to the building but by all accounts, from eyewitness observers, there was damage to the building. If it wasn't causing a loss of capacity what was that, what was being caused?

A. Well what was being, what was the result of that earthquake was damage to non-structural elements which are not accounted for in the overall capacity of the structure so damage to non-structural components does not represent a structural loss of capacity of the building.

Q. Well those conclusions all relate to one design event causing, in your view, no loss of capacity. At what point would there be a loss of capacity? Would there be a loss of capacity after, for example, two events?

A. Not necessarily.

Q. What about five?

A. It would be difficult to say because every earthquake is different and the effects of each of those five earthquakes on the building would have to be studied independently to determine what the effects are so that you can't make any general assumption as to the number of design or equivalent design events and what effect they would have on a building.

Q. And to do that sort of assessment would you need to undertake calculations, look at the seismic records, perhaps undertake some invasive inspections to see effect those events were having on the building?

A. Well, after each of those events you would do the same type of triage procedure that's normally done post-earthquake assessments. You would look visually for evidence of changes in the building and if there are, is enough evidence that there has been substantial change or damage then yes you would probably want to do some invasive procedures or some analysis to determine what the effects are of those earthquakes, but given that there's, if there's no change in the condition even after five events there'll be no reason to do that.

Q. At what point do you undertake those more invasive structural assessments? How many design events do you need to go through?

A. Well whether it's a design event or not is irrelevant. It's whether the earthquake caused any change in the building, any substantial structural change in the building. Without any evidence of substantial structural change in the building you wouldn't need to go through any of those procedures.

Q. As a matter of logic though you could go through conceivably 100 such events and an inspection would not show any significant structural damage? If there's no significant structural damage shown after one event logically go through 100 such events if there's no structural damage observed then you wouldn't need to do any further structural assessment. Is that correct?

A. Yes.

Q. So, again, just an extension of that, you would, your evidence must be that for there to be a loss of capacity there must be a level of force exerted to the building which is in some way extraordinary such as the high accelerations experienced on the 22nd of February?

A. No, what I'm saying is that for there to be substantial structure damage there has to be evidence of that structural damage. Many buildings are designed or actually built and constructed with actual capacities that are far in excess of what they were designed for so a design event would not in those cases cause any structural damage so you have to look at what the real capacity and the real performance of the building and not in the theoretical capacity.

Q. And in the case of this building when Mr Coatsworth inspected of course he didn't find anything that alerted him to concern about structural damage?

A. That's correct.

Q. Have you read Dr Reay's evidence?

A. No, I have not.

Q. Could I ask that you do read a few passages of his second brief? We'll just bring it up. It's REA0002 at pages 10 and 11 and if you're able to bring up pages 10 and 11 together that would be helpful. Now I want you to read please paragraphs 45 to 50? 45 is just a context paragraph, 46 to 50 are what I want you to read and consider, you tell me when you're finished. This is about reinforcing strained hardening, the effects on steel.

**WITNESS READS PARAGRAPHS 45-50 OF DR REAY'S BRIEF OF EVIDENCE**

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Q. Are you familiar with the concept of strain hardening that Dr Reay is giving evidence about here?

A. Yes.

Q. Do you accept that it is a phenomenon that exists?

A. Yes.

Q. Would you also accept that if the phenomenon is operating and in fact the reinforcing steel in a building such as the CTV building has been affected by reinforcing strain hardening? Um, that internal damage to the structure could be caused which is simply not capable of observation, visual observation?

A. Well in order for strain hardening to occur there has to be a large amount of strain to occur in the reinforcing steel. For that strain to occur there necessarily has to be cracking in the concrete which would be visible, so in order for strain hardening to occur, there would be visible evidence of cracking.

Q. And I think the evidence from Mr Coatsworth was that there was visible cracking in the shear walls wasn't there?
A. Yes, but the level of cracking that he reported in the shear walls is nowhere near what would be required for strain hardening.
Q. There were certainly a lot of areas of the building that he didn't inspect weren't there?
A. I wouldn't characterise it as a lot of areas but there were areas that he did and did not inspect.
Q. Well, for example, approximately 50 internal beam column joints he didn't inspect, all those parts of the shear walls that were behind gib board or other internal furnishings?
A. Yes that's correct.
Q. So there may have been additional cracking in the building which was simply not capable of being observed by him in the context of the inspection that he undertook?
A. That's correct but the effect of large cracking that might occur would be visible elsewhere in the finishes and in other portions of the building so the fact that he didn't see cracking in some areas is more of an indication that any cracking that may have occurred was going to be small and not large cracking.
Q. And the fact that the internal finishes were indeed cracked significantly in some areas, and Mr Coatsworth did take a note of that, does that make no difference to your evidence in this respect?
A. Well I wouldn't characterise the cracking that he observed as being significant. There were noticeable cracks mostly hairline or greater. The type of cracking that would need to be visible for strain hardening to occur would be in the order of, would be much, much larger.
Q. Do you have any specific expertise in strain hardening?
A. Ah, no I don't.
Q. If I could just take you to your evidence at paragraph 4.13. Here you say that Mr Coatsworth did not undertake any numerical calculation of the expected seismic capacity, and you wouldn't expect them to be done. Now Mr Coatsworth concluded in his two reports that firstly on

the 6th of October the building had performed reasonably well and you'll be familiar with that conclusion?

A. Yes.

Q. And, secondly, on the 19th of October in his email that he considered the building was still structurally sound. You'll recall that?

A. Yes.

Q. In his evidence Mr Coatsworth expected that those phrases really meant that the capacity of the building is not significantly reduced and that is effectively Code safe in normal circumstances. Do you recall that evidence?

A. Yes.

Q. In circumstances where that sort of conclusion is being reached can you safely reach such a conclusion without undertaking numerical calculations of the expected seismic capacity? Can you do that?

A. Yes.

Q. Without taking into account seismic records and understanding the drawings and plans can you make that assessment?

A. Yes, it's done all the time.

Q. If that assessment is made would it normally be accompanied with a recommendation to undertake a more detailed structural assessment?

A. Not without the evidence of damage there would be no need to undergo that assessment.

Q. In paragraph 4.16 of your evidence you deal with this issue agreeing with his conclusions that the damage that he observed was minor and did not warrant structural repairs and then you say, "In those circumstances we do not believe that a structural analysis or that further detailed inspection of the CTV building was indicated." That word "indicated" what do you mean by that – do you mean warranted or required?

A. That's correct.

Q. In the reporting that Mr Coatsworth did, particularly his 6 October report which was his main one, he didn't provide any qualifications as to the nature of his report in terms of what it did and what it didn't do. It didn't, for example, state clearly that it wasn't a structural assessment. Would it be desirable for such qualifications to be included in a report like the one he did?

A. Ideally it would be desirable to have that unless there was some other method of that information being conveyed to the building owner or manager in this case.

Q. We don't know what level of qualification he gave to the building owners though do we?

A. No, we don't.

**CROSS-EXAMINATION: MR ZARIFEH**

Q. Mr Kehoe can I just start by following on from the point Mr Palmer was just making or asking you about and that's this issue of resilience of a building such as the CTV building and you said yesterday when we finished in answer to a question of whether you would expect cumulative fatigue or low cycle fatigue to have an effect on the reinforcement of such buildings, you mentioned research that suggested that that was not necessarily the case. I'm just wondering if you can explain a bit more and perhaps tell us a bit more about this research because to the layperson anyway it seems counter-intuitive that that would be the case?

A. Yeah I understand that that's counter-intuitive and when this, ah, research programme was first started that developed a theme of 306, 307, 308 documents, there was a lot of debate among the team that was putting this together as whether that would be the case or not which was the reason why we undertook this, one of the members of the team, undertook this analysis to go through numerical simulations of various buildings, various types of modelling of earthquakes and behaviour to determine whether an earthquake damaging a building would affect its ability to withstand another earthquake, because that was a very key point to this research and with all these numerical simulations what we found out was that it really doesn't have too much of an effect on the future performance. There is a little bit more variability but in general it's the same. In some cases buildings that were damaged actually performed better in the second earthquake because they were a little bit softer and the amount of forces generated in the building during the earthquake was actually less than during the first earthquake. In some cases it was a little bit worse, so there was no real evidence that damage in one event would always cause buildings to perform worse in a future earthquake but on average the results were about the same.

Q. And was that testing done on the basis of subsequent earthquakes being the same or less force or accelerations?

A. They were based on the future earthquakes being the same as the first one.

Q. Because the position in Christchurch was different of course because of the February earthquake, as you've pointed out, wasn't it?

A. That's correct.

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Q. And that's perhaps an unusual phenomenon?

A. Yes, very unusual phenomenon.

Q. So the CTV building if it had been subjected to a design level earthquake at or close to that level would you expect it to have a certain amount of resilience left even after such an earthquake?

A. Yes.

Q. And have you seen that in other reinforced concrete buildings, after similar earthquakes?

A. Yes.

Q. So Mr Coatsworth's inspection not observing damage that would indicate substantial serious structural damage would not be a surprise to you?

A. No, it would not be a surprise.

Q. Can you say that without looking at the drawings of the building though?

A. Absolutely, yes, because many buildings that we look at after earthquakes, we, are old buildings or its drawings don't exist or other buildings where we just don't have access to the drawings so in those cases there is no, there's no other choice such as Mr Coatsworth in this case he had to make a decision and a conclusion based on not reviewing the drawings.

Q. Right. I'll come back to that in a moment I just want to talk a bit more about this issue of the, the resilience, you said that you hadn't read Dr Reay's evidence, have you read any of the other evidence in this hearing that's been prepared and logged on to the website for this hearing?

A. Yes.

Q. What have you read can you…?

A. I've read Dr Priestley's evidence and Dr Mander's.

Q. All right. Just dealing with Dr Mander's, and this issue that we're dealing with about the resilience, you'll be aware that he says that because the CTV building had experienced a design level earthquake the September the 4th earthquake it would have been, there would have been significant damage that might not have been observable from a visual inspection?

A. Yes I recall reading that.

Q. And that therefore the inspecting engineer Mr Coatsworth should have, essentially should have red stickered it on that basis alone?

A. Yes.

Q. I take it you don't agree with that?

A. No I don't.

Q. Because of what you've said?

A. Yeah because what I said earlier is that buildings normally will have more strength than they were designed for so the fact that they experience a design level event doesn't necessarily mean that the, that event caused the, the level of damage which might be expected when it reaches its capacity.

Q. Dr Mander goes even further doesn't he? And says that in fact simply on the basis that there's been a design level earthquake for the, that building, that there doesn't even need to be an inspection, it should have been red stickered by the Council inspectors at the beginning without even the need for an inspection?

A. Yes, I –

Q. Presumably you wouldn't agree with that?

A. No, I do not agree with that.

Q. What do you think would follow generally if that was applied?

A. Well in order for that to be applied the people doing the level 1 inspection would need to know what the design level was for every building that they look at so you'd have to have that information, you'd have to know what the ground motion is at that site to be able to compare it to that design level to know whether at each location each building had experienced this design level event, it's not a practical solution at all.

Q. And presumably it would result in a lot of buildings being closed that might not need to be?

A. Absolutely, yes.

Q. Is that something that is applied in the States?

A. No.

Q. Have you ever heard of it being applied or promoted?

A. No I've never heard that position taken before.

Q. Is there anything else in Dr Mander's brief that you take issue with?

A. Well he spent some time talking about the effects of vertical acceleration on the building and I don't believe that, particularly in the 4 September earthquake, that vertical accelerations would have had any effect on the building because the, the amount of vertical acceleration was relatively small compared to the design level gravity forces that would have been, been able to be resisted by the structural elements.

Q. It was a different story in February though wasn't it?

A. Well yes and no, the, the vertical accelerations in February were quite a bit higher but I still don't believe that the vertical accelerations were, would have been a primary cause of the failure because the vertical accelerations are very rapid type of movements and whether they overcame the gravity load or, or the design gravity loads of the elements, I'm not convinced that it would have happened based on studies that I've done in the past.

Q. Right, what kind of studies?
A. I did analysis of several buildings for the effects of vertical acceleration and found that even with large vertical accelerations the effects on the structural elements is very minor compared to the design loads.
Q. Okay, but again to make conclusions about a building you'd have to look at how it was put together?
A. Yeah.
Q. You'd look at structural drawings?
A. Yes.
Q. Right. Anything else in relation to what you read from Dr Mander that you want to comment on?
A. I can't recall anything else right now.
Q. All right. And what else have you read of the material, you said that you've read the eye-witness accounts of how the building felt?
A. Yes.
Q. The Department of Building and Housing report?
A. Yes.
Q. And you mentioned Dr Priestley?
A. Yes.
Q. All right, from the material that you've read have you been able to form any view as to the collapse scenarios that are put forward?
A. Well we haven't done any numerical calculation to discount or reaffirm any of the scenarios that have been put forward. I do believe that the joints in the interior were probably susceptible to damage but what effect that had on the overall collapse, I, I can't answer that.
Q. You're talking about the beam column joints?
A. Yes the beam column joints.
Q. Yes. And what, because of the degree of reinforcement in there or not in there?
A. Yeah, the, the lack of reinforcement in the beam column joints.
Q. Okay, thank you. You said before that often a building will be designed to a greater level or greater strength, if that's the right word, than the Code required?

A. That's correct.
Q. And that's one of the reasons that there can be this resilience?
A. Yes.
Q. The difficulty though is that you won't know that unless you know how it is actually, how it was actually constructed, designed and constructed, the particular building correct?
A. Well yes and no, it, you could do an analysis to figure that out or you could look to see how the building actually performed in an earthquake. As I said the brief, you know, the real earthquake is a much better indication of what the actual strength than any numerical analysis.
Q. I understand that but the, the visual damage-based inspection that Mr Coatsworth did, and as I understand most engineers do following an earthquake, relies on the premise that serious or substantial structural damage will be visible on the outside of the linings?
A. Well not necessarily on the outside of the linings but it will be obviously visible in the structural elements and many times that will also show up in the linings.
Q. All right, sorry, and I was going to add or visible structural elements, but that's the premise isn't it?
A. Yes.
Q. And I think in your brief you say that normally it would?
A. Yes.
Q. So are there circumstances where it might not?
A. Not in my experience I haven't, and I've looked at hundreds of buildings following earthquakes and I can't recall anywhere I've seen that there was structural damage that was not readily apparent from visual observation of the finishes or from the accessible structural elements.

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Q. You heard me, I think you were in the hearing yesterday, you heard me put to Mr Coatsworth the evidence of Professor Priestley?
A. Yes.
Q. And his evidence or his opinion that the mesh could well have become fractured in the September earthquake?

A. Yes.

Q. If that had happened, and I accept that there's probably no way of knowing that now, but if that had happened his evidence is that that could have been caused by the crack of as little as 2 millimetres wide? That's something that might not be detected if one didn't lift floor coverings for example?

A. Well if, if the crack and the fracture of the mesh would happen at a displacement of 0.2 millimetres what would happen after that is there's nothing to restrain that crack from opening up further so it may start at .2 but it would not end up at .2, it'd be much larger than that because there's nothing to restrain that crack anymore. So, yes, you could get a fracture at that little of the displacement but subsequently in the rest of the earthquake that crack would open up wider than that so, and that would be visible.

Q. But what if the earthquake was enough to fracture the mesh but the other elements that secured the floors to the shear-core, thinking about the drag bars that were put in and there's also some steel bars to the west end of the north core, they could have held the wall and the end result of the September earthquake say was just this fractured mesh, which might account for liveliness in the floor, but until the February earthquake didn't collapse?

A. Well if the mesh fractured and there was other structural elements that were able to hold things together then those would be sufficient to prevent the liveliness from that crack causing liveliness of the floor or anything else so the secondary path that would restrain it should be sufficient to prevent any noticeable damage.

Q. Right, but do you accept that that could happen? That scenario?

A. It could happen that the mesh fractured and the crack was restrained by other elements. I don't accept the fact that that would have caused a noticeable change in the liveliness of the floor.

Q. Right. But if that did happen do you accept that –

**JUSTICE COOPER:**

But if what did happen?

**CROSS-EXAMINATION CONTINUES: MR ZARIFEH**

Q. If the mesh had fractured but the other elements had restrained the floors do you accept that, given what Professor Priestley's saying, that that may well not have been observable if one didn't conduct an invasive inspection?

A. Well a crack of that size probably, may not have shown up through the vinyl covering on the floor in the lift lobby area.

Q. Right.

A. So it may not be, may not have been as observable but if there were any larger crack probably would have been observable.

Q. And do you agree with me that if one, if an engineer inspecting the CTV building had access to the drawings there would have been quite a lot of information to be learnt from those? Have you seen the drawings?

A. Yes, I've seen the drawings.

**COMMISSIONER FENWICK:**

Excuse me; can I just clarify one point? I think there might be some confusion between the two of you. The mesh, we found cracks at 2 to 3 millimetres, not .2 to .3 millimetres in width. It goes at a strain of about 2 per cent on 150 millimetres, so you were commenting on .2 while I think Mark Zarifeh, I think, was referring to mesh at 2 to 3 millimetre width for the mesh to crack. Excuse me for interrupting this; want to make sure that point is clear.

**MR ZARIFEH:**

Thank you, no, I think that's right.

**CROSS-EXAMINATION CONTINUES: MR ZARIFEH**

Q. I'd said 2 millimetres, I'm referring to Professor Priestley's evidence at paragraph 80, he says, "Crack widths of only 2 millimetres are required to induce mesh fracture." Did you misunderstand me when I said 2?

A. No, I didn't.

Q. Thank you, so I was saying; if an engineer had access to the CTV structural drawings a lot of information could have been gathered from those in relation to how the building was constructed and whether there was what degree of reinforcing for example was there?

A. Yes.

Q. And perhaps an example of that was you'll recall Mr Coatsworth yesterday saying that he assumed that the beam column joints were constructed in a standard fashion with shear reinforcing carrying on through the joints?

A. Yes.

Q. So an engineer would have found from looking at the drawings that that wasn't necessarily the case, correct?

A. That's correct.

Q. Looking at the drawings one would have seen that there were, there was a problem with the, as designed, with the connections from the, of the floors to the, floor slabs to the north core?

A. Yes.

Q. So do you agree with me that this building anyway is a good illustration of why it's a good idea for an engineer to, who is doing a post-earthquake inspection, to have access to structural drawings?

A. Well, yes, I would agree that certainly in this case having access to the structural drawings would have been enlightening, whether it would have changed Mr Coatsworth's opinion following his review of the building's visual observations in October I, I don't believe they necessarily would have changed his overall conclusions. It may have raised some issues with him and he may have decided to, you know, request some additional work but I don't believe that reviewing the drawings would have changed any of those conclusions.

Q. Well can you really say that? I know it's hypothetical because he didn't have access to them but if you had been inspecting that building and you'd got the structural drawings and you'd seen things like that, for a start, wouldn't it have made you much more cautious about the building and wanting to do a more in-depth inspection?

A. Yes I probably would have, having reviewed the drawings, probably wanted to look a lot more carefully at the connection between the floor slabs and the north core.

Q. And want to lift floor coverings and linings, wouldn't you?

A. Maybe look at more interior beam column joins if, in those areas, yes.

Q. Right, so it would be for a start a more invasive inspection?

A. Probably not at first an invasive inspection, it would still be relatively not invasive it was just lifting some floor tiles or some ceiling tiles to gain access to those areas.

Q. Right, but when I say invasive I mean lifting floor coverings as being perhaps not that invasive but as a start it would involve that kind of thing?

A. Yes.

Q. In the United States do you have any requirement that drawings of such buildings are held electronically?

A. No.

Q. Is there any move towards that?

A. Not that I'm aware of.

Q. Do you agree with me that that would be a good idea if it was possible?

A. Yes, I agree.

Q. And if that was the case do you think that in an inspection such as this it would become the norm to get to, to have access to drawings?

A. Yes, in fact what's happening in the United States is a, ah, some cities are implementing a procedure where the owner of the building will hire an engineer in advance of the earthquake and have – that engineer would have access to the drawings, review the design and be on-call after an earthquake so that's the engineer who is already familiar with the design of the building would be able to go out to that building know where to look for the critical elements and be able to assess the building in a much more knowledgeable fashion than a standard building inspector following an earthquake and that engineer would then be allowed to placard the building with the same authority as the local jurisdiction.

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Q. So is that just something that is becoming common or is there a move to make it a requirement?

A. It's common, it's voluntary at this point in a number of large cities such as San Francisco but I think there is a lot of building owners who see the advantage of that.

**JUSTICE COOPER:**

Q. Where else is it happening, are all the cities where this is happening in California?

A. That is all that I am aware of is in California.

**CROSS-EXAMINATION CONTINUES: MR ZARIFEH**

Q. I just wanted to ask you a bit about training and compare the USA and New Zealand to see if we can learn anything from your knowledge of that. We have had an example in this hearing of – and in other hearings of a lack of training for people conducting post-earthquake rapid assessments, building inspectors but even suggestions that engineers, perhaps it is through lack of opportunity and opportunity for carrying them out or for being trained, don't have the required expertise and that is something that has come out pretty clearly. Is that the case in the States or not?

A. Oh, in the United States there's a lot of jurisdictions where they have implemented training, as I mentioned in the brief Robert Bruce in our office has done hundreds of trainings for ATC-20 across the United States in a lot of jurisdictions. I have done some training and other people have too so it is very, very wildly implemented within the United States at this point.

Q. And, you mentioned in your brief that there are no guidelines in New Zealand as to what to look for particularly for different buildings, different types of construction?

A. I don't think I said that specifically.

Q. I thought you said that there are no – there don't appear to be any guidelines, no written guidelines as to what to look for, or are there to your knowledge?

A. Well other than the guidelines for territorial authorities I don't believe there is anything else and I don't – and in looking at that document I don't recall seeing specifics in there about what to look for, for different types of buildings.

Q. And in the United States, you mentioned this ATC-20.

**JUSTICE COOPER:**

You may have had in mind his paragraph 3.23 Mr Zarifeh.

**CROSS-EXAMINATION CONTINUES: MR ZARIFEH**

Q. Yes it was 3.23. And is that what you meant that there is nothing in writing of any detail to provide guidance?

A. Yes.

Q. And the ATC-20, have a manual in what is called a field manual, you are familiar with that?

A. Yes.

Q. And that appears to me as a lay person anyway, to set out in some detail not only the process, the rapid assessment process, but also what to look for in relation to different types of buildings?

A. That is correct.

Q. Is that used as a matter of course?

A. Yes.

Q. And do you think that from what you have seen that that is something that could be adopted, or something similar could be adopted in New Zealand?

A. Yes I would think so.

Q. And perhaps should be?

A. Yes I would think so.

Q. One of the other things that has been highlighted in this, in the hearings that we've had and in particular in relation to unreinforced masonry buildings is that there can be a Level 1 rapid inspection, so external visual inspection, that results in a green placard because there is no apparent damage or no significant damage and that that can be the last or the only assessment of a building, talking about post-September that occurs and then in February the building has collapsed. Now in terms of the green placard level 1 and the equivalent in the States being the last thing or the only thing that happens is that the situation that occurs frequently in the States?

A. I wouldn't say it occurs frequently but it can occur when the level 1 or equivalent is done from the outside only which is why it is incumbent on the owner of the building if it has got a green placard and the owner of the building goes inside and sees if there is something amiss that the owner can either request a more detailed inspection or hire an engineer to do independent assessment.

Q. In New Zealand with the Level 1 as you will be aware there is a recommendation for the owner to get their own engineer assessment?

A. Yes.

Q. On the placard – is that the case in the States?

A. It is – it's not required but it is suggested that an engineer, that the owner does hire an engineer, yes.

Q. And is there anything other than that to encourage owners to do that?

A. Not that I am aware of.

Q. So have there been any problems, thinking litigation or anything like that, as a result of a building being green placarded or the equivalent in, the falling down subsequently and there being any comeback to the inspector?

A. I am not aware of any case where that has occurred.

Q. Do you not see that a potential problem?

A. Yes it is a potential problem.

Q. And have you got any ideas from your experience as to what can be done about that?

A. Well, like I said, I think the impetus is on the building owner if they have a green placard and they see something in the building that concerns them then they are the one that needs to be responsible for getting either the building department or an engineer to come make another assessment of the building.

Q. What about the standard of the inspectors then, there seems to be more training from what you have said of building inspectors and the like and engineers as well. Is there a certain standard required before someone can be involved in inspecting buildings?

A. Yes, anybody that does the inspections is supposed to go through training and have a, in a lot of cases, a card or certificate that they have gone through the training. What usually happens is that the inspections are done in teams of at least two people and the requirement is usually at least one of them needs to have been trained and be – in some cases they are required to be a licensed engineer but at least have the training.

Q. And so presumably that gives, in terms of the system, more confidence in the actual inspection that is carried out?

A. Yes.

Q. What about liability for inspectors, is there an indemnity provided?

A. Yes if they, an engineer is volunteering to be an inspector for a city then they are deputised by the city and they are provided with indemnification.

Q. So just going back to any possible solutions to this problem of level 1 being the last say. Do you think that if there was a different test, something or perhaps a requirement for level 2, perhaps with access and structural plans that that might overcome any potential problems such as the ones that we've seen in Christchurch?

A. I would think that at least a level 2 would be desirable because there is many times where the damage may not be visible on the outside but once you go inside the building and outside the building you are normally able to find any potential serious hazards so I think as a minimum interior and exterior access would be very desirable.

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Q. For all buildings?

A. For all buildings, yes.

Q. And when you say a level 2, in the States that's called a detailed evaluation isn't it?

A. Yes.

Q. If the level 2 was, if there was a level 2 done in the States is there a requirement that one, at least one of the team be an engineer?

A. I believe in most jurisdictions that is the requirement.

Q. If that was the case then do you think that having a requirement that there be access to structural plans unless there are none would also be a good idea?

A. Well I think that for the level 1 or level 2 the team would spend more time looking at the drawings than they would actually looking at the building and that would slow down the process substantially and in many cases they may not learn enough from looking at the drawings so I would tend to discourage the implication that you have to look at drawings for the level 2.

Q. But do you agree with me that essentially what Mr Coatsworth was doing in this, with the CTV, was a level 2?

A. Well it's a level 2 with, as we said, it's some additional work that he did that would not be normally done in a level 2.

Q. In terms of mapping of cracks, et cetera? A level 2 plus but it's not the next step is it?

A. Right.

Q. And certainly with a building like CTV as we've discussing, having access to the plans would give you information that would be relevant and important?

A. Yes.

Q. So if that, this is just one building as an example, if it could be done there should be access to plans?

A. I think for an engineer hired by the owner if plans are available I think that should be encouraged and like I said before that's not always possible so I don't think, I wouldn't say that it should be a requirement.

Q. All right, the FEMA 306, my understanding is that's a procedure that's followed from research after the ATC-20 guidelines were established?
A. Well –
Q. Is that correct?
A. In the timeline, yes, it was, it was a document or series of documents that were produced after ATC-20 had already existed.
Q. And what, is the FEMA 306 guidelines, are they applied by engineers doing post-earthquake inspections?
A. Yes.
Q. They appear from what I've seen of them to require assessment of structural drawings?
A. Yes. You do need structural drawings in order to apply that.
Q. So Mr Coatsworth's inspection was not one that would be along the lines of the FEMA 306?
A. That's correct.
Q. The FEMA 306 is that the next step? What we would call or what's called in the States an engineering, is it engineering evaluation?
A. Well an engineering evaluation is a wide range of what could be done from what Mr Coatsworth did to FEMA 306 and I think there's a lot in-between that could also qualify as an engineering evaluation. It's really left up to the professional judgement of the engineer to look at that continuum of what could be done to decide what's appropriate for the building, and FEMA 306 is one tool in that range of evaluation techniques that could be done.
Q. And again are you aware of anything similar in New Zealand?
A. No I'm not aware of anything similar.
Q. Are you aware of engineers in New Zealand applying or using the FEMA 306?
A. I'm not aware of that having been utilised in New Zealand. I wouldn't be surprised if it had because some of the research that was the basis for FEMA 306 came from New Zealand.
Q. Sorry, I'm just looking; I don't want to repeat anything Mr Palmer's already put to you. Just finally, this issue of the language used by

engineers and what's understood by the public. Mr Coatsworth, you'll recall, thought that that could perhaps be improved in terms of the communication; I take it you agree with that?

A. Yes I do. Engineers are not always very good about translating engineering thoughts and ideas to the layperson in a way that the layperson can understand. It's something that's struggled with everywhere to make sure that the engineers are communicating in a way that the public's going to understand.

Q. Right, because in this case the owner and occupiers obviously reliant on the inspection that had been done, were wanting to know is it okay, is it safe for us to be in here, and effectively that's what Mr Coatsworth's report was concluding although it wasn't in those terms, do you agree with that?

A. Well I'd say that the impression given by Mr Coatsworth was that the building was safe to occupy and I wouldn't disagree with that and I think the impression that the people had that the building was safe to occupy was probably correct. It was as safe as any other building following that earthquake for an earthquake of equivalent magnitude and there was no indication in that statement that if the next earthquake is bigger that there may be a problem.

Q. And there's also perhaps a need as illustrated by this case for there to be clearer communication of what's involved in an inspection so the difference between structural analysis and structural assessment?

A. Yeah, it appears to be a need for clarification of that also.

Q. Well so that the owner for a start knows what's involved and the occupiers as well, or the tenants. This is what's being done, this is the level it's being done to, it could go higher but it hasn't, so that there's an acceptance of whatever risk is involved?

A. Yes.

Q. An informed acceptance?

A. That's correct, yes.

Q. Yes, thank you.

**JUSTICE COOPER:**

Q. In New Zealand over the last 40 or so years as a result of various Court decisions there has come to be quite a reliance by members of the public on actions taken by Councils when they consent building and liability has gradually been expanded in those areas for Councils that issue building consents when there were a reason they shouldn't have done so. Is there a similar set of liabilities that arise in the United States?
A. No, the way it works in the United States is that the city or other authority that grants a consent or in the United States they're called a building permit is exempt from liability. They're only responsible for a general review. They're not responsible for ensuring that the building met the design requirements.

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Q. One consequence of the New Zealand situation is, I think, probably compared to the United States, given your answer, much greater reliance here on what the Council has done so an inspection system that results in a green sticker is likely probably to be more relied on in New Zealand than an equivalent sticker issued in the United States because New Zealanders look to Council to be responsible for such things. Would you care to comment on that?
A. Well, not being that familiar with New Zealand practice I would say you're probably correct but if there's this reliance on the Council providing the consent and taking that responsibility then a green sticker issued by the Council would probably have a lot more weight to it after an earthquake than the equivalent in the United States.
Q. All right, well it's something worth thinking about.

**CROSS-EXAMINATION: MR ELLIOTT**

Q. Mr Kehoe, the families of those who died are interested in identifying where possible that lessons are learned from this whole thing and for that purpose identifying where people may have done something different or better. You were asked to address whether the assessment undertaken by Mr Coatsworth was appropriate and I think your evidence says that it was. Is that right?

A. That's correct.

Q. And you were also asked to consider whether Mr Coatsworth should have undertaken a seismic analysis of the 'as built' condition of the building and you say no to that. Is that right?

A. That's correct.

Q. And you were asked whether based on his assessment his conclusions and recommendations were properly made and your answer is yes. Is that right?

A. That's correct.

Q. You're familiar with the Code of Conduct that you refer to in your brief of evidence are you?

A. Yes.

Q. And in particular your obligations to the Royal Commission. You're aware of those obligations?

A. Yes.

Q. You're aware that you have an overriding duty to assist the Court impartially on relevant matters within your area of expertise?

A. Yes.

Q. And that you're not to be an advocate for Mr Coatsworth. You understand that?

A. Yes.

Q. Is it your position that you and Mr Paret and Mr Poulson would have done exactly what Mr Coatsworth did in assessing and reporting on this building?

A. Essentially, yes, we would have done the exact same thing following an earthquake. I've done this hundreds of times after earthquakes and would have followed, based on my experience, would have followed the same procedure essentially that Mr Coatsworth did.

Q. And did each of the three authors of your brief agree on every aspect of it?

A. Yes.

Q. And so there is nothing at all that you think Mr Coatsworth could have done differently or better?

A. Well I think there are some things that could have been done differently, um, I wouldn't necessarily say it was better but I think maybe he could have looked at a different area of the building than what he did, um, that would have been different but I don't know that he would have come to any different conclusion.

Q. What different area of the building do you think he could have looked at?

A. I don't know. He said he looked at one, opened up one ceiling tile in one of the upper floors. If he would have looked at a different ceiling tile that would have been a different location, I'm not saying he would have come to any different conclusion but he could have looked at more places in the upper floors.

Q. At the beam column connections in particular do you mean?

A. Yes.

Q. Is that all?

A. I can't think of any other thing that I would have done differently at this point.

Q. As I've mentioned, you said that there was no reason in your view for Mr Coatsworth to carry out a seismic analysis of the building and that an assessment of diminished capacity was appropriate. That's right isn't it?

A. Yes.

Q. Just assuming for a moment that's correct, and we'll come back to that briefly in a minute, but just considering how he might have carried out that assessment of diminished capacity you've already commented to some extent. Do you agree that that assessment wasn't part of any triage or rapid assessment was it?

A. Well, yes, it is. It's still part, there's still a triage aspect of that. You're looking for obvious signs of damage to decide whether you need to look further so it may or may not be the final step so it is part of a triage process.

Q. Mr Coatsworth agreed that his assessment included consideration of whether the building should continue to be occupied during ongoing aftershocks, you appreciate that don't you?
A. Yes.
Q. And if, in the course of considering that question, you were just confining yourself to an assessment of diminished capacity focusing on damage, you would want to know as much as possible about the presence and extent of possible damage wouldn't you?
A. Yes.
Q. The FEMA document that you've referred to, the purpose of the project leading to that document was to develop technically sound procedures to evaluate the effects of earthquake damage on buildings with primary lateral force resisting systems consisting of concrete or masonry bearing walls or frames wasn't it?
A. That's correct.
Q. And in that document it says, "Original architectural and structural construction drawings are central to an effective and efficient evaluation of damage" doesn't it?
A. Yes.
Q. So Mr Coatsworth should have made it clear to Mr Drew that the drawings should be provided to him as soon as they're available shouldn't he?
A. No, as I said before there's many cases where drawings are not available and you can still do an effective damage evaluation without access to the drawings and in this case in the absence of evidence of structural damage the need for looking at the drawings was not there.
Q. The evidence is that the drawings or the file from the Council became available on the 6th of October. Are you saying that Mr Coatsworth shouldn't have tried again with the Council at the time his report was delivered or afterwards or asked Mr Drew to continue initiating attempts to get the drawings?
A. If he had seen more substantial structural damage then, yes, he probably should have asked again for access to the drawings but, given

the limited damage that he saw, there's no evidence of substantial structural damage and no need to really look at the structural drawing.

Q. So this really means examination of the damage and diagnosis of the meaning of the damage is critical?

A. Well I'd say the examination of the damage is critical. The identification of damage is critical. The meaning of the damage is sometimes important and sometimes it's not.

Q. You made the point that in deciding not to pursue enquiries for the drawings Mr Coatsworth would have needed to have considered the damage to the building. Is that right?

A. Yes.

Q. And so he would have needed to know how much damage there was wouldn't he?

A. Yes.

Q. And he also would have needed to consider what the damage that he saw meant?

A. Yes.

Q. If an earthquake had been severe enough it could have potentially deformed the reinforcing steel within the columns of the CTV building. Is that right?

A. Well, yes, if the earthquake was large enough it would have deformed the reinforcing, yes.

Q. And the bolts on the drag bars and the connections at the north core could have potentially been fractured by a large earthquake couldn't they?

A. Yes.

Q. Mr Coatsworth didn't see those because he didn't get into the liftwell did he?

A. That's correct. He didn't see that but he did look for evidence of damage in those areas to the non-structural element.

Q. Cracks may have been developed within concrete elements of the CTV building at the time he looked at the building couldn't they?

A. I'm not understanding what you're talking about.

Q. Well there could have been cracks inside the concrete when he did his inspection?
A. Are you talking about cracks within the concrete that are not expressed on the surface?
Q. Well are there cracks within the concrete that are not expressed on the surface?
A. Not if they're related to damage.
Q. So are you saying that cracks within the concrete are always evident on the surface?
A. Yes.
Q. And does one then have to simply diagnose the meaning of the crack based on what they see?
A. Yes.

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Q. Isn't a person in that position likely to want more information about what the cracks might look like inside the element as opposed to just relying on what they assume from what they see?
A. It depends on whether the cause of that cracking is readily apparent without knowing what happens inside.
Q. As I mentioned to Mr Coatsworth yesterday a section in FEMA provides guidelines for the use of typical tests and inspections doesn't it, to assess the consequences of earthquake damage. Is that right?
A. That's correct.
Q. And that section 3.8 includes a number of methods for assessing an earthquake damaged building only one of which is visual inspection but then there are others, is that right?
A. That's correct.
Q. Now this was a major commercial building in which Mr Coatsworth did not have the structural drawings so have you considered whether these other methods outlined there would have been of some assistance to Mr Coatsworth?
A. Well the first step in any type of earthquake damage assessment is to use visual observation and those other techniques are usually only

employed when there is need to do so and that's why they're presented there. In most cases those other techniques are usually not required.

Q. So looking to the future should an engineer carrying out that type of assessment be equipped with that sort of equipment or not?

A. I would say that it is inappropriate to equip engineers with all of that equipment most engineers don't have the training to use some of that equipment it's very specialised and the time required to, to use that is usually not warranted until there is some evidence from the visual observations that there is some reason for doing further investigation in a given area.

Q. And dealing now with the question of seismic analysis which you say was not something which Mr Coatsworth needed to do. The seismic analysis that you were meaning there is considering the capacity of a building to withstand a future hypothetical earthquake or earthquakes. Is that right?

A. Yes.

Q. Now in your evidence you say that in this case the absence of visible physical damage, I'm sorry, the absence of visible physical evidence of some diminution of capacity would be all the more significant if the intensity of earthquake shaking that prompted the assessment was near that of a design level event and you also say in the absence of evidence of physical damage the building can be assumed to be capable of withstanding another earthquake of equivalent force to the earthquake that resulted in the inspecting taking place. So isn't, is it inherent in those comments that this assessment of diminished capacity which Mr Coatsworth carried out in effect included consideration of a hypothetical earthquake, namely, a design level event?

A. Well it included the consideration that the building should be able to withstand an earthquake of equivalent intensity to what it just experienced and if there was knowledge obviously that that was a design level event then, yes, the answer would be, yeah, it can withstand a design level event again.

Q. So in that sense he has formed a view about the capacity of the building to withstand a particular event namely a design level event?
A. Yes.
Q. And he's not turned his mind to the possible performance of the building in an event above design level?
A. That's correct.
Q. This assessment of seismic capacity in a hypothetical event; Mr Coatsworth gave some evidence yesterday that it would have cost, I think $25,000, he said to carry out. The assessment of seismic capacity can take place in a number of different ways can't it, ranging from quite straight forward to more complex, would that be right?
A. That's correct.
Q. And at the more straight forward end of the spectrum we have I think the, it is the initial evaluation procedure which I think you've referred to in your evidence. Is that right?
A. Yes.
Q. And that's a procedure which an engineer can undertake which can result in an assessment of the extent to which the building complies with new building standard, is that right?
A. That's my understanding yes.
Q. And that assessment can be carried out in hours rather than days would that be right?
A. That's correct.
Q. And at the other end of the spectrum and more expensive end of the spectrum complex, computer modelling. Is that right?
A. Yes.
Q. The FEMA 306 document refers to the use of seismic capacity assessment in assessing earthquake damaged buildings doesn't it, is that the underlying principle?
A. Yes.
Q. And what it contemplates is using the earthquake damage to consider the way the building performed, is that right?
A. Yes.

Q. And your capacity spectrum method that you referred to in one of your papers is a tool which can be used for that purpose as well, is that right?
A. That's correct.
Q. So does one work out theoretically how the building would have performed in a particular earthquake and then compare it to how it did perform in that same earthquake?
A. That's the intent of the FEMA procedure, is you want to, your analysis model and prediction of what it would have done should be, should be close to what it actually experienced and if there's a difference then obviously the anal – there's something wrong probably in the analysis but yeah, the idea would be that you want to perform an analysis of the building and have the prediction of what that analysis would say would be the result should be close to what the building actually experienced in the earthquake.
Q. Can you then use that information to determine at what capacity level the building might fail?
A. Yes, if you have the information to do that type of prediction you should be able to come close to predicting at what level the building might fail.
Q. So a building owner or tenants could be told well in this particular building at this particular level of earthquake we could expect the building may well fail?
A. Yes.
Q. There's no particular reason why Mr Coatsworth couldn't have said to Mr Drew and/or the owners, "I can do more than just look at diminished capacity, I could actually work out the capacity of this building in a future earthquake", is there?
A. Well I think the answer to that is that obviously he could but he would be remiss in telling the owner "I can spent $25,000 of your money to tell you that what the capacity of the building is" when it's a relatively modern building which anybody might expect to have capacity equal to, equal to or greater than the design level event. There may not be a perceived need to, to do that analysis.

Q. But it's evident there's a continuum that an engineer has available to him or her in assessing a building ranging from very brief external visual assessments right through to a computer based capacity assessment is that right?

A. Yes.

Q. And that what Mr Coatsworth has done is to draw a line somewhere along that continuum and say to the owner, "That is what I propose to do", correct?

A. Well he not only did that he also said in his proposal if there is damage then he would, then he would see the need of doing the analysis, doing an analysis but what he said is, "In the absence of damage I don't think it's necessary to do this second step".

Q. There is more he could have done; he just made the decision not to do it?

A. And that's consistent with standard practice is, you don't go into a building that's been damaged and offer to do a lot of analysis for a building that has little or no damage.

Q. But isn't the problem here that it was he who decided that and not the owner and the tenants?

A. Well he proposed that and the owner accepted it so the owner if the owner wanted to have more analysis or thought more analysis was needed the owner could have requested him doing so or if the owner questioned, "Well why aren't you doing the analysis?" he could have explained why.

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Q. You don't think that engineers in this position generally should explain to the layperson I can do all these things to determine whether your building is acceptable to occupy, and we'll discuss what they are, and you will decide where the line is rather than me?

A. That certainly can be done and in a lot of cases it is done. My personal experience is that when given a range of options most owners choose the one that's going to cost less at the beginning and will then only authorise additional expense when the engineer explains to them that

it's needed. Most owners don't opt for a lot of additional analysis without any justification from the engineer.
Q. I see, that is when given the range of options, is that right?
A. That's correct.
Q. Thank you.

**RE-EXAMINATION: MS BRYANT**

Q. Just one small clarification Mr Kehoe. You may recall earlier when you were discussing the evidence of Professor Priestley a question was raised whether or not you were talking about a crack of .2 millimetres or 2 millimetres, do you recall that discussion?
A. Yes.
Q. Would a crack of 2 millimetre width refract through the vinyl lining?
A. I would say that 2 millimetres may or may not reflect through vinyl lining, it would depend on a lot of circumstances. It's possible it could or it could just show up in separations between the vinyl tiles.
Q. Thank you.

**QUESTIONS FROM COMMISSIONER FENWICK – NIL**

**QUESTIONS FROM COMMISSIONER CARTER:**

Q. Mr Kehoe, I understand that you visited New Zealand twice on behalf of US agencies, once after the 4th of September and again after the 22nd of February events, is that correct?
A. That's correct.
Q. And you had the opportunity I think you said to listen in to some briefings on one or more of those occasions, is that right?
A. That's correct.
Q. I'm interested in the way the practice of post-earthquake inspections are carried out here in comparison with what might have been used to in the United States. We have been told that there is a document that produced by the New Zealand Society of Earthquake Engineers which is

the guideline which has got wide practice in application in New Zealand. Do you see that that's generally modelled on the ATC-20 document?

A. I think the general concept is very much similar to the ATC-20 document, yes.

Q. Thank you. There's two aspects of it that I'd just like to hear your views on. One is the possibility of alerting engineers to what scale of aftershock that they should be considering when they report on their examinations Is it common practice to identify the level of aftershock that might be considered by the inspecting engineers?

A. Yes, it's commonly assumed that there will be aftershocks and they would normally be up to one order of magnitude less than the main earthquake.

Q. So that's the practice in the United States at the moment?

A. Yeah, and that's in the ATC-20 document it says that.

Q. And we have heard that that's what our inspecting teams were told. Did you hear anybody tell our teams that that was what they were looking for?

A. No I didn't hear that specifically.

Q. Thank you. There's one other aspect and that was reported in your evidence in 3.5 where you noted that the NZSEE guidelines talk about the colours of tags and what they mean and you use the words that green (safe to occupy), yellow (restricted use), red (unsafe) is the way you described it here. Are those words used in the American practice, USA practice?

A. Red (unsafe) is the same, yellow has two meanings, either restricted use or limited entry and green there was, there's been some change in the terminology for green, whether it's safe to occupy was the original language but now the common language is just that it's been inspected.

Q. Thank you so does that indicate there's been some cause for concern over interpretation of what a green sticker conveys to the public in the United States?

A. Yes.

Q. Thank you.

**QUESTIONS FROM JUSTICE COOPER - NIL**

**WITNESS EXCUSED**

**MR PALMER CALLS**

**DANIEL LEE MORRIS (SWORN)**

Q. Mr Morris, is your full name Daniel Lee Morris?

A. It is.

Q. Do you reside in Christchurch?

A. Yes.

Q. And are you a company director and do you run a concrete grinding and polishing business?

A. Yes I do.

Q. Do you have with you your statement of evidence that has been provided to the Commission?

A. I do.

Q. Could you please read that evidence starting at paragraph 2, and I'm going to ask you a question at the end of paragraph 3?

A. Okay.

**WITNESS READS BRIEF OF EVIDENCE AT PARAGRAPH 2**

A. "I previously owned and operated a concrete cutting business which traded as Knock Out Concrete Cutters. I started the business in about '91 and sold it in 2000. For a period of around five years in the 1990s Knock Out was involved in drilling a number of holes in concrete beams and floors in the CTV building. Of course in those days the building was not known as the CTV building. Knock Out was contracted out to carry out the drilling for the purpose of installing cabling, plumbing and other purposes. The work was done on a number of occasions for different tenancies and across a number of floors of the building. The holes will have ranged in size from 40 millimetres to 100 millimetres. We were not often contracted to drill holes smaller than this. Our equipment was needed for these larger holes which is why we were brought in."

Q. If I could just ask you a question there. Are you able to say which members of your firm did the work?

A. No.

Q. And you know about the work then because of your role as manager of it, is that correct?

A. Yes.

Q. Could you continue at paragraph 4 please?

**WITNESS CONTINUES READING BRIEF OF EVIDENCE AT PARAGRAPH 4**

A. "At the CTV building and many others sometimes when we were drilling such holes we would hit the reinforcing in the concrete beams. We would stop work and speak to the head contractor about shifting the hole so as to avoid the reinforcing. Sometimes it was possible to shift the marked hole, a marked out hole but sometimes it was not practical and the head contractor would just tell us to cut right through the reinforcing which we did. It would take is two to three hours to cut these holes through reinforcing. All of our work was done specifically as directed by head contractors on an 'all care no responsibility' basis. These contractors included plumbers and electricians. We always spent a reasonable amount of time in the roof space because that is where the beams are therefore it was generally easy to see the beams and the holes cut in them by contractors including ourselves. The holes we drilled were not necessarily in a line but placed depending on the requirements. Many holes were cut in concrete beams. I could not say exactly how many but the drilling was extensive in the CTV building and there were, they were all over the place. My best estimate is that we would have drilled about 200 holes in the CTV building, about 50 of those would have been through the beams. I do not know how many holes were drilled by other contractors before and after our work there. I recall pricing a job at the CTV building a short time before I sold the Knock Out business. We did not get the job on that occasion but whoever did would have drilled more holes."

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Q. Could I just ask you a question there as well, you have referred to a figure of approximately 200 holes. How do you know that, where do you get that evidence from?

A. Well it was a long time ago and it is a bit of a guesstimate there really, but we were called to that building a number of times and 10, 20 holes a time would be the normal.

Q. So that is your best guess assessment?

A. Yes.

Q. And then you've referred to at least, oh, sorry, about 50 of those having been through beams. How do you make that assessment?

A. Once again it is just by knowing the general way things work, um, out of say 200 holes I would have thought 50 would go through the breams, mainly they are through the floors but then they extend their electric work and that out through the floors from you floor openings.

Q. Thank you, continue at paragraph 7 please.

"Cutting holes in concrete beams in this manner was not unique to the CTV building and still occurs around town today. Often it included cutting through reinforcing, around one-third of all holes would require some cutting of reinforcing. The reinforcing cut in such drilling included high tensile reinforcing. This would require longer time and involve greater damage to our tools and therefore cost to contractors we worked for. We used to charge a triple rate for excessive steel holes. Collapse of the CTV building. When I heard about the collapse of the CTV building I immediately thought about all the holes that had been drilled in the beams. I mentioned this to others which led to someone from the Department of Building and Housing, DBH, contacting me to discuss what had happened. I spoke to a couple of people by telephone from or on behalf of the DBH in around May or June 2011. I do not recall their names. I told them about the size and number of holes that were drilled. I was not asked for an interview and I do not know what came of the information I passed on."

**CROSS-EXAMINATION: MR REID – NIL**

**CROSS-EXAMINATION: MS BRYANT - NIL**

**CROSS-EXAMINATION: MR ZARIFEH**

Q. Mr Morris, you appreciate I imagine that it is important for the Royal Commission to get accurate information and as much as detail as possible?
A. Sure.
Q. So I want to take you through your brief and some of the paragraphs and see if we can get any more clarification of what you say. Firstly you say that you operated Knock Out Concrete between '91 and 2000, so is that the years that you are talking about the drilling of these holes?
A. No I believe it was probably from '95 up to 2000.
Q. 'Cos you talk about a period of five years in the 90s?
A. Yes.
Q. When you say you, "Believe it was then," are you not sure?
A. Not 100%, no.
Q. Why is that?
A. Oh, just a long time ago, we did a lot of jobs.
Q. So you think it's '95 to 2000?
A. Yes.
Q. And how many people did you have working for you at that time?
A. Twenty, 25.
Q. But you were the manager, you were managing director?
A. Yes.
Q. And so you are aware of all the jobs that were on the go?
A. Yes.
Q. And obviously you're invoicing and payments?
A. Pretty much, yes.
Q. You'd be aware of who you did the jobs for?
A. I can't remember, no, can't recall.
Q. Right, so you can't tell us any of the head contractors in the CTV building that you would have done the drilling for?
A. Oh, it was electric, one was an electrical outfit but I can't remember what one.
Q. Have you got any records still?

A. No.
Q. What, 'cos of the time has elapsed?
A. Yes.
Q. Would you have provided invoices the contractor?
A. Yes.
Q. You don't have those now?
A. No.
Q. Why is that?
A. I sold the company in 2000 I think they have downsized; I haven’t approached them of course but...
Q. But when you sold it, did you provide all your records, did they – would you not have kept your records?
A. I would imagine yes.
Q. What, that you kept them or you provided them?
A. That we’d provided them.
Q. What, even invoices that had been paid in the past, surely you’d keep those?
A. Well, not that I recall.
Q. So you are not sure whether you kept them or not –
A. No.
Q. – but you haven’t got them?
A. No.
Q. All right. Well what about the tenancies that you worked on, which tenancies were they?
A. I don't know.
Q. You can't name any?
A. No.
Q. But you are sure that it was the CTV building?
A. Yes.
Q. Well how do you know that?
A. Well I dropped off some equipment there one day, I think I took a man down there and I went back and it was the CTV building, wasn’t called that then but...

Q. And what was that job?
A. Drilling.
Q. So that was one occasion?
A. Yes.
Q. What was being drilled then?
A. Holes.
Q. Silly question?
A. I don't know where, I didn't go into the building.
Q. Right. So you just dropped someone off?
A. Yes.
Q. One of your workers?
A. Yep.
Q. To do a, presumably a drilling job?
A. To meet someone else that was there to help him. yes.
Q. But you have got no idea who it was for or even exactly when –
A. I can't recall, no.
Q. Right. So is that the only one occasion that you had any connection with the CTV building yourself –
A. Yes.
Q. – in terms of drilling?
A. Yes.
Q. So, what are you relying on for your estimate of the number of holes that would have been drilled?
A. Ah, just, just memory. I can't recall who they were for, exactly what years we did it.
Q. Right, but what I am trying to understand to be, because we want accuracy if we can?
A. Yes.
Q. Is what is your memory based on, what facts?
A. Um, well it's just on memory.
Q. Right, I understand that but what do you remember, do you remember people telling you they drilled holes there or do you remember seeing invoices for particular holes being drilled?

A. I remember phone calls and sending the boys down there, yeah.
Q. And what you remember, 200 times that happening?
A. No. No, you'd send a driller down there he might be maybe there for a day and he may drill 20 or 30 holes.
Q. Right and you say he may but you can't actually remember that, right?
A. Not 100%, no.
Q. So, as you said the 200 is a guess?
A. Yes.
Q. It could be wildly out?
A. Yes.
Q. The 50 into beams is a guess?
A. Yes.
Q. That could be out as well?
A. Could be, yep.
Q. And did you say that a third of those 50 would have been steel?
A. On average, yep.
Q. Why do you say, what makes you say that?
A. Ah, it is just the nature of drilling through beams.
Q. So you are just saying that from your experience –
A. Yes.
Q. – with other jobs, not your knowledge of the CTV building?
A. That's right, yep.
Q. All right, so that could be inaccurate too, you accept?
A. Yes could be.
Q. So – because if it was 200 that is a lot of work isn't it on that building?
A. No.
Q. No?
A. Not really.
Q. How long does it take to drill a hole in a structure like a beam, a concrete beam?
A. Ah, half an hour.
Q. Unless you hit steel?
A. Unless you strike serious reinforcing, yep.

Q. And have to go through it?
A. Mmm.
Q. When you are drilling a hole through a structural member of a building, do you have to apply for a permit?
A. No. Not back then you didn't, I believe it may have changed now.
Q. When did it change?
A. Not sure.

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Q. 'Cos we heard from someone from the Council the other day who said, one of the managers who said, you do need to get a permit, you think that is a recent thing?
A. I think that's recent. I don't believe that was the case in the '90s, no.
Q. So presumably you never got any permits for the holes that were drilled?
A. No.
Q. And whose responsibility would it have been to get them if they were required?
A. Ah, whoever's directing the...
Q. The contractor that was...
A. We were all care no responsibility. We cut where the main contractor marks, um, and it's up to them to have got permission or gained the permits or...
Q. Did you ever enquire as to whether that had happened?
A. Well, no. It was common cause to be drilling through beams back then as far as I was aware.
Q. How many beams have you personally drilled through, or been present when...?
A. Hundreds, not in that particular building but...
Q. No, I'm talking generally.
A. Hundreds.
Q. Right. And has that ever caused you any concern?
A. Yes.
Q. Why?

A. Well because the structural integrity of the building is compromised when it happens. However, it was that common that, um, it was just a done thing.
Q. Right. And not just with your concrete company?
A. With all concrete cutting companies.
Q. Did you ever raise your concerns?
A. I believe I did but I can't recall who with. I just used to say, "Hey, look this isn't right".
Q. Who did you say it to, what kind of people?
A. The odd foreman or having a beer at work, you know, on a Friday I'd bring it up.
Q. And did you or any of your contractors that you worked for ever get an engineer in when such a hole was going to be drilled?
A. No.
Q. What about when the hole went through the steel?
A. Well, no. We would ask often if we strike major steel we would ask, "Can we move the hole?" Sometimes it was allowed, you know, we were allowed to be moving it but, um, maybe 50 percent of the time it was, "No it has to go there, carry on drilling", which we would and sometimes the steel would rip the teeth off the drill bits, um, but we would carry on putting the holes through where it was marked.
Q. And can you make any guess, again I guess because you won't be able to remember, but can you guess how many buildings around Christchurch your company would have been involved in drilling holes in beams?
A. Um, lots, 50 different buildings say.
Q. And your guess of 200 CTV building, accepting that could be wildly inaccurate as you've said –
A. Yeah.
Q. – did you drill similar kind of holes from your memory in other buildings?
A. Yes.
Q. So CTV wasn't out of the ordinary?
A. No.

Q. And I guess you can't say from what you've said where on the CTV building these holes would have been drilled, where the beams were or you can't remember which tenancies?
A. I didn't go in and see the holes being drilled so no.
Q. How do you know there were holes drilled in beams as opposed to floor slab?
A. I don't. Once again I'm going by the nature of the work we normally did.
Q. So I understand what you're saying that you're going from memory and your memory's hazy and you've got no records. So would you accept that to rely on your memory as to the number of holes and where they were drilled is dangerous?
A. No, memory's memory, I mean...
Q. Right, you see –
A. – I'm not here to – I'm not here to make things up and I specifically remember –
Q. No, no, no I understand that.
A. – being there and I believe it's relevant to the enquiry so...
Q. And I understand that. I didn't mean it as a criticism. What I was meaning was that you've told us yourself that your memory's not that good because you can't recall any of the details but what I'm putting to you is that we need to be cautious about relying on your memory –
A. – yeah, sure.
Q. – as to the number of holes and in fact where they were that's all.
A. Sure, sure, yep.
Q. You see we've heard evidence from two USAR engineers that went to the CTV site after it collapsed in the day or two following the 22 February earthquake and obviously they were interested forensically in looking at the remains of the building and went over them, took a lot of photos, one engineer took over 500 photos. These two engineers gave evidence the other day and they also went out to the Burwood site where the remnants of the building are and they've gone over those again. My understanding is that they haven't seen any beams with drilled holes in them. That's not to say there aren't any beams with

holes but if there were a lot of holes in beams you'd think that they would have seen some evidence of that. What I'm suggesting to you from that and from your memory that perhaps when you heard that they CTV had collapsed and you thought could be all those holes that were drilled in there that you've overestimated the number of holes that were drilled, thinking that that could have been the reason or one of the reasons?

A. They may have all been drilled in the floor but I doubt that.

Q. But you can't be sure?

A. That's right.

**RE-EXAMINATION: MR PALMER**

Q. Just dealing with questions around your estimate, when people give estimates, we all give them; we have a high and a low range for those estimates. Is your estimate on that spectrum between high a low range of high estimate and low estimate? Where do you think your estimate is?

A. It says up to 200 holes. I believe it's between 100 and 200 would be more safe.

Q. Would that be a mid-range estimate?

A. 150 would be...

Q. Mid-range?

A. Yes.

Q. 100 low?

A. Yep.

Q. 200 high?

A. Yep.

Q. Who did the billing. Who originated the bills that you sent out to your clients at the time?

A. Ah, well a couple of managers and myself.

Q. And what was the basis upon which you would render your bills. Was it time-based or hole-based?

A. Um, hole-based, per hole.

Q. Is your memory aided by that process that you adopted?
A. Yes.
Q. After you sold your business in 2000 what did you do, did you stay in Christchurch?
A. Ah, yes. I opened a pub, had a pub for a while, um, started another concrete cutting company, um, but yeah, semi-retired for a while.
Q. Did you take your records with you to the hotel that you opened?
A. No.

**QUESTIONS FROM COMMISSIONERS FENWICK AND CARTER – NIL**

**QUESTIONS FROM JUSTICE COOPER – NIL**

**COMMISSION ADJOURNS: 11.27 AM**

**COMMISSION RESUMES: 12.11 PM**

**MR MILLS:**
We are now going to hear from the joint authors of the report into the collapse of the CTV building that was prepared for the Department of Building and Housing and their evidence is going to be led by Mr Gregor Allen of Crown Law and I will leave it to him to explain what has been going on during the rather long break.

**JUSTICE COOPER:**
There is no need to go into great detail about that but we understand there's been some last minute adjustments to the documents.

**MR MILLS:**
There has and it has been discussed with all of the other affected counsel so I think everyone knows what is going on.

**JUSTICE COOPER:**

Now you are Mr Allen are you?

**MR ALLEN:**

Yes I am, Sir. I appear for the Ministry of Business Innovation and Employment which as of only a few days ago assumed the responsibilities of the former Department of Building and Housing and I am assisted today by Mr Mike Stannard, Chief Engineer of the Ministry.

**JUSTICE COOPER:**

Mr Stannard, yes.

**MR ALLEN:**

And Ms Sharon Reading, a senior solicitor of the Ministry.

**JUSTICE COOPER:**

In house?

**MR ALLEN:**

Yes.

**JUSTICE COOPER:**

And Reading is spelt R-E-A-D-I-N-G?

**MR ALLEN:**

It is Sir, yes.

**JUSTICE COOPER:**

And if you'll just forgive me Mr Allen, just inquire as to whether there are any other counsel wishing to appear at this point?

**MR CLAY:**

I appear for Mr Smith in his personal interests and, if needs be, if I need to become involved I propose seeking leave at that particular –

**JUSTICE COOPER:**

So you will let me know if and when you see the need to say something?

**MR CLAY:**

That is what I am proposing, Sir.

**JUSTICE COOPER:**

That is acceptable, thank you. Is there anyone else? All right, thank you.
So Mr Allen?

**MR ALLEN CALLS**
**CLARK WILLIAM KEITH HYLAND (SWORN)**
**ASHLEY HENRY SMITH (SWORN)**

**MR ALLEN TO DR HYLAND:**

Q. Dr Hyland, if I could just start with you. Your full name CV is on the Commission website so I don't propose dwelling on your credentials. Your full name is Clark William Keith Hyland and you are a director of Hyland Fatigue and Earthquake Engineering?
A. Yes.
Q. And that is a specialist consulting engineering company?
A. Yes it is.
Q. You have a PhD in Civil Engineering from the University of Auckland?
A. Yes.
Q. Been a registered engineer since 1989?
A. Yes.
Q. And a chartered professional engineer since 2004?
A. Yes.

Q. Your professional service extends to membership of New Zealand Standards Committees, one of which you have been a committee chair?
A. That's correct.
Q. You have lectured in structural design in relation to steel structures?
A. Yes.
Q. And you regularly present technical seminars to the profession in the areas of your expertise?
A. Yes.
Q. And most relevance to your evidence today you've authored numerous papers and publications including those concerning seismic resistance of structures following earthquakes?
A. Yes.
Q. And in that regard you've published in relation to the effects of earthquakes in 2009 in Indonesia and in Chile in 2010?
A. Yes.

**MR ALLEN TO MR SMITH:**
Q. Your full name is Ashley Henry Smith?
A. Correct.
Q. And you are a director of Structure Smith Limited which is also a, it's a consulting engineering company specialising in structural engineering?
A. That's right.
Q. You graduated in 1997 with a Master of Civil Engineering (Civil) from University of Auckland?
A. Yes.
Q. And you've practised as a structural engineer since 1981?
A. Correct.
Q. Now amongst your professional memberships is a former presidency of the New Zealand Structural Engineering Society from 2005 to 2008 and you are now a life member of that society?
A. Yes.
Q. Your areas of specialist professional expertise are structural engineering analysis and design, earthquake engineering, wind engineering and

monitoring the construction of building structures to ensure that they are built in accordance with design?
A. Correct.
Q. Amongst the projects that you have been responsible for is the seismic risk assessment for Auckland International Airport?
A. Yes.
Q. And you've been a project engineer for some very significant developments, including a 32-storey Quay Street tower in Auckland known as the PricewaterhouseCoopers Centre, the 41-storey Vero Centre in Shortland Street of Auckland, and the 35-storey Quay West Apartments in Albert Street?
A. That's right.
Q. And you've given expert concerning structural engineering on previous occasions?
A. Correct.
Q. Thank you for that.

**MR ALLEN TO DR HYLAND:**
Q. Now in April of 2011, if I could just direct this to one of you, perhaps Dr Hyland, Hyland Consultants and Structure Smith were engaged to prepare a report into the collapse of the CTV building?
A. Yes, that's correct.
Q. And you were both appointed key personnel in relation to that report?
A. Yes.
Q. Now for completeness, because I'm sure we'll hear of these companies, Compusoft Engineering was nominated to assist with structural analysis?
A. Yes, that's correct.
Q. And the company Tonkin and Taylor were nominated to assist on geotechnical engineering aspects?
A. Correct.
Q. Now that report, I understand, was finalised in January 2012 and has since been made available on the Commission's website?

A. Yes it has.
Q. Now the purpose of your evidence today, gentlemen, is to provide an overview of the report and to that end have you prepared a presentation that will take us through that report?
A. Yes, we have.
Q. And is it your own intention to jointly present that presentation?
A. Yes we do. I'll start with initial introduction then Mr Smith will go through some of the building features. I'll go through another section. He will take another part and then I'll finish off.
Q. Okay, all right, if you can comment with that, that will be appreciated.
A. Right, thank you very much.

**DR HYLAND:**

Thank you Commissioners. Yes, so this is a joint presentation of a joint report and so we are going to break it up into these sections. I'll just outline the presentation. First of all we'll talk about tasks that we were given by the Department of Building and Housing. We'll look at the building features, talk about the investigation approach, discuss how we believe the building collapsed and why the building collapsed. So I will now pass on to Mr Smith.

**MR SMITH:**

I would like to just explain a little bit about the building and the portions of it that we will be referring to during this presentation, so I am looking here at a photograph taken in 2004 from the south-east corner, the corner of Madras and Cashel Street, just identifying some of the features, so let's start down the bottom, there is or there was confusion about the levels in the building.

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We were referring to the ground level as level 1, the building starts at level 1 at ground and levels 2 to 6 above. So level 6 is about here with the, a roof above that. So levels 2 to 6 are suspended concrete floors and level 6 is a lightweight roof, steel framed roof structure. We're looking at the south side where I am at the moment, ah, there's what we call a south wall, one of the main bracing elements with a fire escape stair attached to the outside. We're looking at the columns at each level. You can see a portion of the column and some of it is hidden behind this, what we call precast spandrel panels. Precast spandrel panels are these exposed aggregate sections here between columns. Again this is, um, so we're looking at line F as the east face, we're saying this is one of the line F columns at this point. That happens to be the level 5 column. Um, yeah that's probably all I need to say on that one. Um, so just identifying some of those features. The next slide shows a three dimensional view, it's actually of our computer model, but again just identifying the components at this stage. Um, the main bracing elements being the north core, which is this assembly of walls at the north face. A C-shape arrangement, it's got the lift shaft at this end, a stairway in the centre and amenities, bathrooms, in this area. Ah, these will be referred to, what we're calling drag bars were steel angles that were bolted to connect this floor to the floors. The following construction, we'll talk more about those. On the west face the bottom three levels have what we're calling infill masonry walls, not intended to be part of the primary structure but were built between the concrete frames after they were erected. Those are those blue, that blue shading. Here we have the south wall, looking down on top, a penetration at each level being the exit door onto the fire escape and, um, if we're looking at the, we're calling edge beams along this, these spaces, north, east and south. Internal beams on these lines 2 and 3 and we can also see the foundations here which I'll talk to. That's just an overview of the structure. Now I've got a series of extracts from the building consent drawings that were the basis of a presentation, my presentation to the second meeting, I think it was, of the expert panel. So we just had extracts from the building consent drawings, our first appraisal of what we thought might be key things to investigate and that we wanted to try and capture in our analysis of the building.

**JUSTICE COOPER:**

Just for the record, could you just read the number where the plans which are extracts from the permit drawings begin and end? The first one is number 5.

**MR SMITH:**

Five, number 5. The last one is number 14.

**JUSTICE COOPER:**

Thank you.

**MR SMITH:**

So these are not entire drawings, they're just snapshots of a portion of a drawing, just to, um, okay?

**JUSTICE COOPER:**

Thank you.

**MR SMITH:**

We're looking at a plan view here of the foundations. Basically a foundation is comprised, what we call shallow foundations as opposed to pile foundations. So we have in the centre the internal columns have this square pad footing under each column. The edge columns have a pad footing that extends from this point across to here. And in the centre they've got an up stand beam which is cast as part of that, so it's like an inverted T section around the perimeter. Basically the object is to, quite a stiff beam underneath this bracing wall and to restrain these bracing walls they project back into the next column so it helps to restrain the base of the main bracing shear walls. The internal columns are shaded in blue here, circular, 400 diameter. Okay, next page is a view of a typical floor, suspended floor, levels, I've said levels 2 to level 6. Each one is the same. We start with the floor system. It is a 200 thick, what we call Hi-Bond, which is a profiled metal steel deck which has concrete poured on top and that spans in this direction here, north-south between each of these beams in that direction of those arrows and across here in the core. Ah, circular columns, again, shaded in blue. The beams, internal beams running this way, east-west on these two lines, and then we have edge beams on the north, east and south faces. On this face here we had a very small beam just supporting those masonry infills only on the bottom three levels and no beams above that. So, it was just a slab of each without a beam. I've shaded the main bracing walls orange. This is the north core at this end so that's the arrangement of the bracing walls. One of the features is the elements that I've showed in green are penetrations through the floor slab. So, over here we have the lift shaft, the next bay the stairwell and this is a penetration for a mechanical or some sort of building services to come up through the building. So that was certainly one of the features we noticed was the main bracing element, the north core outside the line of the building and connected by a relatively small area of slab. Ah, at this end here, the south wall, shown by these two orange blocks here, but at rather a better connection with the beam connecting in and continuing along that entire side. Okay, if I go back first of all, the next slide I'll show you is an elevation of this side here. So it's the east side of the north core, with this column adjacent to it. The column we refer to as C18, that's the label on the original drawings. This is an elevation of that. So the wall is shaded in orange. At the bottom we see, here's our deep foundation beam so we've got good anchorage of the wall steel down into that beam. Um, we're seeing that the wall was of this width up to roof level but then projected back across and connected on to that column, C18 that we showed. So it was connected and would, it meant, it was significant because when the seismic loads forced this wall to drift sideways, the fact that it's connected to this column imparts certain forces onto that column. An elevation here of the south wall. Again, the deep foundation beam, anchorage of the bars down into that, a penetration at each floor level where the doorways onto the fire escape, and a lintel beam above that which we call a coupling beam. It couples these two individual walls together so that behaves as a coupled shear wall, and it is a common, um, way of detailing those is to put these diagonal reinforcing bars that's what we can see there. Those are bars passed inside the concrete. Next one a section of two of the beams. Well first of all let's look in the middle, ah, I was explaining before about the Hi-Bond floor system, this is what it looks like if you look at along the north or south axis. Two hundred thickness overall, the profile of the metal deck follows that green line and it has the concrete infill with the reinforcing mesh in it. It does have additional bars over the top of the beams and at the edge. So that view there, if you like, is at 90 degrees to these two other views so here we're looking along the, along the profile of the floor which turned out, it was green it's turned out blue on this view, um, again that's the thickness of the profile, so the construction sequence is to erect this precast beam which we normally refer to as a shell beam, cast in a U-shape to that orange profile. It does have bars embedded in it which are not shown here. That's erected, the metal deck is placed on it and concrete is poured across the metal deck into this beam at the same time.

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Just the construction method. One of the internal beams we call pre-cast log beams they, the shell beam when it's a U-shape, the log beam when it's a solid section there with bars projecting out the top and again the concrete floor is poured across. Now we're looking at details of the columns from the consent drawings, a plan view here, that most of the columns in the project were the same size 400 millimetres diameter circle, 50 millimetres thick concrete outside the steel, that's 50 cover there. It's got six vertical bars which are these H20s running up vertically and it's got a six millimetre spiral wrapped around and the vertical spacing of the spiral is 250 millimetres. It's 250 pitch. Looking at an elevation of a typical circular column we see the vertical bars here and at each floor level the bars are inserted at lengths of approximately five metres so we've got a 3.2 metre storey height and then a portion projects out so the column is poured up to here, it has this projecting, this floor is constructed and then a matching, a bar has come down and lapped on to that so we've got a vertical bar lap just above the floor level at each level. The next view is plan views of what we call beam column joints, the junctions, which we're looking down on a 400 diameter circular column here, with the beans connecting in it each side. One of the things that was in this building, the ends of the pre-cast beam were cast as a semi-circular shape and well projected 25 millimetres into that nominal 400 diameter, it's just, it becomes important when we take, start talking about the joints. What we can see here is the bottom bars of the beam projecting into that joint. The beam from this side has a bar projecting from here to this point. This bit, this beam has a bar projecting from this point to here so there's some overlap, there's no spiral shown at this point here but, as was shown for the column. So that's a 400 diameter. At the west side we have a 400 wide by 300 deep column and looking at the projection of these bottom beam bars into that joint that's quite a short anchorage length, that's one of the issues we looked at. I'll just talk about this one here briefly, on, oh, go back to this one we saw that the bottom beam bars overlap to some extent so from here to here is an overlap of the beam bars, that was not always the case on line F one of the edge beams we have the bottom bars projecting here. They don't cross the column centre line and they don't overlap with the other, the bars from the other side, so again quite a short anchorage of that beam bar into there. Now looking here at an elevation of the west side. In fact, I've only shown one bay between grids 1 and 2, there are actually two more bays identical beside, but I've just shown one at the larger scale. Let's look at the section first, masonry's constructed up to here, there's a pre-cast beam, a very narrow one only the width of the masonry wall erected in the orange and then another level of masonry is built on top of that. We'll be talking about the separation of this, so it wasn't intended to be part of the seismic resisting structure and therefore was to be separated from it and that was achieved by not grouting the, not filling up the blocks right up to the underside of the beam but this top half block or so was left ungrouted but there was a steel dowel that connected the wall to the beam at each level. And I'm saying at level 4 there was no beam it's just connecting to the slab edge and above this level we just had lightweight, lightweight construction so the masonry was three levels high. We look at an elevation of that masonry, the other point is each bay between columns, so there's a column at grid 1 and at grid 2, the masonry is divided again into three separate bays by the, shown by these red arrows so there's no connection of this panel to this one. There is no reinforcing projects through that at that point. And lastly I'll just look at a section through one of the edge beams, again this is the portion that I shaded orange before one of the shelvings, not showing the reinforcement this time but after the floor was constructed they came and bolted on pre-cast spandrels which are this orange profile so it's an L-shaped pre-cast spandrel and at each end between the columns there is an end wall shown by this orange line here and this diagonal line. So it's an L-shape at each end. It has a return wall and the point is it is in between the columns. There's one of the circular columns in the background.

**DR HYLAND:**

Okay thanks Ashley, so now we'll talk about the approach we took the investigation and in this section we will cover over a period of time the collapse condition, the interviews that we had with witnesses, the site examination that was undertaken, the materials testing that followed from that, then we'll look at structural analyses that were undertaken, talk about compliance checks and then the collapse scenario evaluations that we undertook. Now the method of determining our collapse scenario, our preferred collapse scenario, was a developmental approach, an iterative approach. We started by collecting data, some background research getting drawings. We started to develop some hypotheses. What were some of the potential collapse scenarios that may have occurred? We carried out analyses, initially simplified analyses, to see whether these looked reasonable. Looked at the evidence we had at the time, did they seem to match up? Go back to analyses, forcing the conclusions. So it was, it was a - and we were reporting back also to the reference panel, expert panel, DBH panel of experts, on a regular basis, progressively as we went through the approach to say well what is, "This is what we're coming up with what do you think?" Drawing some conclusions, going back, getting more information, doing more testing, eventually coming up with draft reports through to a final, final report.

**MR ALLEN:**

Dr Hyland the hypotheses that you formed and then tested and then reformed were they predicated on vulnerabilities that you discerned from the design plans?

**DR HYLAND:**

I mean the, when we started, we started with the site with just the tower left hour and some debris that had been put aside by the USAR teams so we had to then sort of build up our knowledge base from that point. First thing was to get some drawings or the drawings. There were some things we saw there which were of interest. Had a look at the north core found some issues there that were of interest. Looked at the, the debris that had been left by USAR and we could see some issues there. So, it was gradually just building, building up a, a knowledge base and, and starting to develop these hypotheses.

**MR ALLEN:**
Right, and part of the feedback loop that's shown in there does that entail reconciling the results of the analyses with the observable evidence, the debris if you like and other evidence that you gathered such as from witnesses for example?

**DR HYLAND:**
Yeah definitely, we, the way we looked at it is there needed to be a convergence of the various strands of evidence so there had to be convergence between what people saw.
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**MR ALLEN:**
Yes.

**DR HYLAND:**
What we saw in the debris, what the material testing was showing us and what the analyses were verifying. So these things, we believe, needed to come to a convergence.

**MR ALLEN:**
Thank you.

**DR HYLAND:**

The CTV building after the collapse was important to understand the condition immediately after the collapse and before any debris was removed because if we could turn back time we could put the building back together and we could perhaps see what had failed and how it had failed. So we were able to get from witnesses and people who had been, who had come forward from the public with photos to the Department of Building and Housing, we were able to get photos that were taken mainly on, just, phones and things like that and this is the view from the west, from the Les Mills building at the time. You will see that is the north core, the fire has just started in this photo near the north end. The level 4 to 6, cladding, it has been pushed north. There is little debris otherwise on the side, not much of it fell over the western side of the building. There was some diagonal cracking observed in the masonry infill.

**JUSTICE COOPER:**
Can I just ask, you said the cladding has been pushed north but it looks more like it has been pushed towards the person taking the photograph which would mean being pushed to the west, have I got that wrong?

**DR HYLAND:**
Yeah, well we have got the north there on a, down on the core there, there is some panels that have fallen over the wall but they really just dropped straight down but the fact is the, what I am trying to say here is that the debris has been pushed northwards in this particular photo.

**MR ALLEN:**
From its starting position Dr Hyland or…?

**DR HYLAND:**
Yes from a starting position because the building, the northern edge of the office area of the building was actually on the southern face of that north core. So you can see that some debris has been pushed sort of north of that.

**JUSTICE COOPER:**

It has fallen west and north perhaps, has it?

**DR HYLAND:**

Yeah, a little bit west but north, yeah. But there is no liquefaction seen adjacent to the building on the west side. If we look at the south face, this is a photo taken very soon afterwards also in this case, it appears this was taken before the fire had started so perhaps even before the previous photo. You can see there is a portion of level 5 slab in front of the lifts which is just hanging on there. The level 6 portion of slab in front of the lift doors is still holding up there, the column that was, the C18 column however has disappeared. There is no debris or very little debris on the south side of the building. The cars in the car park are largely untouched. If we go to the south-east corner view, you can see the sign that was above the doorway into the office. There is a little bit of smoke in the background, again just showing that this is an early photo, again there is no liquefaction in this area around the building. There's an eastward throw here, the debris on this side of the building has come across the footpath and actually crushed cars so it has come over a metre or so into the, on to this side of the building.

**JUSTICE COOPER:**

That sign, CTV sign, has fallen slightly out from where it would have been prior to the collapse.

**DR HYLAND:**

I think so, it has probably come, yeah, a little bit south because you can just see the portion of the south wall of level 1 and 2 but that was standing up. So it has been pushed out a bit and maybe the panel there has carried it over with it.

**JUSTICE COOPER:**

Yes as Commissioner Carter points out, it is over the kerb.

**DR HYLAND:**

Interesting here, you can see some fractured columns with the tips of the columns have sort of got, almost spearhead-like shape to them, this is the ones along the line here, sort of poking out. Go now, coming along down the east side, just down Madras Street, this is a photo taken out of Blackwells, was across the road and you can see those columns again, it is line 2 and line 4. You can see a bit of smoke in the background again showing this was an early photo before the debris was taken away from the east side. Again no liquefaction on the side of the building, so there was no liquefaction around the side. This is a view of actually, this is Ron Godkin being helped out of the building.

**JUSTICE COOPER:**

In the red shirt.

**DR HYLAND:**

In the red shirt.

**JUSTICE COOPER:**

We've seen, he has shown us that photo before I think. This is a better quality photo than the one we saw.

**DR HYLAND:**

Now when this interviews, we, the Department of Building and Housing put out a request for people who had seen the building collapse, who had been in the building, who were prepared to talk with us, any other people who may have been in the building or at some time and so we conducted interviews, interviews with 18. We had help from Paul Downey, the Ministry of Social Development who is an investigator who assisted us with interviewing. We had six who were in the building during the collapse and we had people who had views of the collapse from the east-south and the west and rather than develop 3D perspectives and things and this just helped us to try to piece together what they were seeing in the particular portion of the building they were seeing at the time and we found that very helpful. Some of the key witness observations that came through was witness 14 was Euan Gutteridge he observed the building from the south and the east and he noticed that the building was twisting, there was a bursting and there were columns breaking. It was Tom Walker and Penelope Spencer they were quite close to the building on the south face and they noticed the upper portion of the building come down as a unit and that was quite interesting from our point of view because we thought perhaps you would expect the building to start collapsing from the bottom but that sort of highlighted to us that we needed to think about how this might have happened that there may have been an upper portion of the building coming down as a unit. I think you have heard from Mr Williams who also saw the thing coming down, like the Trade Towers, sort of coming down, a portion coming down. We had other saying they felt the building coming down in steps on top of each other. Dennis May, he was in the IRD building across the road more closely aligned with the east face, he saw the buildings lean out to the east and then collapse straight down. During the presentations of our report in February Ron Godkin, who we hadn't been able to interview talked about seeing the floor collapsing from the south and undulating for a sharp lurch, so we interviewed him after that. Okay, now I will go to the site examination. So I would just like to say was very helpful having the witnesses come forward and give their time to us and it is really helped to inform and help us judge some of our analyses results. Okay, the site examination involved initially looking at the salvaged structural components that were set aside on-site by USAR and, again, we are extremely appreciated of that.

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I think Graham Frost and Dr Bob Heywood made special efforts in that regard and it really was a huge help. North core examination, level survey, foundations excavation and then column extraction and testing from what was at Burwood. Just looking at the salvage structural components, these are the ones which were on-site, um, when I arrived there. Just on the top left photo we found at level 5 to 6 there was some soft concrete at the door head. This concrete you could kick with your foot and it would break out. There were smooth constructions between the tops of each lift of the south wall. You can see here it's actually quite a smooth base. Similarly here's another view of it. Normally we would expect this to be a roughened surface so that you'd get good interlock between one level of the wall and the next level to prevent slippage and things like that. Other components, columns. There was some unusual things here that we noticed with the columns. Here's a portion of a 400 diameter column. Now, when I first looked at this I could not understand what had happened here because there was a portion here that had seemed to have hinged and there was a portion here that had fractured. There was a piece that was painted. There was a piece that was unpainted. And initially when I looked I thought this must be the top. This is, perhaps in the middle, but then I realised that this painted portion is what happened to the – and the unpainted bits – would be what happened to columns that were on the outside face of the building, because on the outside face we had the pre-cast spandrel panels fitted in between the columns so no paint was put on them. Internal columns, however, had paint on them. So this, therefore, was the bottom of the column and that is also where you can see there's two sets of vertical reinforcing bars where the vertical reinforced steel lapped from one floor to the other. At the base it's just interesting here to note that the concrete cover has broken away but there's still some concrete in the middle here that's been confined by the reinforcing steel. It's got horizontal sort of cracks in it so even though the concrete had lost its cover there was still some concrete inside the core there that would have been able to sustain some vertical load carrying capacity. When you get to the top or the mid-height this is roughly 1500mm or so above the floor level you can see there's this more of a diagonal, or sort of, call it a spearhead type of fracture had occurred. That's not at the top of the column but that's somewhere further up. Looking back on the drawings that Mr Smith has shown, the reinforcing steel went up from one level from the floor below up about 1500 or so, from the floor level, so there was a termination of those two layers of reinforcing to one level, one layer of reinforcing, roughly around this sort of area. Another question it raised was how would we get a fracture occurring in this column which was going between the floors which were 3.2 metres, how would you get that occurring?

You'd expect, perhaps, the sort of hinging at the base and at the top, but why would you be getting a fracture half way in between? So one of the thoughts was, was this spandrel panel contact initiating this?

**JUSTICE COOPER:**

Q. Can I just ask you, this column that you're depicting can you say where it might have been in the building?

A. No, the problem is we don't know which level it was. We know it must have been on the outside face, probably on the line 1 or line F, the south wall there.

Q. It could be the C1 column could it?

A. It could have been, it could have been. Could have been C1, could have been any columns down line 1 or line F but it wouldn't be the ground floor column, the level 1 column because there wasn't a spandrel panel at that level.

**DR HYLAND:**

Okay, one of the thoughts about spandrel panel contact was also could the spandrel panels have caused a mid-height fracture of the column? There was one photo here from the debris that had been pushed north on the west side of the north core and this may be a level 3 beam or level 4 beam. I think that was discussed a little bit by Dr Heywood I saw in his evidence. It was quite interesting before I thought, um, but you see here it was just really the interest here is that we've got this fracture which would have been above the spandrel level and there's some injury at the top but we've got this quite significant fracture zone here, it will be a bit lower than what's here but it did indicate to me that there was some justification to think that spandrel panels could have restrained the columns enough to cause a fracture in the column. So coming up with a hypothesis around that, we have the column in its straight up and down position. We have some inter-storey displacement. It's item 2 on the diagram and we would see perhaps some hinging starting at the base, hinging starting at the head, which would be what would be expected to hinge top and bottom, perhaps like we saw in that column remnant at the base. Then potentially, as it's a column against contact with a spandrel panel, we could start getting a short column effect that may, which would lead to a amplification of the stresses at the head, a reduction perhaps in the stresses at the base of the column, some increase in stresses in the zone where the termination of the vertical reinforcing bars occurred, maybe not enough in general to force a hinge right at the top of the spandrel panels but given a lowly confined concrete column where it is known that you can get localised spalling effects initiating at termination zones or the reinforcing of that, there's a possibility that perhaps the stresses were raised enough in that termination zone to lead to some loss of concrete cover, a loss of capacity which may have initiated another hinge forming there or another fracture zone. Then had a look at the south wall. What was the condition of that after the earthquake? The south wall had lent over at level 2. There appeared to be some heavy compression spalling at the east end of that wall. Now this was on the outside of the wall. The inside, it could have been explained because of perhaps the north-west fall, perhaps causing some localised compressive stress there, through weak (inaudible 13:00:26) suspending but this is on the outside face so to me that indicated that this was compression fractural damage that had occurred when that wall had been forced eastward. The cracking that was observed there now. You can see there's a sort of a diagonal nature to this which comes back down to the sort of running down to the centre of the wall.

**JUSTICE COOPER:**

Now just confirm for me. So this is the eastern end of the south wall? Is that right?

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**DR HYLAND:**

Yes. Eastern side wall, yep. The doorway here's been filled up with masonry, partly, so there's this cracking, a fanlike cracking like this in a wall is usually typical of flexural type cracking in a cantilever wall, so you'd have cracking on this side and there was fanlike cracking on the western side.

**JUSTICE COOPER:**

I am sorry to interrupt you again. I will be the one asking the obvious questions, but you said the doorway?

**DR HYLAND:**

Yes, yes. This was, Mr Smith sort of pointed out there was these doorway exits, um, located all the way up the south wall where the fire escape came out, so you know at level 2, 3 to 6 there was an actual door inside there.

**JUSTICE COOPER:**

Yes, I understand that but that doesn't look like a door to me for some reason?

**DR HYLAND:**

No this, this particular hole had been filled partially with masonry, reinforced masonry and then a window was put in.

**JUSTICE COOPER:**

Right but –

**COMMISSIONER CARTER:**

Was that at ground level?

**DR HYLAND:**

Yeah, ground level.

**JUSTICE COOPER:**

Ground level but in terms of the original drawings, was it designed like we see it?

**DR HYLAND:**

No we didn't see this masonry infill on the drawings that we had got from the Council.

**JUSTICE COOPER:**

So when it was originally designed it was designed to be a doorway and at some stage it was filled in?

**DR HYLAND:**

Yes, appears to have been.

**JUSTICE COOPER:**

So that was never presented on a plan that you were able to see?

**DR HYLAND:**

No we didn't find any drawings on the Council file that we saw, is that correct?

**MR SMITH:**

Yes I think so.

**JUSTICE COOPER:**

Well just going back to the photograph which is your slide 3, there weren't any at the beginning?

**DR HYLAND:**

Yes.

**JUSTICE COOPER:**

We are looking there at the south wall going to the left, is that right?

**DR HYLAND:**

Yes.

**JUSTICE COOPER:**

Have you got, could this be displayed?

**DR HYLAND:**
Well it's slide 27 now.

**JUSTICE COOPER:**
Slide 3? Yes this one. That, where would that feature that we've just been looking at be on this photograph?

**DR HYLAND:**
Just there I think.

**JUSTICE COOPER:**
I see. So it is below the landings of the staircase and the floors above it?

**DR HYLAND:**
Mmm.

**JUSTICE COOPER:**
Is that right?

**DR HYLAND:**
Yeah you can see that's, um, at the upper level you can see the doorway and the landing.

**JUSTICE COOPER:**
So it may be it's my misunderstanding of the question and answer that you gave me a few moments ago, but if we go back to slide 27 that we were looking at, that's not the eastern end of the south wall, it's the eastern side of the coupled shear wall on the south side?

**DR HYLAND:**
Oh, we, we're, yeah, sorry we're referring to the, the coupled shear wall element just as the south wall, and just calling it the south wall.

**JUSTICE COOPER:**

Yes, but that's what was, I couldn't quite follow and in this slide the portion of the building on the south wall, to the east of what we're looking at has simply collapsed, and that's why it appears to turn a corner there?

**DR HYLAND:**

Yes and, yeah the beams and columns that were connected to that, that wall system have fallen away, and then what's been left is that, um, that south wall element with the fire escapes attached as N4 and has fallen over, although some of the beams were still attached as it fell over according to John Trowsdale and Dr Heywood and photos we've had.

**JUSTICE COOPER:**

Well, this may not be the right time for this question but in terms of the structural performance of the building was the effect of that infilling that occurred significant?

**DR HYLAND:**

It's an interesting question that one. It's, it appears that that wall has behaved at that level like it was an, a cantilever wall with a hole in it rather than a coupled wall at that level. However, the coupling beam at that level from level 1 to 2 is also very deep. This is actually very deep so it's possible that the coupling beam itself may have stiffened that up as well but –

**COMMISSIONER FENWICK:**

So while we're here I would sort out one issue with the columns?

**DR HYLAND:**

Yes.

**COMMISSIONER FENWICK:**

The actual drawings show the spiral going up through the beam column joints. Now, I understand if you tried to put those beams in for spiral in place it would be impossible but I've heard there was not spiral in the beam column joints but that's not quite what's shown on the drawings. Can you confirm there was no spiral in the beam column joints?

**DR HYLAND:**
No.

**COMMISSIONER FENWICK:**
Or was there one and you just didn't see it?

**DR HYLAND:**
No I think there was spirals but we are only talking spirals of 250 centres so the –

**COMMISSIONER FENWICK:**
250 centres.

**DR HYLAND:**
250 centres. So there precast log beams were 350 millimetres deep so within the zone where a precast beam went in there was only perhaps one –

**COMMISSIONER FENWICK:**
No, the spirals are shown going up beyond the end of the columns and the precast beams overlap by 25 millimetres, it was 50 millimetres cover to the main bars, so they would have still, be room for that spiral to be in?

**DR HYLAND:**
Yeah.

**COMMISSIONER FENWICK:**
Now I am not quite sure how you get the spiral in there with the bars coming through but that is what is shown on the drawing, I wanted just to confirm whether you were quite clear there was a spiral in there or not in there?

**DR HYLAND:**

Well my recollection is really based on the photos of that because the debris that we saw had been sort of disconnected so.

**COMMISSIONER FENWICK**:

So you don't know?

**DR HYLAND:**

I can't really say, you know from my visuals, we really just got to look at the photos that were taken at the time.

**COMMISSION ADJOURNS: 1.09 PM**

**COMMISSION RESUMES: 2.19 PM**

**MR ALLEN:**

Dr Hyland, we're just staying on this slide relating to that south coupled wall and you've commented on the vertical fanning that we can see on that wall. You said before lunch that that was occasioned, you think, by east-west, or west-east forces. I'm wondering if you can explain how those forces might have arisen in the CTV building?

**DR HYLAND:**

Right, we found that the, when we were doing analyses of the building that building had a centre of stiffness located in the northern side of the building, near the north core, and so the building had a tendency to rotate about that centre of stiffness which means you would get relatively larger deflections or displacements along the south wall where this, or the south face where this wall was located relative to what you would get when it's at the northern end of the building so...

**MR ALLEN:**
So it is a function of a twisting?

**DR HYLAND:**
Function of a twisting of the building.

**MR ALLEN:**
Thank you, if you could take us from there then please Dr Hyland?

**DR HYLAND:**
Thank you, okay. The north core condition, there was hairline cracking found in the north core on the various faces, there was very little seen on the eastern side of the north core, there was some horizontal type cracking found on the north-west corner –

**JUSTICE COOPER:**
North-east corner did you say or north-west?

**DR HYLAND:**
That's the north-west corner on the top right picture here, so this would be the north-east corner on the left top picture. Inside the lift well between walls D and DE you can see there was diagonal cracking, hairline cracking. There was also a little bit of, it was fire-effects so it was blackened. North core examination. I undertook two examinations –

**COMMISSIONER FENWICK:**
There was some cracking you haven't mentioned in the core. I don't know whether you caught up with this, but there was cracking found by Graeme Smith when he went up above the elevator and he found vertical cracks in the north core in the elevator shaft and they were, he said 1 metre and 1.5 metres from the second to east-most wall. He said they were width of .3 to .5 millimetres so you might like to include those in your assessment.

**DR HYLAND:**
Yes, I was quite interested to hear that, um, so they would have been perhaps in the, in line with this, these rails.

**JUSTICE COOPER:**
So you're indicating the position in the bottom right corner between the rails off towards the left-hand side of that image?

**DR HYLAND:**
Yes. That's where I understand he described the cracks, that, half a metre off that corner and one a half metres off that corner. So going up the north core examination went up there twice with, in a bucket on a platform, second time with Mr Smith, and we inspected each of the ends of the walls and we found that at the ends of the, this was looking down on top at the far eastern wing wall, the one on line DE a number of cases, you know, there wasn't any reinforcing coming out of the ends of that wall where the slab would have connected in and, it was, in some cases of less reinforcing, you know, not much reinforcement coming out of them. The slab appeared to have cracked, broken away, over that sort of profile in the examination and then following examination of photos appeared that the cracks, the slab had broken away from the north core over a profile followed by the red line and that was where the ends of the H12 saddle bars had terminated.

**JUSTICE COOPER:**
Just help me, is south at the top of this page?

**DR HYLAND:**
Yes, yes, so south is the top and the north is on the bottom.

**COMMISSIONER CARTER:**
Just help me distinguishing some cracks with red and some are black. Could you just go over that again Why they're two different colours there?

**DR HYLAND:**

Okay, the black was what was there at the time of my examination so that was at – just with the core as it was when I examined it and the red was inferred from looking at the debris photographs. The drag bars which get talked about, this is an elevation of a drag bar, was a steel, piece of steel 152mm x 152mm, 10mm thick. It was bolted into the face of the wing walls and with epoxy anchors, 24mm diameter, and then to the underside of the Ibon slab which is a galvanised steel warmer with concrete placed on it. It had holes drilled up from the underside into it and epoxyed in to connect the slab into the wall. So those drag bars were connected to some of the walls. This is a photo, I think it's a police photo actually, that was taken and shows the level 6 drag bars still attached, holding up the end of the portion of slab just outside the lift well. It says here, "The level 5 drag bars bent down." Got the level 5 precast concrete beam has come down. Level 4 slab failure along the ends of the H12 bars could be seen along here and the failure surface runs diagonally on to this edge beam which was run from the core eastwards and here we have the line 4DE or C18 column. You can see the joint zones have been pulled out of there. This is a photo taken before the debris was touched. This was the portion of slab that was hanging off at level 5. Level 5 in front of the lifts and you'll recall that Mr Godkin and Mr Horsley were standing at level 4, which would have been right underneath where this portion of slab is sitting now. There's the level 5 failure surface and there's the line 4 where the beam was on line 4 and you can see that the slab was actually 1200, well, you know, in the order of 1200 where the ends of those saddle bars would have been so there's a portion of slab at the doors, where the doors of the lift were, there's the line 4 beam there and then about 1200mm out you can see the edge of the slab. This indicates that's the slab had broken away from this piece of slab before this rotated out.

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**COMMISSIONER CARTER:**

At level 6 two of the survivors were on that level 6 slab near that lift when the building started to come down. They were able to, you know, walk away from the collapse. Did you give some thought to whether or not when those slabs must have come down in connection with that witness's survival?

**DR HYLAND:**
Is that Kendal Mitchell. Is that who you are referring to with the two children?

**COMMISSIONER CARTER:**
I'll just have to check the names of the people but they were outside the lift on that upper level, one floor down was he?

**DR HYLAND:**
Yeah, Ron Godkin, David Hawesly were down at level 4 just right in front of the lifts.

**COMMISSIONER CARTER:**
So in fact the slabs must have fallen after he was able to escape?

**DR HYLAND:**
Yes.

**COMMISSIONER CARTER:**
He was in the building as it was collapsing around him?

**DR HYLAND:**
Yes. I think that's really important because, you know, he was quite clear that if the slab broke away from the core prior to the floor collapsing at the south then he would not be around.

**COMMISSIONER CARTER:**
Okay thank you. So, you've got that in there.

**DR HYLAND:**

Yes. So north core examination, okay, this is taken from photos and is not what I was able to see but we can see here that the slabs are leaning against the north core. This indicates there was a loss of support along line 3 which was out, prior to breaking off the core because the logic being that if the slabs had broken away first and line 3 had still been standing then we'd expect the slabs to have swung away from the core and not be leaning against it. Has a look at the drag bars. The interesting thing about the drag bars that I noticed was that you can see here in the top photo the anchors within the wall on the slab that was within the wall, was still upright and they weren't bent over. So it appeared to me that the slab had rotated off, in sort of, perhaps rotated around the tip of the wall and come away rather than being pulled out horizontally. Some of the drag bars had already been cut off during recovery operations. In the bottom right picture there you can see it just had just been gas-axed off and I understand from the USAR people you've talked to that that was just expedient to have those cut off for safety reasons. Another interesting thing about the north core examination was on the western side it was found that the beams that connected into the north core on line 4 were not fully developed. The reinforcing was not fully developed into the wall as had been specified on the drawings. What you can see here in the middle photo that I'm pointing to is the outer vertical reinforcing steel in that wall which was 300mm thick and if you look carefully you can see the imprint of reinforcing steel just almost next to that, but the reinforcing steel hadn't gone inside the wall. It was expected to have gone in another couple of hundred millimetres so that you can develop the capacity of those bars. When I looked at some of the debris that had been put aside on the site, it was actually, you could find these beams with the reinforcing steel that matched up with the imprints here. You can see how the end bars have been sort of almost bent back to do that. So that occurred on level 3 to 6. So, there's four levels where these bars hadn't been developed into the wall. Also took a level survey just to see if there was any sign of obvious settlement of the ground floor or the slabs during the earthquake. There were questions whether perhaps the site had suffered liquefaction and there had been some tilt or something but there was no discernible variation in the levels that indicated settlement had occurred. It was interesting though there was some displacement found in the northwards direction of the north core. You can see at the eastern end it says it's six metres above the RL. It was 33mm, 12 metres 62, 18.5 metres above the RL was 91mm. At the western end a similar displacement of 34mm occurred at six metres. At 12 metres the displacement was a little bit less at 52 compared to the other side and at 18.5 metres it was 68, compared to 91. It indicated there was actually a slight twist to the wall. We had a look at the foundations to see if there was any evidence of damage to the foundation beams and to see if there was any liquefaction or signs of uplift but we found no liquefaction material around the foundations, no signs of uplift, you know, obvious uplift and there was no signs of damage to the foundation beam. We had a civil engineer, John Snook, who undertook the examination of the foundation beams on our behalf. Materials testing. We undertook concrete cores from walls, slabs and columns. There was reinforcing steel and decking samples were taken and tested at SILI Global, in Christchurch, and the drag bar threaded anchors, there were two of those that I took and had been just harnessed check to see if they had properties consistent with the standards. The results, also used to help assess the drag bar capacities. Concrete quality: the wall concrete appeared to be reasonably consistent with what had been specified. I noted before there was some localised door-head damage that just appeared to be a localised issue. Slab concrete was, ah, there was limited testing able to be done on the slab concrete. You can see here where the cores were taken. By necessity they were in areas where there were cracks in the slab but we took cores out that were competent and we tested those. Slab concrete was, from what we could see, was sort of around about there. Beam concrete: we tested just one core in a pre-cast beam. The concrete seemed to be reasonable. Column concrete, however, we found some highly variable results and we cored the concrete and that led us to, on reporting to the expert panel, to say that maybe we should undertake more testing of concrete columns which we then did. We went out to, well, I went out to Burwood landfill where a secure area had been set aside for debris by Civil Defence and by CERA where debris had been left and on the left there is, just a photo of the extended debris. All this debris was from the CTV site. It appeared to have been placed there by truck load. I went through the debris field with John Snook and we just systematically walked over the debris field and marked out where we could find column remnants. Then I pointed those out to the operator who then went and extracted them for me. And we have here the column remnants that we extracted. So in all we had 26 column remnants that were able to be tested out of I think in the order of 109 columns that were on the CTV building so we had a significant sample. There are, obviously, the rest of the columns are somewhere in there, but we did not extract all of them out of respect to the debris and also just the efficiency of the investigation. The column concrete, ah, we had cores extracted and tested in accordance with the standard. We extended that testing by using rebound hammer testing calibrated to the core tests in accordance with the standard.

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Comparison was then made of the concrete test to the concrete production statistical limits operating at the time as set by the concrete production standard and the densities were checked as well. We found that in some cases the densities appeared to be low. We found that the concrete in the significant portion of the columns may have been strengths less than the minimum specified. What we've got here is a diagram showing the distribution of 25 MPa concrete or concrete with a mixed design for a minimum specified 25 MPa and we've got another one here for minimum specified strength of 30 MPa and another one here of 35 MPa concrete. So this is just the, the distribution of what would be expected for concrete made to a mixed design conforming with the standard for a 25 MPa strength concrete. This is a high grade, high grade curve. I acknowledge that the test, the batching plants; I've understood now, have a special grade things so the curves are just slightly different. However, the key point to note here is the black curve is the distribution of these strengths of the concrete that we, we found from our tests from our core tests and our rebound hammers and they're based on the average strength found in each of the 26 columns and the point here to note is that the, the, the curve is left of where we would have expected the curve to have been, expected perhaps the curve to have been over this direction. And when it came to the deciding how to use these results and the analysis we took this to the expert panel. It was decided well this is actually quite a result we weren't expecting and it was decided well we'll move ahead with the analysis on the basis, the minimum specified plus 2.5 MPa rather than using what would normally be expected which is 1.5 times the minimum. Reinforcing steel: I checked the reinforcing steel again just to see if the reinforcing steel would fit within the expectations of the standards it was produced to. We found that it did. The, one of the interesting things that was found was that the reinforcing steel, it didn't appear to have yielded except for some down the bottom of the south wall so reinforcing steel was extracted at a number of places had similar properties and similar yields but there was a piece of steel removed in this place, in this location at the east side of the south wall that we found had a slightly higher yield strength which was consistent with it having perhaps been elongated a bit during the, during the earthquake aftershock and its elongation was also less than the other steel that we had tested.

**COMMISSIONER FENWICK:**
Excuse me.

**DR HYLAND:**
Yep.

**COMMISSIONER FENWICK:**
This reinforcement was at the base, you said, of the wall?

**DR HYLAND:**
Yes, well near the base it wasn't right –

**COMMISSIONER FENWICK:**
Not in, not in the diagonal coupling beams?

**DR HYLAND:**

No it wasn't in the coupling beams.

**COMMISSIONER FENWICK:**

So you, the reinforcement out of the base and you've said the, the yield stress was higher than you expected?

**DR HYLAND:**

Yes, just in one location on the east side.

**COMMISSIONER FENWICK:**

How many bars did you test there?

**DR HYLAND:**

It was just one on each side so it was just a sample.

**COMMISSIONER FENWICK:**

And both of them you said. What sort of stress, what sort of yield strain were you getting?

**DR HYLAND:**

We were getting on average about 448 megapascals of the reinforcing steel.

**COMMISSIONER FENWICK:**

And it was a high grade bar?

**DR HYLAND:**

It was a grade 380.

**COMMISSIONER FENWICK:**

Grade 380 bar.

**DR HYLAND:**
Grade 380 bar.

**COMMISSIONER FENWICK:**
And a 448 you reckon is too high for it?

**DR HYLAND:**
No, no 448 was, appeared to be just the yield strength that had been produced to which was within the limited for the production.

**COMMISSIONER FENWICK:**
Yes. And the ultimate strain?

**DR HYLAND:**
Ultimate strain.

**COMMISSIONER FENWICK:**
Or peak strain.

**DR HYLAND:**
I can't, I can't remember anyway.

**COMMISSIONER FENWICK:**
Peak strain at peak stress, you can't remember?

**DR HYLAND:**
No, no.

**COMMISSIONER FENWICK:**
I was wondering whether there was an ageing effect coming in.

**DR HYLAND:**

No I don't, well the – with the, we seem to have a fairly consistent yield stress in the tests for the reinforcing steel, which indicated, you know, so there was some that was lower down the wall and some further up the wall

**COMMISSIONER FENWICK:**
Right.

**DR HYLAND:**
And that sort of seemed to indicate that perhaps that was just the, the yield stress of that batch.

**COMMISSIONER FENWICK:**
Okay.

**DR HYLAND:**
Whereas the, the portion I took out of the eastern end of the south wall down the bottom had a higher yield stress at test and a lower elongation than the rest of the reinforcing steel taken out of the wall.

**COMMISSIONER FENWICK:**
So it was either strained hardened or strain aged or both?

**DR HYLAND:**
Yeah, exactly so. It's an indicative point; I thought well that was interesting. Okay, so we now move on to the structural analysis portion. I'll hand over to Ashley.

**MR SMITH:**
This is a slide just, just explaining the various analyses we carried out and the various terms we're referring to them by. ERSA short for elastic response spectrum analysis, the type of analysis that we understand was used to design the building in 1986 and is still common, a common method of analysis for design of multi-storey buildings so that shall we say a design method.

Non-linear time history analysis, NTHA is using a model that has additional features that allow – to model the inelastic behaviour of the structure as it's pushed to quite high loads. Non-linear pushover analysis is using the same model as for the time history analysis but pushing it under a constant static load to gauge the performance of the building under that condition and various software was used to calculate the column drift capacity, moment curvature analyses by various software. In the drift compatibility is a reference to the analyses that was carried out or would be carried out in a design situation to, where you analyse with a response spectrum analysis using the shear walls only, and if the structural frame, the gravity frame is considered to be so called a gravity frame you then have to test the compatibility of that frame to sustain those same displacements. So that's just the –

**COMMISSIONER FENWICK:**
Sorry, can you explain that again, you said, analysis including the shear walls only?

**MR SMITH:**
Okay.

**COMMISSIONER FENWICK:**
What about the floors, what about the torsional inertia component from the floors. It must be a quite major feature when the -

**MR SMITH:**
Sorry, yes.

**COMMISSIONER FENWICK:**
– floors are 30 metres long?

**MR SMITH:**

Yes, no, I understand, okay. I'm saying the design, we understand the design was carried out and it is still a common method to assess the earthquake bracing system of the building carrying out an analysis with the shear, with the main shear walls as the seismic resisting elements and not considering an additional contribution of the gravity frames which are a smaller slender columns and beams.

**COMMISSIONER FENWICK:**

I accept that fully, I just wondered how you included the floors, you said you only had the walls?

**MR SMITH:**

No, sorry, I mean only the walls and not the columns and beams. Certainly the floors are modelled as an elastic bake to distribute those bracing actions.

**COMMISSIONER FENWICK:**

So it included the tortional inertia?

**MR SMITH:**

Yep. First of all I'll talk about the non-linear analysis which was the tool that was used to test the performance under the earthquakes or the, using the earthquake records that we had available. I'll talk about the earthquake records soon but basically this, this is the same 3D picture that we showed earlier. It is a 3D view of the model that was created in Sap2000 software. Compusoft Engineering, a company I share an office with in Auckland carried out the analysis. So we've explained these dots appear to have moved on this one but basically the model has a, what we call inelastic properties so that if we get bracing actions occurring and it goes beyond the yield capacity of the element the non-linear behaviour is modelled, which is not normally, or cannot be taken account of in a response spectrum analysis. This model is, we can see the foundations in blue at the bottom here and beneath that we have a series of springs which model the stiffness of the soil underneath the building and to the bottom of those springs we apply a ground acceleration time history. So we have got records from various sites nearby to the CTV building which I will talk about shortly and we are monitoring what happens to this model throughout that ground acceleration time history. I have just circled in, or I have put in red here, some of the key actions that we were looking for out of this model. I talked about this column at F2 this is grid F along here at grid 2, the column that runs from the top down to the bottom here. Quite a good indicator of what is happening in the north-south direction so we are picking up north-south actions because we have got very stiff beams here that induce stresses into the columns when those north-south actions, when those north-south movements occur as opposed to every other column on that line does not have a beam running north-south so is ready to lean over, if you like, without feeling much stress. So that was what we call one of our indicator columns. In fact this column here was, is virtually identical.

**JUSTICE COOPER:**
So what column is that?

**MR SMITH:**
Well this is line F2, this is line F3. F3 is where the mouse is at the moment.

**JUSTICE COOPER:**
Column C11.

**MR SMITH:**
Oh, sorry, yep, I am talking about grid references. I don't remember the column numbers. The other, looking for the east-west components we monitor this column at the grid intersection D2 so the column that runs down through here. In fact there are actually four columns, 1, 2, 3, 4, that have very similar vertical load on them, they are the most heavily loaded columns in the building, the internal columns get the floor coming from both sides. So that was another key one and that is picking up actions in the east-west direction because that is the direction of the beams that frame into that column.

**JUSTICE COOPER:**

Can we come, consider what you have said again? It is going to be helpful if we know what columns you are going to be referring to because we won't have a record of the perambulations of the mouse.

**MR SMITH:**

I see. Okay.

**JUSTICE COOPER:**

Is there a copy of....

**MR SMITH:**

I don't have the drawing.

**JUSTICE COOPER:**

We had done didn't we? Is that the document now shown to you? Yes. it has the suffix in our records of 0486.1 and has the advantage from my point of view anyway, even if I am the only one in the room, of having north at the top of the page and it has the columns marked on it. See the gridline going across the bottom and up the left-hand axis.

**MR SMITH:**

Okay. Let me just clarify then, grid F2, we are talking about column number C5 and grid D2, we are talking about C7.

**JUSTICE COOPER:**

Yes, all right.

**MR SMITH:**

Okay, let me write that down again.

**COMMISSIONER FENWICK:**

Are you moving on from that model or have you got a bit more to tell us.

**MR SMITH:**

Let me just explain these actions on the north core here. So we are monitoring the walls obviously but these, indicator columns and also this diaphragm connection on to the north core. We talked about the drag bars are able to sustain loads in a north-south direction only, so tension or compression shown by these two red arrows, and then we have the piece of floor slab that was connecting the rest of the wall, is able to take tension compression shear is an east-west direction and also twisting moments. Those were quite key components, we monitored each level throughout the earthquake.

**COMMISSIONER FENWICK:**

Can we talk a little bit more about the model please.

**MR SMITH:**

Okay.

**COMMISSIONER FENWICK:**

You said an inelastic analysis?

**MR SMITH:**

Yes.

**COMMISSIONER FENWICK:**

What type of hysteretic model did you use on this.

**MR SMITH:**

Okay, there might be a certain level that I can get to and a certain level you might need to talk to Compusoft but we had the separate session for the non-linear session but I can explain that the shear walls were modelled as like what they call, shallow limits, non-linear shallow limits so they were explicitly modelling the properties of the reinforcement and the concrete as individual shells that were able to model a stress frame behaviour of steel wall concrete as relevant. The beams and the columns used a moment hinge model and there has been some discussion about the – there was the option of using an axial load moment hinge also, we elected to use a moment hinge for the beams and the columns calibrated to the gravity load on the column because that still gave an accurate representation of the overall storey behaviour but then we had to go and –

**COMMISSIONER FENWICK:**
Was it a bi-linear, decader, or what type of model, what type of model did he apply.

**MR SMITH:**
Decader.

**COMMISSIONER FENWICK:**
How did you model damping?

**MR SMITH:**
Damping?

**COMMISSIONER FENWICK:**
Yes.

**MR SMITH:**
They used, I believe, tangent stiffness damping, that they calibrated to 5% at the point where we thought we were getting the first mode response. As it turns out the original analysis on review we had very low damping values at the frequencies that corresponded to the vertical vibrations, so we think that the further work we're doing with Professor Carr on the non-linear analysis has a higher damping value on vertical accelerations so those effects might

have been slightly over estimated in our initial analysis so there is some refinement being done to the model to –

**COMMISSIONER FENWICK:**
So your damping decreased as you got to the higher modes, did it?

**MR SMITH:**
Higher frequency modes?

**COMMISSIONER FENWICK:**
Yes, it usually goes the other way that is why I am asking.

**MR SMITH:**
It is actually a curve, high damping at high period comes down to a low value that corresponded to the frequency for the floor vibration and then it carries on up again.

**COMMISSIONER FENWICK:**
So you had high frequency, high damping at high modes and it came down and you had 5% at first mode and something below that in-between the two.

**MR SMITH:**
That is correct, yes. Now I am looking here at the – so I talked about the ground accelerations that were applying, were obtaining them from recording stations around the city that are monitored by GNS and this shows the locations of the recorders for the 4th of September 2010 earthquake, the Darfield earthquake which was centred where the green star is here. This represents the proportional scale to the peak ground acceleration so we are looking at the vertical arrows peak vertical acceleration and the horizontal blue arrows, peak horizontal acceleration. This gives you an idea that obviously near the epicentre high magnitude, oh, high intensity, and dropping off as we get further away, the CTV site is here. The three recording stations we used are the three that are coloured yellow and we are subsequently, with

Professor Carr, looking at including the fourth station just here so they are reasonably close to the CTV site. Now this is a – so that's for the September earthquake, the next slide is for the 22nd of February earthquake.

1459

The epicentre much closer to the CTV site, very much higher, a higher magnitude shaking or higher intensity shaking, again dropping off as we fall away but because we're a lot closer there's probably more variability in the shaking, even between these three recording stations. But it just gives a feel for, for the relative intensity of the shaking. The next slide I will show is an animation, if I can just explain a little bit about it first, it is one of the analysis runs we carried out for the 22nd of February 2011 earthquake or aftershock. It's a 3D, it's the same model that I showed that picture of, just an animated version of what that goes through, through the shaking. We can see lateral sideways movement of the floors. We can also see twisting of the floors and we can see strain on the columns indicated by when a circle appears on the column it indicates it's undergoing plastic deformation, bending. So if we can just have a look at that animation? It is quite quick so we might need to play it a couple of times through.

**COMMISSION REFERRED TO ANIMATION SLIDE**

So is, I think we're going to play it again. It is an amplified scale on the horizontal deflection so it's magnified so we can see them more clearly. I'll just get it played once more, the sequence, it's quite important in the sequence that the dots appear on the columns. Initially appearing on the east side and then progressing to the rest of the building. So we have, thank you for that. We have numerical output represented in the report that it basically puts that same information in a numerical format so it's just a visual representation of that. Now can you give me back the PowerPoint, sorry?

**COMMISSION REFERRED TO SLIDE**

This slide is a little model that was built to, just to demonstrate stresses and strains on columns and things. Basically, we're looking at two floors undergoing a lateral displacement or a drift, we call it. So the initial state is here where the column would be vertical. It's been pushed sideways and if we've got rigid beams connecting that column they force the column into this

S-shape here and that generates forces in ax – well, bending moments in shears, at all the time we have a vertical component which is the weight of the building on the column, and we also have additional vertical vibrations from the earthquake.

**COMMISSIONER FENWICK:**
Did you assume rigid beams?

**MR SMITH:**
No, we modelled the elastic flexibility of the beams.

**COMMISSIONER FENWICK:**
You modelled them as elastic?

**MR SMITH:**
With hinges, we ran it to find where the beams are highly stressed and located hinges. So not all the beams have hinges, but the ones that suffered elastic, ah, inelastic deformations had hinges for the model.

**COMMISSIONER FENWICK:**
And how did you model the beam column joints?

**MR SMITH:**
The beam column joints were modelled as linear, ah, with a rigid end zone assumed for half the width of the column which is, shall we say, a standard method in a linear analysis. We, we found it, the current modelling that is being done under Athol Carr is trying to accommodate non-linear behaviour of beam column joints but it is quite a complex area. It is not, there are not well defined procedures and it has taken quite some time for the experts to agree on exactly what properties to model. But our initial model only had linear behaviour in the beam column joint.

**COMMISSIONER FENWICK:**

Thank you, that's a point we might return to later on.

**MR SMITH:**

So this next picture is a, it's modelling some of the effects that we saw in that animation in a different form. We're looking at one particular column, F2 or C5 I think it was, at level 3, and we're looking at the record for the 22nd of February from this recording station CBTS. The wavy lines are plotting the inter-storey drift. So we start at zero and we're following, we're plotting on the blue trace the north-south drift which was critical for the column at F2 because it had the stiff beams framing in that direction. And we've got zero to 11 seconds of record there so this is a time axis, relatively small drifts, we're up to and this is talking about an inter-storey drift up to 1%, 2%, that's the scale so, ah, and we had calculated that the drift capacity of the column was somewhere between about one and 1.3% so that's represented by that pink band through there. The level of drift that we think the column is starting to feel distress and at risk of failure. And we can see that, we're just into the band in these two cycles, but well exceeding that band in these two cycles where it's, particularly where we've shaded orange. So that is a zone we've predicted column failure at this point here.

**COMMISSIONER FENWICK:**

Are you planning to tell us how you calculated those drifts? The limiting drifts?

**MR SMITH:**

Yes I can do that, yeah. We've taken one of the slides out but perhaps I can try and explain it here. The legend on the right-hand side is indicating that the orange zone indicates where the ultimate compressive strain in the column exceeded the value of .004. That's at about 1.3% and the width of the pink band is the effect, the possible effect, where the capacity is reduced by the bracing action of a spandrel between a column, so the upper limit at 1.3 or so where my mouse is now, the pointer, would be reduced if the spandrel fully braces that column down to this line. So we just make a judgement call if we, um. You know, when we back analyse it the full bracing capacity of the spandrel was unlikely to, to happen, and so we think we're nearer to the top end, but somewhere in that band depending on the stiffness you assume for the spandrel.

**COMMISSIONER FENWICK:**
So this is only the case you have got for where you are impacting the spandrel?

**MR SMITH:**
No, I'm saying –

**COMMISSIONER FENWICK:**
You haven't got a case where you are – I thought you were looking at the, one of the columns in the middle that was heavily loaded?

**MR SMITH:**
Oh, yes, this, this is, this particular plot is for the column F2 which is at the edge, with this column C5, sorry.

**JUSTICE COOPER:**
Column C5 that's the one noted, yes.

**MR SMITH:**
Okay, C5. We have similar plots for the internal column but I don't have it here.

**JUSTICE COOPER:**
Is there a slide that you would like to ?

**MR SMITH:**
It's in the report.

**JUSTICE COOPER:**
It's in the report and was it in the larger bundle of slides that's inside.

**MR SMITH:**
It was in the orig – ah, was it in the original? No it wasn't in the original presentation.

**COMMISSIONER FENWICK:**
Can you tell me, when you work into a strain, a limiting strain of .004 what exactly do you mean by that? Over what length does that strain occur and where does it occur?

**MR SMITH:**
Ah, we, well we had hinges modelled at the top and bottom of each column and –

**COMMISSIONER FENWICK:**
But you said they were point hinges?

**MR SMITH:**
But they represent a, a hinge length of round about 300 millimetres, hinge height, sorry.
1509

**COMMISSIONER FENWICK:**
How did you select 300 millimetres?

**MR SMITH:**
Well we, we referred to the Earthquake Society Guidelines. There were guidelines for estimating the length of plastic hinge in the *Earthquake Society Bulletin 2006, Earthquake Assessment Guideline* but there are other sources, um, just can't recall if we reviewed that with Athol Carr. I think we may have

shortened that up slightly since then to round about 200mm so I think we had 300mm originally.  I think we're now working on 200.

**COMMISSIONER FENWICK:**

And that was just purely a value plucked out of the Guidelines.  There was no scientific basis for that value apart from taking it from a published publication, so it's an empirical value?

**MR SMITH:**

Well we understood there was some science in behind the Earthquake Society recommendation so, ah, but that basically it came from there.

**COMMISSIONER FENWICK:**

We'll come back to that later.

**JUSTICE COOPER:**

Are you going to come back Mr Smith next week when we look at the NTHA process.

**MR SMITH:**

Yes.

**JUSTICE COOPER:**

So we need to go on to 52 thank you.

**MR SMITH:**

This is a plot of various actions on the column through the duration of the earthquakes.  We have the time-scale at the bottom 0-11 seconds.  The vertical axis has three different components spotted – bending moment in green, shear force in red or pink and axial load in blue and I think the point of interest is that the shear force and bending moment vary at approximately the natural period of the building so if we've got one and a half seconds period, the building is swaying and causing these moments and shears at that period so we can see these by the difference between these peaks here.  In fact this is all plotted on the positive side so it's every second peak represents one whole cycle.

**JUSTICE COOPER:**
Is this the same column?

**MR SMITH:**
Same column.

**JUSTICE COOPER:**
Column C5.

**MR SMITH:**
No C7, sorry, C7.

**JUSTICE COOPER:**
This is one of the internal columns.

**MR SMITH:**
Internal column, one of the most heavily loaded column at the base level, level 1.   I was explaining so the gravity load on this column is around about 1700 kiloNewtons so that's where it starts out where my pointer is now and these oscillations are caused by the vertical earthquake shaking.  I mentioned before about the damping value, quite a low, only one per cent damping on those vertical accelerations.  Subsequently we've carried out other analyses with higher damping and the vertical vibrations do tend to decay a little bit earlier so, as we say, this probably is a slight overestimate of the effect of the vertical earthquake.  I'm just trying to demonstrate that at any particular time step you have all three components acting at various magnitudes and it's a matter of combining those effects to assess your column at each time step and the point about the vertical being at a much higher frequency, so let's say between this cycle here we have four or five cycles of vertical.

**MR ALLEN:**

And you might not have maximum axial load at the same time as you have maximum shear force or bending moment?

**MR SMITH:**

That's right. So we look at each time step to see, and we're running up the vertical line here to assess how much vertical, how much moment, how much shear. As Gregor said we're looking here, we've got a high bending moment with a little bit lesser axial load. At this point they do almost coincide but this is partly because of our low damping value that we've still got high accelerations and vertical components at this point. We've probably slightly overestimated that I believe. So just some comments on the non-running of time history, um, it is quite a complex analysis but it is still not, ah, not expected to tell us exactly what happened to a building at any instant in time. We're looking for trends and general performance indicators rather than looking at the results and saying literally well at that step in time exactly that thing happened. It's not that accurate. The reasons, it's not, is because we're using earthquake records from other sites. They are nearby sites and they do have similar ground conditions but they are not exactly what occurred at CTV. We're just explaining here that they were used directly without any scaling and then we try – one way of trying to validate the analysis is to run the analysis for September the 4th and then to look at all the evidence we have of damage that occurred on 4th of September and try and gauge whether our model is giving us a realistic answer. That was a difficult process from two points of view – one is the limited reports, I guess. There was plenty of reports of non-structural damage, structural damage related to cracks in columns and walls relatively minor and what can happen is there's quite a big range that that can represent so a small crack in a column could be caused at a very low level of drift or a level of drift that corresponds up to the yield stress of the column and there's a range in between and we can't exactly say from that it's cracked. That was the maximum response during the earthquake so it is quite a difficult thing to calibrate. Some of the answers we got for the diaphragm connections in particular indicated huge forces in those connections. Again, it's to do with the analysis we set up, has approximations on the stiffness of the floor at those locations and how those interact with the walls is quite critical to determining the forces that come out of it. And from my point of view I was convinced that the magnitude of the load in those diaphragm connections, I believe, exceeds even at Code level shaking that the magnitude of the force was quite revealing. They can be very high forces in those diaphragm connections and I think that's an area that we need to look at in the standards, even the current standards for that, but they are quite unpredictable; the analysis cannot accurately tell us exactly what those diaphragm forces are very easily.

**COMMISSIONER FENWICK:**
Excuse me, if you suspected forces were too high because the floor was too stiff then the answer is to repeat the analysis with a softer floor to see how sensitive, so what happened when you did that?

**MR SMITH:**
Yes, the forces do come down but they are still very high so –

**COMMISSIONER FENWICK:**
So you still don't believe them even when you've softened the floor up?

**MR SMITH:**
No, I believe them. I believe that the forces are very high in, what do we say, I believe very high forces are, can be generated in diaphragm connections and that has been shown in our modelling and I believe that's accurate. Exactly what magnitude is quite difficult because we don't know exactly how stiff the floor is, so there is some variables in there that make it hard to determine the magnitude of the force.

**COMMISSIONER FENWICK:**

But you should be able to bracket it.  This is a very stiff floor, this is a very flexible floor, this is what we think it is.  You should be able to get a range of what values are.

**MR SMITH:**
Yes, yes, yes. I think we are looking at that in more detail now I believe again, um, the floor has been modelled softer now than it was in our original model and it is showing some reduction in those forces but they are still very high forces.

**COMMISSIONER FENWICK:**
Have you followed up on any of the recent research into diaphragm actions and forces at Canterbury?

**MR SMITH:**
I'm not, ah, I'm not sure exactly which one you are referring to or which papers or what but...
151905
1519

**COMMISSIONER FENWICK:**
Well there has been a recent project on that topic.

**MR SMITH:**
Okay.

**COMMISSIONER FENWICK:**
Which comes out with a set of recommendations as I understand it for diaphragm forces.

**MR SMITH:**
Okay.

**COMMISSIONER FENWICK:**

So maybe we need to put you in connection so you can chase that through?

**MR SMITH:**

Look at that, okay, sure. Now where did we get to, um, yeah but so, so again about these high forces, ah, we did calculate very high diaphragm connection forces, and even the refined model is still coming up with very high diaphragm forces and exceeded the breaking capacity of that connection, ah, and that led us to believe that the collapse, ah, because, ah, as Clark mentioned there were, there were witnesses that were in the vicinity of that connection and survived, it appeared not to be the first thing that broke, so the collapse we believe, was initiated elsewhere prior to those maximum forces being, being reached.

**DR HYLAND:**

We also did the ERSA analysis to look at compliance with the standards that they were designed to. I mean the primary task that we were given was to look at how did the building collapse, how did it get there, but we were also asked to, um, see how the building complied with the standards. So to do that we needed to use the tools that were used by the designers, ah, or that were required or recommended at that time it was designed. So we took a 3D elastic structural model using ERSA and applied the design spectra from the 1984 loading standard NZS24203. We also just had a look at, well, what is the comparative difference in the response of the building between the September, the December and the February aftershocks to get a feel for the relative, um, you, know response that you might expect, and found that there was, ah, the September earthquake response was two times the response you get from a December record and the February aftershock was 2.2 times the response you'd expect from the September one. So there was a doubling of the demand expected on the structure between February and September. So it's quite a big step in the effect on the structure. In looking at the loadings it's important to recognise the way the loading standard works, in that the loading standard sets out loads that are expected to be able to be sustained without collapse of the structure if it's designed in accordance with the standards, um, with a low probability of them being exceeded in the life of the building and it's set at a 10% probability of exceedance in 50 years. What we have here is just an indication that there's the, that's the 1984 loadings, we'll call it the full load. The way the loading standards are set up is that they are able to – they're able to discount the level of load that you apply to your structure if you increase the level of ductility in the structure. So you increase its ability to, or the resilience of the structure, or the toughness of the structure so it can sustain damage, yet still remain a safe structure. So the, with the structure that was designed, the CTV structure, was actually designed for this lower level of loading down here, indicated by the lower green dotted line, which is 20% of the, of the full design load level. And so it was required to have a high level of ductility built into the structure. So you are able to discount the level of loading applied by trading off with higher a level of toughness, and that is developed by using ductile design, or limited ductile design. So it was designed, we found, using the ductile response, which is the lower level of loading. Now we've got here, on here superimposed some spectra developed to compare the spectra that were recorded in the various events. So we have the September spectra here in light blue. We have the, sorry, there's the September spectra in green. We have the December spectra in red, and we have the February spectra in blue. You can roughly see the proportions at the one second natural period of the building, roughly around one second. You can see that those rough proportions that are talked about, the, the September one being roughly twice the size of that, and the February one roughly twice the size of the September amount there. Now there is a subtlety here and that these are earthquake records and we have a design spectra. The design spectra is an amalgam of study that has been done by experts to work out what responses they expect buildings will form at for a cluster of earthquakes. So they're not actually a specific design earthquake but they are a design response of a structure. If you apply these loadings, the loadings actually derived from these design spectra, you come up with specific demands on structural elements that determine stresses and you size the reinforcing, you size the steel and the concrete, all those demands and, of course, to see whether a structure has met that, met that level of expected demand, you would see what damage has been caused in those members after a particular earthquake.

**COMMISSIONER FENWICK:**
On the right-hand side of that diagram you've got some bits I can't quite read. It looks like to me NZS4203, this is the fourth one down, 1984 –

**DR HYLAND:**
Yes.

**COMMISSIONER FENWICK:**
– S=5, what is "S"?

**DR HYLAND:**
Oh, right, okay the –

**COMMISSIONER FENWICK:**
Are you saying this is a structural type and a material type rolled into one or what?

**DR HYLAND:**
No it's a, it's a ductility factor built into the standard, so in –

**COMMISSIONER FENWICK:**
SM, I know what it is, four over SM, but tell me what does "S" represent?

**DR HYLAND:**
So the "S" is the structural factor, um, so if you have a factor of 5 it means it's, um, fully ductile if it's S1 it is –

**COMMISSIONER FENWICK:**
So when you have S=5 that's the elastic response is it?

**DR HYLAND:**

Sorry, the other way round, yeah, S of 5 is the elastic response, the strength design.

**COMMISSIONER FENWICK:**

Can we look at that, you're talking about a building in Christchurch which at that stage was zoned B in the nought to nought point five second response it would be 12.5% G. Five times that does not look like something close to 9, .9 to me?

**DR HYLAND:**

Right okay, I'll explain, what we've got here is the CTV response. So this has been, um, the spectra has been scaled, um, to be equivalent to what you'd put into your design analysis so that you get an equivalent static response base shear, of equivalent to the, or 90% of the static response. The difference being that with the response spectra analysis what we're doing is we're actually taking a, an average of response at different periods of the structure. So you're getting a, an average response which means that the design spectra actually put into the analysis is actually slightly higher than just the base spectra you'd use for the static.

**COMMISSIONER FENWICK:**

You are not taking the design spectra in the Code, you're taking something different?

**DR HYLAND:**

No, no, it's the design spectra from the Code but because we have to scale the response –

**COMMISSIONER FENWICK:**

You're scaling the response to .9 of the equivalent static value?

**DR HYLAND:**
Yes, yes that's right.

**COMMISSIONER FENWICK:**
And you've included that factor in here have you?

**DR HYLAND:**
Yes, yeah.

**COMMISSIONER FENWICK:**
So you've found your analysis, found your load, found that it's higher or lower than the equivalent static –

**DR HYLAND:**
Yes.
1529

**COMMISSIONER FENWICK:**
– and you have then scaled that back into the response spectrum?

**DR HYLAND:**
Yes, yeah, just to say give a comparison to the spectra so if the reason for doing that was to show, well if you were to take the record and put that spectra into your ERSA you're wanting to sort of get an apple for apples comparison, because you'd use an unscaled spectra, so you use the –

**COMMISSIONER FENWICK:**
Just multiply all your values by your correction factor?

**DR HYLAND:**
Well the, yeah, I think the approach is correct here. It's just, yeah, it's just to show that the factors, you take the Code spectra as your base, you then do, have to manipulate it so that you're getting a base shear which is 90% of the

static at the first mode and so this is just really sort of trying to show all these –

**COMMISSIONER FENWICK:**
What is, what 90% at the first mode?

**DR HYLAND:**
Well your static –

**COMMISSIONER FENWICK:**
Your equivalent static is a value which is dominated first mode but not entirely, okay, it's an envelope value so you're going to 90% of that and you're applying that factor to the response spectra?

**DR HYLAND:**
Yes.

**COMMISSIONER FENWICK:**
Normally I would have said to scale all the actions up by that but it comes to the same thing.

**DR HYLAND:**
The same thing.

**COMMISSIONER FENWICK:**
I just needed to know what you'd done, thank you.

**DR HYLAND:**
Yes, no, thank you. I guess the point I'm trying to make here is we do need to be careful about the way we take an earthquake record and say that is equivalent to a design spectra. There is a subtle difference, one is a deliberate recognised assessment of what demand should be imposed on a building for

design purposes, the other one is, is a spectrum and we're not sure how a building responds directly to that particular spectra.

**COMMISSIONER FENWICK:**
Yes, there are a whole series of questions that arise where you have a ductile wall one end and something which is obviously non-ductile the other end because that's basic strength and you're treating it as a ductile structure which clearly doesn't seem to make sense to me at any rate.

**DR HYLAND:**
Yeah, definitely, and issues of damping will help. We just use a base 5% damping in design but what level of damping is really there.

**COMMISSIONER FENWICK:**
Right, thank you.

**DR HYLAND:**
Okay, compliance checks to the standards, so we used the checks. We found the walls compliant with the inter-storey drift limits within the limitations of the standard there are some issues there. We found the non – the columns weren't complying for the size of expiral reinforcing limits would expect. We found the columns weren't compliant for spiral reinforcing for shear under the imposed drifts. So the requirement of the standard was that you checked the drifts of the primary structures so with the shear walls, that there was a certain amount of stiffness provided that would then be assumed to protect the secondary elements or the secondary frames, call them group 2 elements so that check was normally okay. We then found, there was another check that said at those particular drifts that were imposed the columns were expected to remain subject to elastic theory, with the requirements of elastic theory we're talking about, but we found that they didn't comply with that. Now if you didn't comply with that then the Code required you to then design those frames, not just the columns, the columns, the beams and the beam column joints for a limited ductile requirements or fully ductile requirements which meant you

were required to put more reinforcing, more confinement reinforcing in the columns, the beam column joints and the beams.

**COMMISSIONER FENWICK:**

Sorry to interrupt again but I really want to get exactly what you've done. When you did your analysis inter-storey drift you followed the standard of the time, 1984, you took the gross section of the beams as .5 of the, sorry, the effective section of the beams as .5 of the gross and you took the full gross section for the columns. Is that correct?

**DR HYLAND:**

No, what we did was we took a more generous approach. We said what will be –

**COMMISSIONER FENWICK:**

You didn't follow the recommendations in the standard?

**DR HYLAND:**

No, we did originally but because we were trying to see, well, what was, because the standard had recommendations, it didn't set 'You must do this, you must do that' so there was some guidance there and there was some potential for someone to interpret that in a variety of ways. So we said, well what would be the most extreme way you could interpret the properties of those, the cracked section properties of the column and beams. So we looked at the columns and we used the sort of the more modern method and said, okay, if you did your moment curvature analysis what would be the predicted cracked moment of inertia that you would get and use that.

**COMMISSIONER FENWICK:**

And so you've varied your eye down the length depending on the amount of cracking did you? I mean it's more I gross over three-quarters the length of -

**DR HYLAND:**

Yeah, well basically we used, I used a software package which was Compusoft web package then back worked it from there and said, okay, well there's what I'd say would be a lower bound this'd be, if you really sharpened your pencil, you might be able to get to this level.

**COMMISSIONER FENWICK:**

Now the fact that you're, if you follow the standards you're only checking for, well one can argue it, 55% of the peak drift and this of course 55% presumably went with an I gross value so can you square that one?

**DR HYLAND:**

Well the –

**COMMISSIONER FENWICK:**

If you're suddenly using a much more flexible and modern interpretation I don't know how good your modern interpretation is but you're using a more flexible interpretation then is it right now to still only be checking for 55% of the peak drift?

**DR HYLAND:**

Yeah, this is interesting point. I think in the context of the standards of the day, looking back, the indications were that they were really talking about using a working stress approach design to the columns, so, which seem to be encapsulated in ACI-318-71, where you'd use 55% of the yield stress of your reinforcing as your limit and 45% of your –

**COMMISSIONER FENWICK:**

No 55% comes from the allowable inter-storey drift. It came from the allowable displacement from a boundary which was considered to be, it took initially at 50% but when they introduced the response spectrum they took it up to 55%. That was the allowable displacement, was distance for a boundary on the basis that usually if you had two buildings set at 50% they wouldn't often hit and so they felt going to whole 4 over SM rather than 2 over SM would give too wider a margin, wouldn't be economic on a lot of sites because of the gap that was required and that value was then picked up in 1984 and used for the inter-storey drift, probably without much thought that it wasn't the full value. I don't know. I'm speculating.

**DR HYLAND:**
Yes, it's interesting; I mean we've been trying to work it out –

**COMMISSIONER FENWICK:**
But I mean, I'm just concerned that you can pick a flexible value here and use it with an unconservative value here, it doesn't seem to me quite a logical but, you know –

**DR HYLAND:**
I think the general practice was people would take the conservative approach but it's possible people could have said well we'll use a more flexible approach to our column and they could have been justified, the Code wasn't that specific about how you had to do it.

**COMMISSIONER FENWICK:**
And they wouldn't have had your software available for determining what stiffness the column was.

**DR HYLAND:**
No. They could have used cracked section properties though there was a formula in the Code there for determining crack section properties so I cracked and M cracked –

**COMMISSIONER FENWICK:**
I gross.

**DR HYLAND:**
And I gross, so I don't think it was done however.

**JUSTICE COOPER:**
Have you said all you wanted to say about this slide -?

**DR HYLAND:**
Yes.

**JUSTICE COOPER:**
-as far as you know?

**DR HYLAND:**
Yes I do, I'll move on from there.

**COMMISSION ADJOURNS: 3.38 PM**

**COMMISSION RESUMES: 3.55 PM**

**MR ALLEN:**
Before the break we were discussing the approach you took in order to obtain results that might be considered most favourable to the designer. What were the results of that approach?

**DR HYLAND:**
The results of that approach were that the columns were still non-compliant with the requirements of standard.

**MR ALLEN:**
Did you adopt other approaches that were perhaps less favourable, perhaps more in line with common practice.

**DR HYLAND:**

Yes we did and, of course, that meant it was more obvious that the columns were non-compliant with requirements.

**MR ALLEN:**
All right, well those are my only questions remaining on that slide.

**JUSTICE COOPER:**
Right, so we are going to move on to slide 57.

**DR HYLAND:**
So what we have here is, just shows the, how the drift capacity changes as the level of axial load or compression in the columns increases. You can see here at drifts of columns of, between sort of, .1 to .35% we are getting cracking occurring in the columns.

**MR ALLEN:**
The different colours in this graph, do they represent different axial loads.

**DR HYLAND:**
Yes that's correct, yes. You can see where I have circled the second circle, you can see there the blue line down the bottom is, for example, a column with a low axial load and it has got a drift of near one and a half percent before it reaches the limiting concrete strain of .004, whereas the higher loaded column with the sort of mauve line is only getting up to 1.15% before it reaches that so you can see the, as the axial load increases on the column the ability of the column to sustain drift is reduced. And the implication of the standard was that you should be really be able to get to 1.5%, 1.51 safe drift performance of columns if they were designed in accordance with the standard. So these columns do not appear to have had the performance capability to cope with demands required by the standard. This is showing it in a different way, that drift capacity reduces with increasing axial load and you can see that this is the column F2 which is column C5 at, for 995 kiloNewton load level 2 as the drift increased up to around about .6% it would

then, the concrete would reach a yield strain of .002. As it was pushed further the steel would begin to yield at a drift of about .8% and if the column was then pushed up to 1.2% it would reach its limiting concrete depression strain of .004. So, just shows just a progression of damage that would be expected in that column as it was pushed. If we go to – or what would be the, or what would happen if the concrete strength was less than what had been specified and, for example, if we took a strength right at the lower bound of what we found from our tests of 14.2 MPa, the five percentile, you can see for the same axial load, the same column, C5, the column would reach its .002 yield in the concrete at .48%, a lower level, and it would not yield the steel before the concrete and the column fracture at a drift of 1.50%. You can see the effect there of a lower than expected concrete strength in drift capacity.

**COMMISSIONER FENWICK:**

Sorry to interrupt you again, this, when you reach your .004, which you've taking as your limiting value, does that represent where you lose the lateral load capacity or where the axial load capacity ceases to be effective?

**DR HYLAND:**

Well, okay, so this is a flexural compressive model that's been used in the software, this has come out of the Cumbia software package that was developed by Kowalski who was a student of Priestley and it is sort of, I believe, you know, is sort of right up there in terms of current approaches to it but its recognised that shear, the shear capacity of concrete under axial load and compression is still difficult to quantify accurately. So what we have done is we have used this .004 concrete compression limit as what we think is a reasonable, was the best reasonable approach we have got at the moment, recognising that, you know, it may get, that sort of model may improve in time but this is what we have got, we have got to use the tools we have got. So yeah, and the other thing I think is important to recognise that this model is, this model is also based on research that has been done with columns which had reinforcing contacts, spiral contents actually higher than what we found in the CTV building so the CTV building, this may be actually conservative in terms of measuring the performance of the CTV columns.

**COMMISSIONER FENWICK:**
So you are taking this then as an axial load capacity limit? Fair enough, I just wanted to –

**DR HYLAND:**
Yeah, yeah. Estimates of – what was the drag bar failure estimates and Ashley has said look, this is not an easy area to quantify with the software but one of the things we did find is that there does seem to be a consistent drift that occurs when the drag bars reach their nominal capacity and that was a drift along line F of 1%, about in the order of 1%. We are not talking 2% we are talking around about 1%. So that to me indicates a geometrical, an issue that there is a certain drift, there is a certain, you know, drift at the, where the drag bars were. So we see the 1% drift is from what we have just shown with those columns is actually consistent with the columns having failed before the drag bars let go so it is consistent to say that collapse would have occurred due to column failure on line F before the drag bars detached. And that is important because we do, there is a credible basis for saying that, collapse occurred before we got drag bar failure. If we encounter along line F, effects of spandrel panel contact as well then we found that the spandrel panel contact means collapse would have occurred before that, the drag bar failure level, for a level of 1%. So there is quite a credible basis to say collapse could have occurred before the drag bars detached, before, yeah, I will just leave it at that. So summarising the non-compliances, we found the light spiral binding in the columns of R6 at 250 was non-compliant. The requirements and the standard to utilise ductile, or limited ductile design of those secondary frames would have meant that you must have had greater amounts of spiral reinforcing.
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We found that the, that the spandrel panels did not have adequate separation. There's no specified specification between the spandrel panels and the

columns specifically. There was on the architectural drawings a gap between the ends of each spandrel panels. There wasn't a gap specified between the end of the spandrel panel and the column as required by the concrete standard. You can see the potential contact points there.

**JUSTICE COOPER:**
Did you refer to the architectural plans in making that comment, what about the engineering, the design, the plans of the engineering design?

**DR HYLAND:**
There was nothing specified on the engineering plans that showed a, a gap to be made between the ends of the spandrel panels and the columns.

**JUSTICE COOPER:**
And that's an area of noncompliance with NZS4203 is it?

**DR HYLAND:**
NZS3101.

**JUSTICE COOPER:**
3101, the concrete stand?

**DR HYLAND:**
The concrete stand, mmm.

**MR SMITH:**
I think you could interpret from the structural drawings they had obviously grid measurements, column dimensions and spandrel dimensions. So you could piece them together and determine what the gap would theoretically be, but it wasn't shown in the detail, that it had to separated.

**JUSTICE COOPER:**

Well, are you saying that if you did that you would probably have, in building it, provided for a gap? In other words, you're talking about piecing together various details?

**MR SMITH:**
Um, no, no, I'm saying there was a drawing with spandrels, there's a drawing with columns and there's a drawing showing grid dimensions but, ah, there are construction tolerances with each of those items and there was nothing to say that (inaudible 16:07:12)

**JUSTICE COOPER:**
Okay, you have to specify that in the specification I take it, and it wasn't specified?

**MR SMITH:**
That's right.

**JUSTICE COOPER:**
Is that right?

**MR SMITH:**
That's correct.

**DR HYLAND:**
So, the spandrels panel are, ah, you can see them being put there on the building back in 1987, on top of the beams. The other area of noncompliance was this infill masonry wall. The drawings actually show the top was supposed to be grouted up. It appears that perhaps it wasn't grouted during construction because the workers on the day, Leonard Fortune and Bruce Campbell, found that there were, seemed to be hollow at the top. The most important thing that there were supposed to be 25 millimetre gaps between the masonry and the columns, ah, and 10 millimetre gaps between each of the three panels in each bay. And from the outside we're told by these two

people who were working on it at the time, there were no gaps visible on the outside between the masonry and the columns. They also said there was no obvious damage from September. Now on the other side, on the inside, there were gaps observed, so it appears that perhaps the gap had been perhaps compromised by some mortar or something during construction because on this side of the wall there was a building in place during construction, so they wouldn't have been able to get to the other side to perhaps clean those gaps out. The effect of that masonry wall, um, means that it's possible it could've increased the torsional eccentricity of the building further by bringing the centre of rotation further westward, which would've increased the drifts on the eastern wall, over and above what was already a reasonably eccentric building.

**MR ALLEN:**

Just before you move on from there Dr Hyland. How did you account for the possibility of the existence of gaps in the masonry infill wall when you were modelling?

**DR HYLAND:**

Right. Well for compliance checks we ignored it. We, because we need to do a compliance check just based on, um, what the designer assumed prior to construction, assuming that construction was done in accordance with his design. For the purposes of, just investigating what would be the effect of that, we use different ways of looking at the masonry infill. For the ERSA it was more of an investigative way of looking at things. For the NTHA there was a lot more effort put into trying to consider what effect that would have, and, you know, there's many ways you could possibly think about that. You can say it was very rigid, or you can say it was degrading over, at a certain load or you can say it wasn't even there. Ultimately the design analysis that, or the analysis that was done in the NTHA used two, ones without any masonry infill and ones with a masonry infill of a certain sort. Okay, the drag bars. These were interesting because they were added after completion. The building was constructed in 1986/1987 but the design defect was found in

1990. This was a defect, um, it was quite a serious defect, because the building could separate from the north core in quite a low level of earthquake. So remedial works were designed in 1991 as I've, sort of, given that explanation before. So these steel angels were epoxyed in. Interesting thing, there were no drag bars designed to install to levels 2 or 3 and that was, on my first inspection I'd wondered why there were only drag bars at the upper levels. We looked at the design calculations. It appeared they'd used the parts and portions section of the loading standard NZS4203 1984, and had been able to justify using that, that they only did require them at levels 4, 5 and 6, using their analysis. I'm not saying that their analysis was correct, totally correct in all respects but it did highlight there did seem to be a problem with that portion of the loading standard that related to design of connections of diaphragms into the, um, into shear walls. I think that's an important point. The implication, however, from this is that the diaphragm drag bars were there, um, if the collapse had not occurred prior to them detaching, that's at drifts of 1% or so, in the columns, it's very likely that these would've failed quite soon afterwards. There were, what we found is there were a number of vulnerabilities in this building that were ready to happen, yet, it was just the first one off the block was, well, we believe to be the columns, and then there was a number of things including drag bars and various vulnerabilities that, um, would've been found out if something else had not failed before. The other thing about this is that there wasn't any building consent application on the Council file so we couldn't find this out from the designer; we had to ask questions around, we didn't, sorry, – we didn't find it from the Council files, we had to ask questions. We eventually found it through a review that had been done for a pre-purchase agreement, a pre-purchase inspection so this was, um, information that wasn't in the public realm for people to evaluate, ah, when looking at this building. So, a summary of the vulnerabilities that we identified in this structure, following our analysis. First of all the south wall was much weaker and more flexible than the north core. You can see it's reasonably short length compared to this quite big thing here.

**MR ALLEN:**

Is this the asymmetry that gives rise to that torsional twisting you spoke of earlier?

**DR HYLAND:**

Yes, yes, because this, um, has a low, comparatively low stiffness compared to this item, the centre of stiffness was found to be over, over near the north, over near the north core rather than in a normal building you try to get your centre of stiffness near the middle of the structure, between your structural elements. It was weaker than the north core as well which meant it would yield and deform plastically in a way that was difficult to predict using the elastic response spectrum approach, using the standard at the time. So, it meant that because it's gone outside the symmetry of things, the requirements they'd made this quite vulnerable to unexpected performance.

1615

We found that the column spiral reinforcing was insufficient in the columns. We found that some of the concrete column test strengths were low. We found that the beam column joints were fragile. We found that there was no seismic separation gaps specified between the columns and the spandrel panels so we don't know if some of them were hard up against the columns or if there was a gap. In the interview I had with Bill Jones he said, the construction foreman, he said, in his view they probably just lined them up to get a good line. He wasn't aware of any requirement for a seismic gap. The seismic separation, of the masonry infill wall appeared to have been compromised on the outside face. We found smooth construction joints in the south wall which may have increased potential for slippage. We found the drag bars had limited strength. They may not have been the critical element in this particular collapse but they didn't have enough strength to develop the full capacity of the structure at a, at a high load and so they were, they were a vulnerability. We found that the beam reinforcing had not been connected into the west side of the north core, in the bottom from levels 3 to 6. So those are the vulnerabilities we found from the investigation.

**JUSTICE COOPER:**

Can I just ask you a question about the drag bars, there are two shown, where are they in terms of grid lines?

**DR HYLAND:**

Right, that's, they're actually located, sort of, on the wall, on the wall on each side of the lift, lift shaft, so that's a wall we call a wall on line D.

**JUSTICE COOPER:**

Yes.

**DR HYLAND:**

And wall, that's half way between gridlines D and E so we've it D/E, D slash E.

**JUSTICE COOPER:**

Yes.

**DR HYLAND:**

They were just attached to those, those two.

**JUSTICE COOPER:**

And their location there that's where those four what have been finger walls where they don't actually go out to meet the floor the way the ones on the other side of the toilets a, is that – ?

**DR HYLAND:**

Yeah, that's quite correct, the slab there, yeah, there's, yeah, they don't go, reach out to the beam or the column out that the, on line four, they, they just had a slab there that –

**JUSTICE COOPER:**

So they're designed to supplement the strength of the structure at that point for that reason, I mean what, why only the two?

**DR HYLAND:**

Only the two.

**JUSTICE COOPER:**

And not more once the problem was identified?

**DR HYLAND:**

I think what they, what they would have seen was there was a vulnerability for the floor to twist and break away from the eastern side of the core at those two points so putting those drag bars in well then meant you could engage the, engage the, the core there.

**JUSTICE COOPER:**

Yes.

**DR HYLAND:**

Whereas there was a slab in the area between C and, yeah, well the toilet area.

**JUSTICE COOPER:**

Yes.

**DR HYLAND:**

So there was, sort of, a route for the loads to get in and get out of –.

**JUSTICE COOPER:**

Right.

**DR HYLAND:**

–there, which was more competent.

**JUSTICE COOPER:**

Okay, right I follow thank you.

**COMMISSIONER CARTER:**

I've a couple of questions sir that you could perhaps help me with, one is regarding your assumption of the fixing of the spandrel when you considered that the column might impact on the spandrel and therefore fold around the spandrel causing the fracture you showed us. That assumes that the spandrel was able to resist that force. The spandrel itself's got blanked end walls in it which are fastened just by two pretty simple brackets to the floor slab.

**DR HYLAND:**

Yes.

**COMMISSIONER CARTER:**

The columns on one, all along one wall would presumably all be deflecting about the same amount, so I'm assuming that for the spandrel to stay put and the column to push against it there would have to be enough strength in the fixing of the spandrel to the floor to sustain that force. Did you actually go through that little bit of analysis?

**DR HYLAND:**

Yes, yes we've done a number of things there. The, um, there's a number of things there, first of all you've got the bolts, you had four bolts there that if they all engaged and were able to develop their strength, there was more than enough strength there to develop the (inaudible 16:20:07) necessary to cause that fracture in the northern core.

**COMMISSIONER CARTER:**

And the end walls could convey that load down from the sill level down to the floor when you've fastened on to those blanked off walls at the end of each spandrel?

**DR HYLAND:**

Yeah, that, I mean that's, that's where it gets a little tricky because you could say with those end walls, sort of, bend and flex and so not perhaps pick up as much trauma, so much load into those particular bolts –

**COMMISSIONER CARTER:**

Okay, all right, so I think we're talking about the same subject.

**DR HYLAND:**

Yeah, yeah.

**COMMISSIONER CARTER:**

The other thing, the west wall block work each bay of block work was divided into three panels which there were construction joints anyway, two intermediate construction joints. Did you assume that in your analysis in which the, there was a, supposedly a gap between those panels, there was certainly an elastic filler against the columns? Did you assume there was also one in between the, in the construction joints between the three segments?

**DR HYLAND:**

Yes, yeah, we did, we, in each case we said, we broke it into three panels. The interesting area is what is the effect of the, the header beams on the performance of those panels because there wasn't a gap specified at the top.

**COMMISSIONER CARTER:**

No.

**DR HYLAND:**

So for those, for those to, to drift even if there was a gap there you'd be preventing rotation at the top of the panels which would have stiffened them up so you could get a stiffening of it, you know, beyond just a simple flexural sort of model.

**COMMISSIONER CARTER:**

(Inaudible 16:21:55) some mortar up onto the beam.

**DR HYLAND:**

(Inaudible 16:21:56) model.

**COMMISSIONER CARTER:**

Okay, thank you.

**DR HYLAND:**

Yeah, so I mean there's been quite a bit of discussion around this area I imagine.

**COMMISSIONER FENWICK:**

Can I just follow up on that question? You have the column going over and bearing against the spandrel, presumably you have a plastic hinge at the top and the bottom of the column but you have to apply quite a high force from that spandrel to the column, then that force then, as you say, have got to be transmitted to those four bolts, two at each end through a 100 millimetre thick diaphragm reinforced with three 12 millimetre bars horizontally and three vertically, which have to be bent within the 100 millimetre reinforcing, so they actually go round the corner and they have to cut the corner.

**DR HYLAND:**

Mhm.

**COMMISSIONER FENWICK:**

So you actually have quite a lot flexural capacity at that corner. Now can you give me an idea of what sort of magnitude force did you require to actually form the third plastic hinge in that beam?

**DR HYLAND:**

Well the thing is, we didn't, yeah, we didn't need to form a third hinge, all we're saying is all we needed is for those spandrel panels to cause contact, to then start the short column effect off so that we could get the, the hinge at the top of the column starting to increase at a much more rapid, you know, much more increased rate. And then for perhaps the vulnerability at the ends of the reinforcing bars at the end of the splicers to perhaps lead to some localise spalling and loss of capacity at that point. So yeah, we, we couldn't find that you could justify a plastic hinge right at the top and the bottom of the column or at the top of the spandrel panel, at the same time, but we could, well, I believe, you can justify there being a stress raiser developed at the end of those vertical reinforcing bars that, along with the enhanced development of the hinge at the top of the column, then led to a loss of capacity and a loss.

**COMMISSIONER FENWICK:**

So you're predicting a shear failure are you, is that you're getting at?

**DR HYLAND:**

I'm just looking at a, at a –

**COMMISSIONER FENWICK:**

Not a flexural failure it's something different then?

**DR HYLAND:**

Yeah it's a, a failure at the termination of your vertical reinforcing bars initiated by the bars being unconfined or not having significant confinement being able to perhaps, sort of, move a little bit, hop off the cover locally, leading to a loss of compressive strength at that point, which then sort of just started a…

**COMMISSIONER FENWICK:**

So if I've got you right the, the lapping position you say weakens the flexural strength because you'll lose the bars and lose them at –

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**DR HYLAND:**

Yeah, the bars –

**COMMISSIONER FENWICK:**

That's the theory is it?

**DR HYLAND:**

Yeah, so it's just a weakened point, which I think that's an explanation, I'm just trying to find something that perhaps explains what we saw then with those columns.

**COMMISSIONER FENWICK:**

Yes it does look as though those end panels are very lightly reinforced and when you look at the reinforcement in them you think, well, it can't take much when the bolts sure can take a lot but getting it through that end panel looks, looks pretty weak.

**DR HYLAND:**

(inaudible 16:25:38) weak, but then if you think of it as a, you've got a long – if you think of it like a diaphragm that's been braced with two end panels and two walls and that there is a, there is a reasonable rigidity to that element.

**COMMISSIONER FENWICK:**

Yes, you can do a yield line allowances for it.

**DR HYLAND:**

You can do, yes, even with two bolts what we've found is that you can get enough enhancement out of that panel to, if you just took the bolts down the outside face along the edge of the structure you can get enough there to force increased demand on the column head to push it to capacity at less than the 1% drift. So we're saying we can get column failure at 1% and if we have enhancement from the spandrel panels it just makes it worse if we can get failure even less than that, at drifts less than the 1%.

**COMMISSIONER FENWICK:**

Right, thank you.

**MR ALLEN:**

Dr Hyland, that was the point that I was about to make. How did you account for the uncertainty as to whether or not there was interaction with the spandrels and your evidence just now is that you modelled both scenarios with and without spandrel contact and found in both instances, failure?

**DR HYLAND:**

Yes, that's right, so there was, the capacity of the column was, were limited and they were, didn't look like they could get to a 1.5% drift, it failed at 1% in that area. You put the spandrel effect on it, makes it worse, they could have failed at less than 1%. If you added lower than reduced concrete strength makes it less again, if you add in vertical accelerations as its less again so you know it just –

**MR ALLEN:**

Just another possible factor that could account for the collapse?

**DR HYLAND:**

Yes, yes, correct. So having looked at all those vulnerabilities, having discussed everything with colleagues, panel looking at the evidence from witnesses, from the materials, we built up these collapse scenarios and there's basically what we found is that everyone agreed that internal column failure at a mid to low level was the ultimate cause of the collapse. How you got there, there was debate and there remains debate with people. Did you start with diaphragm disconnection from the north core? Did you start with east wall failure of columns? Did you start with a column in the middle of the building failing because of high axial load? Perhaps low concrete strength in common with drift? Did you get detachment of the north core up at the higher levels starting it? There's even, you know, did the floors themselves, there's another scenario, did the floors fracture and break which I understand Professor Mander's got a view on. But ultimately it comes down to a column failure. There's a range of things which build in it; we've got all those other factors which effect capacity, the detailing, the gaps between the columns and spandrel panels, the concrete strength, the vertical seismic loads, there's also the issue of the ability of the structure to resist progressive collapse so once one thing failed was there ability of the structure to cope with that and isolate the collapse to just a few elements rather than for the whole building to go down. So the scenarios, the collapse initiation scenario is where we looked at was, okay, east or south face column failure initiating which is scenario one; scenario two was internal column failure on line 2 or 3, so this is where there is just the collapse started with a weak column or something going wrong internally. Level 3 was column failure following floor slab diaphragm disconnection at the north core at levels 2 and 3. This was an option brought forward by Dr Clifton who assisted me when I looked at this and –

**JUSTICE COOPER:**
Doctor who, sorry?

**DR HYLAND:**
Clifton. The fourth one was column failure followed by floor slab disconnection at the north core level 4, 5 and 6. So we looked at all of these because they're all quite valid scenarios. All of these are quite valid and, you know, people will champion one or the other, you know, but ultimately they're all valid, they're all quite reasonable to look at and what we really had to do was say well how can

we evaluate these and come up with what we believe to be the most consistent with the physical evidence that we've seen, our analyses and come up with a conclusion. And our preferred scenario, the scenario preferred by the panel at a majority level was scenario one, and this was what we presented during the presentation report to the public in February. I'll just talk about the scenario one because of that –

**JUSTICE COOPER:**
Just before you do, scenario one as you described it, was east or south face column failure on line F41. The discussion that you are about to go on to, this document is east face column failure initiation and what happened to the south face column initiation? Is that still of equal weight in your view as the scenario based on east face column failure?

**DR HYLAND:**
The initiation of column failure on the south face would seem to be probably occurring at the same time as the east face but the issue for development of progressive failure from there is harder to justify. The reason being that on the south face the south wall itself was able to hold up the beams that were attached to it and that was found in the collapse evidence that there was still some of those beams were still attached to the south wall even after it had fallen down so while the column might have collapsed or failed it may not have then led to the progressive –

**JUSTICE COOPER:**
... that failed part, I mean the stacking of the materials on the site too, as described by Messrs Heywood and Frost, suggested to me anyway that south wall seemed to collapse somewhat after other things had collapsed. As to that you wouldn't have had the benefit of the organised statements that we got from Messrs Frost and Heywood earlier in the hearing presumably?

**DR HYLAND:**

No, we, I interviewed Graeme Frost early on in the investigation but I hadn't seen Dr Heywood's evidence until his submission was put online.

**JUSTICE COOPER:**

So you've read it now?

**DR HYLAND:**

Yes, yes.

**JUSTICE COOPER:**

And does that, what does that? How do you react to that in terms of sequencing of failure?

**DR HYLAND:**

Yeah, well, I found it very interesting because I hadn't seen a lot of debris evidence down the west side and at the north-western end which he had a lot of. So I was very interested to look at that and I saw what stood out to me first of all was that he showed a slide where he said there was a distinct eastward shift of the level 5 and 6 floor slabs by about 1.5 metres from the western side at the south end and he also said that there was a northward push and it appeared to me that that was also consistent with the level 5 and 6 swinging down on to the level 4, perhaps pushing that out northward.

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He also mentioned that he had perhaps a three to three and a half metre difference between the slab that had been north by, by that three metres possibly around line 2 in that zone there he, I saw he said he hadn't perhaps, he sort of talked about a valley or something of, you know, where that rupture zone had been and I believe that is consistent with the view of some of the witnesses where they saw some upper levels coming down as a unit and I wasn't able to identify specifically whether it was level 5, level 6, level 3, what they were talking about, but there did seem to be a consistent view of a portion of the building coming down and seeing the eastward displacement in the slab of 1.5 metres that's at the south end of level 5 or 6; to me that indicates level 5 and 6 where perhaps the portion that dropped as a unit. The rupture in the slab also, is also consistent with the collapse scenario one I believe, and that the, as the building started to drop, as the collapse progressed along line 2 that would've have pulled the slabs downwards, was of quite high tensile, the stresses in the slab back to the core and back to the south wall pulling those down perhaps then leading to a rupture of the slab at some point away from line 2 and going (inaudible 16:36.58) so its, yes, so I feel it is reinforced it, given more information. I was interested in the evidence by Ron Godkin because he also mentioned this undulating slab as the building was collapsing and shows there would be more significant vertical accelerations under going on as the building was more shaking and yeah. Okay, so this is the collapse sequence that we've preferred, so we are saying, we are talking about hinging developing top and bottom of the columns on the east face and again on the west, on the south face. Then, as perhaps, as we are getting contact with spandrel panels the demand is increasing at the hinges at the tops of the columns, getting to, we can get up to the drifts of the 1% and then getting a loss of load capacity. It is possible also that we may have initiated some failure also at the termination zones of the reinforcing in the columns and just wanting to point out this is not a, this is not meant to be a plastic hinge here, it is just showing contact and a bending effect. So we move to this so we get a drop or a loss of load carrying capacity on that line F which then means that those loads then have to be taken somewhere else and that load then spreads into the columns adjacent to them internally in the building; so we'd be talking about column C6 and C12 then having to pick up higher loads than what they were intended to take. So we see a loss here, the load is then transferring down the column into the more vulnerable area of the column at the lower levels and then collapse occurring there, so we are saying the line F scenario is a load, a loss of load, not necessarily a collapse occurring on that line, but a loss of load which then causes load to be shifted into the internal columns which then start to overload and we get this progressive collapse sequence starting where the, and a drop also too, or a tilt in the upper level as that upper level starts to gradually sink, that may be the way the east would drift or the tilt to the east developed and so we just get a gradual, like a stack of cards the load just progressively move along the structure towards the west; gradually moving over perhaps then pulling the beams out of the columns on the western wall. Now then looking, so what I was just showing there was a section through the building along line 2, east-west, so you are looking northwards if you were standing in the building you were looking northwards. At the same time as the collapse progressing westward from the east face it is also progressing northwards through the building. So we have the starting point of one and then the line 2 column losing capacity and you can see this pulling or this dropping as the column drops the stress is developing in the slabs leading to, which is shown now, I mean, I didn't show this here because I hadn’t picked that up but having seen what Dr Heywood has shown that then this, will then lead to these slabs here and here starting to tear away, or break off as the –

**JUSTICE COOPER**:
So what line is that?

**DR HYLAND:**
It will be line 2. So quite possibly pulled away there and then the loading, I mean as those drop then the load shifts to line 3 and then everything just starts to pull down.

**MR ALLEN:**
Dr Hyland that slide there and the previous two are demonstrating the, what you earlier described as the preferred collapse scenario, that you preferred, along with Mr Smith following discussions with the expert panel. Each of those slides is headed up, “Likely collapse sequence.” Now that’s a loaded word, likely, not everyone understands it in the same way. I note the DBH report describes it as a, “Possible collapse scenario.” What do you mean when you speak to this slide with the heading, “Likely”?

**DR HYLAND:**

To me it's the likely or preferred, most preferred, that seems to fit the evidence as I see it.

**MR ALLEN:**

Thank you. I just wanted to clarify the meaning of the word "likely" in this clause.

**JUSTICE COOPER:**

What about Mr Smith?

**MR SMITH:**

I think it's consistent with things we've seen, even the recent witness statements or the experts statements.

**JUSTICE COOPER:**

You are referring to?

**MR SMITH:**

The Frost and –

**JUSTICE COOPER:**

Heywood?

**MR SMITH:**

Heywood, yes.

**JUSTICE COOPER:**

You say it is consistent with what they said but do you agree that with the word "likely" as Dr Hyland has explained it to you?

**MR SMITH:**

Yes, I agree.

**COMMISSIONER CARTER:**

I could just seek a bit of thought to, if you had a crippling of the column by the, by the interference with the spandrels, and just looking at the way those, that mechanism might fill it, would that not occasion a sort of a, a northward or a southward collapse of that frame?

**DR HYLAND:**

Um –

**COMMISSIONER CARTER:**

I am sort of imagining a –

**DR HYLAND:**

– it being a sort of a –

**COMMISSIONER CARTER:**

– swaying towards the direction in which it collapses, having swayed to the point where it fails you'd expect there to be some residual movement in that direction as the building came down, and the way you've drawn it is as if it would happen to the south but it would just as equally apply if you had drawn the sketches so that they collapsed to the north. I just wonder if there's any evidence of northward? I think there wasn't much movement to the north on that wall. There was on the west wall but not on the east wall?

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**DR HYLAND:**

Ah, it's quite interesting. I actually agree with what you're saying there because, and it's something, I've since the report, really, ah, because when we've talked with Ron Godkin he and I've looked at Margaret Aydon's evidence, um, they both talk about a, I think Ron Godkin shows – talks about a stapler being thrown into a window across the corridor and then, you know, the thing collapsing. My view is the stapler thrown eastwards then that's a sign that something's come back the other way, you know, the inertia has thrown it off so he seems to be describing a final eastward lurch of the building as it collapsed. The south wall with its damage on the, the compression damage on the lower side seems to indicate that it may have had a final eastward lurch on it to the east. That is consistent if you have an eastward movement then that's also consistent with that line F moving north, you know, because of the twisting. So, and then, the other interesting thing is the twist in the north core that we found from the survey that there seemed to be more permanent set in the core on the eastern side of the north core than the western so put those together, it would seem that there was perhaps an eastward twist with a northward movement on line F as you say.

So why did the building collapse? So how did it collapse? We've talked about the investigations; so why did it collapse? In our view specific factors that contributed or may have contributed collapse include severe earthquake aftershock. There is no doubt about that. There was, column drift capacity was substandard. The seismic gaps between the columns and the spandrel panels was substandard. Some column concrete test strings were substandard. Unsymmetrical layout and large strength differential between the south wall and north core. There was seismic separation on the masonry infill on the west wall but it appears to have been compromised to some extent and there appears to be substandard construction joints in the south wall. These are conclusions that were in the report. We also said stronger than design level ground shaking there's repeats of things there. Limited robustness in tying together the building was another issue. There wasn't this redundancy or alternative load path that could have happened but really that's the consequence of not getting those requirements for the group to, beams and columns to comply with the limited ductile or ductile design. If that criteria had been enacted then I think we'd have seen different performance in those frames and maybe the collapse wouldn't have progressed as the way it did. So a summary of the findings –

**MR ALLEN:**

Just before we move on to the Summary of Findings, Dr Hyland, you've said that what you've just spoken to now was taken from the report and are conclusions as set out in the report. The slides that follow headed up, “Summary of Findings” are not a summary of findings that we find as such in the report but they’re your own summary as a result of having investigated the collapse?

**DR HYLAND:**
Yeah, they are, sorta, they are a summary I guess, taking into account what we’ve perhaps seen since then as well and on reflection of what we have in the report; there are a number of things in the report there, you know, as you reflect on them you start to see things come out of that as well.

**JUSTICE COOPER:**
That is understood.

**DR HYLAND:**
So you can sort of, I think you can read those, they are all there.

**JUSTICE COOPER:**
Can I just ask, this summary is one the both of you agree with?

**DR HYLAND:**
Ah, I mean I’ve even got refinements to what I think about the conclusions in the report so, broadly speaking, fine. I do have refinements which I can talk to, to clarify that at some stage, yeah.

**JUSTICE COOPER:**
When is that envisaged?

**MR ALLEN:**
Immediately, Sir. You’ve (inaudible 16:49:50) my very next question which was, having brought us to the conclusions of the report, (inaudible 16:49:57) with you Mr Smith in light of what you now know –

**JUSTICE COOPER:**

Sorry to interrupt but we do have people in the back of the room who are very interested in this and don't have this in front of them so I think it would actually be better if these were read by you aloud and we can display the pages as he goes through them.

**DR HYLAND:**

Okay, so the first finding was the earthquake was severe but the building appears to have collapsed at inter-storey drifts less than those expected by the standards. The second one was a number of collapse scenarios were considered. The collapse was most likely initiated in substandard concrete columns along the east face building at levels 3, 4 and 5. I, probably now, having heard the evidence; I'm thinking now it is more likely at level 4. Columns designed in accordance with the standards would have been expected to be safe at drifts of 1.5%. The columns along the north and east face of the building – at levels 2 to 4 – were estimated at drift capacities between 1.15 and 1.45%, which is less than that. It appears that these east face columns may have failed at drifts of less than 1% prior to drag bar failure at the north core. So specific factors that contributed or may have contributed to the column failures include: columns that do not have the amount of spiral confining and shear reinforcing steel required by the design standard. There was no specific seismic gaps between the spandrel panels and the columns so it was really left to chance. The south wall may have begun to yield and lose thickness at drifts as low as .4% due to structural asymmetry. Vertical accelerants may have reduced column drift capacity. Smooth construction joints in the south wall may have slipped and increased inter-storey drifts. It's hard to quantify that. The concrete in some of the columns had test strengths less than the minimum strength specified and the seismic separation gaps between the infill masonry on the west face and the structure appear to have been compromised and may have changed the response of the structure.

**MR ALLEN:**

In the second bullet point there, there was no specified seismic gaps. Is that what you're ....?

**DR HYLAND:**

Yeah, that's correct. Yeah, I mean if they had been placed perfectly and the columns had the same 400mm dimension as specified, whereas we know the columns and construction will have a variance on their diameter, which is allowed within the Concrete Construction Standard, had been placed perfectly then you could have had 10mm gap but we don't know if they were placed perfectly. We found that critical connections of the floors to some of the north core walls were omitted in the original design. They were only identified during a pre-purchase structural review three years after construction. So it was only because of the pre-purchase structural review that it was picked up. The Council did not have any record of the remedial works that were subsequently undertaken. The drag bars installed could not sustain the ultimate design response to the structure so there was limitations in the design approach taken there, which may have been because of the limitations of the requirements of the standard. The building did not appear to have suffered significant structural damage in the 4 September 2011 earthquake or in the aftershock on the 26th of December 2010, as based on the interviews and the witness statements we've seen. There was only one column below level 6 which was found to have a crack in it and that crack was not considered significant enough for it to be repaired. Then, just a summary – this is based on our investigation and our site examination and, you know, that's the limitations of our evidence and just in highlight we discovered the investigation was not to identify culpability but was purely to identify technical reasons for the collapse.

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**MR ALLEN:**

So that's your summary of findings Dr Hyland, to which you subscribe today in light of everything you now know?

**DR HYLAND:**
Sure.

**MR ALLEN:**
If I could ask the same of you Mr Smith? Do you still subscribe to the conclusions of the report you've prepared in light of everything you know?

**MR SMITH:**
Well I'll just, if I can just talk to the slide that's up first, ah, the second bullet point, "The building did not appear to have suffered significant structural damage," I think we just need to qualify that a little bit, that because there were some areas that were not inspected so we don't actually have confirmation were they, were they damaged or not? So I would say there was no evidence of significant structural damage but there were some areas that were not inspected.

**DR HYLAND:**
Oh, okay, yeah, so there's a qualification I want to make there too, in the, with regards to damage, ah, at the time of the report and actually at the time of putting together this presentation I hadn't read the evidence of, is it Michael Higgins?

**JUSTICE COOPER:**
Yes.

**DR HYLAND:**
And where he noticed that there was damage to the connection of C18 into the north core, ah, they interpreted that as being a lintel beam that had been connected in but, um, that was actually part of the north core and so seeing that, to me, that is actually significant structural damage. It is significant structural damage but, um, it wasn't, it did not appear to have caused the actual collapse on the February earthquake.

**MR SMITH:**

Can I just, if I'm going to, to tell the truth, we made these last minute alterations to this presentation and I wouldn't like to be held to the wording in that summary but I'm quite happy to talk to my thoughts on the conclusions in the report because things have shuffled around here. It's Clark's wording and you asked me if I also agree –

**JUSTICE COOPER:**

Yes.

**MR SMITH:**

It's just the, some qualifications on some items I believe as –

**JUSTICE COOPER:**

Well, I would like to, I mean we are going to adjourn shortly but he'll be back on Monday, as you will be, so if you want to part company with anything that is said in the summary, I would like you to tell us what that is?

**MR SMITH:**

I can comment on the conclusions in the report because I have, if you wish, or are we adjourning?

**JUSTICE COOPER:**

Well yes if you wish to do that but there's the process that I have suggested that you deal with this summary, the findings, not likely to elucidate any areas?

**MR SMITH:**

Okay, I'm quite happy to comment on Monday, that's fine.

**JUSTICE COOPER:**

Is that all right?

**MR SMITH:**

Sure.

**JUSTICE COOPER:**

Mr Allen, you have rather had these witnesses taken out of your hands, I'm sorry.

**MR ALLEN:**

I'm grateful for it Sir. I'm happy to –

**JUSTICE COOPER:**

Well, for our purposes it is useful to have a summary of findings like this and if Mr Smith, as he is entitled to do, says, "Well I wouldn't put it quite like that, and would prefer to put it in a different way," well he is on oath giving evidence so that is exactly what he should do.

**MR ALLEN:**

Yes.

**JUSTICE COOPER:**

And then on Monday we will, there may be some questions that the Commissioners will also have before cross-examination concludes, maybe some after the cross-examination concludes as well but this is, unless there is something else that can usefully be achieved now I would say we have probably reached the point where we should adjourn.

**COMMISSION ADJOURNS: 5.00 PM**

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